Precast Prestressed Concrete

BRIDGE DESIGN MANUAL

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PRECAST/PRESTRESSED
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PRECAST/PRESTRESSED CONCRETE INSTITUTE

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BRIDGE DESIGN MANUAL

With the sponsorship of
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(Technical Activities Council)
and the
PCI Bridge Producers Committee
(Marketing Activities Council)

Under the direction of

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Foreword

BACKGROUND

Since the mid-1980s, the PCI Bridge Producers Committee and the PCI Committee on Bridges have led plans for the development of this comprehensive Bridge Design Manual. It was the consensus of bridge designers and producers alike that the 1980 publication, *Precast/Prestressed Concrete Short Span Bridges - Spans to 100 Feet* and the 1985 *Design Supplement to Short Span Bridges* had served their original purposes. This Manual incorporates much more thorough information and revisions and updates needed to reflect the extensive changes in the AASHTO *Standard Specifications*, changes in concrete and manufacturing technologies, and coverage of the requirements of the AASHTO *LRFD Specifications*. It also includes such timely topics as continuity considerations, seismic requirements and spliced-beam innovations. It is intended to have multiple uses including marketing, information and education for bridge owners, practicing engineers, and university students.

Furthermore, regional groups and local producers have the ability to add local information such as available sections, preferred systems, allowable bottom tension, loading levels and concrete properties.

In preparation for development of the contents of this Manual, several surveys of design professionals, owner agencies, regional associations, and precast concrete producers were conducted over a span of ten years, the latest of which was in June 1994. The contents of this Manual includes the top 27 topics identified by these surveys. Much of the needed material does not exist in other publications. For example, hand calculation examples using the *LRFD Specifications*, fabrication and erection, and preliminary design aids appear here in a PCI publication for the first time. The examples and charts are given for concrete strengths representative of the state-of-the-art, rather than the conventional values.

Based on the results of the surveys, a planning report was prepared for PCI in August 1994 by Richard Imper, Maher Tadros and Stephen Zendegui. The information from the report was further developed by the PCI Bridge Design Manual Steering Committee and became the outline and plan for the current Manual. Preparation of the Manual began in June 1995 by a team of twenty-six authors, led by Maher Tadros. The Manual development effort consisted of two phases. The first phase of the Manual was published in October 1997 and the second phase scheduled for a year later. Contents of the two phases as well as plans for future additions are given below.

It should be emphasized that the Manual is intended to be continually expanded and updated. As need arises, other topics from the following list such as integral bridges, segmental bridges and bridge repair will be added. Updates are expected to be issued, or otherwise made available, to registered Manual owners as they become necessary and available.

FOREWORD

The Manual is intended to be a national document reflecting the latest knowledge and successful practices. Since precast, prestressed concrete bridges have been in existence for almost fifty years, designers have resorted to a variety of resources, including company brochures, state highway agency manuals, reference books and computer software. These resources were fully utilized in the development of this Manual. The authors, however, avoided making recommendations based on individual local practices, or on ideas or concepts that have not been implemented in actual field conditions. It is hoped that state highway agencies will use this Manual as their main design guide, and supplement it with local criteria and details as needed.

There are a number of computer programs for design of precast concrete bridges. Neither the authors nor PCI certify or endorse any of these programs. Rather, the Manual is intended to explain the theories and practices of bridge design, regardless of the software used in design. Some of the work presented in the Manual is based on computer runs using several programs for verification purposes. However, the Manual's use by designers does not require accessibility to any of these programs. A popular method of utilizing personal computers in design is the use of spreadsheet software. The examples given throughout the Manual are documented in adequate detail to allow designers to develop their own spreadsheet programs for similar design tasks.

OBJECTIVES

The Manual is intended to provide a comprehensive document for the design, fabrication and construction of bridges using precast or precast and prestressed concrete components, including precast, post-tensioned products. The document is limited to precast concrete elements produced in permanent manufacturing plants. It presents recommendations recognizing all the best current industry practices available for use by designers. Its flexible format allows for changes that occur in the industry. It is intended to provide both advanced information for experienced designers and basic information to designers, students and educators who are not familiar with bridge design. It explains and applies the major AASHTO *LRFD Specifications* provisions pertaining to prestressed concrete beams in addition to the current AASHTO *Standard Specifications* criteria. In addition, it provides preliminary design aids to help in selecting cost-effective bridge systems and in sizing of precast concrete members.

CONTENTS

Chapter 1, Prestressed Concrete Bridges - The High Performance Solution: This is a general promotional chapter with extensive illustrations. It includes the benefits of precast concrete bridges for both new construction and rehabilitation. It gives examples of successful projects. This chapter includes introduction of various types of precast concrete products made nationally for bridge construction, and examples of bridge beam shapes.

Chapter 2, Material Properties: Key properties of all major materials currently used for precast, prestressed concrete bridge structures are explained in this chapter. It also reviews concrete constituent materials and mix requirements for strength and durability, hardened concrete properties, prestressing and post-tensioning reinforcement, nonprestressing reinforcement and concrete grouts. High performance concrete is discussed. The chapter features a reference list of more than 120 relevant standards and publications by AASHTO, ACI and ASTM. A chart cross-references the identical AASHTO and ASTM standards.

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Chapter 3, Fabrication and Construction: This chapter describes the complete fabrication process and implications for design. It will help in educating the design professional about general precast industry practices and explains product components and details. It explains the impact different materials have on production. Quality and fabrication control are described. Also covered are product evaluation and repair, camber, sweep and accelerated curing. Transportation and erection are covered including use of cranes, launching trusses and temporary support towers. Field-placed concrete for decks and diaphragms is discussed.

Chapter 4, Strategies for Economy: Discusses the options that designers have to even further improve the cost-effectiveness of precast, prestressed concrete bridges. Six sections outline and describe topics such as: geometry (span/depth, vertical and horizontal curves, skewed ends and flared spans); designer options (structural system selection, diaphragms, strand profiles, reinforcing details, bearing systems, high strength concrete); fabrication systems; shipping and erection methods; and, the use of other very economical precast products.

Chapter 5, Aesthetics: Sets forth guidelines by which aesthetics can become a part of an engineer's design technique, including geometry, superstructure type, pier shape, abutment shape, surface treatment, signing, lighting and landscaping.

Chapter 6, Preliminary Design: Discusses the criteria that must be considered early in bridge planning. Provides a host of charts and design graphs to assist in the selection of common bridge beams. Issues discussed include structure-type, hydraulics, construction, utilities, safety and aesthetics. Piers, abutments and foundations are discussed. Beam design charts are included for voided and solid slab beams, box beams, AASHTO I-beams, AASHTO-PCI bulb-tee beams, deck bulb-tee beams and double-stemmed beams.

Chapter 7, Loads and Load Distribution: This subject is outlined and discussed according to both the AASHTO *Standard Specifications* and *LRFD Specifications*. Live load effects are emphasized and moments and shear forces discussed. Live load distribution factors are listed and described for common types of precast concrete superstructures. Findings and recommendations are presented concerning refined methods of analysis.

Chapter 8, Design Theory and Procedure: Extensive review of design procedures that includes: introduction to prestressed concrete fundamentals; critical section and fiber locations; estimation of number of strands; cracking moment; ultimate flexure; and maximum and minimum reinforcement limits. Also included: bond, transfer and development lengths; shear; loss of prestress; deflection; slab design and analysis and detailing for creep effects at pier joints.

Chapter 9, Design Examples: Three design problems illustrate the step-by-step process for design. Each design case is solved by both the AASHTO *Standard Specifications* and the *LRFD Specifications*. Bridge types included are a simple-span adjacent box beam bridge, typical simple-span AASHTO-PCI bulb-tee beam bridge and a three-span bulb-tee beam bridge made continuous for live load and impact. Each example solution provides details, explanation and precise reference to the applicable specification section.

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Chapter 10, Bearings: Contains selection and detailing guides for elastomeric bearings and an introduction to other types of bearings. Examples illustrate the AASHTO Standard Method A, Method B and AASHTO LRFD procedures.

Chapter 11, Extending Spans: Describes the effectiveness of various methods of extending span capacity such as the use of high strength concrete, continuity, spliced beams and post-tensioned beams. Discusses effects on substructure geometry. Contains numerical design examples and examples of successful details of recently constructed bridges.

Chapter 12, Curved and Skewed Bridges: Covers ordinary highway and specialty bridges. Emphasizes stringer bridge systems. Looks at effects of skew and curvature on design and detailing of both superstructure and substructure. Issues related to handling and transportation are also covered.

Chapter 13, Integral Bridges: Future addition.

Chapter 14, Segmental Design and Construction: Future addition.

Chapter 15, Seismic Design: Simplified as well as detailed analysis are presented. Connections between precast concrete superstructure and substructure for seismic resistance are covered as well as numerical examples.

Chapter 16, Additional Bridge Products: Covers design and detailing of such precast bridge products as piers, abutments, full-depth deck panels, stay-in-place composite deck panels, piles, pile caps, railings, culverts and earth retaining systems.

Chapter 17, Railroad Bridges: Includes the specific requirements for railroad bridges, and the benefits of precast concrete. Covers typical product details and construction considerations. Includes worked-out examples.

Chapter 18, Load Rating Procedures: Describes strength evaluation including rating factors and load testing. Covers analysis and load distribution methods according to both AASHTO *Standard* and *LRFD Specifications*.

Chapter 19, Repair and Rehabilitation: Future addition.

Chapter 20, Piles: Future addition.

Chapter 21, Recreational Bridges: Future addition.

Appendix A, Notation

Appendix B, AASHTO/PCI Standard Products

Appendix C, Local/Regional Products

Appendix D, Sample Specifications

Appendix E, Glossary

Appendix F, PCI Certification Programs

THE REVIEW PROCESS

This Manual has undergone a series of reviews during its preparation. The process was under the direction of the Project Manager who in turn worked with the guidance of the Manual Steering Committee. Formal reviews were performed by a Manual Review Panel and a Blue Ribbon Panel. The members of the Manual Review Panel were not asked to review all chapters of the Manual. Each member volunteered to review the chapter or chapters where they felt they could provide expertise. Some members of the MRP did review all chapters. The process was developed to ensure that the Manual meets the quality standards of the Precast/Prestressed Concrete Institute, includes state-of-the-art information and is representative of industry practices and procedures. Members of the Manual Steering Committee, the Manual Review Panel and the Blue Ribbon Panel were approved by the PCI Board of Directors and are identified in the Acknowledgments section of this Manual.

The outline of the contents of the Manual was first submitted to the Manual Review Panel (MRP) for review. The revised outline was then provided to the principal chapter authors for their use in planning content. All communication between PCI and the individual authors was conducted by the Manual's Principal Author. The principal chapter authors generally employed the services of two or more co-authors to assist in writing and review.

Upon completion of a first draft, or "Author's Draft," the manuscript was submitted to PCI for review by the MRP. As comments were received, they were sent back to the authors for consideration for inclusion.

A second draft, or "Ballot Draft," was again reviewed by the MRP. This time, the review accompanied a formal ballot which either requested or required the consideration of the review comments to the satisfaction of the reviewer. The revised manuscript was presented to PCI for editing and the final review.

At this stage of development, the Manual manuscript was layed-out and typeset in its final format. It was then submitted to the Blue Ribbon Panel (BRP). Members of the BRP were selected by the PCI Technical Activities Council (TAC) and the Manual Steering Committee. Selection was based on the members' expertise and to satisfy the requirement by PCI policies that all technical publications be reviewed and approved by TAC.

The BRP review was returned with a formal ballot (with the same rules as the earlier Ballot Draft by the MRP). BRP comments were incorporated, the manuscript given a final edit, then released for printing.

Future changes to the Manual will undergo a similar review process. However, the MRP will be replaced with the combined members of the PCI Bridge Producers Committee and the Committee on Bridges. The BRP review will be undertaken by TAC.

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Note: Each chapter contains its own thorough table of contents.

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Precast, Prestressed Concrete Bridges – The High Performance Solution

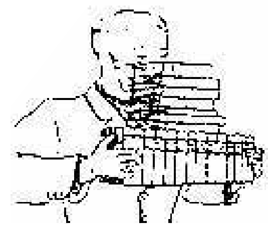
Since its introduction in the United States in 1949, precast, prestressed concrete has rapidly become the preferred composite material for bridge design and construction. Today, it remains the solution of choice for transportation agencies and their bridge designers across the country. This growth came, and will continue to come, from the commitment of precasters to develop, improve, and implement advanced materials, products and technology all aimed at enhancing the performance of these bridges and the options available to the designer.

This publication is intended to provide the designer with an understanding of the precast, prestressed concrete industry and an introduction to the application of this material to bridge design and construction.

Growth of the Industry

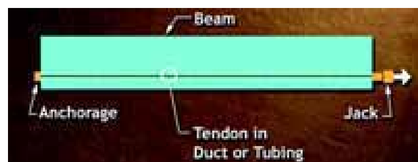
The combination of prestressed high strength steel to counteract tensile stresses, and high performance concrete to provide compressive strength, makes this unique composite material adaptable to many situations, especially to the design and construction of bridges.

Professor Gustav Magnel, one of the pioneers of prestressed concrete, explained it very simply to his students by using a stack of books. When concrete is precompressed, as the lower row of books are, it can carry not only its own weight but also a significant amount of superimposed loads, represented by the books on top.

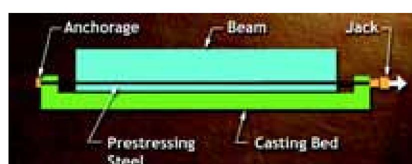


There are two ways of introducing prestress into a concrete member:

• Post-tensioning applies to concrete where steel strands or bars are tensioned against the concrete after the concrete has hardened. Cement grout is usually pumped to fill the duct.



• Pretensioning applies to concrete where steel strands are tensioned between abutments before the concrete is placed in the forms. After the concrete has hardened, force in the strands is transferred to the concrete by releasing anchors at the abutments. The transfer of force occurs through the bond between concrete and steel.



Walnut Lane Memorial Bridge
Photo: ©Lawrence S. Williams, Inc.

The single most important event leading to the founding of the precast, prestressed concrete industry in North America was the construction, in 1949 and '50, of the famed Walnut Lane Memorial Bridge in Fairmont Park, Philadelphia, Pennsylvania. From a technical perspective it is innovative, and from an historical perspective, it is fascinating that the Walnut Lane Memorial Bridge was constructed with prestressed concrete. Consider that there was very little published information on the subject and no experience with linear prestressing in this country. The bridge became a reality through a fortunate sequence of events, and the vision, courage and persistence of a few extraordinary individuals.

The 1950s was the decade that saw the introduction of 7-wire prestressing strand, plant pretensioning, long-line steel casting beds, chemical admixtures, high early-strength concrete, steam curing and many other innovations. These developments coupled with the technical and logistical support provided by the Precast/Prestressed Concrete Institute (PCI), chartered in 1954, fostered the rapid growth of the industry. Applications of precast and prestressed concrete designs quickly began to appear in a wide variety of impressive structures. By 1958, there were more than 200 prestressing plants in the United States.

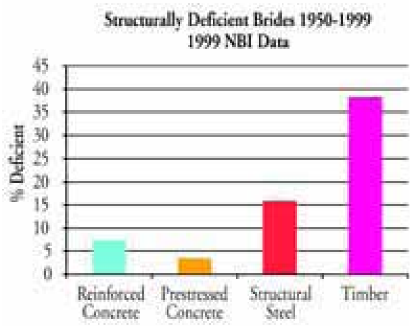
Precast and prestressed concrete products, while designed in accordance with evolving engineering standards, gained an excellent reputation because the industry, early on, recognized the need for quality above all else. PCI's Plant Certification program quickly became an integral part of plant production. PCI Plant Certification assures specifiers that each manufacturing plant has been audited for its processes and its capability to consistently produce quality products.

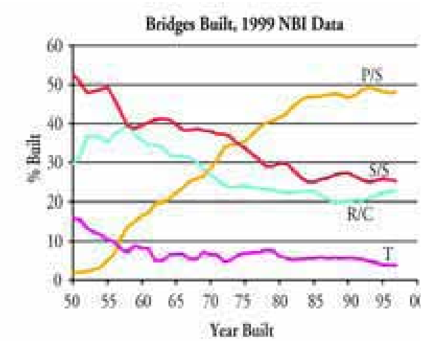
Performance of Prestressed Concrete Bridges

The National Bridge Inventory, maintained by the Federal Highway Administration (FHWA), reveals that of about 475,000 bridges with spans of 20 feet and more, 173,000 are rated as substandard.

The fact that a bridge is "deficient" does not imply that it is unsafe or is likely to collapse. It may be either structurally or functionally deficient. A deficient bridge may need significant maintenance, rehabilitation or sometimes, even replacement. Proper load posting, restricted use and various other methods of traffic control can allow these bridges to continue to be used.

What is causing the nation's bridge problem? One contributing factor is age – the average age of all bridges is now about 45 years. Another factor is increasing vehicle sizes and weights, as well as traffic volumes, that are well beyond what many structures were designed for when they were put into service. A third major factor was limited corrosion resistance in coastal regions and the increasing use of de-icing salts in cold climates. These salts seep through and under the bridge decks, corroding reinforcing bars in decks, in beams and in substructures. Salts readily attack exposed steel members.





Source: National Bridge Inventory Data

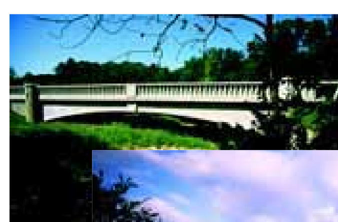
Studies of the National Bridge Inventory data clearly indicate the superior performance of prestressed concrete bridges when compared to the performance of other materials of an equal age.

In addition, owners and designers have long recognized the low initial cost, low maintenance requirements and extended life expectancy of prestressed concrete bridges. This is reflected in the increasing market share of prestressed concrete, which has grown from zero percent in 1950 to about 50 percent now. It's the only structural material to have experienced continuous growth during this period.

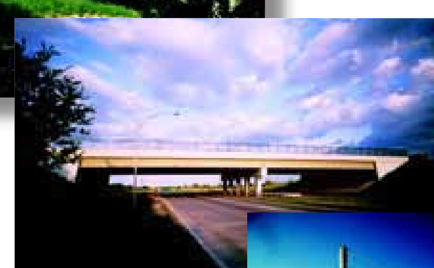
This growth is not only reflected in short-span bridges, but is also now occurring for spans over 150 feet. These spans have been the exclusive domain of structural steel for many years.

Precast concrete bridges have also been shown to be highly durable and fire resistant, and they have excellent riding characteristics. Precast concrete bridges can be installed during all seasons and opened to traffic more rapidly than any other permanent type of bridge. In addition, very slender bridges can be achieved with solid slabs, box beams, multi stemmed units and I-beams. The clean, attractive lines of concrete beams help bridge designers meet the most demanding aesthetic requirements.

Since 1950, tens of thousands of prestressed bridges have been built and many are under construction in all parts of the United States. They range in



size from short spans



to medium spans

to some of the largest bridge projects in the world.

Advantages of Prestressed Concrete Bridges

There are several good reasons why precast, prestressed concrete bridges have gained such wide acceptance. Some bridge designers are surprised to learn that precast, prestressed concrete bridges are usually lower in first cost than other types of bridges. Coupled with savings in maintenance, precast bridges offer maximum economy. Case-after-case can be cited at locations throughout the United States, and these bridges are attractive as well as economical.

Low Initial Cost

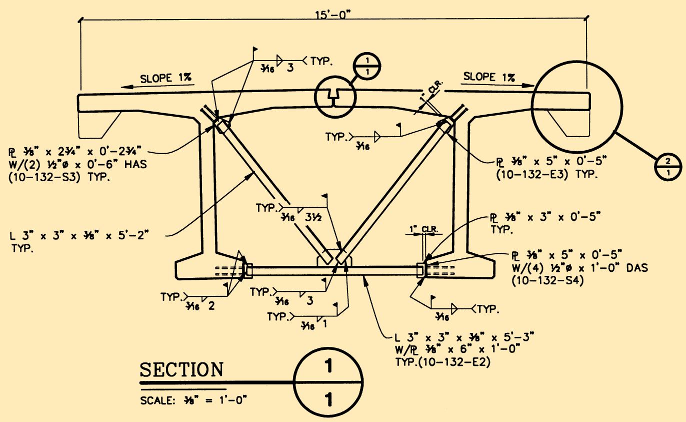
The state of Minnesota saved more than 16% – half a million dollars – by planning for a prestressed alternate to a steel bridge. The 700-foot-long bridge is jointless up to the abutments and is the longest continuous bridge in the state. It also contained the state's longest single concrete span. A Minnesota transportation official stated, "Originally, we didn't think concrete was suited to this…bridge. However, the fabricator showed us it was a viable alternative. Everything went smoothly…we're well satisfied…"

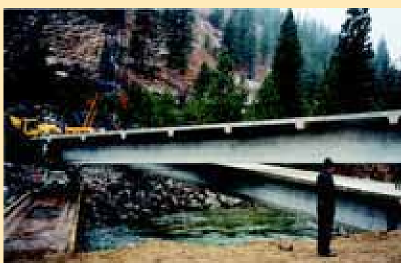
The overall economy of a structure is measured in terms of its life-cycle costs. This includes the initial cost of the structure plus the total operating costs. For stationary bridges, the operating cost is the maintenance cost. Precast, prestressed concrete bridges designed and built in accordance with AASHTO or AREMA specifications should require little, if any, maintenance. Because of the high quality of materials used, prestressed members are particularly durable. Fatigue problems are nonexistent because traffic loads induce only minor net stresses.

Minimal Maintenance

On the Illinois Toll Highway System, during 1957 and 1958, the superstructures of more than 250 bridges were built with precast prestressed concrete I-beams. They span up to 90 feet and some of them have precast stay-in-place deck panels, precast diaphragms, and 94 use spun-cast, hollow cylinder pile column bents. They have withstood heavy traffic, severe weathering and very high salt applications. Yet, these bridges have required very little maintenance. Other projects in all parts of North America have exhibited similar experience – little or no maintenance has been required on precast prestressed concrete bridges.

Of course, no painting is needed. Some bridge engineers estimate the life-cycle cost of re-painting steel bridges to be 15 to 25% of the initial cost. Painting bridges is environmentally unfriendly and can be especially expensive when accomplished over busy highways, streams and railroad rights-of-way, or in rugged terrain.



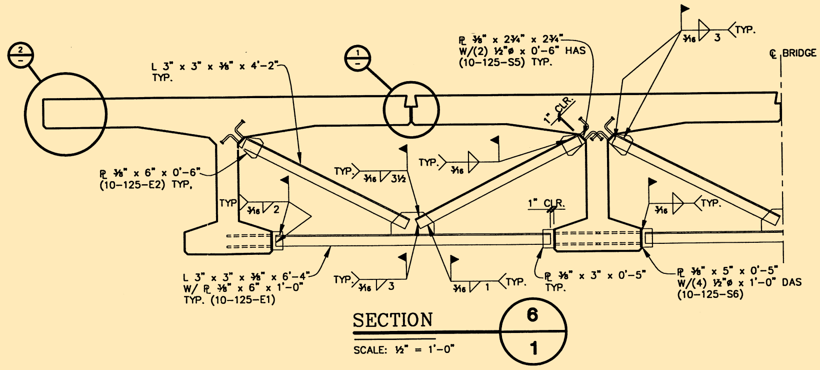


Durable Concrete

One of the reasons for selecting prestressed concrete beams with integral precast decks for this bridge was the durability of prestressed concrete and the resulting low maintenance requirements. As a result of a winter flood, the single lane bridge on a major forest road was washed out, cutting access to a U.S. highway for a half dozen homes…including one with an elderly resident needing continuing medical care. In only 15 days of receiving plans, the precaster had fabricated the two, 135-foot-long spans with 7'-6"-wide integral decks, and the bridge was opened to traffic 3 days later – 18 days in all. The U.S. Forest Service stated that the bridge was least expensive, fastest and the best solution.

Precast, prestressed concrete bridge components are easy to erect, particularly when the tops of the units form the entire deck slab – called an integral deck bridge. Formwork and site-cast concrete are eliminated. Connections between these adjacent units often consist of welding adjoining plates and grouting a continuous keyway. Carefully planned details speed the construction process and result in overall economy.

Simple Solution



Replacing this bridge on US Route 95 in Idaho illustrates another example of the advantages of very fast, yet simple construction:

New Year's Day: Rains and melting snow washed out this bridge over the Little Salmon River linking the northern and southern parts of the state.

January 4: The Idaho Department of Transportation contacted the precaster to investigate solutions. They determined that the fastest way to replace the three spans was to use a single 80-foot span comprised of bulb-tees with an integral deck. The top flange would be 8-inches thick and 8'-6" wide. The diaphragms would also be precast onto the ends of the girders.

January 8: Engineers in the Bridge Section approved shop drawings and tensioning calculations.

January 18: Bulb-tees were shipped 240 miles and set in place…just 17 days after the flood! Included in the shipment were intermediate steel diaphragms, guardrail posts and guardrail…all the components to complete the structure.

January 25: The project was completed. The bridge was in service just 24 days after the flood!

Integral deck bridges can be set on precast or other abutments and erected through practically any weather. They can be opened to traffic very rapidly.

All Weather Construction

In Ketchikan, Alaska, a bridge on the only highway to the north was washed out when an old dam gave way on October 26. Integral deck girders were selected for the 85-ft span. The 12 girders were designed and precast in the state of Washington, then shipped by rail and barge to Alaska. The girders were installed and the bridge was completed and opened to traffic on December 19 - only 54 days after the washout - despite the problems of design, remote location, great distances, and adverse weather conditions during the onset of an Alaskan winter!

The planned replacement of substandard bridges can be accomplished easily with precast prestressed sections. In some cases, existing abutments can be used, but in others, it is easier and more economical to build new ones, or to utilize precast abutments and wing walls supported on cast-in-place footings.

Fast Construction

Mitchell Gulch Bridge, southeast of Denver, was scheduled for replacement with three, 10 ft by 6 ft cast-in-place box culverts. This would require three months of traffic detour on a key commuter route carrying 12,000 vehicles per day. A contractor-suggested alternate resulted in the replacement of the bridge in less than 48 hours – requiring traffic interruption only from Friday night until Sunday.

The project required driving H-piles in advance of closure, dismantling the old bridge, then installing a precast wingwall and abutment system. Next, prestressed voided slabs were installed and grouted along the joints. Fill was placed over the slabs and compacted. Finally, asphalt paving was laid and the bridge opened to traffic. Commuters on Monday morning weren't any the wiser – exactly as planned!

The replacement of bridges may not always be easy to plan in advance. Fires, floods and accidents are but a few reasons for emergency replacements or repairs. Precast concrete and industry manufacturers have consistently demonstrated response to disasters large and small.

Emergency Response

In1996, the bridge over Salt Creek on I-75 near Venice, Florida, was damaged beyond repair when a tanker loaded with diesel fuel crashed and rolled underneath. The five-span, 330-foot-long bridge required 25 AASHTO beams, 65 ft, 3-1/2 in. long. Exposed precast piles were salvaged by cutting them just below ground line, then splicing on precast extensions. The extensions arrived on-site just two days after they were ordered. The first five beams were delivered and erected four days after production began, and all 25 beams arrived within seven days. The new bridge was reopened to traffic just 18 days after the accident.

In May 2002, two barges hit and collapsed four spans of the I-40 bridge over the Arkansas River near Webber Falls, Oklahoma. Fourteen people were lost. Originally steel, three spans were replaced with 36, 72-in.-deep precast bulb-tee beams, 130-feet long. After a spectacular effort by the entire design and construction team, the bridge was opened to traffic in just 65 days. State officials stated that, "…precast concrete offered us a speed advantage over replacing the entire bridge with steel."

Interstate 65 in Birmingham, Alabama was brought to a standstill on a Saturday morning in January 2002, when a tanker load of gasoline crashed and burned under a steel bridge. The state quickly designed a replacement bridge and construction began only 16 days after the accident. Prestressed concrete bulb-tee beams, 54-in. deep and 140-ft long, were used in the new bridge, which was both wider and some 20-ft longer to provide for additional future lanes. Using high strength concrete that achieved 8,500 psi in 14 days, the span-to-depth ratio is an impressive 31:1. Fabrication of the beams required only 15 days. The new bridge was opened to traffic just 65 days after the accident and 36 days after construction began. A state designer said that precast concrete "…could be cast and delivered to the jobsite before steel fabricators could even procure material and start fabrication." The general contractor said, "There was no way we could have gone with steel girders because the lead time was prohibitive. The precast was on site within a very short period of time."

A common requirement of bridges is that the superstructure be as shallow as possible in order to provide maximum clearance with minimum approach grades. Through the technique of prestressing, the designer is able to utilize the maximum possible span-to-depth ratio. Span-to-depth ratios as high as 35:1, or even more, can be achieved with solid slabs, voided slabs, box beams, multi-stemmed units, I-beams or bulb-tee sections, each within their respective span ranges. Even though deeper sections will require less prestressing steel, the overall economy of a project may dictate the shallowest available section.

Slender Bridges

The Sedley Bridge provides a crossing for county Rt. 475W over the Norfolk Southern/CSX Railroad tracks in Porter County, Indiana. Faced with severe clearance and approach embankment constraints, the designer chose a unique through-girder solution that resulted in a 112-foot span having an effective structure depth of just 14 inches.

The Yale Avenue Bridge carries Interstate 25 over Yale Avenue, a busy urban arterial in Denver, Colorado. The structure was Colorado's entry in the Federal Highway Administration's High Performance Concrete Showcase program. It is designed for traditional Interstate highway loading. The adjacent, single-cell box beams measure 67 in. wide by 30 inches deep and use 10,000 psi concrete (at 56 days). The bridge has two continuous spans (for live load) of 100 and 114 ft and is 138-ft wide. Composite topping has a minimum thickness of 5 in. for a total structure depth of 35 in. and a span-to-depth ratio of 39:1.

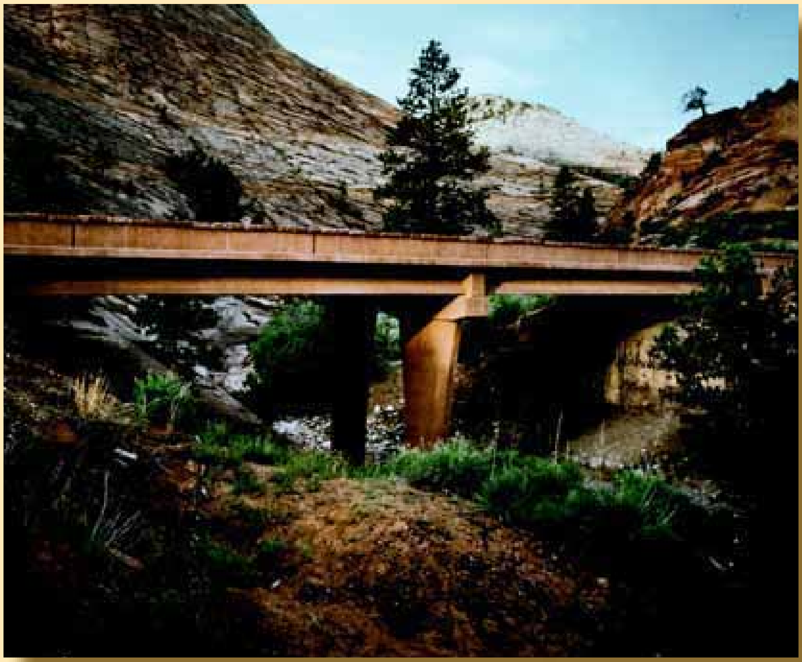
The San Angelo (Texas) Bridges, carrying U.S. 87 over the North Concho River and South Orient Railroad, are parallel, eight- and nine-span structures. One bridge used primarily conventional concrete and the other, high performance concrete as part of the Federal Highway Administration's HPC Showcase program. Designed as simple spans, one used 0.6-in.-diameter strands with 13,500 psi concrete to achieve a length of 157-ft with 54-in.-deep beams plus 3-1/2-in.-thick precast concrete deck panels plus 4-1/2-in. cast-in-place composite concrete topping to achieve a 30.4:1 span-to-depth ratio.

The Clarks Viaduct located in Omaha, is a four-span bridge over U.S. Highway 30 and the Union Pacific Railroad. It has a 52-degree skew and spans of 100, 151, 148 and 128.5 ft. The superstructure is a modified Nebraska 1100 beam, 50-in. deep, using 8,500 psi concrete. The beams sit on unique, individual cast-in-place pier tables to extend their spans. The beams are made fully continuous for superimposed dead loads and live load by splicing high-strength reinforcement extended from the ends of the beams through the cast-in-place tables between the ends of the beams. Including the 7-1/2-in. deck, the span-to-depth ratio is 31.5:1.

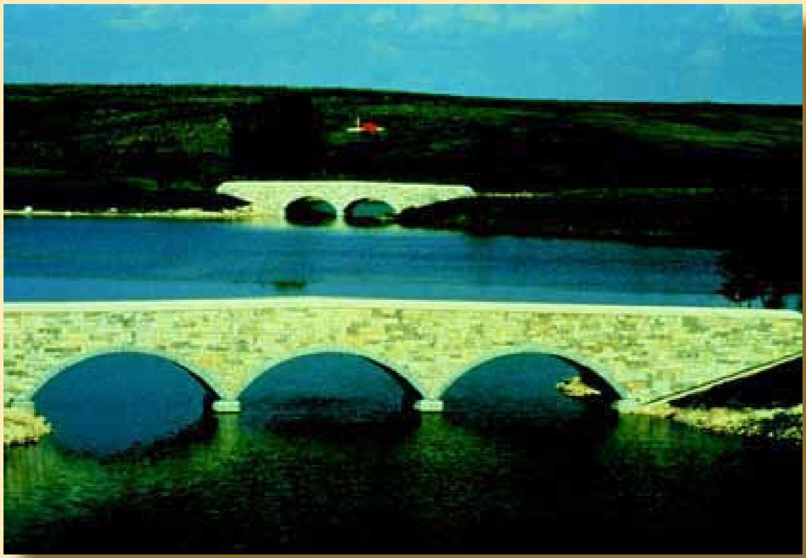
Beams that include integral decks, such as this one, can achieve exceptionally high span-depth ratios. In addition, they can be installed very quickly while requiring little site-cast concrete.

Regardless of how they are viewed, prestressed concrete bridges are attractive from above, below, and from the side because of the simple and clean shapes of the members used. The high span-to-depth ratios made possible with prestressing, result in strong, tough, durable and yet graceful bridges.

Aesthetic Bridges



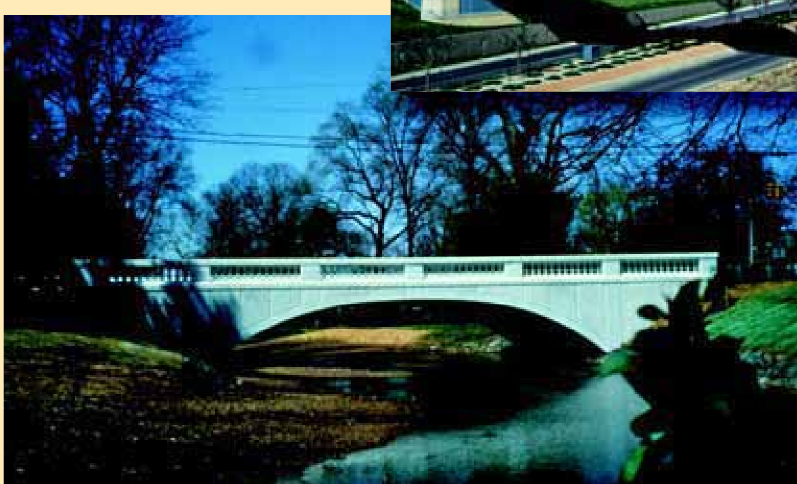
Two very different parks use precast concrete in special ways. The Bridge over Clear Creek, Zion National Park, Utah, uses colored aggregate, sandblasting and pigments to match the bridge to the surrounding native stone. Costing just $60/SF, the project was considerably less than either steel or CIP.

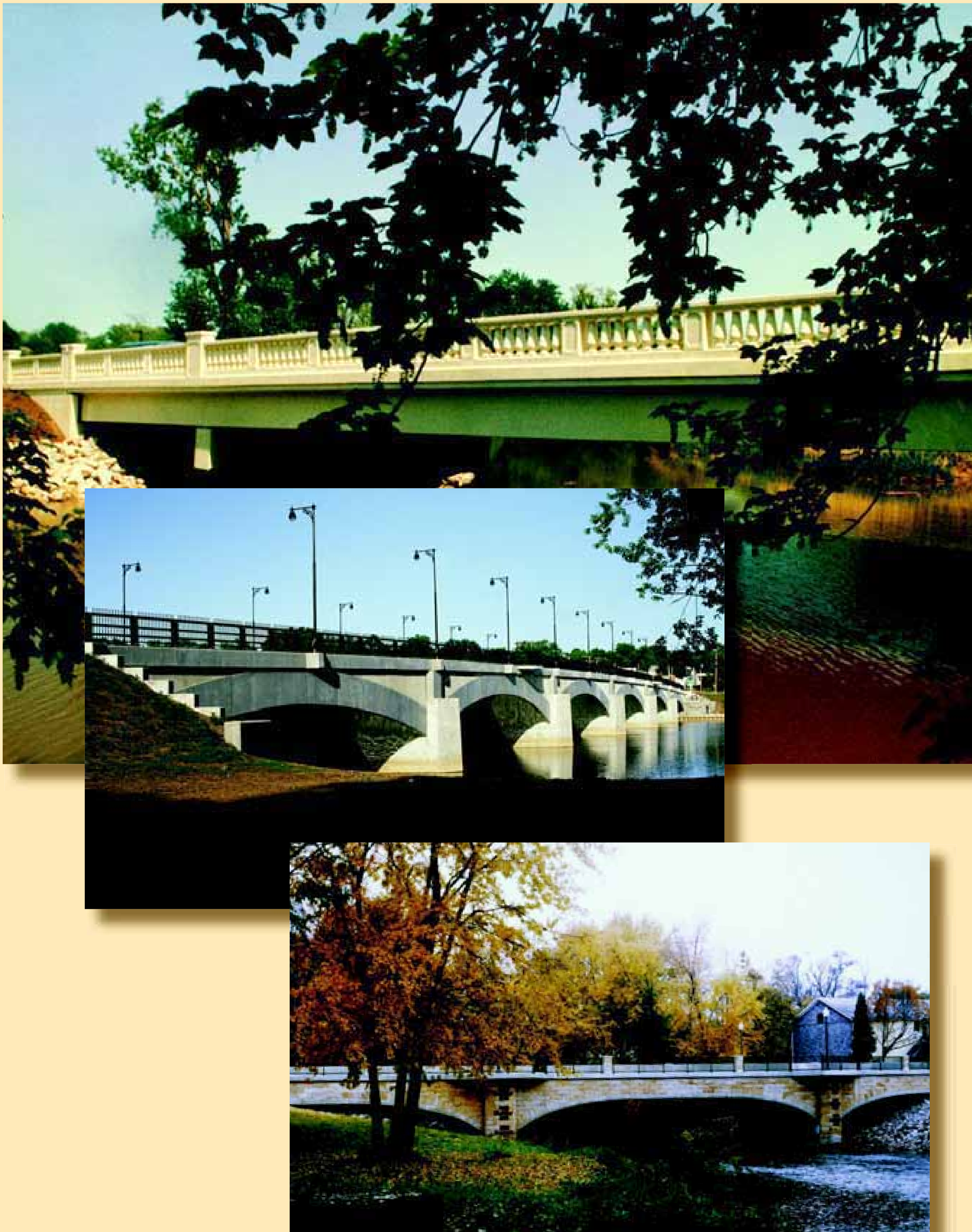


Two bridges in Kil-Cona Park in Winnipeg provide an attractive compliment to these family recreational surroundings.

Attractive Bridges

More and more often, designers are adding architectural and aesthetic treatments to precast bridges. These include panels that create an arch appearance or decorative railings. Some solutions are shown in the accompanying photos.





Bridges are subjected to a hostile environment as well as repeated impact loadings. Some must endure intense sun, high temperatures and brackish water. Others must withstand not only the freezing and thawing provided by nature but also the potential for damage induced with the use of de-icing chemicals. High strength prestressed concrete has excellent freeze-thaw and chemical resistance. Also, prestressed concrete bridges are not easily damaged by fire.

Fire Performance

The Washington State Route 509 Bridge over the Puyallup River near Tacoma was damaged in December, 2002, when a railroad car containing 30,000 gallons of methanol burned beneath span number 8. The span is 146 ft in length and uses 15 lines of 74-in.-deep bulb-tee beams. An investigation revealed that the fire reached temperatures of 3,000 degrees F. The study showed that no significant amount of prestress was lost. A plan was immediately developed for repairs that would permit the bridge to remain in service.

After this timber deck truss bridge burned, an extremely busy 2-lane link was severed between two major population areas.

It was replaced by a safe, low maintenance, prestressed concrete bridge with a record span for this area of 141 ft. It was erected without falsework over an environmentally sensitive, salmon-bearing river. It opened seven months after bid.

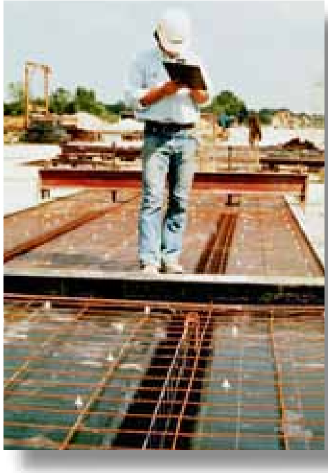
Steel girder bridges frequently exhibit disturbing vibrations. The natural frequency of vibration of these bridges can coincide with the frequencies of traffic and then resonance occurs. There are documented cases that show that light bulbs in fixtures installed on steel bridges burn out more rapidly because of such vibrations. There are indications that concrete decks on steel bridges need replacement significantly sooner then concrete decks cast on concrete girders. The natural frequency of vibration of prestressed girder bridges, because of their mass and stiffness, does not coincide with vehicle frequencies. The public will feel safe, secure and comfortable when riding on prestressed concrete bridges. Owners report that decks are less likely to crack prematurely when built on stiff concrete bridges.

Excellent Riding Characteristics

The public will not only be safe but they will feel more secure and comfortable on a concrete bridge that holds traffic vibrations to an absolute minimum. Long continuous spans and integral abutments eliminate or reduce expansion joints for a smoother ride and reduced maintenance.

Quality Assurance

Prestressed concrete is economical because it is an efficient composite of high-strength steel and high performance concrete. To take advantage of this efficiency, precasting plants have developed sophisticated quality control programs that assure the customer that products meet exacting specifications.



Precast prestressed concrete products are rigorously inspected and quality is controlled at the precasting plant. In fact, each operation in the manufacturing process provides for a point of scheduled inspection and control.

During fabrication and handling, portions of prestressed concrete beams are subjected to some of the highest stresses they will ever encounter as structural members. So, in a sense, prestressed members are load-tested during fabrication, handling and installation.

Engineers put their professional reputation on the line whenever they specify a structural material. This requires that they work with the most reputable and qualified sources.

A plant that is PCI Certified tells the engineer several important things:

- The facility has demonstrated production and quality control procedures that meet national industry standards.
- A nationally recognized, independent consulting engineering firm conducts at least two unannounced annual audits. The auditors are accredited engineers. The firm is engaged by PCI for all audits nation-wide.
- Each plant must maintain a comprehensive Quality System Manual (QSM) based on national standards and approved by PCI. The QSM is available for review by owner agencies.

The rigid audits cover more than 150 items. Standards are based on the Manual for Quality Control for Plants and Production of Structural Precast Concrete, PCI manual MNL-116. The audits evaluate concrete materials and stockpiles, concrete mixing, transporting, placing, consolidation and finishing. Procedures are inspected for tensioning of strands and transfer of prestress; concrete curing and temperature controls; product stripping, handling and storage. In-house QC procedures are reviewed thoroughly. In addition, engineering, shop drawings, record keeping and many other practices related to quality production are examined.

- QC personnel must be PCI-Certified, attained by passing written and practical examinations.
- The designer will know that the producer has PCI confirmed capabilities and that the producer stands behind their products.

Failure to maintain acceptable standards makes loss of certification mandatory.

Totally Precast Concrete Bridges

Work zones and detours are difficult problems faced by highway agencies. Using precast concrete and with techniques such as integral deck bridges, traffic interruptions can be minimized because of the availability of plant-produced sections and the speed of erecting and completing the bridge.

The versatility of the precast, prestressed concrete industry provides the designer with many options. Can one use precast bridge components to build an "Instant Bridge"? Almost! There are many ways to put a bridge together with precast concrete products.

In addition to the well known superstructure elements – girders and deck slabs – substructure components can be precast.

Precast concrete piles are quite popular in many parts of the country. They come in different sizes and shapes, ranging from 10-inch-square piles to 66-inch-diameter cylindrical piles such as this 172-ft-long unit.

In addition, pile caps can be precast.

Piers and abutments can also be made of precast concrete pieces quickly assembled in the field.

There are many benefits to using precast concrete elements to construct prefabricated bridges. They include:

- A single contractor working with only one familiar material can control the schedule for erection of the entire bridge.
- Precast concrete structural elements are made in manufacturing plants under controlled conditions in advance of need and stockpiled for "just-in-time" delivery and erection.
- No need for curing cast-in-place concrete: precast bridge piers can be erected in one working day and beams can be erected immediately following the piers.
- Corrosion resistance and excellent concrete quality is provided through in-plant manufacture of all of the structural elements.
- Fully cured precast concrete structural elements can be delivered to the site. These elements contain little potential for additional shrinkage or creep.
- Owner agencies complete more work in a shorter period of time, resulting in:

  Reduced cost of handling traffic
  Reduced accident exposure
  Reduced inconvenience to the traveling public
  Fewer motorist complaints
- Contractors benefit from:

  Reduced exposure of personnel to traffic hazards
  Greater dollar volume of work accomplished in a shorter period
  Fewer delays due to weather conditions
  Less dependence on remote delivery of ready-mixed concrete
- Lower costs for:

  Forms
  Cranes
  Skilled field labor
  Scaffolding and shoring
- The same crane already needed on the job site for erecting beams and girders may be used for erecting bridge piers and other elements.

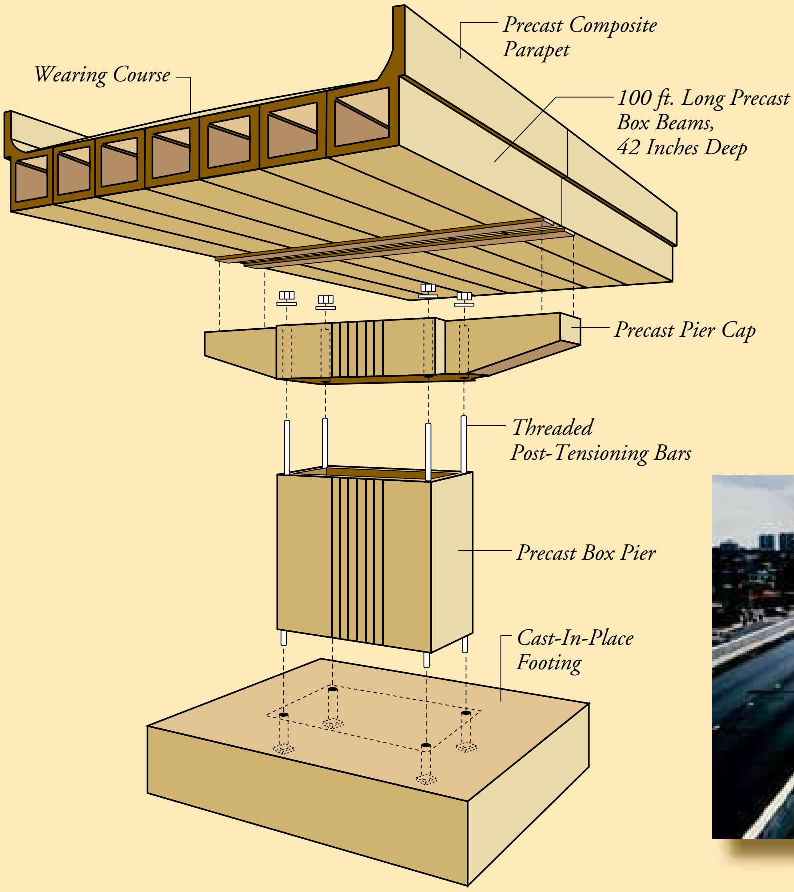
- Reduction of motorist delays, complaints and accidents. According to a report by the Texas Transportation Institute, costs incurred by drivers passing through a work zone, along with engineering costs, can be $10,000 to $20,000 per day. In urban areas, a federal report states that the cost of work zones can reach $50,000 per day.

Minimal Traffic Disruption

In San Juan, Puerto Rico, the four, totally precast concrete Baldorioty de Castro Avenue bridges were built in record-setting time, attractively and economically.

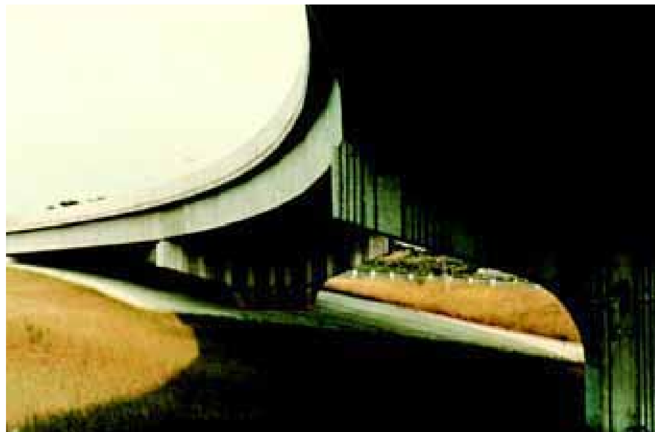
Each of four bridges, ranging in length from 700 to 900 feet, was erected in less than 36 hours – that's from the time traffic was re-routed on Friday night until traffic resumed over the new bridge on Saturday or Sunday! This included the piers, the superstructure, the overlay and lighting. It was well within the owner's construction allowance of 72 hours per bridge; a condition established to minimize disruption to one of the city's most highly traveled corridors.

In addition to speed, the bridges also met the city's budgetary needs. The four box-beam bridges were constructed for $2 million less than the next lowest bid for another material. In addition, the bridges will prove durable and maintenance-free, adding value to this investment.



The Future

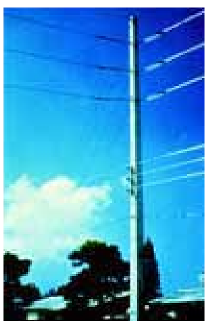
Innovation in bridge construction has been, and will continue to be the ongoing focus in the precast concrete industry. The development of horizontally curved precast concrete bridges is one such example out of the past.



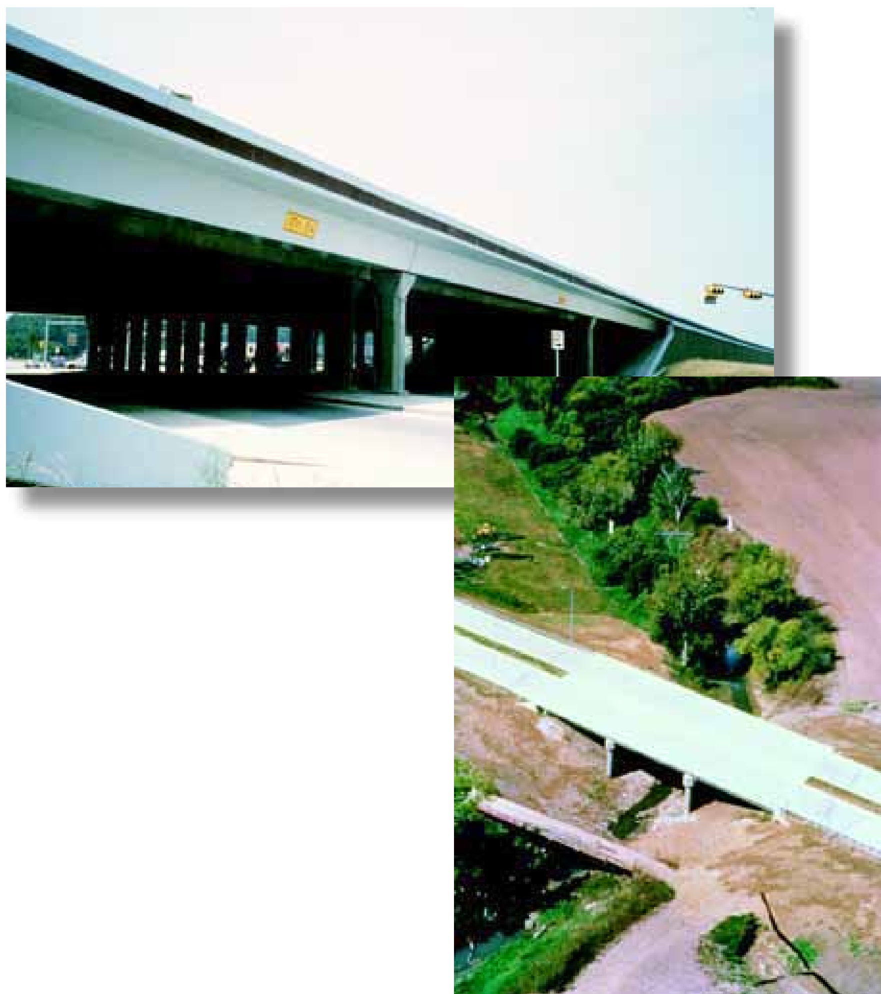
Another development was the use of precast deck panels. Used as stay-in-place forms, the panels improve safety on the jobsite, reduce field placement of reinforcing steel and concrete for bridge decks, resulting in considerable savings. The panels become composite for live loads with the field-placed concrete and are now common in many states.

Material properties, such as corrosion resistance, fire resistance and durability have been improved in a process of continuous evolution. These inherent qualities of precast, prestressed concrete together with a high degree of design flexibility also make it ideal for a wide variety of other applications such as poles, storage tanks, retaining walls, railroad sleepers and sound barriers. All have benefited from plant standardization and the production repetitions achieved from it.




Concrete in the 12,000 to 14,000 psi range is already commercially available. The Louetta Road Bridge in Houston, Texas and the 120th Street and Giles Road Bridge in Sarpy County, Nebraska, both completed in 1996, are examples of bridges with 12,000 to 14,000 psi concrete girders and 5,000 to 8,000 psi concrete decks. Further, the Louetta Road Bridge utilizes high strength precast concrete hollow segmental piers. The Federal Highway Administration, jointly with PCI and numerous states, has consistently promoted the use of High Performance Concrete in bridge applications. High Performance Concrete often involves higher than average compressive strength. But other factors, such as stiffness, permeability and abrasion resistance, in addition to strength, may be requirements of High Performance Concrete. This often depends on the geographic location of the bridge and the component for which it is used.

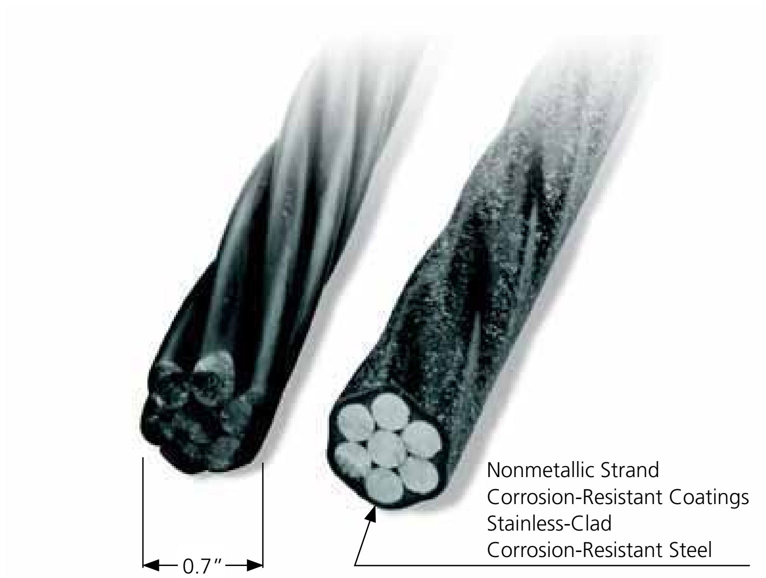


The benefits of High Performance Concrete include: 1) reduced initial construction costs resulting from wider beam spacing and, 2) longer spans and reduced long-term costs that result because of fewer replacements and fewer repairs.

Lightweight aggregate concrete with strengths in the 7,000 to 10,000 psi range is possible. Lightweight concrete reduces dead loads and results in lower seismic forces.

Synthetic, organic and steel fibers have been shown to improve toughness and shrinkage cracking. Recent developments in high performance fiber-reinforced concrete hold promise in terms of performance and cost-effectiveness.

Strands of larger diameters and higher strengths will become more common as higher strength concretes are used and the demand for higher prestress force increases. When 0.6-inch diameter strands are used in conjunction with high strength concrete, in the 10,000 to 12,000 psi range, standard I-beams and other products have significantly increased span capabilities. Standard products can be stretched to spans never thought possible before. Epoxy-coated and enhanced strands will further increase product versatility.



Nonmetallic reinforcement such as glass, carbon and aramid fiber composites will be increasingly used for special applications. A recent demonstration project has shown the compatibility of carbon fiber strands for prestressing a double-tee bridge. Both internally bonded pretensioning and external unbonded prestressing systems were used.

Prestressed concrete got its start as a unique composite material. Further developments by the industry and its suppliers have continued to refine the performance of the product for a wide range of bridge applications.

Today, it gives the public extraordinarily good value for their money.

The reputation of the precast, prestressed concrete industry has been built on the strength, imagination, consistency and integrity of its people and products alike. These attributes will continue to make prestressed concrete the solution of choice for the nation’s bridges... not only today, but far into the future.

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NOTATION

A = constant

A_s^* = nominal area of prestressing steel

B = constant

$C(t, t_0)$ = creep coefficient at a concrete age of t days

C_u = ultimate creep coefficient

$(E_c)_t$ = modulus of elasticity of concrete at an age of t days

f'_c = specified concrete compressive strength

f'_{ci} = the concrete compressive strength at time of initial prestress

$(f'_c)_t$ = concrete compressive strength at an age of t days

$(f'_c)_{28}$ = concrete compressive strength at an age of 28 days

f_f = fatigue stress range in reinforcement

f_{min} = minimum stress level in reinforcement

f_{ps} = stress in prestressing strand

f_r = modulus of rupture

f'_s = ultimate strength of prestressing steel

H = annual average ambient relative humidity

k_c = product of applicable correction factors = k_{la} x k_h x k_s

k_{cp} = correction factor for curing period

k_h = correction factor for relative humidity

k_{la} = correction factor for loading age

k_s = correction factor for size of member

k_{sh} = product of applicable correction factors = k_{cp} x k_h x k_s

K = constant

r/h = ratio of base radius to height of transverse deformation on reinforcement

S = surface area of concrete exposed to drying

$S(t, t_0)$ = shrinkage strain at a concrete age of t days

S_u = ultimate shrinkage strain

t = age of concrete

t_{la} = loading ages

t_0 = age of concrete at the end of the initial curing period

V = volume of concrete

w_c = unit weight of concrete

ε_{ps} = strain in prestressing strand

λ = concrete weight factor taken as 1.0 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete

Material Properties

2.1 SCOPE

This chapter contains a description of the properties of all major materials currently used for precast, prestressed concrete bridge structures. It includes a discussion of concrete constituent materials, mix requirements, hardened concrete properties, pretensioning and post-tensioning reinforcement, nonprestressed reinforcement and grouts used between precast members and other components. Recent developments in high performance concrete and nonmetallic reinforcement are also introduced. Discussion of the materials used in fabrication and construction is included in Chapter 3.

2.2 PLANT PRODUCTS

The production of precast concrete components in a plant environment offers several advantages compared to on-site production. Many of these advantages occur because one company is responsible for quality control throughout production. This results in closer monitoring of raw materials, steel placement, concrete production and delivery, concrete curing and shipment. The overall effect is to produce a product with more consistent material properties than can be achieved with site-cast concrete.

2.2.1 Advantages

In many aspects, the material properties of precast components are superior to those of cast-in-place members. Precast concrete components are required to achieve a minimum concrete strength for release and removal from their precasting beds at an early age (12 to 18 hours). This often results in a concrete that has a 28-day compressive strength in excess of the specified 28-day strength. Consequently, the concrete has a higher modulus of elasticity and less creep than would occur if the actual strength were equal to the specified strength. The use of accelerated curing to achieve the release strength also results in less shrinkage and creep. From a durability aspect, precast concrete members have a low permeability and, therefore, are better suited for use in aggressive environments such as coastal areas and areas where deicing salts are used.

2.3 CONCRETE MATERIALS

The five major component materials of concrete produced today are cement, aggregates, chemical admixtures, mineral admixtures and water.

2.3.1 Cement

Cement for use in bridge construction generally conforms to one of the following specifications:

AASHTO M85 Portland Cement

AASHTO M240 Blended Hydraulic Cement

2.3.1.1 AASHTO M85

The AASHTO Specification M85 lists eight types of portland cement as follows:

Type I	Normal
Type IA	Normal, air-entraining
Type II	Moderate sulphate resistant
Type IIA	Moderate sulphate resistant, air-entraining

Type III High early strength

Type IIIA High early strength, air-entraining

Type IV Low heat of hydration

Type V High sulphate resistance

Type I portland cement is a general purpose cement suitable for all uses where the special properties of other types of cement are not required. Type II portland cement is used where precaution against moderate sulphate attack is important. Type II cement can also be used to reduce the heat of hydration. Type III portland cement provides high strengths at an early age and is particularly appropriate for obtaining high release strengths. Type IV portland cement is used to reduce the heat of hydration and is particularly beneficial in mass concrete structures. Type V portland cement is used in concrete exposed to severe sulphate attack. Types IA, IIA and IIIA, correspond in composition to Types I, II and III respectively, except that small quantities of air-entraining material are included in the cement.

2.3.1.2 AASHTO M240

The AASHTO Specification M240 lists six classes of blended cement as follows:

Type IS Portland blast-furnace slag cement

Type IP Portland-pozzolan cement

Type P Portland-pozzolan cement

Type S Slag cement

Type I (PM) Pozzolan-modified portland cement

Type I (SM) Slag-modified portland cement

Blended hydraulic cements are produced by intergrinding and/or blending various combinations of portland cement, ground granulated blast-furnace slag, fly ash and other pozzolans. These cements can be used to produce different properties in the hardened concretes. Types IS, IP, I(PM) and I(SM) are used for general concrete construction. Type P is used where high early strengths are not required. Type S is used with portland cement in concrete or with lime in mortar but is not used alone in structural concrete.

2.3.1.3 Restrictions

The *Standard Specifications* generally restrict cement to portland cement Types I, II or III; air-entrained portland cement Types IA, IIA or IIIA; or blended hydraulic cements Types IP or IS. It should also be noted that not all types of cement are readily available and that the use of some types is not permitted by some states.

2.3.2 Aggregates

Aggregates for concrete consist of fine and coarse materials. Fine aggregate for normal weight concrete should conform to the requirements of AASHTO M6. Coarse aggregate for normal weight concrete should conform to the requirements of AASHTO M80. Lightweight aggregate for use in lightweight or sand-lightweight concrete should conform to the requirements of AASHTO M195. The maximum size of aggregate should be selected based on mix-requirements and the minimum clear spacing between reinforcing steel, clear cover to reinforcing steel and thickness of the member in accordance with AASHTO specifications. If aggregates susceptible to alkali-aggregate reactivity are used in prestressed concrete members, special precautions must be observed. These include the use of low alkali cements, blended cements or pozzolans.

2.3.3 Chemical Admixtures

Chemical admixtures are used in precast, prestressed concrete to provide air entrainment, reduce water content, improve workability, retard setting times and accelerate strength development. Chemical admixtures, except air-entraining admixtures, should conform to the requirements of AASHTO M194. This specification lists the following types of admixtures:

Type A Water-reducing
Type B Retarding
Type C Accelerating
Type D Water-reducing and retarding
Type E Water-reducing and accelerating
Type F Water-reducing, high range
Type G Water-reducing, high range and retarding

2.3.3.1 Purpose

Water-reducing admixtures and high range water-reducing admixtures are used to allow for a reduction in the water-cementitious materials ratio while maintaining or improving workability. Accelerating admixtures are used to decrease the setting time and increase the early strength development. They are particularly beneficial in precast concrete construction to facilitate early form removal and release of prestressing. Since admixtures can produce different results with different cements, and at different temperatures, selection of admixtures should be based on the plant materials and conditions that will be utilized in production. Compatibility between admixtures is also important and should be specifically addressed when using combinations of admixtures produced by different companies.

2.3.3.2 Calcium Chloride

Calcium chloride has been used in the past as an accelerator since it is very effective and economical. The use of calcium chloride in concrete promotes corrosion of metals due to the presence of chloride ions. Consequently, calcium chloride should not be permitted in prestressed concrete members. Accelerators without chlorides may be used.

2.3.3.3 Corrosion Inhibitors

Corrosion-inhibiting admixtures are also available for use in concrete to protect reinforcement from corrosion. These admixtures block the passage of chloride ions to the steel reinforcement and, thereby, reduce or eliminate corrosion of the reinforcement. Corrosion-inhibiting admixtures are more likely to be effective in cast-in-place bridge components that are directly exposed to chloride ions than in precast concrete bridge girders that are already highly impermeable.

2.3.3.4 Air–Entraining Admixtures

Air-entraining admixtures are used in concrete primarily to increase the resistance of the concrete to freeze-thaw damage when exposed to water and deicing chemicals. They may also be used to increase workability and facilitate handling and finishing. Air-entraining admixtures should conform to AASHTO M154. The air content of fresh concrete is generally determined using the pressure method (AASHTO T152) or the volumetric method (AASHTO T196). The pressure method should not be used with lightweight concrete. A pocket-size air indicator (AASHTO T199) can be used for quick checks but is not a substitute for the other more accurate methods.

2.3.4 Mineral Admixtures

Mineral admixtures are powdered or pulverized materials added to concrete to improve or change the properties of hardened portland cement concrete. Mineral admixtures are used in concrete to increase early strength development or to reduce the heat of hydration. They may also be used to improve the resistance of concrete to reactive aggregates and to replace cement. They have also been used in high strength concrete to produce higher strengths at later ages. The use of mineral admixtures may affect the workability and finishing characteristics of fresh concrete.

2.3.4.1
Pozzolans

AASHTO M295 lists three classes of mineral admixtures as follows:

Class N Raw or calcined natural pozzolans
Class F Fly ash
Class C Fly ash

High-Reactive Metakaolin (HRM) is a manufactured white powder that meets the requirements of a Class N pozzolan. HRM has a particle size significantly smaller than that of cement particles, but not as fine as silica fume. Fly ash is a finely divided residue that results from the combustion of pulverized coal in power generation plants. Class F fly ash has pozzolanic properties; Class C has some cementitious properties in addition to pozzolanic properties. Some fly ashes meet both Class F and Class C classifications. Selection of these materials will depend on their local availability and their effect on concrete properties.

2.3.4.2
Silica Fume

Silica fume meeting the requirements of AASHTO M307 may also be used as a mineral admixture in concrete. Silica fume is a very fine pozzolanic material produced as a by-product in electric arc furnaces used for the production of elemental silicon or ferro-silicon alloys. Silica fume is also known as condensed silica fume and microsilica. The use of silica fume can improve the early age strength development of concrete and is particularly beneficial in achieving high release strengths in high strength concrete beams. The use of silica fume in concrete generally results in concrete that has low permeability. The use of silica fume increases the water demand in concrete. Consequently, it is generally used in combination with a water-reducing admixture or a high range water-reducing admixture. Concrete containing silica fume has significantly less bleeding and the potential for plastic shrinkage is increased. Therefore, early moisture loss should be prevented under conditions which promote rapid surface drying such as low humidity and high temperatures.

2.3.5
Water

Water used in mixing concrete must be clean and free of oil, salt, acid, alkali, sugar, vegetable or other injurious substances. Water known to be of potable quality may be used without testing. However, if there is doubt, water should meet the requirements of AASHTO T26. Mixing water for concrete should not contain a chloride ion concentration in excess of 1,000 ppm or sulfates as SO_4 in excess of 1,300 ppm.

2.4 SELECTION OF CONCRETE MIX REQUIREMENTS

This section discusses various aspects of concrete mix requirements that need to be considered by the owner or the owner's engineer. Selection of concrete ingredients and proportions to meet the minimum requirements stated in the specifications and contract documents should be the responsibility of the precast concrete producer. Wherever possible, the mix requirements should be stated on the basis of the required performance and not be over-restrictive to the producer. The producer should be allowed to show through trial batches or mix history that a proposed mix design will meet or exceed the specified performance criteria. Consequently, prescriptive requirements such as minimum cement content should be avoided.

2.4.1
Concrete Strength at Release

For prestressed concrete bridge beams, the Engineer generally specifies minimum strengths at time of release of the prestressing strands and at 28 days, although ages other than 28 days may be used. The Engineer may also specify a minimum compressive strength at time of beam erection, or a minimum compressive strength at time of post-tensioning if a combination of pretensioning and post-tensioning is utilized. For most prestressed concrete bridge beams, the specified strength at time of release will control the concrete mix proportions. Based on AASHTO specifications, the release strength is selected so that the temporary concrete stresses in the beam, before losses due to creep and shrinkage, do not exceed 60% of the concrete compressive strength at time of release in pretensioned members and 55% of the concrete compressive strength at time of stressing of post-tensioned members. In addition, the strength is selected so that, in tension areas with no bonded reinforcement, the tensile stress will not exceed 200 psi or $3\sqrt{f'_{ci}}$ where f'_{ci} is the compressive strength of concrete at time of initial prestress in psi. In areas with a specified amount of bonded reinforcement, the maximum tensile stress cannot exceed $7.5\sqrt{f'_{ci}}$.

2.4.2 Concrete Strength at Service Loads

The design of most precast, prestressed concrete members is based on a concrete compressive strength at 28 days of 5,000 to 6,000 psi. However, because the mix proportions are generally dictated by release strengths, concrete strengths at 28 days are frequently in excess of the specified 28-day value and actual strengths of 8,000 psi or more are often achieved. Consequently, mix requirements are generally based on the release strengths and the precaster only has to ensure that the mix will provide concrete with a compressive strength in excess of that specified for 28 days.

2.4.3 High Performance Concrete

Concrete with a compressive strength in excess of 8,000 psi has not been commonly specified for precast, prestressed concrete bridge beams. There is, however, a trend toward the greater utilization of higher strength concretes to achieve more durable and economical structures. Some states are using the higher strength characteristics of high performance concrete to stretch spans or widen beam spacings by using beams with concrete strengths in excess of 10,000 psi. In such cases, strength is typically specified at 56 days because of the strength gain that is possible in higher strength concretes between 28 and 56 days.

The minimum compressive strength, in some cases, may be controlled by the need to meet a minimum requirement for special exposure conditions as discussed in Section 2.4.6.2.

2.4.4 Durability

Durability is a concern when bridges are exposed to aggressive environments. This generally occurs where deicing salts are utilized on highways during winter or in coastal regions where structures are exposed to salt from sea water. The Engineer must be concerned about the deleterious effects of freezing and thawing, chemical attack and corrosion of embedded or exposed metals. The ideal approach is to make the concrete as impermeable as possible. In this respect, precast, prestressed concrete has inherent advantages over cast-in-place concrete since it is produced in a controlled environment that results in high quality concrete. In addition, the mix proportions needed to achieve a relatively high strength concrete often produce a relatively impermeable concrete. As a result, precast, prestressed concrete bridge beams have an excellent record of performance in aggressive environments.

2.4.4.1 Freeze–Thaw Damage

Freeze-thaw damage generally manifests itself by scaling of the concrete surface. This occurs as a result of temperature fluctuations that cause freezing and thawing when the concrete is saturated. Freeze-thaw damage is magnified when deicing chemicals are present. To minimize freeze-thaw damage, a minimum air content is generally specified. The presence of entrained air provides space for ice to expand without developing high pressures that would otherwise damage the concrete. **Table 2.4.4.1-1**, based on ACI 211.1, provides the required air

Table 2.4.4.1-1
Total Air Content for Frost-Resistant Concrete

Nominal Maximum Aggregate Size, in.	Minimum Air Content*, percent	
	Severe Exposure	Moderate Exposure
3/8	7-1/2	6
1/2	7	5-1/2
3/4	6	5
1	6	4-1/2
1-1/2	5-1/2	4-1/2

*The usual tolerance on air content as delivered is ±1.5 percent content for severe and moderate exposure conditions for various maximum aggregate sizes. Severe exposure is defined as a climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts come in contact with the concrete. This includes bridge decks. Salt laden air, as found in coastal areas, is also considered a severe exposure. A moderate exposure is one where deicing salts are not used or where concrete will only occasionally be exposed to moisture prior to freezing. This is generally the case for bridge beams. It should be noted that some state highway departments specify air contents that are slightly different from those shown in **Table 2.4.4.1-1**. In addition, many states do not require air entrainment in prestressed concrete beams because beams are sheltered by the deck or other conditions exist such that air entrainment is not required for good performance.

2.4.5 Workability

The ease of mixing, placing, consolidating and finishing freshly mixed concrete is called workability. Concrete should be workable but should not segregate or bleed excessively. Excessive bleeding increases the water-cementitious materials ratio near the top surface and a weak top layer of concrete with poor durability may result. For prestressed concrete bridge beams, particular attention should be paid to ensure that concrete has adequate workability so that it will consolidate around the prestressing strands, particularly at end regions of beams where a high percentage of nonprestressed reinforcement is present. It is also important that concrete can be placed in the webs of beams without segregation. Workability can be enhanced through the use of water-reducing admixtures, high range water-reducing admixtures and air entraining agents. No standard test exists for the measurement of workability. The concrete slump test is the most generally accepted method used to measure consistency of concrete but it should not be used as a means to control workability.

2.4.6 Water-Cementitious Materials Ratio

The water-cementitious materials ratio is the ratio of the amount of water, exclusive of that absorbed by the aggregate, to the amount of cementitious materials in a concrete or mortar mixture. As such, the amount of water includes that within the admixtures and that in the aggregate in excess of the saturated surface-dry condition. The amount of cementitious material includes cement and other cementitious materials, such as fly ash and silica fume. The total cementitious materials content for compressive strengths from 4,000 to 8,000 psi can vary from 600 to 1,000 pcy and will also vary on a regional basis.

2.4.6.1 Based on Strength

When strength, not durability, controls the mix design, the water-cementitious materials ratio and mixture proportions required to achieve specified strength should be determined from field data or the results of trial batch strength tests. The trial batches should be made from actual job materials. When no other data are available, **Table 2.4.6.1-1,** which is based on ACI 211.1, may be used as a starting point for mix design procedures for normal weight concrete.

Table 2.4.6.1-1 Approximate Ratios for Trial Batches

Compressive Strength at 28 days, psi	Water-Cementitious Materials Ratio By Weight	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
6,000	0.41	–
5,000	0.48	0.40
4,000	0.57	0.48

Table 2.4.6.2-1
Maximum Requirements for Various Exposure Conditions

Exposure Condition	Maximum Water-Cementitious Materials Ratio for Normal Weight Concrete
Concrete intended to have low permeability when exposed to water	0.50
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45
For corrosion protection for reinforced concrete exposed to chlorides from deicing chemicals, salt, salt water or brackish water, or spray from these sources	0.40

2.4.6.2
Based on Durability

When durability is a major consideration in the concrete mix design, the water-cementitious materials ratios for various exposure conditions should be limited to the values specified in ACI 318 and shown in **Table 2.4.6.2-1**. For precast, prestressed concrete members exposed to deicing salts or spray from sea water, the maximum ratio will generally be 0.40.

2.4.7
Unit Weight

2.4.7.1
Normal Weight Concrete

The unit weight of normal weight concrete is generally in the range of 140 to 150 pcf. For concrete with compressive strengths in excess of 10,000 psi, the unit weight may be as high as 155 pcf. The unit weight will vary depending on the amount and density of the aggregate and the air, water and cement contents. In the design of reinforced or prestressed concrete structures, the combination of normal weight concrete and reinforcement is commonly assumed to weigh 150 pcf but may be assumed as high as 160 pcf.

2.4.7.2
Lightweight Concrete

Lightweight concrete and sand-lightweight concrete (also called semi-lightweight concrete) may also be utilized in precast, prestressed concrete bridge construction with the use of suitable lightweight aggregates. Lightweight aggregate concretes generally have a unit weight of 90 to 105 pcf. Sand-lightweight aggregate concretes have a unit weight of 105 to 130 pcf with a common range of 110 to 115 pcf. When lightweight concrete is used in prestressed concrete members, special consideration must be given to using mix design procedures for lightweight concrete as given in ACI 211.2.

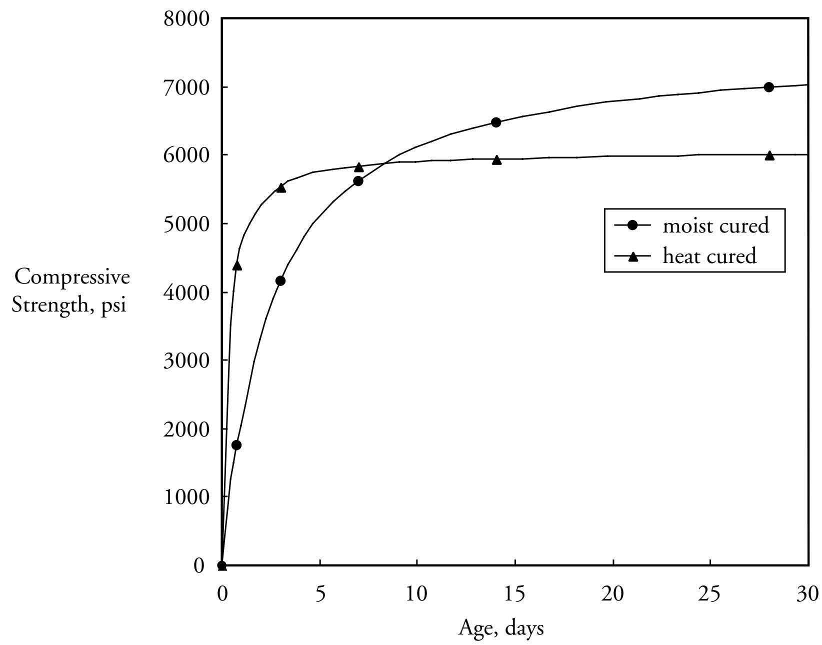
2.4.7.3
Blended Aggregates

Where suitable lightweight aggregates are available, a common practice is to blend lightweight with normal weight aggregates to achieve a desired concrete unit weight. This is done to control beam (or other product) weights to satisfy shipping limitations, jobsite conditions such as crane size or reach limits, or plant or erection equipment capacities.

2.4.8
Effect of Heat Curing

Because of the need for early strength gain, Type III cement is often used in precast concrete so that forms may be reused on a daily basis. This generally requires that the release strength be achieved no later than 18 hours after the concrete is placed and may be achieved at 12 hours or less. To accelerate the strength gain, it is often necessary to raise the temperature of the concrete. In some situations, such as with high strength concrete, the increase in temperature can be provided by the internal heat of hydration. However, in most situations, it is necessary to utilize an external source of heat, such as steam or radiant heat, to reach the necessary release strengths. The use of external heat causes the concrete temperature to be higher at an earlier age than would be achieved from the natural heat of hydration. A consequence of achieving a high release strength is a reduction in the later age strengths compared to strengths that would have been obtained if the concrete had not been heat cured. This is illustrated in **Figure 2.4.8-1.** The effect of heat curing on the concrete compressive strength development must be taken into account in the selection of mix requirements and in the preparation of trial mixes.

Figure 2.4.8-1
Effect of Curing on Concrete Compressive Strength Gain



2.4.9 Sample Mixes

Sample concrete mixes for six different concrete compressive strengths are shown in **Table 2.4.9-1.** These are concrete mixes from different precasting plants. It should not be assumed that these mixture proportions will always produce the same concrete compressive strengths when used with different materials.

2.5 CONCRETE PROPERTIES

2.5.1 Introduction

Concrete properties such as modulus of elasticity, tensile strength, shear strength and bond strength are frequently expressed in terms of the compressive strength. Generally, expressions for these quantities have been empirically established based on data for concretes having compressive strengths up to 6,000 psi. With recent research, these empirical relationships have been reevaluated for concrete compressive strengths up to 10,000 psi. Unless indicated otherwise, the relationships in this section may be assumed applicable for concrete with compressive strengths up to 10,000 psi. Where alternative expressions are available, they are discussed in each section. For concretes with compressive strengths in excess of 10,000 psi, the recommendations given in ACI 363 and Zia et al (1991) should be considered.

2.5.2 Compressive Strength

Compressive strength is generally measured by testing 6x12-in. cylinders in accordance with standard AASHTO or ASTM procedures. The precast concrete industry also uses 4x8-in. cylinders. Some state highway departments permit the use of either 6x12-in. or 4x8-in. cylinders for quality control. For high strength concretes, the use of smaller size cylinders may be necessary because of limitations on testing machine capacities. For precast, prestressed concrete members it is particularly important that the concrete cylinders used to determine release strengths be cured in an identical manner to the bridge members. In general, this is accomplished by curing the concrete cylinders alongside the prestressed concrete member until release of the prestressing strands. A more advanced technique of match curing is also available. In this procedure, the cylinders are enclosed in a container in which the temperature is controlled to match the temperature of the concrete member. The test cylinders then undergo the same time-temperature history as the concrete member.

Table 2.4.9-1
Sample Production Concrete Mixes

Mix	A	B	C	D	E	F
Specified Strength, psi						
Release	3,500	4,000	5,000	6,000	6,000	8,800
28 Days	5,000	6,000	7,500	7,500	10,000	13,100
Quantities per cu yd						
Cement, lb	705	705	850	750	750	671
Fly Ash, lb	0	0	0	140	0	316
Silica Fume, lb	0	0	0	0	95	0
Sand, lb	1,055	1,085	935	1,085	1,030	1,029
Coarse Aggregate, lb	1,790	1,920	1,770	1,980	1,870	1,918
Water, lb	270	285	300	230	230	247
Air Entrainment, fl. oz.	5	0	17	0	3	0
Water-Reducer, fl. oz.	25	53	29	0	10	0
High Range Water-Reducer, fl. oz.	125	0	145	160	85	200
Concrete Properties						
Water-Cementitious Ratio	0.38	0.40	0.36	0.26	0.31	0.25
Slump, in.	3-1/2	4-3/4	4	6	5	9
Unit Weight, pcf	141.5	147.8	140.0	145.0	147.4	UNKN
Air Content, %	6.0	N/A	6.0	N/A	5.0	N/A
Release Strength, psi	3,800	4,350	5,300	6,700	9,070	8,800
28-day Strength, psi	5,700	6,395	8,000	9,400	10,450	13,900
56-day Strength, psi	UNKN	UNKN	UNKN	UNKN	UNKN	15,200

UNKN – Unknown; NA – Not Applicable

2.5.2.1 Variation with Time

The variation of concrete compressive strength with time may be approximated by the following general calculation:

$$(f'_c)_t = \frac{t}{A + Bt}(f'_c)_{28} \qquad \text{(Eq. 2.5.2.1-1)}$$

where:

$(f'_c)_t$ = concrete compressive strength at an age of t days

$(f'_c)_{28}$ = concrete compressive strength at an age of 28 days

A and B = constants

The constants A and B are functions of both the type of cementitious material used and the type of curing employed. The use of normal weight, sand-lightweight or all lightweight aggregate does not appear to affect these constants significantly. Typical values recommended by ACI 209 are given in **Table 2.5.2-1**. The constants for current practice shown in **Table 2.5.2.1-1** are based on the sample mixes shown in **Table 2.4.9-1**. These mixes have release strengths that vary from 63 to 87% of the 28-day strength.

Table 2.5.2.1-1 Values of Constants A and B

Source	Curing	Cement	A	B
ACI 209	Moist	I	4.00	0.85
ACI 209	Moist	III	2.30	0.92
ACI 209	Steam	I	1.00	0.95
ACI 209	Steam	III	0.70	0.98
Current Practice	Heat	III	0.28	0.99

2.5.2.2 Effect of Accelerated Curing

As shown in **Figure 2.4.8-1**, a concrete that is heat cured will have higher initial strengths but lower strength at later ages when compared to the same concrete that is moist cured. It should be emphasized that these are general relationships and variations will occur for different concretes and curing procedures. When fly ash is used as a mineral admixture, it may be appropriate to determine the compressive strength at 56 days to take advantage of the later strength gain. Therefore, it is important that the strength gain relationship be established through trial mixes or previous experience using local producer data. This is particularly important for release strengths which can occur as early as 12 hours. If the relationship is unknown, the values listed in **Table 2.5.2-1** for current practice will give an approximate relationship.

2.5.3 Modulus of Elasticity

The modulus of elasticity is the ratio of uniaxial normal stress to corresponding strain up to the proportional limit for both tensile and compressive stresses. It is the material property that determines the amount of deformation under load. It is used to calculate camber at release, elastic deflections caused by dead and live loads, axial shortening and elongation, prestress losses, buckling and relative distribution of applied forces in composite and non-homogeneous structural members. Modulus of elasticity is determined in accordance with ASTM C 469.

2.5.3.1 Calculations (E_c)

For concrete compressive strengths less than 8,000 psi, the following calculation may be used to predict the modulus of elasticity:

$$(E_c)_t = 33(w_c)^{1.5}\sqrt{(f'_c)_t} \qquad \text{(Eq. 2.5.3.1-1)}$$

where:

$(E_c)_t$ = modulus of elasticity of concrete at an age of t days, psi

w_c = unit weight of concrete, psi

$(f'_c)_t$ = concrete compressive strength at an age of t days, psi

The above equation was based on an analysis for concrete strengths up to about 6,000 psi. According to ACI 363, the above calculation tends to over-estimate the modulus of elasticity for higher strength concretes. Several alternative equations have been proposed for the calculation of modulus of elasticity and the following by Martinez (1982) has received general acceptance:

$$(E_c)_t = \left(40{,}000\sqrt{(f'_c)_t} + 1{,}000{,}000\right)\left(\frac{w_c}{145}\right)^{1.5}$$

2.5.3.2 Variations (E_c)

Deviations from predicted values are highly dependent on the properties and proportions of the coarse aggregate used in the concrete. Consequently, where local producer data are available, they should be utilized in place of the values determined from these standard equations. This is particularly important in computing the camber at release as these modulus of elasticity equations have not been developed specifically for determination of the modulus of heat cured concrete at an early age.

2.5.4 Modulus of Rupture

The modulus of rupture is a measure of the flexural tensile strength of the concrete. It can be determined by testing, but the modulus of rupture for structural design is generally assumed to be a function of the concrete compressive strength as given by:

$$f_r = K\lambda\sqrt{f'_c} \qquad \text{(Eq. 2.5.4-1)}$$

where:

f_r = modulus of rupture, psi

K = a constant, usually taken as 7.5

λ = 1.0 for normal weight concrete

0.85 for sand-lightweight concrete

0.75 for all-lightweight concrete

For high strength concretes, a value of K greater than 7.5 has been proposed. However, for most applications, a conservative value of 7.5 is still used for high strength concretes.

2.5.5 Durability

Durability refers to the ability of concrete to resist deterioration from the environment or service conditions in which it is placed. Properly designed concrete should survive throughout its service life without significant distress. The following test procedures may be used to check the durability of concrete made with a specific mix:

Freeze-thaw resistance	ASTM C 666, C 671 and C 682
Deicer scaling resistance	ASTM C 672
Abrasion resistance	ASTM C 418, C 779 and C 944
Chloride permeability	AASHTO T277 or T259
Alkali-aggregate reactivity	ASTM C 227, C 289, C 342, C 441 and C 586
Sulphate resistance ASTM	C 452 and C 1012

It is not necessary to perform all the above tests to prove that a concrete will be durable. In general, a concrete that has a low permeability will also have a high resistance to freeze-thaw cycles and surface scaling. It should also be noted that a concrete that does not perform very well in the above tests will not necessarily perform poorly in the field. Concrete that performs well in the above tests, will nearly always perform well in an actual structure. This is the case for precast concrete members that are produced under controlled factory conditions.

2.5.6 Heat of Hydration

Heat of hydration is the heat generated when cement and water react. The amount of heat generated is largely dependent on the chemical composition of the cement but an increase in cement content, fineness or curing temperature will increase the heat of hydration. Heat of hydration is particularly important in heat-cured concretes where the heat generated by the chemical reaction of the cement in conjunction with heat curing can be used to accelerate the development of compressive strength. The heat of hydration can be measured using ASTM C 186. When prestressed concrete beams are heat cured, the heat generated by hydration cannot escape from the surface of the member. Consequently, under this condition, the beams may be considered as mass concrete. Procedures for determining the temperature rise in mass concrete are described in ACI 207.1. However, as an approximate calculation, it can be assumed that a temperature rise of 10F will occur for each 100 lb of cement used in the concrete. More precise calculations can be made using the actual concrete mix proportions, specific heat of the concrete and heat generated per unit mass of cement.

2.5.7 Shrinkage

Precast concrete members are subjected to air drying as soon as they are removed from the forms. During this exposure to the atmosphere, the concrete slowly loses some of its original water, causing shrinkage to occur. The amount and rate of shrinkage vary with the relative humidity, size of member and amount of nonprestressed reinforcement.

2.5.7.1 Calculation of Shrinkage

Procedures to calculate the amount of shrinkage and creep have been published in the *LRFD Specifications*, by CEB-FIP (1990) and ACI 209. These procedures are based on the recommendations of ACI 209 which are summarized in this section.

Shrinkage after 1 to 3 days for steam-cured concrete:

$$S(t,t_0) = \frac{(t - t_0)}{55 + (t - t_0)} S_u \qquad \text{(Eq. 2.5.7.1-1)}$$

Shrinkage after 7 days for moist-cured concrete:

$$S(t,t_0) = \frac{(t - 7)}{35 + (t - 7)} S_u \qquad \text{(Eq. 2.5.7.1-2)}$$

where:

$S(t, t_0)$ = shrinkage strain at a concrete age of t days

S_u = ultimate shrinkage strain

t = age of concrete, days

t_0 = age of concrete at the end of the initial curing period, days

Although Eq. 2.5.7.1-1 was developed for steam-cured concretes, it may be applied to radiant heat-cured concretes if more specific information is not available.

In the absence of specific shrinkage data for local aggregates and conditions, the following average value for the ultimate shrinkage strain is suggested:

$$S_u = 545\ k_{sh} \text{ x } 10^{-6} \qquad \text{(Eq. 2.5.7.1-3)}$$

where:

k_{sh} = product of applicable correction factors

$= k_{cp} \text{ x } k_h \text{ x } k_s$ (Eq. 2.5.7.1-3a)

k_{cp} = correction factor for curing period

k_h = correction factor for relative humidity

k_s = correction factor for size of member

Table 2.5.7.1-1
Correction Factor k_{cp}
for Initial Curing Period

Moist Curing Period, days	Shrinkage Factor, k_{cp}
1	1.20
3	1.10
7	1.00
14	0.93
28	0.86
60	0.79
90	0.75

For shrinkage of concrete moist-cured for other than 7 days, the curing correction factor, k_{cp} may be taken from **Table 2.5.7.1-1**.

The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2.** A relative humidity map taken from *LRFD Specifications* is shown in **Figure 2.5.7.1-1**.

Table 2.5.7.1-2
Correction Factors k_h
for Relative Humidity

Average Ambient Relative Humidity, %	Shrinkage Factor, k_h	Creep Factor, k_h
40	1.43	1.25
50	1.29	1.17
60	1.14	1.08
70	1.00	1.00
80	0.86	0.91
90	0.43	0.83
100	0.00	0.75

The above correction factors are based on the following equations:

Shrinkage: $k_h = 2.00 - 0.0143H$ for $40 \leq H \leq 80$ (Eq. 2.5.7.1-3b)

$= 4.286 - 0.0429H$ for $80 < H \leq 100$ (Eq. 2.5.7.1-3c)

Creep: $k_h = 1.586 - 0.0084H$ for $40 \leq H \leq 100$ (Eq. 2.5.7.1-3d)

where H = annual average ambient relative humidity in percent

The size correction factor, k_s, depends on the volume to surface area of the member and may be taken from **Table 2.5.7.1-3.** The volume to surface area ratio for long members may be computed as the ratio of cross-sectional area to section perimeter.

Figure 2.5.7.1-1
Average Annual Ambient Relative Humidity

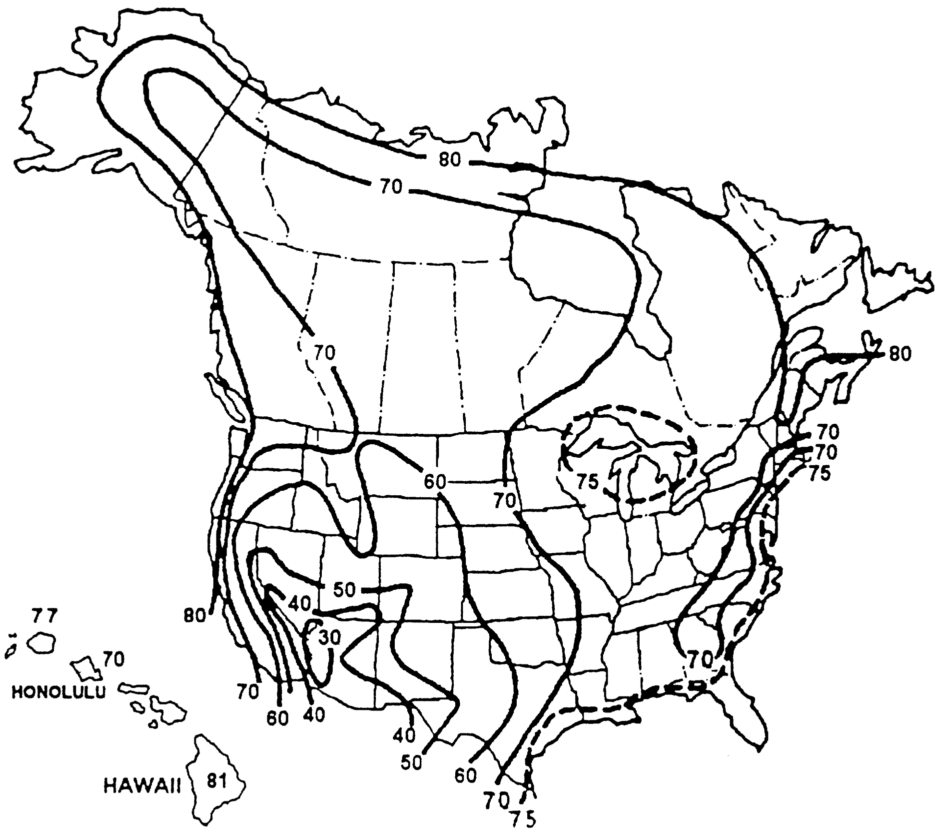


Table 2.5.7.1-3
Correction Factors k_s for Size

Beam Section	Volume/Surface (in.)	Shrinkage Factor, k_s	Creep Factor, k_s
Type I	3.05	0.83	0.81
Type II	3.37	0.80	0.79
Type III	4.06	0.74	0.75
Type IV	4.74	0.68	0.73
Type V	4.44	0.71	0.74
Type VI	4.41	0.71	0.74
BT-54	3.01	0.84	0.82
BT-63	3.01	0.84	0.82
BT-72	3.01	0.84	0.82

The above correction factors are based on the following equations:

Shrinkage: $k_s = 1.2e^{-0.12V/S}$ (Eq. 2.5.7.1-3e)

Creep: $k_s = 2/3\ (1+1.13e^{-0.54V/S})$ (Eq. 2.5.7.1-3f)

where:

V = volume of concrete, in.^{3}

S = surface area of concrete exposed to drying, in.^{2}

2.5.8 Creep

Prestressed concrete beams are subjected to the effects of creep as soon as the prestressing force is released in the plant. Creep of concrete results in time-dependent changes in camber and prestress forces. The amount and rate of creep vary with the concrete age at loading, stress level, relative humidity, size of member and amount of nonprestressed reinforcement. The following calculations are based on ACI 209.

2.5.8.1 Calculation of Creep

Creep strains are determined by multiplying the elastic strains by a creep coefficient, $C(t, t_0)$.

For steam-cured concrete loaded at 1 to 3 days and moist-cured concrete loaded at 7 days:

$$C(t,t_0) = \frac{(t-t_0)^{0.6}}{10+(t-t_0)^{0.6}}C_u \qquad \text{(Eq. 2.5.8.1-1)}$$

where: C_u = ultimate creep coefficient.

Although Eq. 2.5.8.1-1 was developed for steam-cured and moist-cured concretes, it may be applied to radiant heat-cured concretes if more specific information is not available.

In the absence of creep data for local aggregates and materials, the following average value is suggested :

$$C_u = 1.88k_c \qquad \text{(Eq. 2.5.8.1-2)}$$

where:

k_c = product of applicable correction factors

$= k_{la} \times k_h \times k_s$ (Eq. 2.5.8.1-2a)

k_{la} = correction factor for loading age

k_h = correction factor for relative humidity

k_s = correction factor for size of member

For loading ages later than 7 days for moist-cured concrete and 1 to 3 days for steam-cured concrete, the loading age correction factor, k_{la}, may be taken from **Table 2.5.8.1-1.**

Table 2.5.8.1-1
Correction Factors k_{la} for Loading Age

Loading Age, days	Steam Cured, Factor k_{la}	Moist Cured, Factor k_{la}
7	0.94	1.00
10	0.90	0.95
14	0.88	0.92
28	0.83	0.84
60	0.76	0.77
90	0.74	0.74

Correction factors are based on the following equations:

For steam-cured concrete: $k_{la} = 1.13(t_{la})^{-0.094}$ (Eq. 2.5.8.1-2b)

For moist-cured concrete: $k_{la} = 1.25(t_{la})^{-0.118}$ (Eq. 2.5.8.1-2c)

where: t_{la} = loading age, days

The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2.** A relative humidity map taken from the *LRFD Specifications* is shown in **Figure 2.5.7.1-1.**

The size correction factor, k_s, depends on the volume to surface area of the member and may be taken from **Table 2.5.7.1-3.**

2.5.9 Coefficient of Thermal Expansion

The coefficient of thermal expansion of concrete varies with the aggregate type as shown in **Table 2.5.9-1**, which is based on ACI 209. The range for normal weight concrete is generally 5 to 7 x 10^{-6} per °F when made with siliceous aggregates and 3.5 to 5 x 10^{-6} per °F when made with calcareous aggregates. The range for structural lightweight concrete is 3.6 to 6.0 x 10^{-6} per °F depending on the type of aggregate and the amount of natural sand. For design, coefficients of 6 x 10^{-6} per °F for normal weight concrete and 5 x 10^{-6} per °F for sand-lightweight concrete are frequently used. If greater accuracy is needed, tests should be made on the specific concrete. Since the coefficient of thermal expansion for steel is also about 6 x 10^{-6} per °F, the thermal effects on precast, prestressed concrete members are evaluated by treating them as plain concrete and utilizing the coefficient of thermal expansion for concrete.

Table 2.5.9-1 Coefficients of Thermal Expansion of Concrete

Rock Type	millionths/ °F
Chert	6.6
Quartzite	5.7
Quartz	6.2
Sandstone	5.2
Marble	4.6
Siliceous Limestone	4.6
Granite	3.8
Dolerite	3.8
Basalt	3.6
Limestone	3.1

2.6 GROUT MATERIALS

2.6.1 Definitions and Applications

When precast, prestressed concrete members are placed adjacent to each other, load transfer between adjacent members is often achieved through a grouted keyway. The keyway may or may not extend for the full depth of the member. The keyway is grouted with one of several different grouting materials which are described in this section. In some bridges, no additional deck work is performed after grouting. In other bridges, a composite concrete deck may be cast on the members or the top surface of the members may be coated with a waterproofing membrane and overlaid with an asphaltic wearing course.

2.6.2 Types and Characteristics of Grout

ASTM Specification C 1107 covers three grades of packaged dry hydraulic-cement grouts (non-shrink) intended for use under applied load. These grouts are composed of hydraulic cement, fine aggregate and other ingredients and generally only require the addition of mixing water for use. Three grades of grout are classified according to the volume control mechanism exhibited by the grout after being mixed with water:

Grade A – pre-hardening volume-adjusting in which expansion occurs before hardening

Grade B – post-hardening volume-adjusting in which expansion occurs after the grout hardens

Grade C – combination volume-adjusting which utilizes a combination of expansion before and after hardening

2.6.2.1 Performance Requirements

Performance requirements for compressive strengths and maximum and minimum expansion levels are given in ASTM C 1107. Although these grouts are termed non-shrink, the intent is to provide a final length that is not shorter than the original length at placement. This is achieved through an expansion mechanism prior to any shrinkage occurring.

2.6.2.2 Materials

Different cementitious materials may be used to produce grout. These include portland cement, shrinkage-compensating cement, expansive portland cement made with special additives, epoxy-cement resins and magnesium ammonium phosphate cement (Gulyas et al 1995).

2.6.3 ASTM Tests

The properties of grout are determined using the following ASTM test methods:

- C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)
- C 138 Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete
- C 157 Test Method for Length Change for Hardened Hydraulic-Cement Mortar and Concrete
- C 185 Test Method for Air Content of Hydraulic Cement Mortar
- C 827 Test Method for Change in Height at Early Ages of Cylindrical Specimens from Cementitious Mixtures
- C 1090 Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout

2.6.4 Grout Bed Materials

The same materials that are used for grouting keyways between precast concrete members may be used for grout beds to support structural and non-structural members. In some cases, the grout will be very stiff and is referred to as dry pack. Dry pack will often have a very high compressive strength because of the low water-cementitious materials ratio. It is often compacted by hand tamping.

2.6.5 Epoxy Resins

Epoxy-resin grouts can be used between precast concrete members where increased bonding and tensile capacity is required. When these are used, consideration should be given to the higher coefficient of thermal expansion and the larger creep properties of epoxy grouts.

2.6.6 Overlays

When concrete overlays are placed on precast concrete members, a 1/16- to 1/8-in. thick layer of grout consisting of one part cement, one part sand and enough water to make a thick, creamy, paint-like consistency is brushed onto the concrete surface. The grout is placed a short distance ahead of the overlay concrete. The grout should not be allowed to dry prior to the overlay placement. Otherwise, the dry grout may act as a poor surface for bonding. It is particularly important that the concrete surface be clean and sound and that the grout be well brushed into the concrete surface.

2.6.7 Post–Tensioned Members

Grouting of post-tensioned members is described in the PTI *Post-Tensioning Manual* (1990).

2.7 PRESTRESSING STRAND

Although prestressed concrete may be produced with strands, wires or bars, prestressed precast concrete bridge members are generally produced using seven-wire strand conforming to ASTM A 416 (AASHTO M203). Seven-wire strand consists of a straight center wire that is wrapped by six wires in a helical pattern. Strand sizes range from 3/8-in. to 0.6-in. diameter, as shown in **Table 2.11.1**. The larger size strands are used in prestressed concrete beams because this results in fewer strands. The use of 0.6-in. diameter strand is essential to take full advantage of high strength concrete.

2.7.1 Strand Types

Two types of strands are covered in ASTM A 416: "low-relaxation" and "stress-relieved" (normal-relaxation). However, in recent years, the use of low-relaxation strand has progressively increased to a point that normal-relaxation strand is seldom used. Two grades of strand are generally used in prestressed concrete construction. These are Grades 250 and 270, which have minimum ultimate strengths of 250,000 and 270,000 psi, respectively. In general, Grade 270 is used in prestressed concrete bridge beams. Grade 250 strand may be used where lower levels of precompression are required. In addition to smooth, uncoated strands, epoxy-coated strands are available.

2.7.1.1 Epoxy-Coated Strand

Epoxy-coated strand is seven-wire prestressing strand with an organic epoxy coating which can vary in thickness from 25 to 45 mils. Two types of coatings are available. A smooth type has low bond characteristics and is intended for use in unbonded, post-tensioned systems, external post-tensioned systems, and stay cables. An epoxy-coated strand with particles of grit embedded in the surface is used in bonded pretensioned and post-tensioned systems.

In addition to the strand having an external coating, it can also be manufactured with the interstices between the individual wires filled with epoxy. This prevents the entry of corrosive chemicals, either by capillary action, or other hydrostatic forces. This type of strand should be specified when there is risk of contaminants or moisture entering at the ends of tendons. Epoxy-coated strand should comply with ASTM A 882. This specification requires that all prestressing steel strand to be coated shall meet the requirements of ASTM A 416.

2.7.1.1.1 Effect of Heat

For pretensioned applications with epoxy-coated strands where accelerated curing techniques are employed, the temperature of the concrete surrounding the strand at the time of prestress transfer should be limited to a maximum of 150 F and the concrete temperature should be falling. The epoxy-coating will not be damaged if this recommended temperature is not exceeded during the curing cycle. Concrete temperatures under sustained fire exposure conditions will most likely be considerably higher than the epoxy can withstand. This could result in a complete loss of bond between the strand and the concrete. Although bridge structures may not require a specific fire resistance rating, the likelihood of vehicle fires and subsequent effects of elevated temperatures should be evaluated. More specific information on the use of epoxy-coated strand is given in the report by the PCI Committee on Epoxy-Coated Strand (1993).

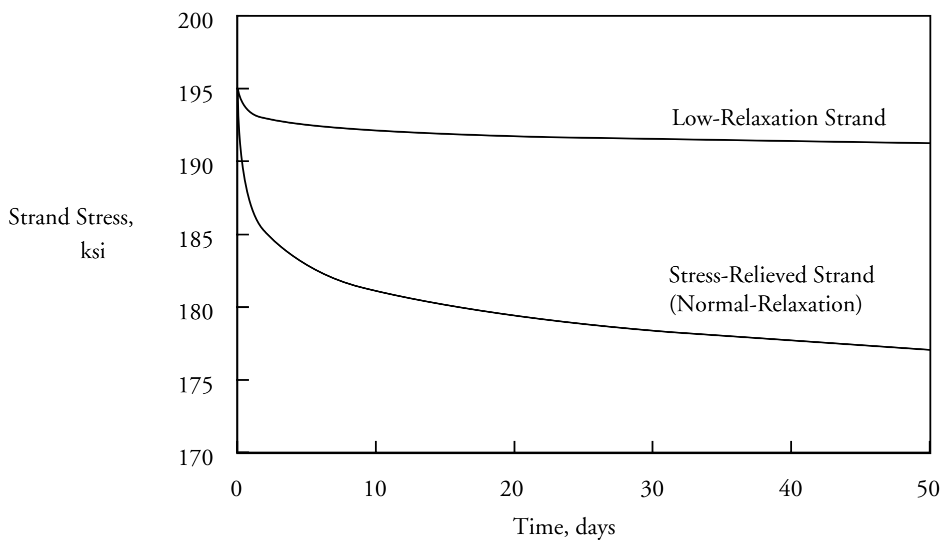
2.7.2 Material Properties

Cross-sectional properties, design strengths and idealized stress-strain curves of Grade 250 and 270 low-relaxation seven-wire strands are given in Section 2.11. Also, see Chapter 8, Section 8.2.2.5.

2.7.3 Relaxation

Relaxation is the time-dependent reduction of stress in a prestressing tendon. When a strand is stressed and held at a constant length, the stress in the strand decreases with time, as illustrated in **Figure 2.7.3-1**. Relaxation losses increase with stress level and temperature. The relaxation losses of low-relaxation strand are considerably less than the losses in normal-relaxation strand. Relaxation of a prestressing strand depends on the stress level in the strand. However, because of other prestress losses, there is a continuous reduction of the strand stress, which causes a reduction in relaxation. Therefore, several complex and empirical relationships have been proposed for the determination of relaxation losses. Several of these methods are based on the loss that would occur if the strand were under constant strain. This loss is then reduced by the effects of elastic shortening, creep and shrinkage. Early research work on relaxation was performed by Magura (1964). Subsequently, many other design recommendations have been made. The most recent recommendation is in the *LRFD Specifications*.

Figure 2.7.3-1
Comparison of Relaxation Losses



2.7.3.1
Epoxy–Coated Strand

Tests of epoxy-coated, low-relaxation strands have shown the relaxation to be significantly higher than that of uncoated strand. The use of relaxation losses equal to double the relaxation loss calculated for uncoated strand have been recommended by manufacturers. Individual manufacturers of epoxy-coated strand should be consulted for suitable relaxation loss values.

2.7.4
Fatigue Strength

If the precompression in a prestressed concrete member is sufficient to ensure an uncracked section at service loads, the stress range in the strands is not likely to be high enough for fatigue of the strand to be a critical design factor. Fatigue considerations have not been a major factor in the specification of prestressing strand for bridges because bridge beams are designed to be uncracked. The actual and allowable fatigue life of prestressing strand depend on the stress range and the minimum stress level. The stress range may be affected by the strand radius of curvature, particularly in harped strand.

2.7.4.1
Stress Range

The following design provisions for fatigue were introduced in the *LRFD Specifications:*

The stress range in prestressing tendons shall not exceed:

- 18,000 psi for radii of curvature in excess of 30 ft and
- 10,000 psi for radii of curvature not exceeding 12 ft

A linear interpolation may be used for radii between 12 and 30 ft

2.7.5
Surface Condition

In a pretensioned member, the prestressing force in a strand is transferred from the strand to the concrete by bond. Strand surface condition has long been recognized as a primary factor affecting bonding of concrete to prestressing strand. An increase in the surface roughness, such as a light surface rust, increases the bond between the concrete and the strand and results in a shorter development length. However, researchers have found it difficult to consistently quantify the effects of surface characteristics (Buckner 1994). This means that the increase in bond strength can possibly provide an extra margin of safety, but is not always consistent and should not be counted on to provide a shorter development length unless tests are conducted with specific strand. Chemicals on the strand surface can result in a reduction in bond between the concrete and strand and longer development lengths. Consequently PCI recommends that "Prestressing strand shall conform to the requirements of ASTM A

416 and shall be certified by its manufacturer to bond to concrete of a normal strength and consistency in conformance with the prediction equations for transfer and development lengths given in both ACI and AASHTO specifications."

2.7.6 Splicing

Lengths of prestressing strand can be connected using specialized strand connectors. Generally, this is not necessary in precast, prestressed concrete bridges. In situations where splicing of strands is necessary, consult the specific manufacturer's literature for details. The use of splice chucks in plant production is described in Chapter 3.

2.8 NONPRESTRESSED REINFORCEMENT

Nonprestressed reinforcement generally consists of deformed bars or welded wire reinforcement (previously refered to as welded wire fabric). Material properties and sizes of nonprestressed reinforcement are given in **Tables 2.11-2** through **2.11-4** and **Figure 2.11-1**.

2.8.1 Deformed Bars

Reinforcing bars should be deformed except plain bars may be used for spirals or for dowels at expansion or contraction joints. Reinforcing bars are generally specified to have yield strengths of 40,000 or 60,000 psi (Grade 40 or Grade 60 respectively). In some situations, a yield strength of 75,000 psi (Grade 70) may be specified, although this would be unusual in bridges.

2.8.1.1 Specifications

Reinforcing bars should conform to one of the following ASTM specifications:

A 615 Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

A 616 Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement

A 617 Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement

A 706 Specification for Low-Alloy Deformed Bars for Concrete Reinforcement

A 767 Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement

A 775 Specification for Epoxy-Coated Reinforcing Steel Bars

The most widely used type and grade of bars conform to ASTM A 615 Grade 60 and include bars with sizes from No. 3 through No. 11, No. 14 and No. 18. When welding is required or when more bendability and controlled ductility are required, as in seismic-resistant design, low-alloy reinforcing bars conforming to ASTM A 706 should be considered.

2.8.1.2 Corrosion Protection

When coated reinforcing bars are required as a corrosion protection system, the bars may be either zinc-coated or epoxy-coated and conform to ASTM A 767 or ASTM D 3963 (AASHTO M284), respectively. Epoxy-coated reinforcing bars are generally used in bridge decks exposed to a salt environment.

2.8.2 Mechanical Splices

The most common method for splicing reinforcing bars is the lap splice. However, when lap splices are undesirable or impractical, mechanical or welded connections may be used to splice reinforcing bars. In general, a mechanical connection should develop, in tension or compression, at least 125% of the specified yield strength of the bars being connected. This is to ensure that yielding of the bars will occur before failure in the mechanical connection.

2.8.2.1 Types

Mechanical connections can be categorized as compression-only, tension-only and tension-compression. In most compression-only mechanical connections, the compressive stress is transferred by concentric bearing from one bar to the other. The mechanical connection then serves to hold the bars in concentric contact. Various types of mechanical connections are available that will handle both tension and compression forces. These connectors use a variety of couplers that may be cold swaged, cold extruded, hot forged, grout filled, steel filled or threaded. Tension-only mechanical connections generally use a steel coupling sleeve with a wedge. This is only effective when the reinforcing bar is pulled in tension. Most mechanical connection devices are proprietary and further information is available from individual manufacturers. Descriptions of the physical features and installation procedures for selected mechanical splices are described in ACI 439.3R.

2.8.3 Welded Wire Reinforcement

Welded wire reinforcement (WWR) is a prefabricated reinforcement consisting of cold-drawn wires welded together in square or rectangular grids. Each wire intersection is electrically resistance-welded by a continuous automatic welder. Pressure and heat fuse the intersecting wires into a homogeneous section and fix all wires in their proper position. WWR may consist of plain wires, deformed wires or a combination of both. WWR can also be galvanized or epoxy coated. WWR conforms to one of the following ASTM standard specifications:

A 185 Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement

A 497 Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement

A 884 Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

Wire sizes are specified by a letter, W or D, followed by a number indicating the cross-sectional area of the wire in hundredths of a square inch. Plain wire sizes use the letter W; deformed wire sizes use the letter D. Wire sizes from W1.4 to W45 and D2 to D45 may be specified. Wire spacings generally vary from 2 to 12 in. Common stock styles and wire sizes are listed in **Table 2.11-3**. The Engineer should check on availability of styles before specifying because all sizes may not be locally available.

2.8.4 Fatigue Strength

The *Standard Specifications* limits the allowable stress range in straight reinforcement caused by live load plus impact at service load to:

$$f_f = 21 - 0.33f_{min} + 8(r/h) \quad \text{(Eq. 2.8.4-1)}$$

where:

f_f = stress range between maximum tensile stress and minimum stress

f_{min} = minimum stress level, tensile stress is positive, compressive stress is negative

r/h = ratio of base radius to height of rolled-on transverse deformation, taken as 0.3 when actual values are not known.

2.9 POST–TENSIONING MATERIALS

Post-tensioning systems may be conveniently divided into three categories depending on whether the stressing tendon is wire, strand or bar. For bridge construction, wire systems are generally not used. Further information on post-tensioning systems has been published by the Post-Tensioning Institute (1990). For details of proprietary systems, the manufacturers' literature should be consulted.

2.9.1 Strand Systems

Strand systems utilize the same strand and strand types that are used for pretensioned concrete members. In post-tensioning systems, the strands are generally combined to form a complete tendon and may consist of any quantity from a single strand to 55 strands. Anchorages for strand systems utilize the wedge principle in which the individual strands are anchored with wedges into a single tendon anchorage. In a post-tensioned multi-strand system, all strands are tensionsed at the same time. Strand tendons may be tensionsed in the plant, on the construction site, or in the finished structure.

2.9.2 Bar Systems

Bar systems generally utilize a single bar in a post-tensioning duct. The surface of the bar may be smooth with rolled threads of the required length at both ends, or the thread deformation may be rolled-on over the entire length of the bar during manufacturing. This permits the bar to be cut at any point and threaded fittings added. The bars are anchored using a threaded nut. Different types of anchorages are used at the tensioning and dead end anchorages. Bars for use in post-tensioning systems should conform with ASTM A 722 This specification covers both plain and deformed bars.

2.9.3 Splicing

Various proprietary systems are available for splicing both strand and bar systems. Couplers are required to develop at least 95% of the minimum specified ultimate strength of the tendon without exceeding the specified anchorage set (Post-Tensioning Institute, 1990).

2.9.4 Ducts

Ducts for post-tensioning systems may be either rigid or semi-rigid and made of ferrous metal or polyethylene. They may also be formed in the concrete with removable cores. The use of polyethylene ducts is generally recommended for corrosive environments. Polyethylene ducts should not be used on radii less than 30 ft because of the polyethylene's lack of resistance to abrasion during pulling and tensioning the tendons. The inside diameter of ducts should be at least 1/4 in. larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct should be at least twice the net area of the prestressing steel. Where tendons are to be placed by the pull-through method, the duct area should be at least 2.5 times the net area of the prestressing steel.

2.10 FIBER REINFORCED PLASTIC REINFORCEMENT

2.10.1 Introduction

A newly emerging technology, with potential application in prestressed concrete, consists of prestressing bars and tendons made from fiber reinforced plastic (FRP) composites. This class of material consists of a polymer matrix such as polyester, vinyl ester, epoxy, or phenolic resin which is reinforced with fibers such as aramid, carbon or glass. These composites have tensile strengths similar to conventional strand and bar systems and are particularly suitable for applications where weight, durability, corrosion resistance and resistance to electromagnetic currents are relevant. Details of FRP composites are given in ACI 440.

2.10.2 Mechanical Properties

The mechanical properties of FRP vary significantly from one product to another. Factors such as type and volume of fiber and resin play a major role in establishing the characteristics of the product. The mechanical properties of all composites are affected by loading history, loading duration, temperature and moisture. Furthermore, standardized tests for determination of the mechanical properties have yet to be developed to the same extent that exists for steel products. Despite these limitations, a comparison of properties with conventional steel strands and bars can still be made.

2.10.2.1
Short–Term

The tensile strength, modulus of elasticity, coefficient of thermal expansion and unit weight for several types of FRP composites are given in **Table 2.10.2.1-1,** which is based on ACI 440. Because an FRP bars and tendons are anisotropic, the mechanical properties are those measured in the longitudinal or strong direction. Unlike steel, the tensile strength of FRP bars is a function of bar diameter. Due to shear lag, the fibers located near the center of the bar cross section, are not stressed as much as those near the outer surface of the bar. This results in reduced strength in larger diameter bars. FRP bars and tendons reach their ultimate tensile strength without exhibiting any yielding of the material. Consequently, fiber reinforced plastic composite bars and tendons do not possess the ductility of steel tendons. However, design methods are being developed to ensure that members reinforced with FRP composites will possess adequate ductility.

Table 2.10.2.1-1
Comparison of Properties of Steel Strand and FRP Reinforcement

Property	Steel Strand	Glass Fiber Bar	Glass Fiber Tendon	Carbon Fiber Tendon	Aramid Fiber Tendon
Tensile Strength (ksi)	270	75-175	200-250	240-350	170-300
Modulus of Elasticity (ksi)	29,000	6,000-8,000	7,000-9,000	22,000-24,000	7,000-11,000
Coefficient of Thermal Expansion (millionth/ °F)	6.5	5.5	5.5	0	-0.5
Unit Weight (pcf)	490	94-125	150	94-100	78

2.10.2.2
Long–Term

Fibers such as graphite and glass have excellent resistance to creep whereas the resins exhibit high creep. The orientation and volume of fibers have a significant influence on the creep and performance of the composites. Consequently, relaxation losses may be much higher with FRP composite bars and tendons. FRP bars and tendons exhibit good fatigue resistance.

2.10.3
Applications

Despite the above limitations, composite materials have already been used in a variety of civil engineering applications in the field on a limited basis. Further details of these applications are given in ACI 440.

2.10.4
Products

According to ACI 440, nine companies have marketed FRP composites as concrete reinforcement in North America. At the present time, there is a rapid evolution and considerable research underway on fiber reinforced plastics. Consequently, the reader should verify current products and their availability with individual manufacturers.

2.11 REINFORCEMENT SIZES AND PROPERTIES

Table 2.11-1
Properties and Design Strengths of Prestressing Steel

Seven-Wire Low-Relaxation Strand Grade 270 (f'_s = 270 ksi)

Nominal Diameter (in.)	3/8	7/16	1/2	1/2 Special	9/16	0.6
Nominal Area (A^*_s, in.^{2})	0.085	0.115	0.153	0.167	0.192	0.217
Nominal Weight (plf)	0.29	0.39	0.52	0.53	0.65	0.74
Minimum Tensile Strength (kip)	23.0	31.0	41.3	45.1	51.8	58.6
Minimum Yield Strength (kip)	20.7	27.9	37.2	40.6	46.6	52.7
$0.70f'_sA^*_s$ (kip)	16.1	21.7	28.9	31.6	36.3	41.0
$0.75f'_sA^*_s$ (kip)	17.2	23.3	31.0	33.8	38.9	44.0
$0.80f'_sA^*_s$ (kip)	18.4	24.8	33.0	36.1	41.4	46.9

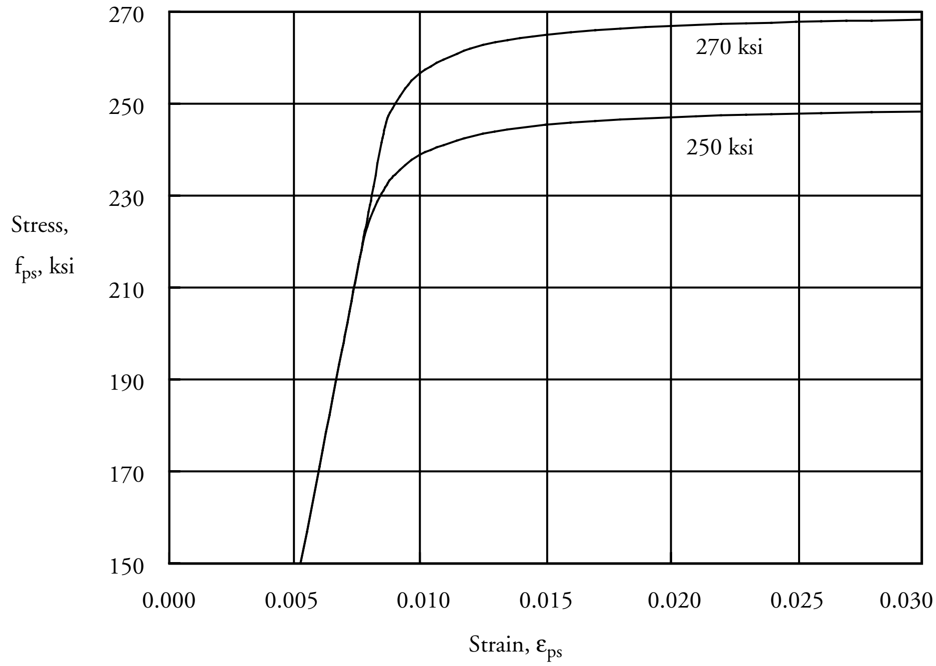
Seven-Wire Low-Relaxation Strand Grade 250 (f'_s = 250 ksi)

Nominal Diameter (in.)	3/8	7/16	1/2	0.6
Nominal Area (A^*_s, in.^{2})	0.080	0.108	0.144	0.216
Nominal Weight (plf)	0.27	0.37	0.49	0.74
Minimum Tensile Strength (kip)	20.0	27.0	36.0	54.0
Minimum Yield Strength (kip)	18.0	24.3	32.4	48.6
$0.70f'_sA^*_s$ (kip)	14.0	18.9	25.2	37.8
$0.75f'_sA^*_s$ (kip)	15.0	20.3	27.0	40.5
$0.80f'_sA^*_s$ (kip)	16.0	21.6	28.8	43.2

Deformed Prestressing Bars Grade 150 (f'_s = 150 ksi)

Nominal Diameter (in.)	5/8	1	1-1/4	1-3/8
Nominal Area (A^*_s, in.^{2})	0.28	0.85	1.25	1.58
Nominal Weight (plf)	0.98	3.01	4.39	5.56
Minimum Tensile Strength (kip)	42.0	127.5	187.5	237.0
Minimum Yield Strength (kip)	33.6	102.0	150.0	189.6
$0.70f'_sA^*_s$ (kip)	29.4	89.3	131.3	165.9
$0.75f'_sA^*_s$ (kip)	31.5	95.6	140.6	177.8
$0.80f'_sA^*_s$ (kip)	33.6	102.0	150.0	189.6

Figure 2.11-1
Idealized Stress-Strain Curve for Seven-Wire Low-Relaxation Prestressing Strand



These curves can be approximated by the following equations:

250 ksi strand
For $\varepsilon_{ps} \leq 0.0076$: $f_{ps} = 28{,}500\, \varepsilon_{ps}$ (ksi)
For $\varepsilon_{ps} > 0.0076$: $f_{ps} = 250 - 0.04/(\varepsilon_{ps} - 0.0064)$(ksi)

270 ksi strand
For $\varepsilon_{ps} \leq 0.0086$: $f_{ps} = 28{,}500\, \varepsilon_{ps}$ (ksi)
For $\varepsilon_{ps} > 0.0086$: $f_{ps} = 270 - 0.04/(\varepsilon_{ps} - 0.007)$(ksi)

Table 2.11-2
Reinforcing Bar Sizes

ASTM Standard Reinforcing Bars				
Bar Size Designation No.	Weight (plf)	Nominal Dimensions		
		Diameter (in.)	Area (in.^{2})	Perimeter (in.)
3	0.376	0.375	0.11	1.178
4	0.668	0.500	0.20	1.571
5	1.043	0.625	0.31	1.963
6	1.502	0.750	0.44	2.356
7	2.044	0.875	0.60	2.749
8	2.670	1.000	0.79	3.142
9	3.400	1.128	1.00	3.544
10	4.303	1.270	1.27	3.990
11	5.313	1.410	1.56	4.430
14	7.650	1.693	2.25	5.320
18	13.600	2.257	4.00	7.090

Table 2.11-3
Common Stock Styles of
Welded Wire Reinforcement

Style Designation		Steel Area ($in.^2$/ft)		Approximate Weight
Former Designation (By Steel Wire Gage)	Current Designation (By W-Number)	Longit.	Trans.	(lb/100 ft^2)
12x6-10x7	12x6-W1.4xW2.5	0.014	0.050	23
12x6-8x4	12x6-W2.0xW4.0	0.020	0.080	35
12x6-10x6	12x6-W1.4xW2.9	0.014	0.058	27
6x6-10x10	6x6-W1.4xW1.4	0.029	0.029	21
4x12-8x12	4x12-W2.1xW0.9	0.062	0.009	25
6x6-8x8	6x6-W2.1xW2.1	0.041	0.041	30
4x4-10x10	4x4-W1.4xW1.4	0.043	0.043	31
4x12-7x11	4x12-W2.5xW1.1	0.074	0.011	31
6x6-6x6	6x6-W2.9xW2.9	0.058	0.058	42
4x4-8x8	4x4-W2.1xW2.1	0.062	0.062	44
6x6-4x4	6x6-W4.0xW4.0	0.080	0.080	58
4x4-6x6	4x4-W2.9xW2.9	0.087	0.087	62
6x6-2x2	6x6-W5.5xW5.5	0.110	0.110	80
4x4-4x4	4x4-W4.0xW4.0	0.120	0.120	85
4x4-3x3	4x4-W4.7xW4.7	0.141	0.141	102
4x4-2x2	4x4-W5.5xW5.5	0.165	0.165	119

Availability of styles should be verified by the local supplier.

Table 2.11-4
Sizes of Wires used in Welded Wire Reinforcement

Wire Size Number		Nominal Diameter*	Nominal Weight*	Area (in.^{2}/ft of width)						
				Center to Center Spacing (in.)						
Plain	Deformed	(in.)	(plf)	2	3	4	6	8	10	12
W45	D45	0.757	1.530	2.700	1.800	1.350	0.900	0.675	0.540	0.450
W31	D31	0.628	1.054	1.860	1.240	0.930	0.620	0.465	0.372	0.310
W30	D30	0.618	1.020	1.800	1.200	0.900	0.600	0.450	0.360	0.300
W28	D28	0.597	0.952	1.680	1.120	0.840	0.560	0.420	0.336	0.280
W26	D26	0.575	0.884	1.560	1.040	0.780	0.520	0.390	0.312	0.260
W24	D24	0.553	0.816	1.440	0.960	0.720	0.480	0.360	0.288	0.240
W22	D22	0.529	0.748	1.320	0.880	0.660	0.440	0.330	0.264	0.220
W20	D20	0.504	0.680	1.200	0.800	0.600	0.400	0.300	0.240	0.200
W18	D18	0.478	0.612	1.080	0.720	0.540	0.360	0.270	0.216	0.180
W16	D16	0.451	0.544	0.960	0.640	0.480	0.320	0.240	0.192	0.160
W14	D14	0.422	0.476	0.840	0.560	0.420	0.280	0.210	0.168	0.140
W12	D12	0.390	0.408	0.720	0.480	0.360	0.240	0.180	0.144	0.120
W11	D11	0.374	0.374	0.660	0.440	0.330	0.220	0.165	0.132	0.110
W10.5		0.366	0.357	0.630	0.420	0.315	0.210	0.158	0.126	0.105
W10	D10	0.356	0.340	0.600	0.400	0.300	0.200	0.150	0.120	0.100
W9.5		0.348	0.323	0.570	0.380	0.285	0.190	0.143	0.114	0.095
W9	D9	0.338	0.306	0.540	0.360	0.270	0.180	0.135	0.108	0.090
W8.5		0.329	0.289	0.510	0.340	0.255	0.170	0.128	0.102	0.085
W8	D8	0.319	0.272	0.480	0.320	0.240	0.160	0.120	0.096	0.080
W7.5		0.309	0.255	0.450	0.300	0.225	0.150	0.113	0.090	0.075
W7	D7	0.299	0.238	0.420	0.280	0.210	0.140	0.105	0.084	0.070
W6.5		0.288	0.221	0.390	0.260	0.195	0.130	0.098	0.078	0.065
W6	D6	0.276	0.204	0.360	0.240	0.180	0.120	0.090	0.072	0.060
W5.5		0.264	0.187	0.330	0.220	0.165	0.110	0.083	0.066	0.055
W5	D5	0.252	0.170	0.300	0.200	0.150	0.100	0.075	0.060	0.050
W4.5		0.240	0.153	0.270	0.180	0.135	0.090	0.068	0.054	0.045
W4	D4	0.225	0.136	0.240	0.160	0.120	0.080	0.060	0.048	0.040
W3.5		0.211	0.119	0.210	0.140	0.105	0.070	0.053	0.042	0.035
W3	D3	0.195	0.102	0.180	0.120	0.090	0.060	0.045	0.036	0.030
W2.9		0.192	0.098	0.174	0.116	0.087	0.058	0.044	0.035	0.029
W2.5		0.178	0.085	0.150	0.100	0.075	0.050	0.038	0.030	0.025
W2.1		0.162	0.070	0.126	0.084	0.063	0.042	0.032	0.025	0.021
W2	D2	0.159	0.068	0.120	0.080	0.060	0.040	0.030	0.024	0.020
W1.5		0.138	0.051	0.090	0.060	0.045	0.030	0.023	0.018	0.015
W1.4		0.134	0.049	0.084	0.056	0.042	0.028	0.021	0.017	0.014

* Based on ASTM A 496

2.12 RELEVANT STANDARDS AND PUBLICATIONS

The following list of standards and manuals is provided for the convenience of the reader because not all documents are referenced in the text of this chapter. The complete serial designation of each document includes a year of adoption. However, since these documents are updated on a frequent basis, the year has been omitted. The reader is referred to the respective organizations for the latest revisions and year of adoption.

2.12.1 AASHTO Standard Specifications

HB *Standard Specifications for Highway Bridges*

LRFD *AASHTO LRFD Bridge Design Specifications*

HM *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*

M6 *Fine Aggregate for Portland Cement Concrete*

M31 *Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*

M32 *Steel Wire, Plain, for Concrete Reinforcement*

M42 *Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*

M43 *Sizes of Aggregate for Road and Bridge Construction*

M53 *Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*

M54 *Fabricated Deformed Steel Bar Mats for Concrete Reinforcement*

M55 *Steel Welded Wire Fabric, Plain, for Concrete Reinforcement*

M80 *Coarse Aggregate for Portland Cement Concrete*

M85 *Portland Cement*

M144 *Calcium Chloride*

M154 *Air-Entraining Admixtures for Concrete*

M194 *Chemical Admixtures for Concrete*

M195 *Lightweight Aggregates for Structural Concrete*

M203 *Steel Strand, Uncoated Seven-Wire for Concrete Reinforcement*

M204 *Uncoated Stress-Relieved Steel Wire for Prestressed Concrete*

M205 *Molds for Forming Concrete Test Cylinders Vertically*

M221 *Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement*

M225 *Steel Wire, Deformed, for Concrete Reinforcement*

M240 *Blended Hydraulic Cement*

M275 *Uncoated High Strength Steel Bar for Prestressing Concrete*

M284 *Epoxy Coated Reinforcing Bars*

M295 *Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*

M302 *Ground Iron Blast-Furnace Slag for Use in Concrete and Mortars*

M307 *Microsilica for Use in Concrete and Mortar*

2.12.2 AASHTO Standard Methods of Test

T24 *Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*

T26 *Quality of Water To Be Used in Concrete*

T106 *Compressive Strength of Hydraulic Cement Mortar (Using 2 in. or 50 mm Cube Specimens)*

T131 *Time of Setting of Hydraulic Cement by Vicat Needle*

T137 *Air Content of Hydraulic Cement Mortar*

T152 *Air Content of Freshly Mixed Concrete by the Pressure Method*

T160 *Length Change of Hardened Hydraulic Cement Mortar and Concrete*

T161 *Resistance of Concrete to Rapid Freezing and Thawing*

T196 *Air Content of Freshly Mixed Concrete by the Volumetric Method*

T199 *Air Content of Freshly Mixed Concrete by the Chase Indicator*

T259 *Resistance of Concrete to Chloride Ion Penetration*

T277 *Electrical Indication of Concrete's Ability to Resist Chloride*

2.12.3 ACI Publications

207.1 *Mass Concrete*

209R *Predictions of Creep, Shrinkage and Temperature Effects in Concrete Structures*

211.1 *Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete*

211.2 *Standard Practice for Selecting Proportions for Structural Lightweight Concrete*

212.3R *Chemical Admixtures for Concrete*

213R *Guide for Structural Lightweight Aggregate Concrete*

215R *Considerations for Design of Concrete Structures Subject to Fatigue Loading*

221R *Guide for the Use of Normal Weight Aggregates in Concrete*

223 *Standard Practice for the Use of Shrinkage-Compensating Concrete*

226.1R *Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete*

226.3R *Use of Fly Ash in Concrete*

308 *Standard Practice for Curing Concrete*

315 *Details and Detailing of Concrete Reinforcement*

318 *Building Code Requirements for Structural Concrete*

343R *Analysis and Design of Reinforced Concrete Bridge Structures*

345R *Guide for Concrete Highway Bridge Deck Construction*

363R *State-of-the-Art Report on High Strength Concrete*

423.3R *Recommendations for Concrete Members Prestressed with Unbonded Tendons*

439.3R *Mechanical Connections of Reinforcing Bars*

440 *State-of-the-Art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures*

2.12.4 ASTM Standard Specifications

A 82 *Specification for Steel Wire, Plain, for Concrete Reinforcement*

A 184 *Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement*

A 185 *Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement*

A 416 *Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete*

A 421 *Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete*

A 496 *Specification for Steel Wire, Deformed, for Concrete Reinforcement*

A 497 *Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement*

A 615 *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*

A 616 *Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*

A 617 *Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*

A 706 *Specification for Low-Alloy Deformed Bars for Concrete Reinforcement*

A 722 *Specification for Uncoated High Strength Steel Bar for Prestressing Concrete*

A 767 *Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement*

A 775 *Specification for Epoxy-Coated Reinforcing Steel Bars*

A 882 *Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand*

A 884 *Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement*

C 33 *Specification for Concrete Aggregates*

C 94 *Specification for Ready-Mixed Concrete*

C 150 *Specification for Portland Cement*

C 260 *Specification for Air-Entraining Admixtures for Concrete*

C 330 *Specification for Lightweight Aggregates for Structural Concrete*

C 470 *Specification for Molds for Forming Concrete Test Cylinders Vertically*

C 494 *Specification for Chemical Admixtures for Concrete*

C 595 *Specification for Blended Hydraulic Cements*

C 618 *Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*

C 845 *Specification for Expansive Hydraulic Cement Concrete and Mortar*

C 989 *Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*

C 1107 *Specification for Package Dry, Hydraulic-Cement Grout (Nonshrink)*

C 1240 *Specification for Silica Fume for Use in Hydraulic Cement, Concrete and Mortar*

D 98 *Specification for Calcium Chloride*

D 448 *Specification for Standard Sizes of Coarse Aggregate for Highway Construction*

D 3963 *Specification for Fabrication and Jobsite Handling of Epoxy-Coated Reinforcing Steel Bars*

2.12.5 ASTM Standard Test Methods

C 42 *Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*

C 109 *Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)*

C 138 *Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete*

C 157 *Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*

C 173 *Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method*

C 185 *Test Method for Air Content of Hydraulic Cement Mortar*

C 186 *Test Method for Heat of Hydration of Hydraulic Cement*

C 191 *Test Method for Time of Setting of Hydraulic Cement by Vicat Needle*

C 227 *Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)*

C 231 *Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*

C 289 *Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)*

C 342 *Test Method for Potential Volume Change of Cement-Aggregate Combinations*

C 418 *Test Method for Abrasion Resistance of Concrete by Sandblasting*

C 441 *Test Method for Effectiveness of Mineral Admixtures of Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction*

C 452 *Test Method for Potential Expansion of Portland Cement Mortars Exposed to Sulfate*

C 469 *Test Method for Static Modulus and Poisson's Ratio of Concrete in Compression*

C 512 *Test Method for Creep of Concrete in Compression*

C 586 *Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)*

C 597 *Test Method for Pulse Velocity Through Concrete*

C 666 *Test Method for Resistance of Concrete to Rapid Freezing and Thawing*

C 671 *Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing*

C 672 *Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*

C 682 *Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures*

C 779 *Test Method for Abrasion Resistance of Horizontal Concrete Surfaces*

C 803 *Test Method for Penetration Resistance of Hardened Concrete*

C 805 *Test Method for Rebound Number of Hardened Concrete*

C 827 *Test Method for Change in Height at Early Ages of Cylindrical Specimens from Cementitious Mixtures*

C 900 *Test Method for Pullout Strength of Hardened Concrete*

C 944 *Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method*

C 1012 *Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution*

C 1090 *Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout*

C 1202 *Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*

2.12.6 Cross References ASTM-AASHTO

This list of cross references is provided for ease of comparing two similar documents. In many cases, the two documents are not identical and should not be interchanged without review of their content.

ASTM	AASHTO	ASTM	AASHTO	ASTM	AASHTO	ASTM	AASHTO
A 82	M32	A 616	M42	C 185	T137	C 618	M295
A 184	M54	A 617	M53	C 191	T131	C 666	T161
A 185	M55	A 722	M275	C 231	T152	C 989	M302
A 416	M203	C 42	T24	C 260	M154	C 1202	T277
A 421	M204	C 109	T106	C 330	M195	C 1240	M307
A 496	M225	C 150	M85	C 470	M205	D 98	M144
A 497	M221	C 157	T160	C 494	M194	D 448	M43
A 615	M31	C 173	T196	C 595	M240		

2.12.7 Cited References

AASHTO LRFD Bridge Design Specifications, First Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1994

Buckner, D. C., "An Analysis of Transfer and Development Lengths for Pretensioned Concrete Structures," FHWA, U.S. Department of Transportation, Report No. FHWA-RD-94-049, 1994, 108 pp

CEB-FIP Model Code 1990, Bulletin d'Information No. 213/214, Comite Euro-International du Beton, Lausanne, Switzerland, 1990

Gulyas, R. J., Wirthlin, G. J. and Champa, J. T., "Evaluation of Keyway Grout Test Methods for Precast Concrete Bridges," PCI JOURNAL, V. 40, No. 1, January-February 1995, pp. 44-57

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PCI Committee on Epoxy-Coated Strand, "Guidelines for the Use of Epoxy-Coated Strand," PCI JOURNAL, V. 38, No. 4, July-August 1993, pp. 26-32

Post-Tensioning Manual, Post-Tensioning Institute, Phoenix, AZ, 1990

Standard Specifications for Highway Bridges, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996

Zia, P., Leming, M. L., and Ahmad, S. H., "High Performance Concretes, A State-of-the-Art Report," Report No. SHRP-C/FR-91-103, Strategic Highway Research Program, National Research Council, Washington, DC, 1991

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NOTATION

FABRICATION AND CONSTRUCTION

A_s = area of a prestressing strand, in.^{2}

A^*_s = total prestressing steel area, in.^{2}

D = prestressing steel elongation, in.

E_s = modulus of elasticity of prestressing steel, ksi

f'c = specified compressive strength of concrete, ksi

f_{pu} = specified tensile strength of prestressing steel, ksi

L = total length of prestressing steel from anchorage to anchorage, in.; length of member, ft

P_s = design jacking force, kips

Fabrication and Construction

3.1 SCOPE

This chapter describes materials and techniques used in the fabrication, handling, transportation and erection of precast, prestressed concrete bridge components. It also discusses how the components are integrated into the completed structure. Familiarity with this chapter will enable bridge designers to take advantage of the flexibility and economy of precast, prestressed concrete products. It will help to avoid the pitfalls that make precast systems less cost-effective. In addition to fabrication, quality control procedures are described which maximize product quality, making plant-cast components superior to site-cast construction.

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3.2 PRODUCT COMPONENTS AND DETAILS

Precast, prestressed concrete bridge products generally consist of concrete, reinforcement and various embedments used for temporary or structural connections. Variations in these components affecting cost and constructability are summarized in this section. These descriptions are not intended to be all-inclusive, and the reader is directed to the references for further information.

3.2.1 Concrete

Plant-cast concrete bridge products are structurally efficient sections which are relatively thin and congested with reinforcement and embedments. It is therefore imperative that fresh concrete (portland cement, fine aggregate, coarse aggregate, water, and admixtures) have sufficient workability to fill all spaces without voids, honeycombing or segregation. The following sections describe variations in individual concrete constituents that can be beneficial or detrimental to concrete placement, consolidation, and finishing, but discounting the influence of other components in the mixture. In reality, the behavior of fresh concrete will depend on the interaction of all constituents. Both fresh and hardened concrete properties vary widely due to the availability and nature of local materials. Most Precast/Prestressed Concrete Institute (PCI) Certified Plants have standard in-house mixtures with proven histories of placeability, strength and durability. Bridge designers should consult their local producers for information on their experience with local materials.

3.2.1.1 Cement

The quantity and fineness of cement play important roles in the behavior of fresh concrete, as described in ACI 225R. Lean mixtures (those using less cement) with coarsely ground cement are generally harsh and difficult to consolidate and finish. As the quantity or fineness of the cement increases, the mixture becomes more cohesive. Very rich mixtures with finely ground cement can be overly cohesive or sticky. AASHTO M85 Type III cement, normally used in precast products for its high-early strength characteristics, is the finest grind of portland cements available. As the fineness of the cement increases, the cement content which produces optimum workability with minimum water, is reduced.

3.2.1.2 Aggregates

The behavior of fresh concrete can be significantly affected by the physical properties of the aggregates, as described in ACI 221R. The maximum size and gradation of the fine aggregate, as well as the shape and texture of both the fine and coarse aggregates, affect the water content required to produce workable concrete. Rough, angular aggregates require more cement and water for workability than smooth, rounded aggregates. Too many flat or elongated pieces of coarse aggregate can result in a harsh mixture. Porous aggregates will affect the water demand if not sufficiently saturated at the time of batching.

The maximum coarse aggregate size should be smaller than the tightest space the concrete is expected to fill. ACI 318 limits the maximum coarse aggregate size to one-fifth the narrowest dimension between form sides, one-third the depth of slabs, or three-quarters the minimum clear dimension between reinforcement. The smallest practical maximum coarse aggregate size is 3/8 in., however, this should not be interpreted as permitting a 1/2 in. minimum clear dimension between reinforcement. Other restrictions apply. See Section 3.3.3.4 for discussion on steel spacing.

3.2.1.3 Admixtures

All admixtures in a given concrete mix must be compatible with each other as well as with the cement. Combinations of admixtures can accentuate or mitigate placement and finishing problems. The admixture manufacturer should be consulted before combinations are used.

3.2.1.3.1 Water-Reducing Admixtures

In precast plants, fresh concrete mixes are augmented with one or more admixtures. The purpose of an admixture is to produce a desired property of concrete, either in its fresh or hardened form. ACI 212.3R provides a detailed description of commonly available chemical admixtures.

Strength and durability considerations of hardened concrete for bridge applications normally dictate concrete mixtures with low water-cementitious material ratios. Without chemical admixtures, these mixtures can exhibit poor workability. Normal water-reducing admixtures decrease water demand from 5 to 12 percent for the same workability, or increase workability for the same water content. High range water-reducing admixtures (superplasticizers) decrease water demand from 12 to more than 30 percent. Under most conditions, water-reducers are used for both purposes; to reduce water demand and provide optimum workability. The ability to produce workable concrete while maintaining low water-cementitious material ratios aids in the early strength gain necessary for a daily production cycle. Concretes using water-reducing admixtures are also less likely to segregate during placement.

3.2.1.3.2 Retarders and Accelerators

Water-reducing admixtures normally do not increase the working life of fresh concrete, and frequently decrease it, particularly with high range water-reducers. Rapid loss of workability can often be controlled by the addition of a retarding admixture. Water-reducing admixtures have also been known to retard the set of concrete. This can be controlled by the introduction of a non-chloride accelerating admixture.

3.2.1.3.3 Air-Entraining Admixtures

In some cases, high range water-reducers make finishing more difficult because of the lower water content and the resulting lack of bleed water which normally rises to the surface. This can also be true of air-entraining admixtures. At low and moderate cement contents, air-entraining admixtures make fresh concrete more workable and cohesive, reducing segregation and bleed water. However, at high cement contents, the mixture can become overly cohesive or sticky. Air-entrainment also reduces concrete strength in approximate proportion to the amount entrained, unless the mix proportions are readjusted. Excessive air contents can affect both early- and long-term concrete strengths, and should be avoided.

3.2.1.3.4 Corrosion Inhibitors

Corrosion inhibitors are occasionally specified for the protection of embedded steel, and have various affects on the behavior of fresh concrete. Calcium nitrite, for example, accelerates the set of the concrete, reduces the amount of entrained air, and increases the likelihood of plastic and drying shrinkage cracking. When these chemical admixtures are used, proper mix adjustments and curing techniques should be specified in accordance with the manufacturer's recommendations.

3.2.1.3.5 Mineral Admixtures

Mineral admixtures are sometimes used for economy, strength or durability considerations. Fly ash can be an economical alternative if used to replace cement, and will usually increase the workability of concrete. However, its set retarding properties generally make it unattractive for use in a daily production cycle. ACI 226.3R provides guidance on the use of fly ash.

Fresh concrete with up to 5 percent silica fume by weight of cement will normally behave much like conventional concrete. However, higher dosages can result in overly cohesive mixtures, difficult finishing due to lack of bleed water, longer setting times and increased shrinkage. As with all concrete constituents, this detrimental behavior can be controlled with good mix design, batching, placing and curing practices. The report by the PCI Committee on Durability (1994), provides helpful information on the use of silica fume.

3.2.2 Prestressing Steel

Most precast concrete bridge components are prestressed for added strength and serviceability. Prestressing is achieved by one of two methods: pretensioning or post-tensioning. The primary difference between the two methods is the point in production at which the prestressing tendons are tensioned.

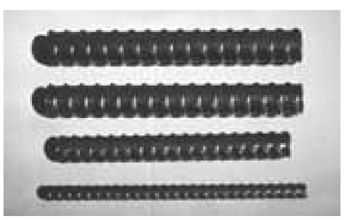
Pretensioning is most economical for plant-cast products, since much of the necessary material used in post-tensioning is eliminated. Post-tensioning may be required in the plant if pretensioning equipment or facilities are inadequate or not suited for the project. Bridge designers should consult their local producers for information on plant capabilities.

Combinations of pretensioning and post-tensioning within the same member have proven to be cost-effective. For example, combinations of pre- and post-tensioning may reduce the concrete strength required at transfer of prestress. In other cases, pretensioned strands have been designed to carry a predetermined percentage of the dead load in simple span (e.g., its own weight plus that of the cast-in-place deck without shoring). Then, post-tensioned tendons continuous over several spans are added for strength for subsequent dead and live loads.

Figure 3.2.2-1a and **b** shows several types and sizes of prestressing tendons. For pretensioning, the common tendon material is 7-wire strand, whereas in post-tensioning, single- or multi-strand tendons or high strength bars are commonly used.

Figure 3.2.2-1
Prestressing Steel

a) Post-Tensioning Bars



b) 7-Wire Prestressing Strands
From the left: 3/8 in.; 3/8 in. indented; 1/2 in.; 9/16 in.; 0.6 in.; 0.6 in. epoxy-coated with embedded grit

3.2.2.1 Pretensioning

In pretensioning, strands are first jacked to a specified force in a predetermined profile. Concrete is placed in direct contact with the tensioned strands and then cured. When the concrete achieves the specified release strength, forms are stripped and the tension in the strands is released. For some products, tension in the strands is first released, then the product is removed from fixed forms. See Section 3.3.6 for more information on form or product removal. The force in the strands is transferred to the product by the bond which develops between the concrete and surface of the strands.

3.2.2.2 Post-Tensioning

Post-tensioning is a method where the prestressing force is introduced into the concrete after it has been cast and cured. The tendons are then jacked between anchorages embedded in the concrete. Post-tensioning tendons may be internal or external to the concrete cross section.

For internal tendons, ducts or sleeves are provided in the concrete into which the prestressing tendons are inserted. Internal tendons may remain unbonded after stressing, or may be bonded by pressure grouting the ducts. Another type of internal tendon is a single strand that has had a factory application of grease followed by insertion into a plastic hose-like sleeve. These remain unbonded and the grease protects against corrosion.

External tendons, although outside of the concrete cross section, are normally contained within the structure. For example, tendons within the voids of box girders are considered external. External tendons are normally draped between anchorage points to achieve the desired profile. By definition, external tendons are unbonded. All unbonded tendons, whether internal or external, should be permanently protected against corrosion.

3.2.2.3 Strand Size and Spacing

Seven-wire prestressing strand meets the requirements of AASHTO M203 and is used for both pretensioned and post-tensioned applications. It is available in the sizes and grades shown in Chapter 2, **Table 2.11.1**. The predominant size and grade used for pretensioning is 1/2 in. diameter, uncoated, 270 ksi. Most proprietary post-tensioning systems accommodate either 1/2 in. or 0.6 in. diameter, 270 ksi strand. Two- and three-wire strands are also available, but their use, along with the other sizes and grades of seven-wire strand, is much less common for bridge applications.

The minimum clear distance between pretensioned strands, as required by the AASHTO *Standard* and *LRFD Specifications,* is 3 times the nominal strand diameter, or 1.33 times the maximum aggregate size, whichever is greater. However, smaller center-to-center spacing of strand (1.75 in. for 1/2 in. strand, 2 in. for 0.6 in. strand) has been used successfully in various parts of the country for many years. A Federal Highway Administration Memorandum (1996) permits the lesser spacings as the result of numerous research studies. Post-tensioned tendons, which are mechanically anchored and do not rely on bond to the concrete at transfer, are exempt from these requirements.

3.2.2.4 Strand Anchors and Couplers for Pretensioning

A typical anchor for pretensioned strand is shown in **Figure 3.2.2.4-1**. Normally referred to as a "strand chuck," the device consists of a hardened steel barrel with a machined conical core. This barrel receives the jaw or wedge assembly. Wedges are used in sets of 2 or 3 pieces. They are held in alignment by a rubber "O-ring" and are tapered to match the comical shape of the barrel. The wedges have machined serrations or "teeth" that bite into and grip the strand, distributing the radial load to the barrel. The cap is spring loaded to keep the wedges in place during jacking or tensioning.

Figure 3.2.2.4-1
Strand Chuck Showing Internal Components

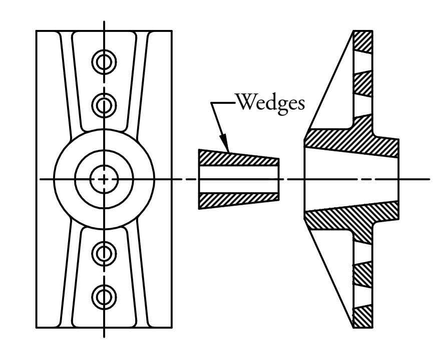
Figure 3.2.2.4-2
Strand Splice Chuck Showing Internal Components

A coupler, or "splice chuck," as the name suggests, is used to splice two lengths of strand together. As shown in **Figure 3.2.2.4-2**, they are essentially the same as strand chucks, with the exception that in place of the spring loaded head, they are furnished with male and female threads, enabling them to screw onto each other back-to-back. Couplers are not used within precast members, but rather are used to connect strand between members or strand passing through the member with "bridle" strand. See Section 3.3.2.4 for a description of "bridle" strand.

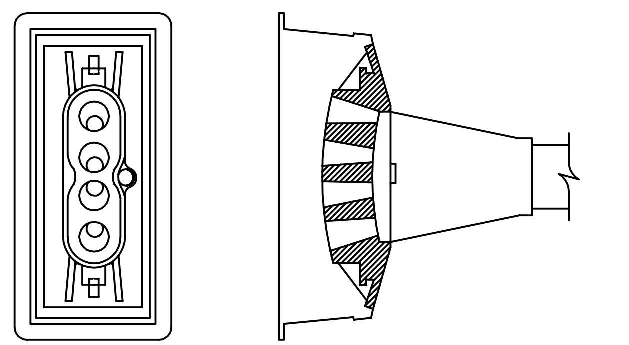
3.2.2.5
Strand Anchors and Couplers for Post-Tensioning

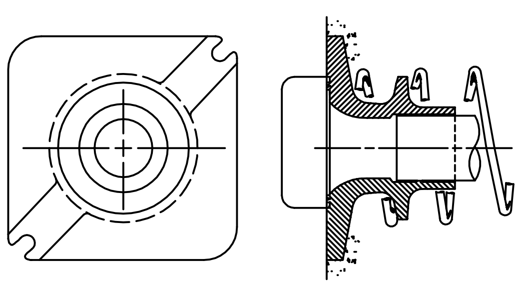
Most anchors for post-tensioned strand are proprietary, but generally use wedges similar to pretensioning anchors. These anchorages are embedded in the concrete prior to stressing, and are reinforced to resist the bursting stresses associated with high localized concentrated loads. In many cases, the wedges are hydraulically pressed into conical holes in the anchor head to reduce seating losses after jacking. Post-tensioning tendons vary from single strand tendons to multiple strand tendons which occupy the same duct and anchorage device. **Figure 3.2.2.5-1a-1d** shows typical post-tensioning anchorages.

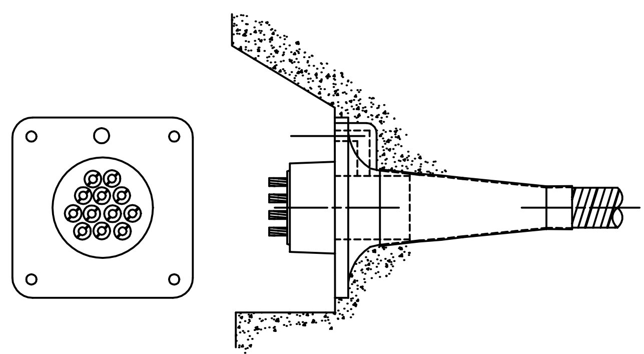
Figure 3.2.2.5-1a-1d
Types of Post-Tensioning Anchorages



a) Monostrand Anchorage

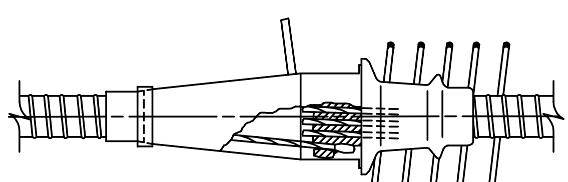

b) "Flat" Multi-Strand Anchorage


c) "Special" Cast Multi-Strand Anchorage


d) "Basic" Plate Multi-Strand Anchorage

Proprietary post-tensioning couplers are also available to join a new tendon to one which has already been placed and stressed. One such coupler is shown in **Figure 3.2.2.5-2**.

Figure 3.2.2.5-2
Post-Tensioning Coupler



3.2.2.6
Epoxy-Coated Strand

Seven-wire prestressing strand with an organic coating meeting the requirements of ASTM A882 is available for conditions which require a higher degree of corrosion protection. In pretensioned and bonded post-tensioned applications, this normally applies to exposure conditions which are particularly harsh, such as direct exposure to seawater. For unbonded post-tensioned applications, unless an alternate system of corrosion protection is employed, the epoxy coating provides the only barrier between the uncoated strand and the environment. The following sections present information and procedures for the use of epoxy-coated strand which are different from, or in addition to, those for uncoated strand. These sections are not intended to be all inclusive. A report by the PCI Ad Hoc Committee on Epoxy-Coated Strand (1993) provides excellent guidance on the use of epoxy-coated strand.

It should be emphasized that the use of epoxy-coated strand has significant cost implications. The cost of coated strand is approximately three times the cost of uncoated strand, and the set-up labor costs can increase by as much as 30 percent. For most bridge applications where the prestressing tendons are bonded, the plant-cast quality of the concrete, the concrete cover, and the limits on tensile stresses under service loads, provide excellent corrosion protection without coated strand.

3.2.2.6.1
Types of Epoxy Coating

Two types of coatings are available. For pretensioned or bonded post-tensioned applications, the coating is embedded with aluminum oxide grit to aid in the bond of the concrete to the coating. Coating without the grit is smooth and will not allow concrete bond. It is intended for unbonded post-tensioned, external post-tensioned or cable stay applications. The thickness of the coating for strand meeting ASTM A882 may vary from 25 to 45 mils. Strand with less variable coating thickness is also available, and may be necessary for compatibility with stressing hardware. Manufacturers of epoxy-coated strand should be consulted. For pretensioned applications, holes in the stressing abutments will usually need to be enlarged to accommodate the coating thickness.

Coatings with grit are extremely abrasive and appropriate precautions must be taken during handling. All workers should wear heavy protective gloves when handling the strand. Dragging the strand over steel form soffits or through holes in stressing abutments can abrade forms and elongate holes. This can result in out-of-tolerance strand positioning. Holes should be checked periodically. Dragging the strand over inappropriate surfaces or through unchamfered holes can cause damage to the coating or erosion of the grit. Any coating damage should be repaired in accordance with the manufacturer's recommendations. Loss of the grit will reduce effectiveness of the concrete bond.

3.2.2.6.2
Anchorage of Epoxy-Coated Strand

Special anchors with "bite-through" wedges designed specifically for epoxy-coated strand must be used for stressing and seating. Once seated, wedges should not be allowed to unseat during stressing, since the serrations can become contaminated with epoxy coating. Therefore, final stressing of epoxy-coated strand should be accomplished with a single stroke of the jack. Anchorage seating losses are typically higher for epoxy-coated strand than for uncoated strand (see Sects. 3.3.2.7.1 and 3.3.2.7.4). This should be considered in the stressing and elongation calculations. Wedge assemblies must be thoroughly inspected and cleaned prior to reuse. Epoxy-coated strand should not be gripped in locations where it was damaged, heated or previously gripped.

When the length of the concrete member is substantially shorter than the length of the stressing bed between abutments, a technique is used to save material costs. Uncoated "bridle" strand is often coupled to the epoxy-coated strand for the stressing bed length outside the member. See Section 3.3.2.4 for more discussion on "bridle" strand. This coupling can be done by one of two methods. The epoxy coating can be stripped from the end of the strand using a device specifically designed for this purpose. The strands can then be joined using a standard splice chuck. Alternatively, a special splice chuck can be manufactured to grip epoxy-coated strand on one side, and uncoated strand on the other.

3.2.2.6.3 Protection of the Epoxy Coating

Sharp deflection of the strand profile, such as harping in pretensioned or external post-tensioned applications, should be minimized. Friction at the deflection point during tensioning can cause damage to the coating. Tensioning the strands in a straight profile, then pulling or pushing them into the deflected position, minimizes damage. Cushioning materials can also help alleviate such damage. In internal post-tensioned applications, galvanized spiral-wound metal duct is not recommended, as damage to the coating can result from abrasion at the duct seams. Smooth polyethylene duct will minimize damage.

3.2.2.6.4 Epoxy Coating and Elevated Temperatures

At elevated temperatures like those sometimes used during accelerated curing of the concrete, the stability of the coating may be reduced, which can lead to a partial or total loss of prestress at release. ASTM A882 requires the epoxy coating to be capable of withstanding temperatures up to 150F without reduction of bond. Accordingly, the temperature of the concrete surrounding the strand must be below 150F and falling prior to release of prestress. The possibility of exposure of the members in the structure to fire, and the subsequent potential loss of prestress, should be evaluated when specifying epoxy-coated strands.

3.2.2.7 Indented Strand

Seven-wire prestressing strand with small indentations in the outer wires conforming to ASTM A886 is available in the sizes and grades shown in **Table 3.2.2.7-1**. This material is identical to normal prestressing strand meeting the requirements of AASHTO M203 with the exception of the indentations. The purpose of the indentations is to increase bond between concrete and strand and decrease the transfer and development length of pretensioned strand.

Indented strand is only used in short members where rapid transfer of the prestress force is critical. One common application is in precast, prestressed concrete railroad ties. Nearly all bridge products are of sufficient length to accommodate the transfer and development length provided by normal strand. However, some short span prestressed bridge members (e.g., stay-in-place deck forms) may benefit from the use of indented strand. Due to the decreased transfer length of indented strand, splitting

Table 3.2.2.7-1
Properties of Indented Strand

Nominal Diameter of Strand			Tensil Strength of Strand		Normal Steel Area of Strand		Nominal Weight of Strand	
in.		(mm)	lb	(KN)	in.^{2}	(mm^2)	lb/1000 ft	(kg/1000 m)
Grade 250								
1/4	(0.250)	(6.35)	9,000	(40.0)	0.036	(23.22)	122	(182)
5/16	(0.313)	(7.94)	14,500	(64.5)	0.058	(32.42)	197	(294)
3/8	(0.375)	(9.53)	20,000	(89.0)	0.080	(51.61)	272	(405)
7/16	(0.438)	(11.11)	27,000	(120.1)	0.108	(69.68)	367	(548)
1/2	(0.500)	(12.70)	36,000	(160.1)	0.144	(92.90)	490	(730)
	(0.600)	(15.24)	54,000	(240.2)	0.216	(139.35)	737	(1,094)
Grade 270								
5/16	(0.313)	(7.94)	16,500	(74.3)	0.061	(39.36)	210	(313)
3/8	(0.375)	(9.53)	23,000	(102.3)	0.085	(54.84)	290	(432)
7/16	(0.438)	(11.11)	31,000	(137.9)	0.115	(74.19)	390	(582)
1/2	(0.500)	(12.70)	41,300	(183.7)	0.153	(98.71)	520	(775)
	(0.600)	(15.24)	58,600	(266.7)	0.217	(140.0)	740	(1,102)

and bursting forces at the ends of pretensioned members will increase compared to members using normal strand.

3.2.2.8 Prestressing Bars

Prestressing bars conforming to AASHTO M275 are fabricated from high-strength steel with a minimum ultimate tensile strength of 150 ksi. The bars are either plain or deformed. Available sizes of deformed prestressing bars are shown in Chapter 2, Table 2.11.1. Plain bars are not commonly used in prestressing.

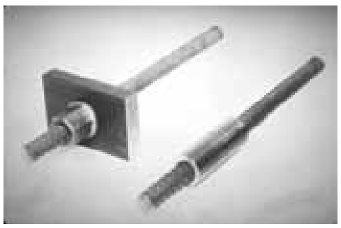


Figure 3.2.2.8-1 Prestressing Bar Anchor and Coupler

Deformed bars are generally used for post-tensioned applications where the tendon profile is straight and relatively short. In this application, the deformations are not specifically intended to provide bond with the concrete, as with mild reinforcement, but rather to allow the bars to be anchored or coupled with screw-on devices specifically designed for this purpose. Anchorage devices are normally of the plate variety, and are installed prior to casting the concrete to distribute the post-tensioning force during stressing. **Figure 3.2.2.8-1** shows a typical anchorage device and coupler system. Prestressing bars are normally not used in pretensioned applications.

Due to the relatively short lengths and large bar areas, the stressing operation is characterized by short elongations, which at times are difficult to measure and compare to theoretical values. Accurate ram calibrations are important for proper stress application. Prestressing bars may be bonded by grouting, or may be left unbonded with appropriate corrosion protection measures.

The uses of prestressing bars include transverse post-tensioning of bridge decks, diaphragms and precast multi-beam decks (flat slabs, slab beams, box beams, etc.), as well as the connection of precast members to other precast members or cast-in-place construction. **Figures 3.2.2.8-2 through 4** illustrate some of these applications.

Note that in **Figure 3.2.2.8-2**, the detail shown is often used for skewed bridges with skew angle greater than 20 degrees. For bridges with skew angle less than 20 degrees, or zero, lateral post-tensioning extends from edge-to-edge of bridge. (See Sect. 3.6.6.3).

Figure 3.2.2.8-2 Voided Slab Beams Connected through Diaphragns with Threaded Post-Tensioning Bars

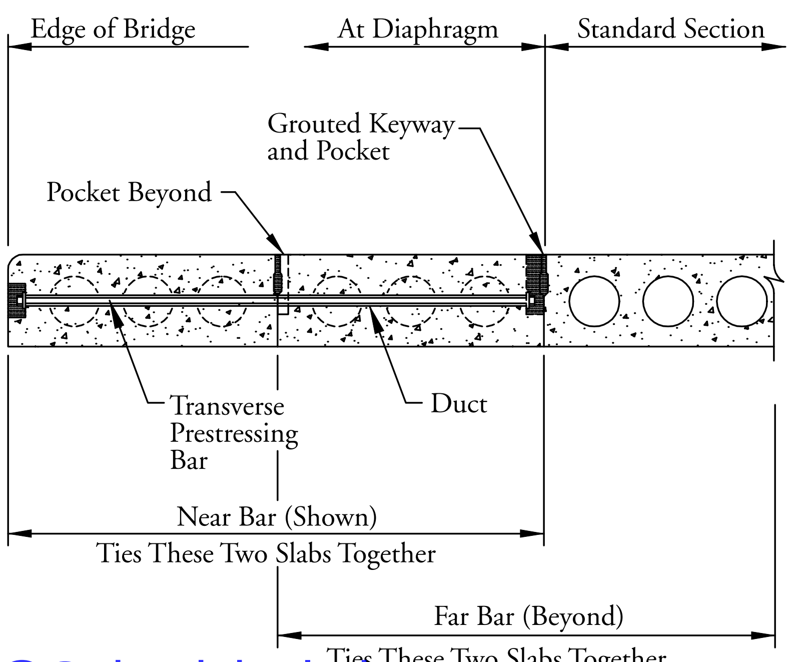


Figure 3.2.2.8-3
Box Beams Connected through Diaphragns with Threaded Post-Tensioning Bars

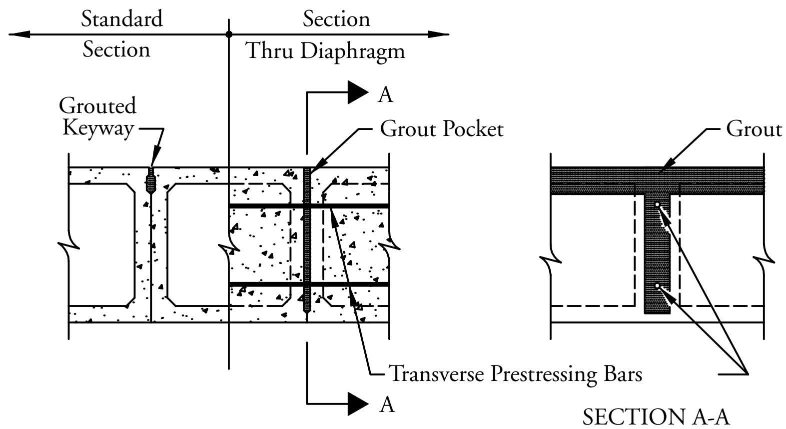
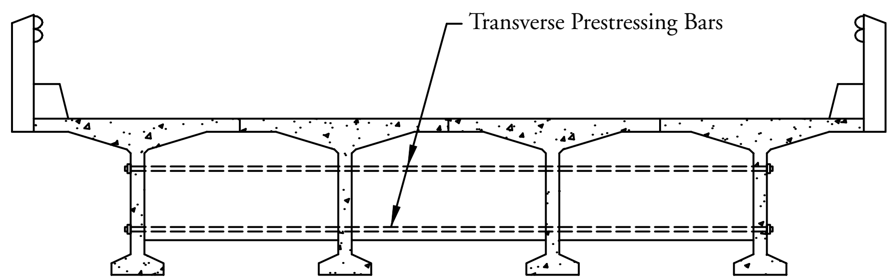


Figure 3.2.2.8-4
Deck Bulb-Tees Connected through Diaphragns with Threaded Post-Tensioning Bars



3.2.3 Nonprestressed Reinforcement

Precast, prestressed concrete bridge products are nearly always supplemented with nonprestressed reinforcement, widely referred to as “mild steel,” “mild reinforcement,” or “conventional reinforcement.” This material conforms to AASHTO M32, AASHTO M225, AASHTO M31 or ASTM A706. AASHTO M32 and AASHTO M225 address cold-worked steel wires which are smooth and deformed, respectively, and used primarily as spiral reinforcement for piles and columns. They are also used in the fabrication of welded wire reinforcement. AASHTO M31 is the most common type of deformed reinforcing bar (although this specification also includes plain bars, they are rarely used as concrete reinforcement). ASTM A706 applies to low-alloy steel deformed bars which are intended for circumstances where embrittlement, sometimes associated with AASHTO M31 bars, must be avoided. This can apply to field bent bars, or to bars to be welded. However, in some parts of the country, the availability of ASTM A706 bars is limited, particularly in small quantities. Procedures for field bending of AASHTO M31 bars, as well as proper preheating to permit welding are widely used. Consideration should be given to availability when specifying ASTM A706 reinforcement.

Prestressing steel is usually provided for all positive moments in flexural members, but may be supplemented with nonprestressed reinforcement. In many cases, negative moments at the supports of continuous spans are resisted entirely by mild steel, either in the cast-in-place deck, or in connections between precast members. Axial loads can be resisted entirely by prestressing steel, nonprestressed reinforcement, or a combination of both. Shear and torsion effects generally require the use of nonprestressed reinforcement. Flexural stresses transverse to the prestressing steel, bursting forces due to development of the prestressing forces, tensile stresses in the top flange of eccentrically prestressed members during handling, and confinement of the core of concrete piles and columns are all resisted by mild reinforcement. The following sections suggest configurations of nonprestressed reinforcement that are compatible with prestressed concrete members, and are considered standards in the industry.

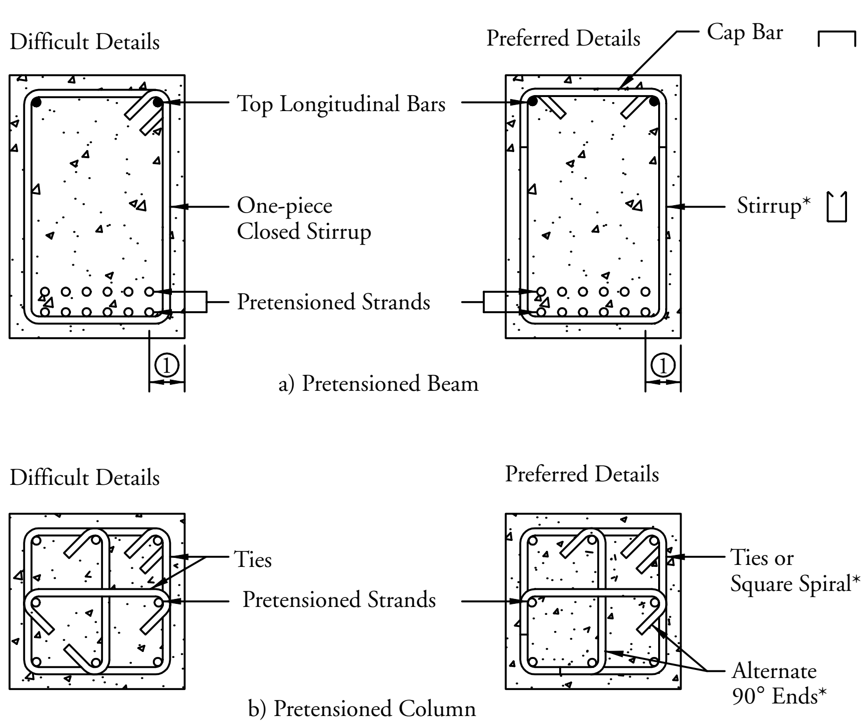
3.2.3.1 Reinforcement Detailing

In pretensioned applications, prestressing strand must be pulled from reels through one stressing abutment, over the casting bed, and into the opposite stressing abutment (or coupled into "bridle" strand already anchored to the opposite abutment). This is done either by hand, or using a winch system which can pull several strands at a time. In either case, threading the strand through closed mild steel configurations, such as shown in **Figure 3.2.3.1-1a**, becomes labor intensive, particularly when the reinforcement cannot be bundled into compact groups and spread after stressing (such as with heavy spiral reinforcement or some welded wire reinforcement cages). Whenever possible, mild reinforcement should be detailed for placement after the prestressing strand has been strung in the bed and stressed. If this is not feasible, the bars should be open at the top to allow the strands to be pulled over them. The bars may be capped after stressing if necessary. Only where mild reinforcement is required for torsion or confinement should closed bars or spirals be considered.

Care must also be taken when specifying single bar ties with bends at both ends , such as used in compression members (shown in **Figure 3.2.3.1-1b**) and in the anchorage zones of prestressed concrete flexural members. These bars should be detailed with the assumption that they are the last ones placed in the assembly, and that the prestressing strands cannot move to accommodate them. Bars with 90° bends at one end, and 135° bends at the other, with the bends alternating from side to side of the member, are generally satisfactory for placing after stressing. Section 3.2.3.5 offers suggested nonprestressed reinforcement configurations for common prestressed concrete bridge products. Note that 135° hooks are required in the AASHTO specifications in regions requiring seismic resistance or for members resisting torsion.

When detailing bars which enclose prestressing strands, proper consideration should be given to the bend radius. The dimension from the edge of the member to the strand must be sufficient to allow for both the bend radius and the required concrete cover.

Figure 3.2.3.1-1a-1b
Reinforcement Details Showing Fabrication Considerations



① Minimum Dimension = Concrete Cover + Stirrup Bar Diameter + Stirrup Bend Radius

* See text regarding 135° hooks

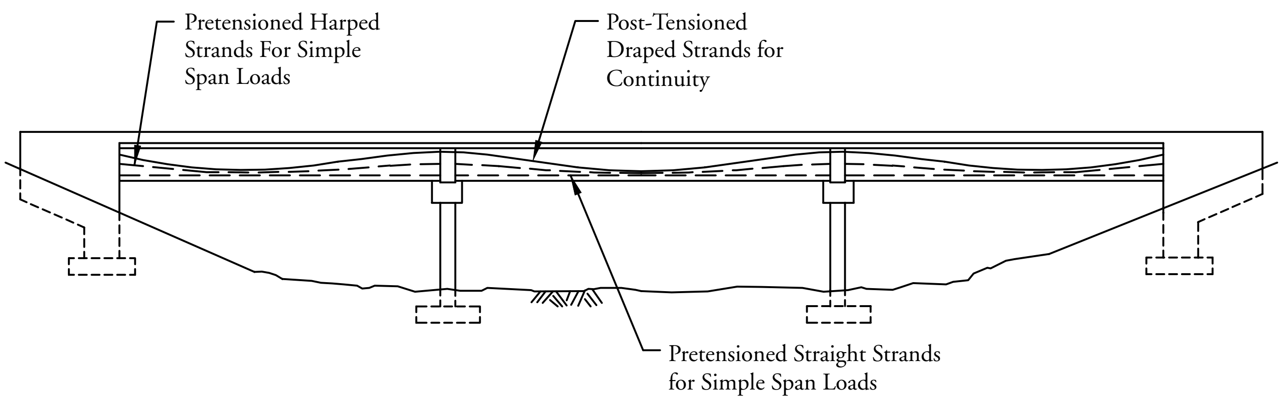
3.2.3.2
Developing Continuity

Several methods are available for developing continuity in adjacent spans with precast concrete bridge members. These are discussed in Sections 3.2.3.2.1 through 3.2.3.2.3 and specifically address development of the negative moments over the interior piers. Often, positive moments must also be considered over the piers. The most economical means of developing positive moments over the piers is by extending the necessary number of strands from the bottom flange of the precast member, and anchoring them into the pier by bending them up to provide sufficient development length. These strands may also be anchored by mechanical means, but this option is more expensive.

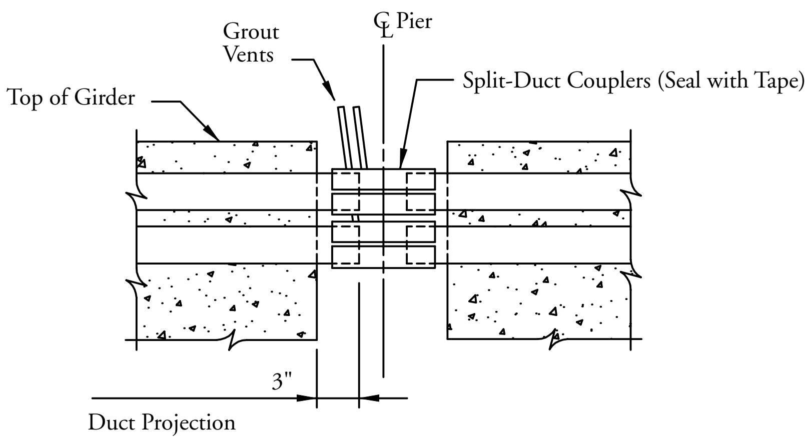
3.2.3.2.1
Continuity with Post-Tensioning

Continuity of precast, prestressed concrete spans can be achieved in several ways. The solution shown in **Figure 3.2.3.2.1-1a** could be considered the most structurally efficient. The precast members are pretensioned for the portion of the dead load imposed prior to developing continuity, and post-tensioning is added for all subsequent loads, with the tendon profile following the continuous span moment envelope. However, considering that only a limited number of standard section depths are readily available, that site conditions usually limit the range of span lengths, and that post-tensioning carries a higher cost than pretensioning, this may not be the most cost-effective alternative.

Figure 3.2.3.2.1-1a-1b
Continuity Developed with Post-Tensioning



a) Bridge Elevation Showing Tendon Profiles

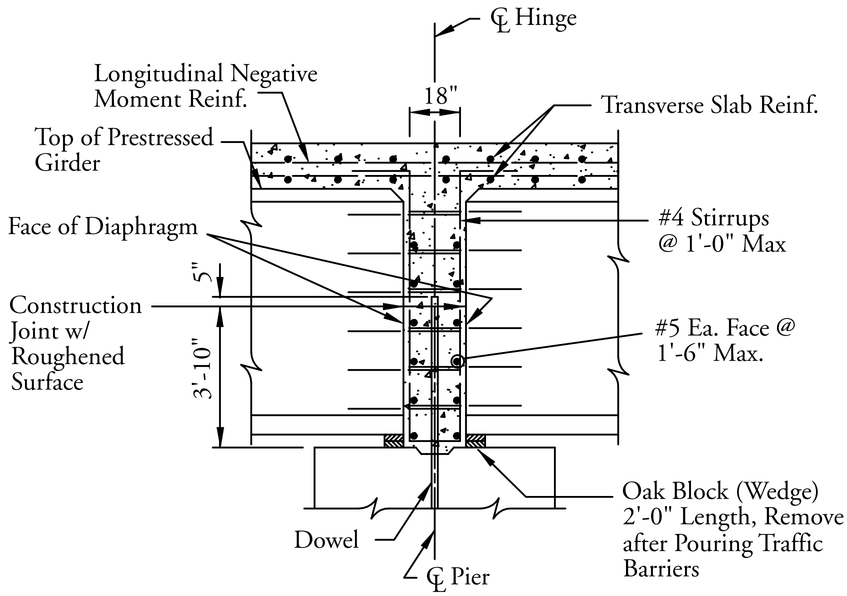


b) Duct Splices Over Pier

3.2.3.2.2 Continuity with Nonprestressed Reinforcement

A simple solution for bridges with cast-in-place decks is to proportion the longitudinal nonprestressed reinforcement in the deck over the piers to resist the negative moments. This is a very common and cost-effective method of developing continuity because it involves only straight reinforcing bars which are easily placed and spliced. A typical detail used in the State of Washington is shown in **Figure 3.2.3.2.2-1**. The detail is not typical of most agencies in the method used to transfer end reactions.

Figure 3.2.3.2.2-1
Example of Continuity Developed with Conventional Deck Reinforcement (typical Washington State Detail—see text)



3.2.3.2.3 Continuity in Full-Depth Members

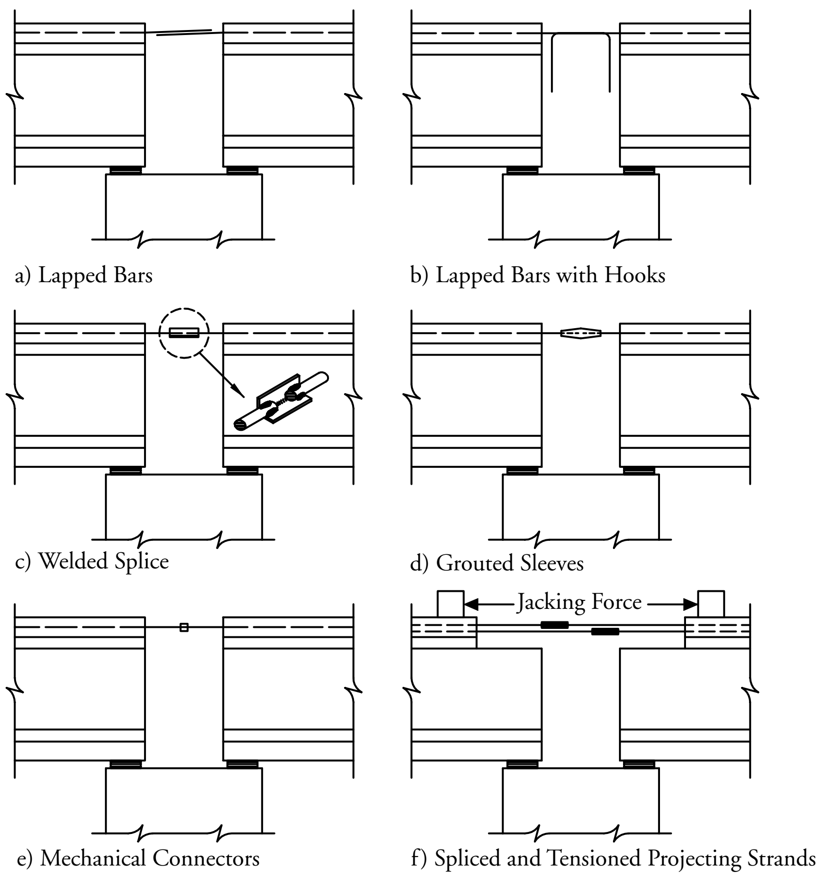
Precast members which are full depth, or are topped only with a non-structural wearing surface, must be post-tensioned over the piers as described earlier, or must have projecting reinforcement spliced in some manner to provide negative moment capacity. Several methods have been successfully employed in splicing the projecting reinforcement, some of which are illustrated in **Figure 3.2.3.2.3-1**.

If the pier is of sufficient width, the simplest and least expensive method is to provide a non-contact lap splice of mild reinforcement extending from the top of the precast members (**Fig. 3.2.3.2.3-1a**). These bars may also be hooked to aid with development (**Fig. 3.2.3.2.3-1b**). In this case, the bars should be allowed to be field bent, since the form normally extends past the end of the member, and may interfere with the placement of pre-bent bars. In both cases, the bars should be staggered horizontally to avoid interference with bars from the facing member, and with reinforcement projecting from the pier.

When the pier does not provide sufficient width for lapped or hooked bars, nonprestressed reinforcement projecting from the top of the precast members may be spliced mechanically by welding, with grouted splice sleeves, or with mechanical splices. Some of the more common splice details are shown in **Figures 3.2.3.2.3-1c through 1e.** A wide variety of generic and proprietary splicing details are available. Each detail has advantages and disadvantages with respect to material cost, labor cost, tolerances for fabrication and erection, and the degree of quality control required to properly execute the splice. Bridge designers should consult local producers for information on the splice details favored by builders in the local area.

Yet another solution involves coupling prestressing strands that extend from the top of the precast members. After coupling the strands, the members are jacked apart at the pier to induce required tensile forces in the coupled strands. This is shown in **Figure 3.2.3.2.3-1f** and reported by Tadros, et al (1993) and Ficenec, et al (1993). With the members held apart, the cast-in-place closure is made. Once the closure concrete attains design strength, the jack is released to apply compression across the joint. The resulting continuous spans behave much the same as with the post-tensioned solution, without the expense of the post-tensioning hardware.

Figure 3.2.3.2.3-1a-1f Methods to Establish Continuity



3.2.3.3
Coated Nonprestressed Reinforcement

Reinforcing bars coated for corrosion protection are naturally more expensive than uncoated bars, both in material and labor costs. Epoxy coating conforming to AASHTO M284 or ASTM A934 and galvanizing conforming to ASTM A767 roughly double the cost of uncoated material. Special epoxy coatings, such as designed for the protection of steel pipe, raise material costs even higher when specified for use on reinforcing bars. Increased development lengths of epoxy-coated bars will slightly increase the amount of material required.

The quality of plant-cast concrete, the control of concrete cover, and the limits on tensile stresses for prestressed concrete members under service loads provide excellent corrosion protection for uncoated reinforcement under normal exposure conditions. Coated reinforcement should only be considered for severe exposure conditions.

3.2.3.3.1 Epoxy-Coated Nonprestressed Reinforcement

The effectiveness of epoxy coatings in preventing corrosion is only as good as the integrity of the coating, as summarized by D'Arcy, et al (1996). Consequently, specifications for the bars for shipping, handling, placing, and protection during concrete placement are increasingly restrictive. Labor costs are increased for these types of bars due to the special handling required to prevent damage to the coating, and to repairing areas damaged due to cutting, bending or handling. When specifying bars to be cut and bent prior to coating, consideration should be given to constructability. For example, hooked bars projecting from the ends of precast members may interfere with the formwork at the member ends. In many cases, field bending of bars is the best option for constructability.

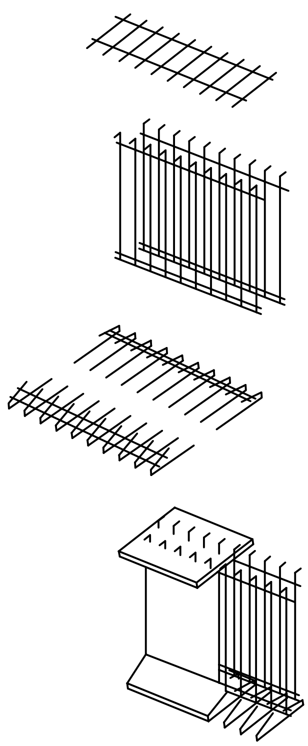
3.2.3.3.2 Galvanized Nonprestressed Reinforcement

When specifying reinforcement to be bent prior to hot-dip galvanizing, the specifications should direct the fabricator to ASTM A767, which dictates larger finished bend diameters for #7 bars and larger than is standard for uncoated bars. This is important in preventing embrittlement of the steel during the hot galvanizing process.

3.2.3.4 Welded Wire Reinforcement

Welded wire reinforcement (WWR) has long been the standard for reinforcing floor slabs, wall panels and other flat-cast products. Recently, this material has gained popularity as a substitute for nonprestressed reinforcing bars in precast concrete bridge products. Depending on the configuration of the sheets, the cost of this material can be from 50-100 percent higher than mild reinforcing bars. However, savings in labor and inspection costs can more than offset the material cost disadvantage. The price and availability of WWR suitable for precast bridge products varies in different geographical regions. Local producers should be consulted for information on the cost effectiveness of WWR in their area.

Figure 3.2.3.4-1 Welded Wire Reinforcement in a Bulb-Tee



Sheets of WWR can be made in virtually any configuration up to approximately 14.5 ft wide, with smooth (AASHTO M55) or deformed (AASHTO M221) wires up to 5/8 in. in diameter (W31 or D31, the equivalent of a #5 reinforcing bar). The sheets are normally shipped flat and bent at the precast plant, since shipping pre-bent sheets can result in a large amount of "ghost" freight. The sheet lengths are limited by the length of available benders, which is currently a maximum of about 30 ft.

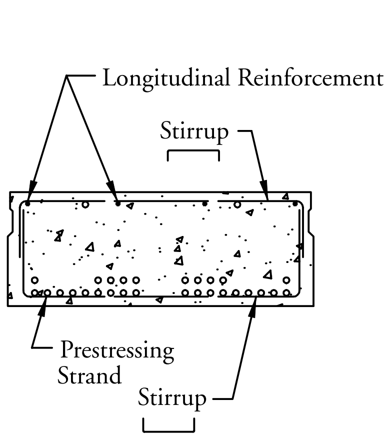
Figure 3.2.3.4-1 shows the end reinforcement of a typical prestressed concrete bridge beam using WWR. The labor savings results from eliminating the need to tie individual bars into the required configuration. Improvement in quality can also be expected, since the bar spacing of WWR is much more precise than can be expected from tying individual bars.

The key to efficient use of WWR is standardization. Purchasing is most economical when ordering in truckload quantities. Therefore, unless the project is large, the precast producer must be reasonably assured that any WWR that ends up in inventory will be usable on future projects. WWR configurations should also be detailed to allow installation after the prestressing strands have been strung and tensioned. Much of the savings attributed to the use of WWR will be lost if the strands must be pulled through long runs of enclosed reinforcement.

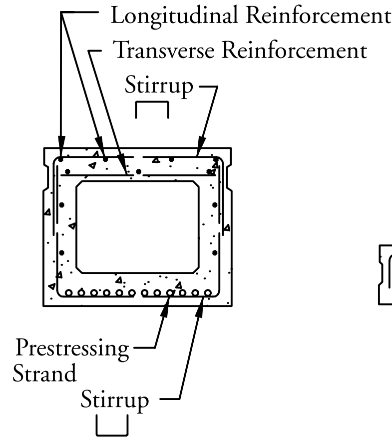
3.2.3.5 Suggested Reinforcement Details

Figure 3.2.3.5-1 shows suggested nonprestressed reinforcement configurations for various types of standard precast concrete bridge products. WWR cages can be patterned after these configurations.

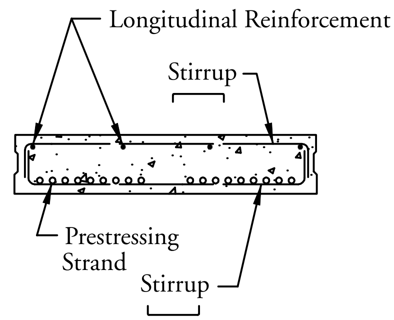
Figure 3.2.3.5-1a-1g
Recommended Reinforcement Configurations for Standard Products



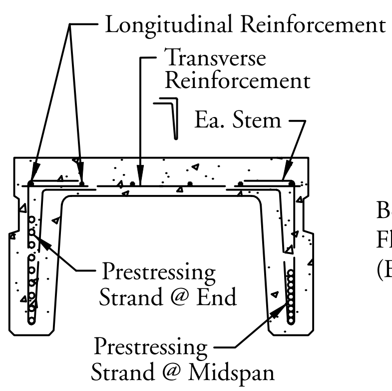
a) Solid or Voided Slab Beam



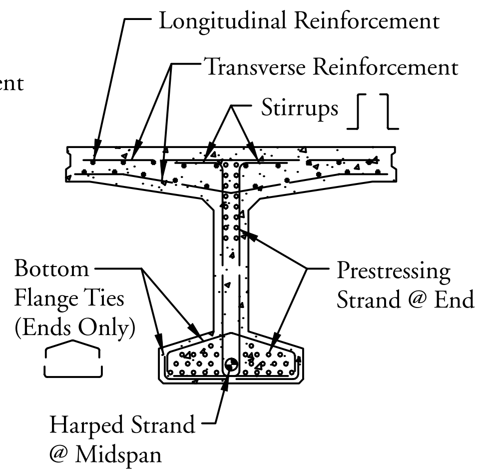
b) Box Beam



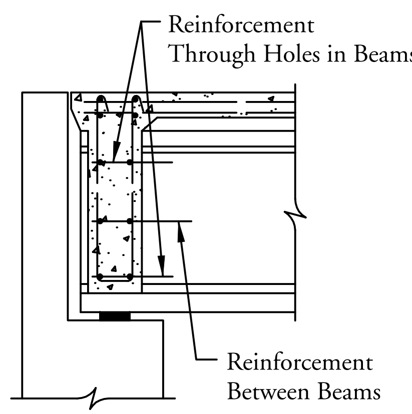
c) Full-Depth Deck Panel



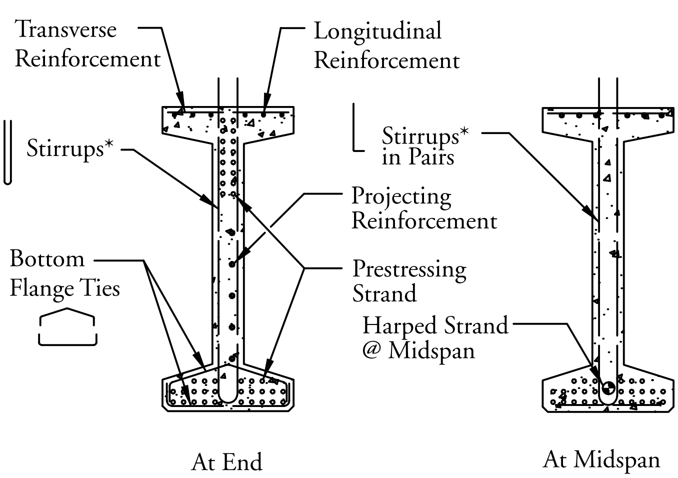
d) Channel Beam



e) Deck Bulb-Tee



f) Cast-in-Place Diaphragm



* Alternate types shown; use one type throughout

g) I- and Bulb-Tee Beams

3.2.4
Embedments and Blockouts

Embedments and blockouts in precast concrete bridge products are used typically for the following:

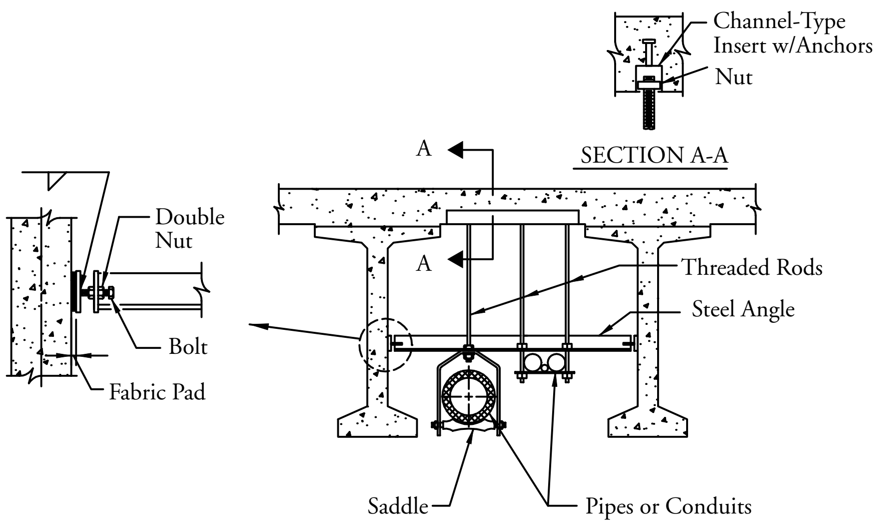
- hanging utilities
- for connecting the members to other members of the structural system
- for attaching cast-in-place concrete formwork
- for handling and shipping the members

The following sections describe common embedments and blockouts used for these purposes. A wide range of details are used throughout the country. Bridge designers should consult with local producers for preferred details.

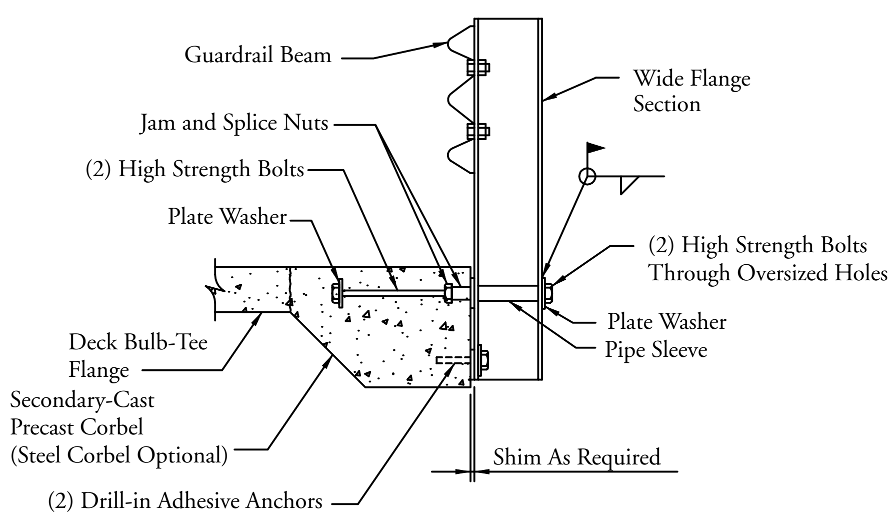
3.2.4.1
Embedments and Blockouts for Attachments

Numerous types of embedments are available for connecting miscellaneous items, such as utilities and guardrails, to precast concrete members. These embedments range from simple threaded inserts to complex welded assemblies. Depending upon

Figure 3.2.4.1-1a-1b
Common Attachments



a) Utility Connections



b) Guardrail Connection

the anticipated exposure, the embedments can consist of uncoated steel, coated steel, stainless steel, plastics or any other material which is both suitable for the intended purpose and compatible with both the concrete and reinforcing steel. Examples of common connections are shown in **Figure 3.2.4.1-1a-1b**.

The combined tolerances for all parts of the system should be considered when detailing attachments to inserts embedded in precast members. Slotted or oversized holes are highly recommended wherever possible. Section 3.4.6 provides information on industry standard tolerances.

3.2.4.2 Embedments and Blockouts for Diaphragms

Embedments for diaphragms depend on the type of diaphragm used, ranging from threaded inserts and holes for reinforcement for cast-in-place concrete and attachment of temporary diaphragms, to welded assemblies for precast concrete and steel diaphragms. A detailed discussion and examples of typical diaphragms are included in Section 3.7.

3.2.4.3 Embedments and Blockouts for Deck Construction

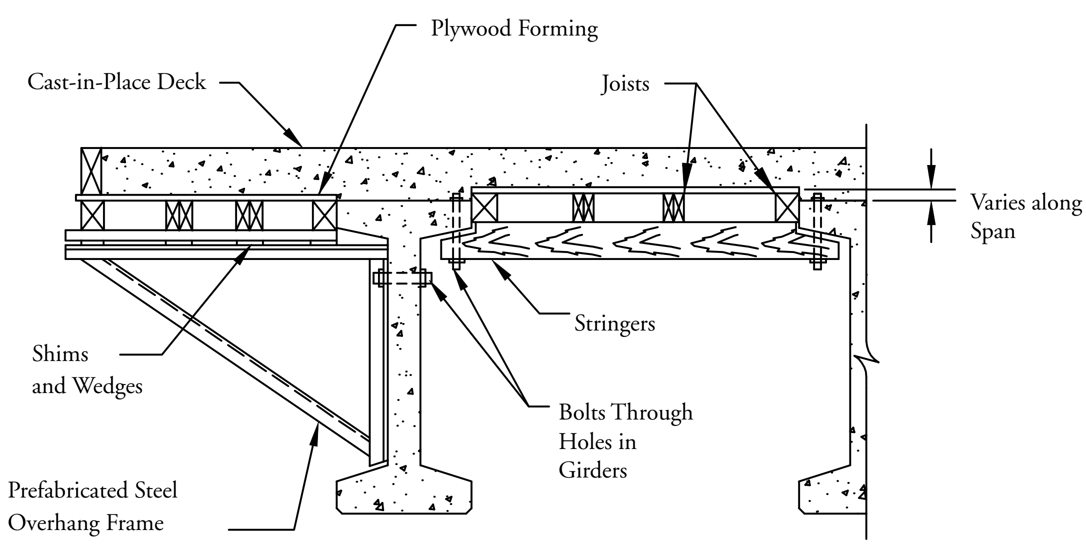
Deck construction usually falls into one of three methods:

- cast-in-place concrete over bulb-tees or I-beams
- cast-in-place concrete over composite stay-in-place deck panel forms
- no cast-in-place concrete but simply connecting totally precast concrete members (normally referred to as adjacent precast multi-beam decks)

Embedments and blockouts required for decks vary for each type of construction.

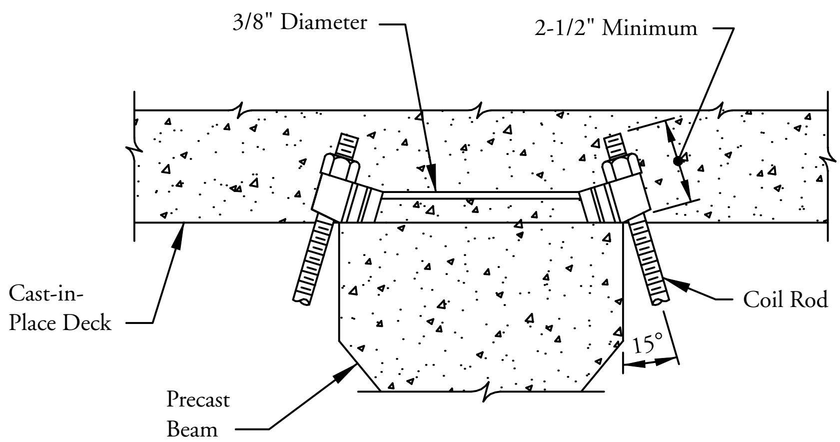
Placing cast-in-place concrete over bulb-tees or I-beams requires supplementary formwork, which is normally hung from the girders. This can be done economically with a series of holes and bolts, through either the girder flange or web, as shown in **Figure 3.2.4.3-1**. Form attachment can also be accomplished with proprietary sys-

Figure 3.2.4.3-1
Typical Cast-In-Place Deck Forming Methods



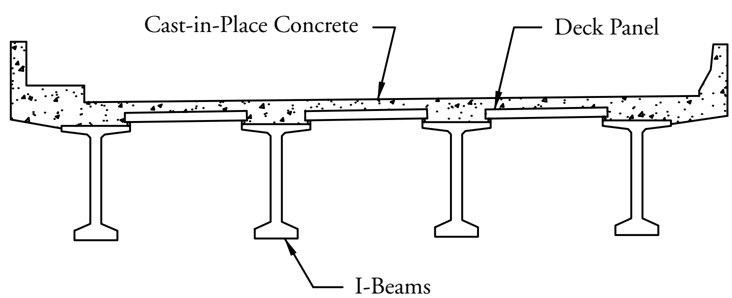
tems, such as that shown in **Figure 3.2.4.3-2**. Other methods can also be employed to attach the formwork.

Figure 3.2.4.3-2
Proprietary Cast-In-Place Deck Forming Method

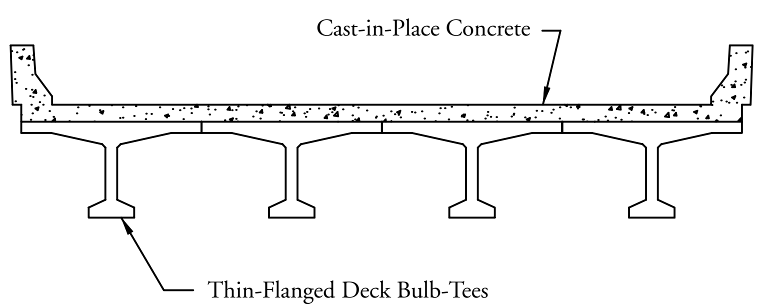


Composite stay-in-place deck forms fall into two general categories. The first is thin prestressed concrete deck panels designed to span between spread box beams or flanges of bulb-tees, I-beams or steel beams. The second is thin, wide flanges cast integrally with bulb-tees in the plant. The bulb-tees (or similar wide-flanged products) are abutted in the field. These are shown in **Figure 3.2.4.3-3a-3b**. Neither system requires embedments or blockouts in the beams, except for forming the edge overhang slab in deck panel sys-

Figure 3.2.4.3-3a-3b
Composite Bridge Deck Systems



a) Precast Concrete Composite Deck Panels



b) Thin-Flange Deck Bulb-Tee Deck System tems. Typical embedments in deck panels are discussed in Section 3.8. Thin-flange deck bulb-tees require no embedments or blockouts for deck construction, except perhaps inserts for attachment of the formwork at the edge of the cast-in-place deck.

Methods to connect multi-beam decks vary depending on the type of members being joined. Connections are normally welded or post-tensioned. Section 3.6.3 discusses typical details and considerations.

3.2.4.4 Lifting Devices

Lifting devices embedded in precast concrete bridge members normally consist of strand lift loops, bolts or proprietary metal inserts. For very heavy lifts, prestressing bars have also been used. The type of lifting device employed depends upon several considerations:

- configuration of the member
- load on each device
- angle of the lifting strap
- distance of the embedment from the edge of the concrete
- preference of the precast plant

The strength of the embedded lifting device is governed by the weakest link in its load path. This can be the strength of the device itself, the bond between the device and the concrete, or for shallow embedments, the strength of the shear cone that can be pulled from the concrete. Most precast plants and vendors of proprietary lifting devices have extensive experience in the design and use of lifting devices.

Bent reinforcing bars should not be used as lifting devices. Bending a bar decreases the ductility of the material in the region of the bend. Highly concentrated loads from a shackle or crane hook, have been known to fracture bent bars in a brittle manner. In addition, bent reinforcing bars do not have the flexibility of strand lift loops. When picking with angled cables, which is very common in both plant handling and field erection, flexibility is critical in distributing the load uniformly to all legs of the loop.

3.2.4.4.1 Strand Lift Loops

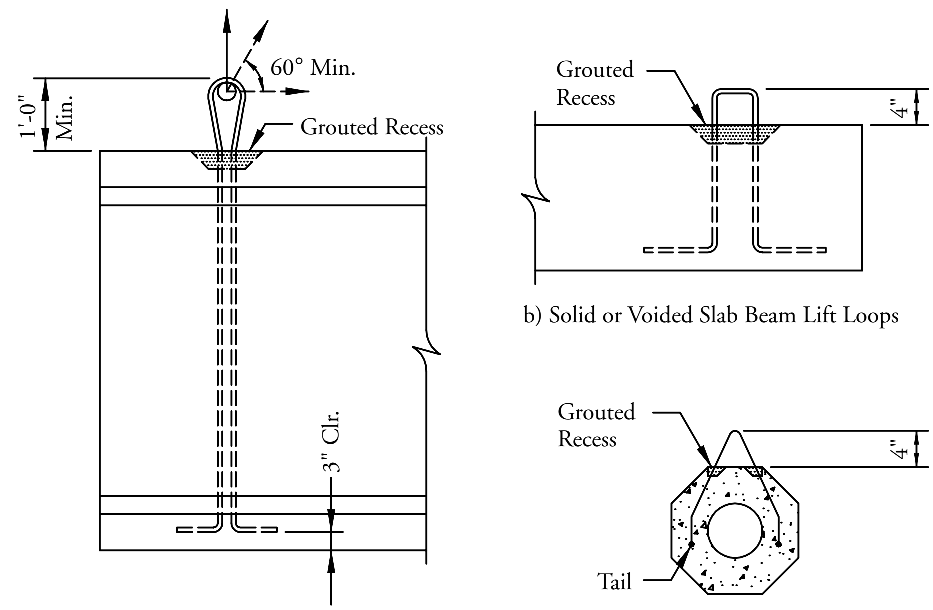
Prestressing strand lift loops are widely used due to their high strength and flexibility. Loops can be bent into nearly any configuration suited to the intended application. They are also economical since in many cases they are made from what would otherwise be "waste" strand, such as the tail end of a strand pack, or tails cut from a production run. Strand which has been damaged by gripping jaws or pitted with rust should not be used for lift loops.

The capacity of strand lift loops is governed by the following:

- size and grade of the strand
- configuration of the loop
- length of embedment into the concrete
- diameter of the pin used through the loops during lifting

Typical lift loop configurations are shown in **Figure 3.2.4.4.1-1**. The capacity of lift loops embedded with angled legs, as shown in **Figure 3.2.4.4.1-1c**, are reduced due to the increased resultant force in each leg. Loops lifted with angled cables are similarly reduced, particularly when their projection from the concrete is insufficient to allow the loops to flex to the same angle as the cable. Strand embedment must be of sufficient length to preclude bond failure. Tails can be added to the ends of the loops to increase embedment, such as shown in **Figures 3.2.4.4.1-1b and 3.2.4.4.1-1c**. The surrounding concrete should be adequately reinforced to prevent splitting and loss of bond. Small diameter shackle pins or hooks, when used through strand lift loops, can significantly decrease the capacity of the loop.

Figure 3.2.4.4.1-1a-1c
Typical Lift Loop Configurations



a) I-, Box or Bulb-Tee Beam Lift Loops

b) Solid or Voided Slab Beam Lift Loops

c) Hollow Pile Lift Loops

3.2.4.4.2
Other Lifting Embedments

Bolts used for lifting perform much the same as headed studs. Embedment must be sufficient to prevent concrete shear cone failure, and edge distance must be considered when it encroaches on the shear cone.

A wide range of proprietary lifting devices also is available. For bridge applications, these are normally limited to products that are relatively thin and light such as precast wing walls, barriers and soundwalls.

3.2.4.5
Blockouts for Shipping

Precast members are normally secured to the truck, railcar or barge with chains or straps (tie-downs), which are draped over the top of the member and tightened. Wide, thin top flanges can sustain damage if the tension in the chain or strap is applied to the tip of the flange. Some producers provide blockouts in the flange adjacent to a web for tie-downs to pass through. Section 3.5.3.5 discusses these blockouts in further detail.

3.2.5
Surface Treatments

Since most bridge products are cast in precision-made steel forms, it makes sense to design and fabricate the form so that the precast member requires minimum additional surface treatment. In most cases, this finishing needs to be performed by hand, so the most obvious savings are economic. Finishing operations such as removing lips or fins at form joints, patching paste bleed at form joints, and sacking bugholes, add time to the production cycle and increase the production cost of the product.

Theoretically it is possible, using rigid steel forms and compaction of the concrete by vibration, to produce members that are uniform in appearance and with a "glass-like" finish. This finish will not only be aesthetically pleasing, but it will produce a surface with maximum durability. A densely compacted cement-paste surface finish produced against steel forms results in a surface with minimum porosity and permeability, and maximum long-term durability. Any additional surface treatments or patching are unlikely to improve, or even match, the durability of densely compacted concrete. In reality, however, some areas on pretensioned products will require surface treatment in spite of the best possible form design.

This section does not cover patching of major "honeycomb" areas or large voids. It also does not cover the repair of structural cracks or large spalls caused by form removal. These subjects are discussed in Section 3.4.2.

3.2.5.1 Protecting Product Ends

The parts of a prestressed product that typically require surface treatment are the ends where the prestressing strands exit and have been trimmed-off after detensioning. Usually for production expediency, and because of the physical constraints of the bulkheads, the projecting strands are initially cut off during stripping about 6 in. from the concrete surface. Then, during the finishing process, the strands are trimmed flush to the member. Depending upon the method of incorporation into the bridge structure, there are two techniques used to patch the strand ends.

3.2.5.1.1 Ends Cast into Concrete

If the ends of the member are incorporated into the pier or abutment with cast-in-place concrete, the treatment of the strand ends is not critical. The producer only needs to protect them from corrosion during storage. In this case, after the strand is cut off flush, it is satisfactory to simply paint them with a two-component epoxy. A thin coat, approximately 1/8 in. thick, will suffice and only in a 2 in. square area over each strand. In applying the epoxy, it is important that the strand and surrounding concrete be relatively clean. Often, if the strands are burned off with a poorly adjusted oxy-acetylene torch, the surrounding area is blackened with acetylene soot and melted slag, which should be removed prior to epoxy application. It is for this reason that many producers prefer to grind off the projecting strand with a hand-held high-speed disc grinder. This method leaves a clean area for the epoxy coating.

3.2.5.1.2 Exposed Ends

Strand ends which are to be exposed for the service life of the structure are normally recessed and patched. Various methods are used. A common procedure is to provide a recess with a proprietary expanded foam cube placed on each strand, directly against the inside of the bulkhead. This recess-forming device, sometimes called a "doughnut," is approximately 1 1/2 in. square and 3/4 in. thick, with a hole through the center to accommodate the strand. The doughnut is split on one edge and can be placed over the strand at any location before or after tensioning. During the finishing process, the expanded foam and projecting strand are burned out using an oxy-acetylene torch. The recess is first cleaned-out to remove any remains of the expanded foam and strand slag, then patched flush with the concrete surface.

For maximum durability, the material used to patch the recess should be appropriate. A poorly selected material applied in the recess will soon shrink, deteriorate, or even fall out. This will leave the strands exposed to the environment, promoting corrosion by the capillary action of liquid through the interstices of the seven-wire strand.

3.2.5.1.3 Epoxy Mortar End Patches

Epoxy mortar is often used to patch strand recesses, since it is widely assumed to be less permeable and more durable than portland cement mortar. This is generally true, assuming the epoxy has been appropriately selected and mixed, and the correct epoxy binder-to-sand ratio has been used. However, field experience has shown that the use of incorrect procedures to prepare epoxy mortars, particularly in selection of the sand type, gradation, and mixing procedures, can result in a porous patch that provides inadequate protection of the strand end.

Epoxy mortar sands should to be angular in shape, since sands with rounded particles tend to roll under the trowel, making placement difficult. The sand should also be dry. Two good sand gradations are blends by volume of two parts 12 mesh to one part 80 mesh, or three parts 16 mesh to one part 90 mesh. When graded sands are not available, 30 mesh silica sand works reasonably well. Most epoxy resin suppliers can furnish these sands.

Another disadvantage of epoxy mortar patches is that pure epoxies generally have a higher coefficient of thermal expansion than concrete. Larger patches, particularly those using pure epoxy, can fail due to differential expansion and contraction of the patch and the parent concrete. More than cost reduction, this is the primary reason that a silica sand "extender" is used in epoxy mortar. The incorporation of sand reduces the coefficient of thermal expansion of the epoxy mortar mixture.

3.2.5.1.4 Portland Cement Mortar End Patches

Considering the high demand for quality control and the cost of epoxy mortars, some producers patch the strand recesses with portland cement mortar. This mortar is considerably less expensive, and also has a coefficient of thermal expansion similar to that of the parent concrete. It is easier to work with, and can be matched to the member finish. A patch made with gray portland cement and sand will generally be darker than the surrounding concrete. This outcome can be mitigated by using 25 - 40 percent white portland cement in the patch mix. The usual cement-to-sand ratio is 1:2, and an epoxy bonding compound is applied to the recess before the mortar is troweled in. The "dry-pack" method of placement is also common. Properly executed, these portland cement patches perform as well or better than epoxy mortar patches, and are more economical.

3.2.5.1.5 Patching Ends with Proprietary Products

Proprietary patching compounds can also be used to fill recesses. In general, these are at least as expensive as epoxy mortar. The majority of these materials are portland cement based and contain combinations of accelerators, bonding agents, fillers, and workability, curing and shrinkage-compensating aids. The performance of such material should be carefully evaluated by the producer. In some cases, long-term durability could be sacrificed in favor of ease of initial application.

Other proprietary patching compounds are available that are not portland cement based. Examples are: polyester resin-based materials, high alumina cement-based material, and magnesium phosphate cement-based materials. These materials are often promoted as providing a solution to patching in temperatures below 40 F, where portland cement and epoxy-based mortars are not recommended. However, in practice this is rarely necessary because in cold weather climates, precast members are usually cured with heat. With careful timing, the producer can take advantage of the elevated temperature of the member immediately after stripping to perform the patch and cure the patching material. Use of these proprietary non-portland cement-based patching materials should be evaluated on a case-by-case basis by an accredited concrete laboratory.

3.2.5.2 Intentionally Roughened Surfaces

Another bridge product surface that often requires non-cosmetic treatment is one which is intentionally roughened to promote mechanical bond of cast-in-place concrete to the member. This is usually specified when the member is to be made composite with cast-in-place concrete. Most I-beams and bulb-tees are designed to act compositely with a cast-in-place deck. Section 3.3.9 covers the preparation of these surfaces in detail.

3.2.5.3
Cosmetic Surface Treatments

Surface finishes resulting from good daily production practices will not be entirely "glass-like." Some of the most common imperfections are:

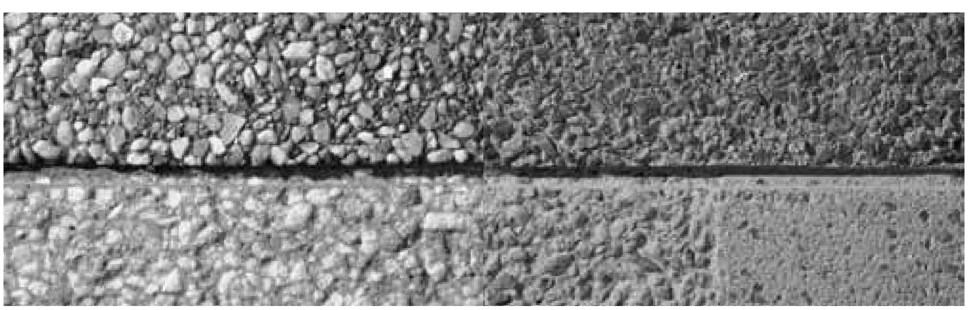
- small surface "bugholes" formed by entrapped water and air bubbles at the form skin, particularly on vertical surfaces
- dark lines and areas denoting high cement paste concentrations
- "pour-lines" due to the overlapping of individual concrete placements
- granular surface areas where the paste has bled out of form joints
- imperfections and offsets at form joints

There are as many theories about the cause of these aesthetic surface blemishes as there are suggested methods to avoid them. The best methods of placement and compaction do not consistently eliminate their occurrence. For this reason, many producers "rub" or "sack" the surface of the member immediately after it is stripped. This involves wetting down the member, hand-applying a 1:1 fine sand and portland cement mortar to the surface using a sponge-faced trowel to fill any bugholes, then curing the application. Often, this surface is then rubbed with a burlap sack and cement powder. This procedure, which is more art than science, benefits greatly from the care, knowledge and diligence of an experienced concrete finisher. In general, unless the bridge is in a high visibility zone, this additional finishing needs only to be done on the exterior surface of the fascia beams. For high visibility zones where a uniform appearance is required, pigmented sealers can be applied.

3.2.5.4
Architectural Finishes

A wide variety of architectural concrete finishes, normally used for building cladding applications, could be used in the production of precast concrete bridge members. These include colored concrete using integral dyes, the use of white cement, exposed aggregate finishes, ribs or other textured surfaces, and the application of stains. Practically, however, except for the application of stains, these treatments are rarely employed for large structural members, such as I-beams or bulb-tees. The need to optimize the concrete's early strength gain normally precludes the use of white cement, which is usually ground to AASHTO M85 Type I specifications. The other processes may also prove difficult to control under large-scale production runs typical of large bridge members. The same is not true of other types of bridge products, such as median barriers or soundwalls, where architectural treatments are standard practice. **Figures 3.2.5.4-1** and **2** show some typical architectural finishes. Local producers should be consulted for economically available architectural treatments.

Figure 3.2.5.4-1a-1b
Architectural Finishes with Exposed Aggregate



a) Top: Surface Retarded & Exposed
Bottom: Medium Sandblast

b) Top: Surface Retarded & Exposed
Bottom Left: Deep Sandblast
Bottom Right: Light Sandblast

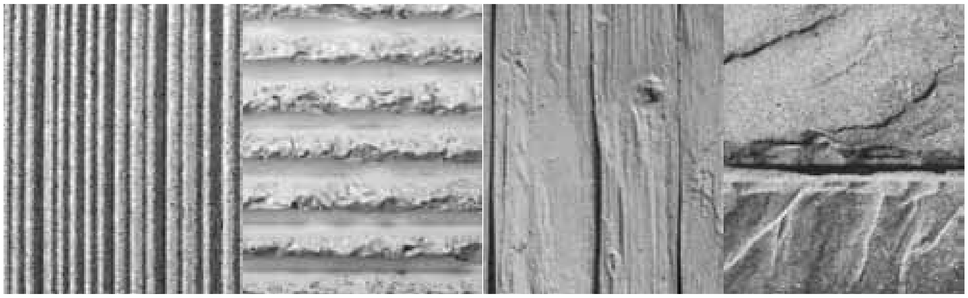


Figure 3.2.5.4-2a-2b Architectural Finishes Made with Formliners

a) Left: Striated
Right: Hammered Rib

b) Left: Cedar Stake
Right: Ashlar Stone

3.2.5.5
Durability-Related Treatments

Some specifications require a final surface treatment be applied to precast concrete bridge members as added assurance of long-term durability. The most common treatment employed is the application of a penetrating sealer, such as a silane or siloxane coating. Various studies conclude that, assuming the material is properly selected and applied, these materials enhance the long-term durability of concrete, as summarized by D'Arcy, et al (1996). From a producer's perspective, one of the difficulties associated with the application of silanes is that the members must be kept dry for a minimum period before application. In rainy climates, this requires the members to be stored and the work performed under cover. Considering the size and number of the products involved, most precast plants do not have facilities appropriate for this operation. The most economical application of these sealers is usually accomplished in the field, since delivery schedules are no longer an issue, and the application can wait for good weather, or can be done under the cover of the completed bridge deck. Epoxy coatings have also been specified to provide surface protection, long-term durability, and wear resistance. Most surface treatments have limited life and need periodic renewal to achieve continued protection.

3.2.5.6
Protection of Exposed Steel

Another issue that should not be overlooked is protection of projecting reinforcing bars, strand and metal hardware embedded in the member. If the products are expected to be stored for a significant length of time, projecting reinforcing bars and strand are normally coated with zinc-rich paint for protection against corrosion prior to incorporation into the structure. If this is not done, the projecting steel quickly develops a surface coating of rust. Although this is usually not detrimental over short storage periods, and can be cleaned off immediately before delivery, wet weather will cause this rust to run down the faces of the member, causing unsightly stains that are difficult to remove.

The most common protection for metal embedments is hot-dip galvanizing before they are cast into the concrete (AASHTO M111). This results in the optimum long-term protection of the embedments. When welding galvanized embedments, it is important to first remove the zinc coating from the area of the weld. Toxic fumes are produced from welding on galvanizing, and the zinc may contaminate the weld metal, which can result in a structurally deficient weld. After the welding has been performed, the damaged coating should be restored, either by "soldering" over the area with zinc rod, or by painting the area with a zinc-rich paint.

For this reason, zinc-rich paints are sometimes specified in lieu of galvanizing. The embedments are given an initial coat of paint before being cast into the concrete, and are given subsequent coats after the welding has been completed. Epoxy-based and other volatile solvent zinc-rich paints were once popular for this application. However, with increasing Hazardous Waste Disposal Regulations, the recent tendency has been towards water-based zinc-rich paints.

3.3 FABRICATION

Precast, prestressed concrete bridge products are fabricated under strictly controlled plant manufacturing conditions to assure the highest level of quality possible in concrete construction. Industry standards demand uniform quality of finished products nation-wide. This section will discuss standard methods of concrete forming, batching, placing and curing, as well as tensioning of the prestressing steel and placement of nonprestressed reinforcement. Fabrication methods and production capabilities differ from plant to plant, with varying consequences for the design of precast concrete bridge members. Bridge designers should consult with local producers for specific information on plant capabilities.

3.3.1 Forms and Headers

Forms used in the precast, prestressed concrete industry are unique to the standard product they are intended to produce, though most forms have three common characteristics. First, forms are fabricated with a constant cross section. Second, they are long and slender, with overall lengths ranging from 20 ft to more than 600 ft. Finally, they must be capable of producing the same concrete shape repeatedly to very accurate dimensional tolerances. For example, the PCI manual, MNL-116, allows a casting tolerance for the width of an I-beam web of only +3/8 in., -1/4 in. Therefore, most bridge product forms use steel construction. **Figure 3.3.1-1** shows typical forms used in the industry.

Figure 3.3.1-1a-1d
Common Precasting Forms

a) AASHTO I-Beam

b) Stemmed Channel Section

c) AASHTO-PCI Bulb-Tee

d) Beam/Piling

3.3.1.1 Self-Stressing Forms

A self-stressing form is used not only to produce the concrete product, but also to resist the initial prestressing force. The form's longitudinal stiffeners and skin are used as the compression member against which the prestressing strands are jacked. This method is often cost-effective, since it eliminates the need for a traditional prestressing bed (see Sect. 3.3.2.1). **Figure 3.3.1.1-1** shows a typical self-stressing form and its associated stressing hardware.

Members of different cross-sections can be cast in the same self-stressing form, as long as the form is designed for the largest and most highly pretensioned section. For

Figure 3.3.1.1-1 Self-Stressing Form for a Stemmed Section

example, it is quite common to cast 24 in. deep stemmed members in a 36 in. deep self-stressing form by using 12 in. "false bottoms" in the stems. Likewise, 8 ft-wide stemmed members can be cast in a 10 ft wide self-stressing form by using "false sides." The prestressing force is still distributed over the original form configuration. This can also be done with other member types.

3.3.1.1.1 Applications of Self-Stressing Forms

Self-stressing forms have become a popular solution for precast members where the location of the prestressing force is not excessively high or eccentric. For bridges, they are used for stemmed members, prismatic or trapezoidal beams, box beams and voided slab beams. There are two basic considerations that limit the use of self-stressing forms. First, the eccentricity of the prestressing force must be small enough to allow the load to be distributed to the form in a reasonably uniform manner. Second, this type of form requires the strands to be jacked against the entire cross-section of the form, including the sides, which must be in place during stressing. Set-up crews must be able to assemble the reinforcement cage and install embedments from the top only. If the form is deeper than the length of a person's arm, it is difficult to place material in the bottom of the form.

Self-stressing forms can also be designed to accommodate harped or draped strands. The vertical reaction from deflected strands can be transferred through the form into the foundation. For this reason, among others, it is usually necessary to mount the form on a reinforced concrete slab. These slabs run full length, and are slightly wider than the form. The self-stressing form is attached to this slab to maintain alignment, to provide intermediate bracing for the compressive force, and to provide anchorage to prevent the form from being lifted off the ground during stripping.

If a product cast in a self-stressing form is to be heat cured, it is essential to recognize that the form will expand as the concrete temperature is elevated. For a 600 ft length of form, it is not uncommon for the form to expand up to 6 in. during the curing cycle. Also, the form will shorten due to the prestressing force imparted during jacking. For these reasons, the form attachment to the slab must not restrain the form in the longitudinal direction. The usual approach is to weld or bolt a 20 to 40 ft section of the form to the slab, either at one end or in the center, and design all the other connections to allow longitudinal movement.

3.3.1.2 Non-Self-Stressing Forms

Tall, slender bridge members such as I-beams, bulb-tees, deck bulb-tees and large stemmed members are usually cast in forms that are not self-stressing. The primary reason for this is that the prestressing strands, nonprestressed reinforcement, and embedments are, by necessity, placed in the form with the sides removed. Also, these types of members usually have a relatively high location (eccentricity) of prestress. Since the bulk of the prestressing force would need to be distributed to in-place form sides, self-stressing forms are not appropriate for these applications.

The prestressing force and deflected strand vertical reactions for this type of member are normally resisted by an independent prestressing bed. These beds are discussed in detail in Section 3.3.2.1.

3.3.1.2.1 Design of Non-Self-Stressing Forms

The design and fabrication of form sides for casting prestressed concrete bridge members are not governed solely by the equivalent fluid pressures induced during concrete placement, but also by the need to minimize temporary and permanent deformations, and to account for the affects of external form vibration, repeated heating and cooling cycles, and repeated use.

In practice, this requires the forms to be fabricated from steel. When using 1/4 in.-thick form skins, continuous vertical and horizontal stiffeners are usually required at no more than 2'-6" on center in each direction. Some manufacturers fabricate forms with 3/16 in.-thick steel skin and the same stiffener spacing. During repeated use, this steel tends to “oil-can” between bulkheads, or suffer premature damage due to fatigue induced by external form vibration. Vertical stiffeners are usually fabricated from 1/4 in. plate with folded flanges. Horizontal stiffeners can be standard steel shapes, or can be fabricated by the form manufacturer. Each form supplier has their own preference and usually provides the form design to accommodate the specified casting procedure.

3.3.1.3 Adjustable Forms

Innovative form design not only facilitates rapid assembly and disassembly on a daily basis, but also provides long life from each form. Most producers purchase forms that are easily modified to accommodate various member sizes with similar cross-sections. For example, I-beam and bulb-tee forms are commonly designed with a standard shape for top and bottom flanges, and a variable web height. This allows the same form to be used for shallow and deep members with varying span lengths. The forms are split horizontally, usually near mid-height, and bolt-in “fillers” are used to vary the girder depth. **Figure 3.3.1.3-1** illustrates a typical adjustable form. Girder widths

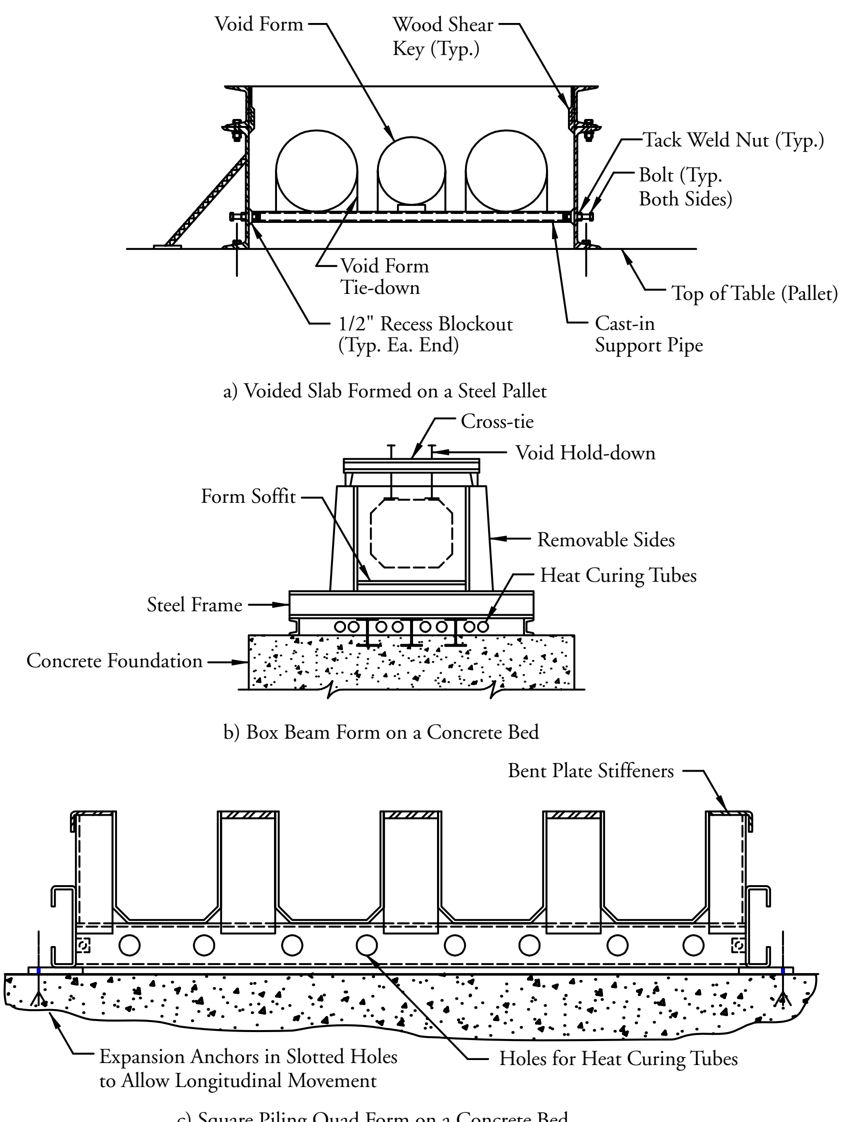
Figure 3.3.1.3-1 Bulb-Tee Form Used to Fabricate the Florida DOT Section. Horizontal Joint is where Form can be Separated for Installation of Fillers to Increase Depth of Section

can also vary by adjusting the width between form sides. However, since the shape of the form sides is normally fixed, other horizontal girder dimensions are affected incrementally. Some manufacturers have forms which can accommodate wider top flanges. Producers should be consulted for dimensions of forms which can vary from the local standard.

I-beam and bulb-tee bottom flanges are formed in two ways. Some producers use flat steel pallets, with the sides and top of the bottom flange form being part of the side form. This allows for easy adjustment of width. Other producers use "pans" which form the bottom and sides of the bottom flange. The top of the bottom flange form is part of the side form. In this case, varying girder widths require different pan widths.

Voided slab beams and box beams are normally cast on horizontal steel pallets with removable side forms. Pallets are usually sized to accommodate the widest member anticipated. Narrower members can be cast with relative ease. Concrete slabs can be used in lieu of steel pallets, although the uniformity of heat for curing the bottom flange of the member becomes less reliable. **Figure 3.3.1.3-2** illustrates typical pris-

Figure 3.3.1.3-2a-2c
Various Form Configurations



matic form configurations. The depth is not as easily varied because most producers do not have a variety of form sides on hand. Bridge designers should consult with local producers for available form depths. Small increases in depth using standard form sides are accommodated by "adding" to the form side, most commonly with steel angles. Decreases in depth can be achieved by manually holding the concrete down below the top of the form. Special "drop" screeds are used for this purpose. Members with mild reinforcement projecting from the top complicate the use of drop screeds.

3.3.1.4
Advantages of Precast Concrete Formwork

A unique and attractive feature of plant-cast bridge members, in contrast to typical jobsite construction, is the elimination of "through-bolts," "she-bolts," or "snap-ties." Such devices are laborious to place, inhibit placement of reinforcement and embedments, and require patching of the resulting recesses. Most forms used in the precast industry are held together only at the top and bottom, resulting in increased productivity, and patch-free vertical surfaces. Also, precast formwork joints are designed to minimize paste bleed during concrete placement.

3.3.1.5
Other Form Considerations

Side forms for deep, thin bridge members tend to be heavy and usually require stripping by crane or some other mechanical device. Since form sides are relatively slender, cranes usually cannot remove them in one piece for the full length of a long member. Form sides are normally bolted together in 20 or 40 ft sections. To reduce the equipment and labor required to strip the forms, some producers install "roll-away" form sides. This system uses side forms mounted on trolleys and transverse rails that allow them to be rolled away laterally from the member during stripping. Long side forms can then stay in one piece for subsequent production. A disadvantage of this system is that the sides need to be retracted far enough to allow the work crews access for production. Many plants do not have sufficient space for this system, particularly those with parallel prestressing beds.

Side forms that are vertical, or which form shear keys in the sides of members, must be removed daily to strip the product. This increases both labor costs and wear and tear on the forms. Where possible, the sides of products should have a minimum draft of 1/8 in. per ft (1/4 in. per ft is preferable) so that the side forms may stay in place during stripping. Concrete members formed in rigid, single-piece forms, always require adequate draft on surfaces that could otherwise meet at right angles.

3.3.1.6
Headers

The terms "header," "endplate," "bucket"and "bulkhead" are used interchangeably in the precast concrete industry to describe devices used to form the ends of precast

Figure 3.3.1.6-1a-1b
Forms for Ends of Sections

a) I-Beam End Plate *b) Stem Bulkheads or Buckets for a Triple-Stemmed Section* members. In this manual, the word "endplate" is used to describe a device that forms the end of a single member, or the last member in a series of members cast end-to-end in a prestressing bed. The word "bulkhead" is used to describe a device that forms the adjacent ends of two members cast in series. "Header" can refer to either an endplate or bulkhead. **Figure 3.3.1.6-1** illustrates typical endplates and bulkheads. They can also be seen in **Figure 3.3.1-1c**.

3.3.1.6.1 Header Configuration

A requirement common to both endplates and bulkheads is the penetration of the prestressing strands. The hole that each individual strand passes through not only controls the strand location, but also may be subject to forces from intentional or unintentional deflection of the strand. Some headers are designed with slots at edges to accommodate placement after some or all of the strands have been tensioned. Holes should be oversized a minimum of 1/16 in. and the edges should be rounded. Sharp edges can damage a strand during tensioning, with potentially catastrophic results. Both endplates and bulkheads must be restrained longitudinally to resist movement during concrete placement, as well as being dragged along with the strands during tensioning. Most producers are reluctant to drill form sides for bolting endplates, due to damage to the forms caused by the penetrations. Various alternate methods are used to secure the endplates, such as cable clamps to fix the bulkheads on the strands already tensioned.

Forms designed to cast several members in series can present problems with the extended projection of strands or reinforcing bars. Header configurations that can present problems are normally limited to shallow members, such as stemmed members or slab beams. These standard bulkheads are just wide enough to allow cutting of the strands between members, and cannot accommodate extended projections. Extended projections can also create problems during stripping of the members. In some cases, it is more economical to use threaded reinforcing bar couplers to provide extended projections.

3.3.1.7 Internal Void Forms

Members such as voided slab beams, hollow box beams and hollow piling require internal voids. Forming can be achieved in one of three ways:

- a retractable "mandrel" system
- a collapsible form which is retracted after the concrete has hardened
- casting over sacrificial forming material

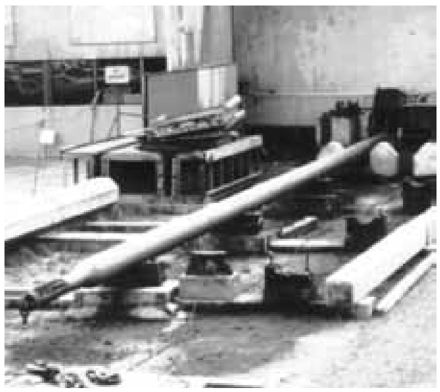
All internal forms must be accurately located and held in place during concrete placement. In monolithic pours, the inner forms will have a tendency to float. The force required to hold the inner form in place can be calculated assuming the concrete to be completely liquefied. Hold-down systems usually consist of vertical compression struts, which react against crossbeams that span the top of the form. These vertical struts are normally tapered, coated with a debonding agent, and withdrawn after the concrete has hardened. In designing such systems, the inner form must be analyzed to determine the spacing of the restraining struts, both for the span of the form material, and for the local stresses imposed by the struts. When expanded polystyrene blocks are used, appropriately sized pads are placed on top of the block under each strut to prevent localized compression failure of the block.

Flotation forces during concrete placement can be controlled to some extent by pouring procedures and timing. If the concrete initially placed directly under the void form is allowed to approach initial set prior to further concrete placement around the vertical form sides, the flotation forces are substantially reduced. In the fabrication of box beams, some producers place the bottom slab as a first stage. After the concrete hardens, normally the next day, the void form is placed, an epoxy bond coat is applied to the cold joint, and the webs and top flange are placed. For void forms with vertical sides, this results in very little uplift.

3.3.1.7.1
Mandrel Systems

A system of forming internal voids employs a vibrating steel slip-form, normally referred to as a mandrel, which is pulled through the concrete during placement. The concrete used with this technique must be designed with a low slump, so that it does not collapse after the mandrel passes. Mandrels must have a constant cross-section throughout their length, and are not easily modified to produce shapes with varying dimensions. Together with the necessary winches and tooling required for their operation, mandrels represent a significant investment for the precast producer. Consequently, their use is normally limited to standard cross sections. **Figure 3.3.1.7.1-1** shows a typical mandrel used for casting hollow prestressed concrete piles. The design of endplates and bulkheads is complicated by the need for the mandrel to pass. Solid sections or diaphragms within a precast member, if required, are usually added with a secondary cast.

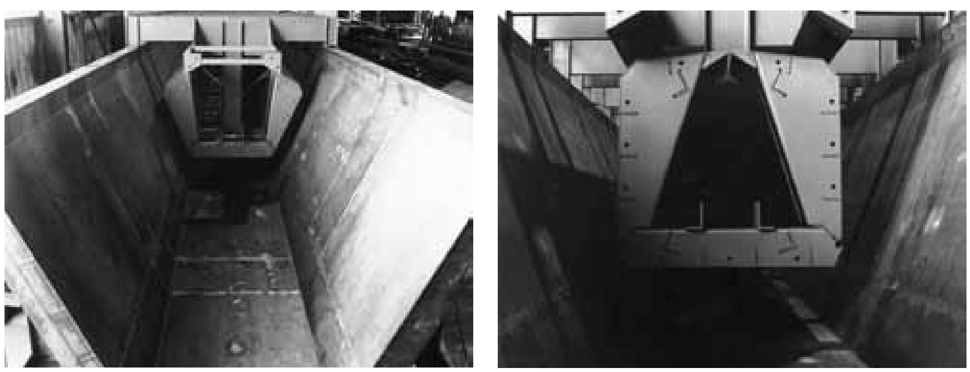
Figure 3.3.1.7.1-1
Cylindrical Slip-Form "Mandrel" for Hollow Piling



3.3.1.7.2
Retractable Inner Forms

Retractable inner void forms are stationary, and the concrete is cast around them to create the void. These forms must be designed to collapse for removal after the concrete has hardened. Articulating forms of this nature, as well as their retraction tooling, are expensive, and are normally reserved for standard shapes with large voids (such as box beams), or for large projects which can tolerate high initial tooling costs. **Figure 3.3.1.7.2-1a-1b** illustrates a typical retractable form.

Figure 3.3.1.7.2-1a-1b
Removable Void Form



a) Void Form Expanded for Casting *b) Void Form Retracted for Removal*

3.3.1.7.3
Sacrificial Inner Forms

Sacrificial inner forms can be made from wax-coated cardboard tubes or boxes, prefabricated plywood boxes, or blocks of expanded polystyrene. The choice of material depends on the size and shape of the voids. For example, voids in a typical 4 ft-wide x 2 ft-deep slab beam are usually formed with cardboard tubes capped with plywood endplates. However, to create the inner void of a large box beam, the choice may be between blocks of expanded polystyrene cut with hot wires, or boxes constructed from plywood. Expanded polystyrene is the most common choice because it

Figure 3.3.1.7.3-1a-1b
Stay-In-Place Inner Forms


a) Waxed Cardboard Tube


b) Polystyrene Foam Billet

eliminates the risk of collapse that can occur with hollow void forms. **Figures 3.3.1.7.3-1a** and **1b** illustrate typical applications of sacrificial inner forms.

3.3.2 Prestressing

Careful control of the prestressing operation is critical to the quality of prestressed concrete products. The following sections describe common types of beds used for pretensioning, typical procedures and controls employed to ensure that the proper level of prestress is delivered to the concrete. Pretensioning procedures apply only to strand, since prestressing bars are not used in pretensioned applications. An article by Preston (1990) describes the manufacture of strand and its corrosion characteristics; precautions during use and for handling; and, special considerations during concrete curing and detensioning.

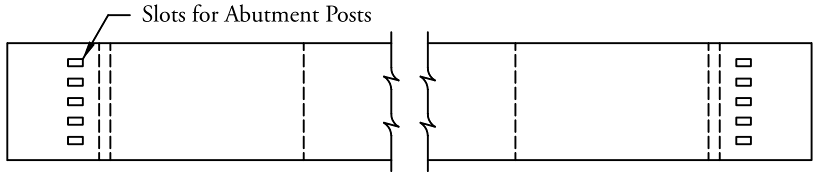
3.3.2.1 Types of Pretensioning Beds

In addition to the self-stressing forms described in Section 3.3.1.1, two basic types of prestressing beds are commonly used in precasting plants. These are generally referred to as “abutment-type” beds, and “strutted” beds. In contrast to self-stressing forms, both types are independent of the formwork used to cast the member.

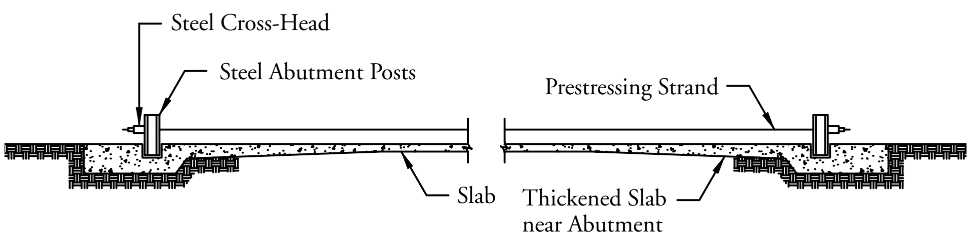
3.3.2.1.1 Abutment Beds

A bed employing abutments normally consists of a full length concrete slab, which is substantially thickened at each end to provide foundations for the support of vertical steel or concrete “uprights” (abutments). A typical profile is shown in **Figure 3.3.2.1.1-1**. The center portion of the slab is designed to carry the highest design axial force from the prestressing operation, as well as vertical forces created by deflect-

Figure 3.3.2.1.1-1
Permanent Prestressing Bed with Fixed Abutments



a) Plan View



b) Elevation View

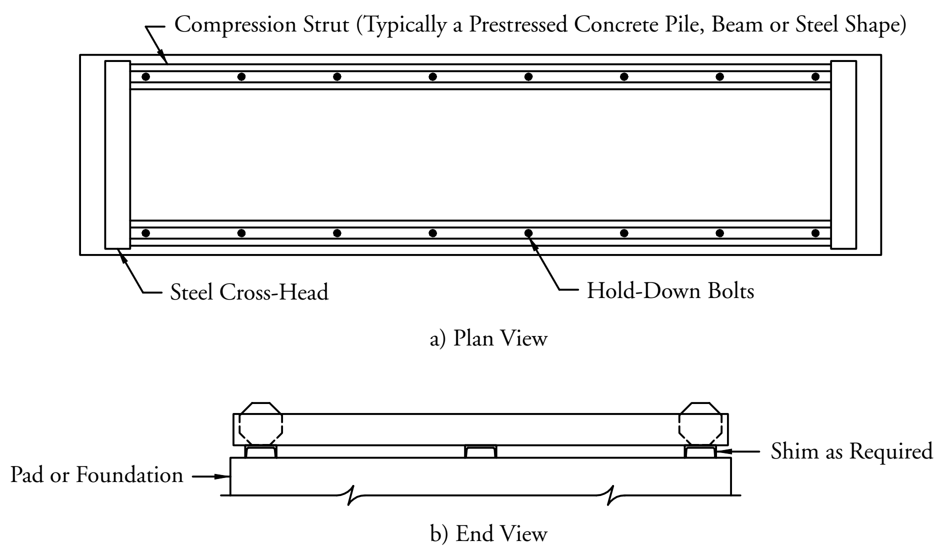
ed strands. The thickened ends are designed to transfer concentrated loads from the abutments into the body of the slab, and to resist overturning moments from the eccentricity of the prestress force. Global overturning is usually countered by the inertial resistance of the concrete foundation's mass. Although abutment beds have the highest capacity among available types, all pretensioning beds are limited in the number of strands that can be accommodated, either due to the total prestressing force, or strand eccentricity from the bed's center of resistance. Bridge designers should check with local producers for pretensioning capabilities.

Steel abutments are usually inserted into "slots" or "trenches" cast into the foundation. Though slots limit the transverse adjustment capability of the abutments, they are more economically incorporated into the foundation design. Trenches provide a large degree of flexibility, but are more expensive to accommodate. The type chosen depends upon the anticipated use of the facility. When necessary, strands are distributed transversely by "cross-heads" spanning horizontally between uprights. "Distributions," or templates, which are independent of the stressing hardware, are also employed to deflect strands vertically or horizontally from the configuration of the holes in the standard stressing hardware to the configuration required for the precast member.

3.3.2.1.2 Strutted Beds

Strutted beds employ independent compression struts running from end-to-end. **Figure 3.3.2.1.2-1** schematically shows a strutted bed. The strands are stressed between cross-heads, which span from strut-to-strut. These set-ups are normally reserved for strand patterns in a few horizontal planes, such as with prestressed slab beams or wall panels.

Figure 3.3.2.1.2-1a-1b "Strutted" Prestressing Bed



3.3.2.2 Strand Profile

Pretensioning strands can project straight through the length of a member, can be deflected in straight segments to a desired profile, or can be a combination of both. Straight strands are the simplest to install and tension, while deflected strands, normally referred to as "harped" or "draped" strands, more closely follow the moment envelope of flexural members. **Figure 3.3.2.2-1** illustrates how varying strand profiles correlate with typical moment envelopes. Post-tensioned strands can be straight, or can be draped in a curved profile to best fit the moment envelope. Post-tensioned bars are normally used for straight profiles only.

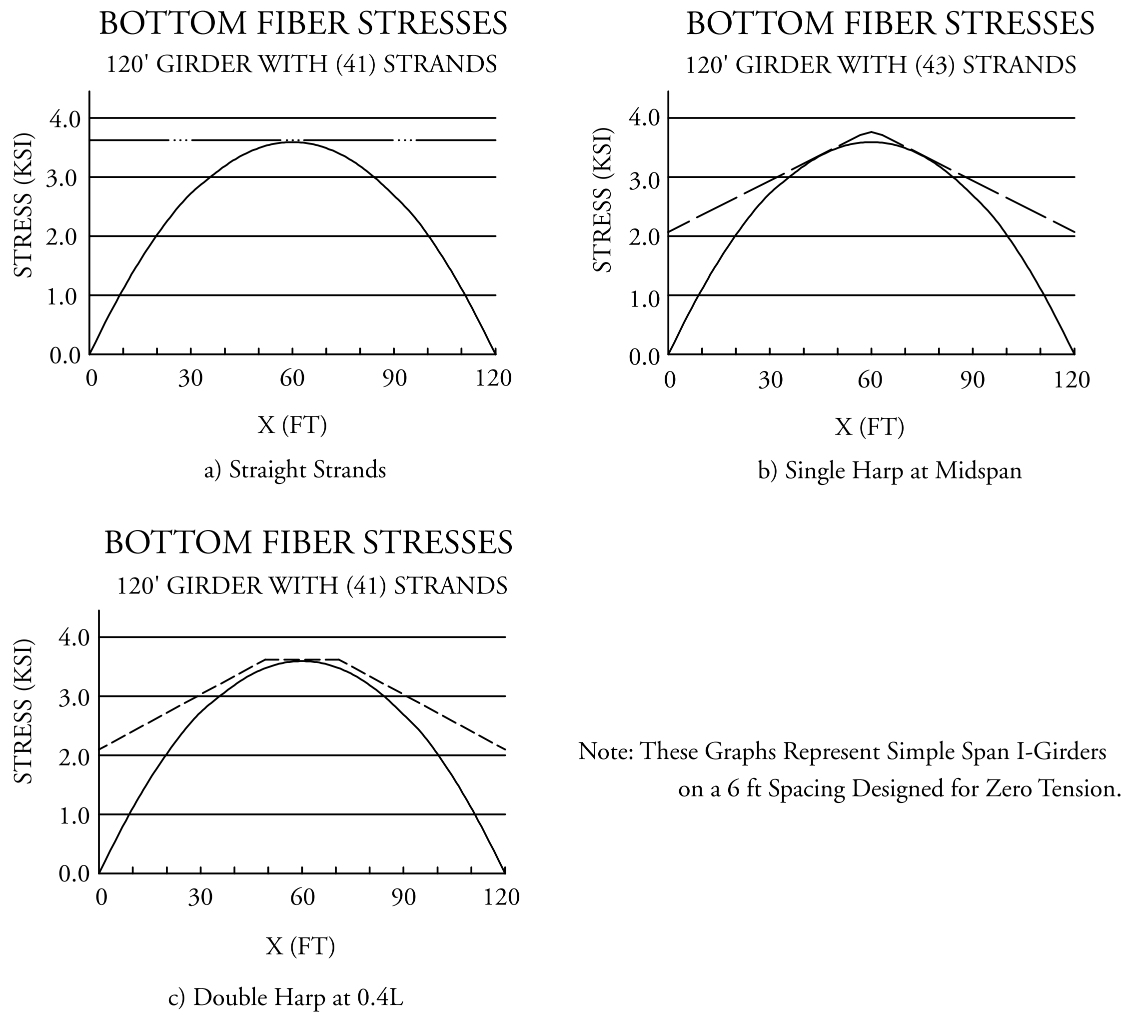


Figure 3.3.2.2-1a-1c Bottom Fiber Stresses for Three Strand Profiles

3.3.2.2.1
Straight Strands

Pretensioned members containing only straight strands are normally wide and relatively shallow, such as voided slab beams and some box beams. The strands are distributed horizontally across the width of the member as uniformly and symmetrically as possible, though they can be offset to some degree to avoid openings or other obstructions. By necessity, the eccentricity of the prestressing force is relatively small. Otherwise, excessive tensile and compressive stresses can develop at the member ends, where these stresses are not offset by the member dead load moment. Straight strands in the top of the member can compensate for this to some degree, but often result in design inefficiencies. The most common approach to control end stresses is to debond some of the strands at the member ends. This method is discussed further in Section 3.3.2.9. Harping strands which are distributed across a wide, shallow member is very difficult and should be avoided.

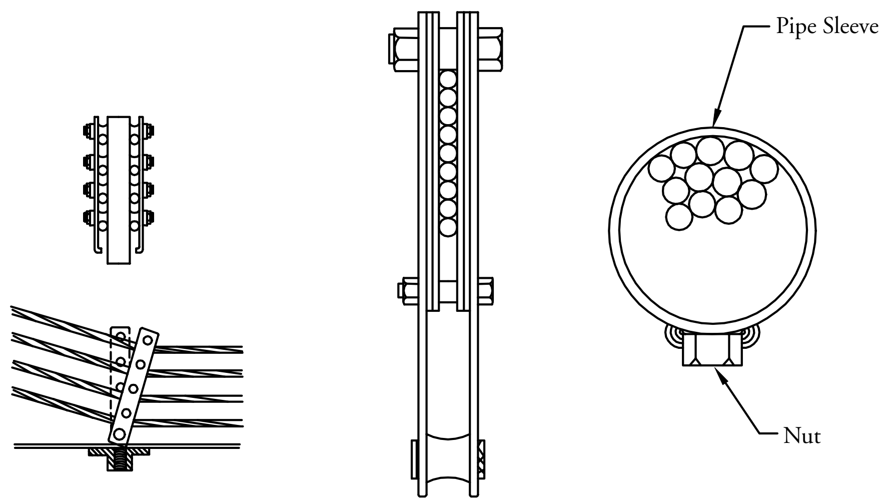
3.3.2.2.2
Harped Strands

Harped strands are most commonly used in the webs of relatively deep members, such as I-beams, bulb-tees, stemmed sections and deep box beams. The resulting reduction of the eccentricity of the prestressing force at the member ends reduces or eliminates the need for debonding. Harped strands can be deflected in one or more locations along the length of the member. As **Figure 3.3.2.2-1** shows, a single midspan harp provides better correlation to the moment envelope than straight strands, but not as good as multiple harp locations in a concrete member. For negative moments created by cantilevers, the strands can be lifted at the support and held along the top of the member for the length of the cantilever. For safety reasons, the slope of deflected strands should not exceed about 9 deg.

3.3.2.2.3
Harping Devices

Devices used to deflect pretensioned strands at the point of the harp vary from plant-to-plant. **Figure 3.3.2.2.3-1** illustrates common types. Some devices maintain the same strand spacing throughout the length of the member, others bundle the strands into one or more vertical rows, and still others bundle the strands at one or more discrete locations. Maintaining constant strand spacing throughout the member is normally not necessary unless the member is unusually short. In fact, it limits the maximum strand eccentricity which could otherwise be achieved. Bundling strands at discrete locations provides optimum eccentricity, though the strands must be splayed between the harp point and the member ends to assure bond development of each individual strand. Different harping devices are used with different products and with different harping methods.

Figure 3.3.2.2.3-1a-1c
Harping Devices



a) Two Columns with Uniformly Spaced Rows

b) Single-Stacked Column

c) Strand Bundle

3.3.2.2.4
Anchorage of Harping Devices

Some forms are designed to accommodate deflected strands, and therefore provide for the attachment of harping devices to the form. This is common with “one-piece” forms without removable sides, such as used to cast stemmed members. With this type, the endplates and bulkheads are designed to hold the strands in position at the ends of products (**Fig. 3.3.2.2.3-1b**). The strands are first stressed straight, then are subsequently pushed down from the top of the form at midspan with “fork” type harping devices using greased, tapered steel pins. The harping hardware can normally be anchored anywhere along the length of the form. After the concrete has cured, the tapered pins are pulled from the top of the members, and the holes are filled. Strands can also be pulled down and anchored from beneath some forms.

For other types of bridge members, such as I-beams, bulb-tees and box beams, most plants anchor harping devices to the concrete floor of the prestressing bed with embedments at a constant spacing. Normally, the member design is not especially sensitive to the harp location, and generous tolerances on the longitudinal location of the harp point (on the order of 18 - 20 in.) should be specified to allow the use of established anchor locations. At the ends of the member, the harped strands are usually not held in position by the endplates, but rather by steel "horses" and blocks which are beyond the endplates of the form. When the harped strands are held at a location away from the endplate, it is difficult to maintain small tolerances on their vertical position at the point they enter the member. Therefore, if the member design is not sensitive to vertical location of the harped strands at the member ends, generous tolerances (on the order of ± 1 in.) should be specified.

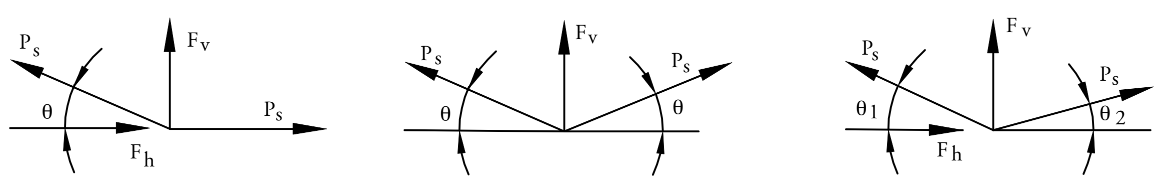
The vertical and horizontal forces developed by the harping operation are shown in **Figure 3.3.2.2.4-1**. Both must be considered when selecting the type and number of harping devices. Local producers should be consulted for harping capabilities on specific products. Vertical harp forces which exceed the capacity of the harping devices can usually be split into two or more locations which straddle the intended location. Horizontal forces occur when the angle of the harped strands differ on each side of the harping device and can be a problem for the "push-down" harping method described above. The tapered pins used in this procedure are relatively long and slender, and are normally not designed for combined flexure and axial loads. For this reason, unless the tapered pins are designed for combined flexure and axial loads, stemmed members manufactured with this method should be designed with straight strands, or a single harp at midspan only. Devices holding harped strands from beneath the form are generally not subject to this limitation.

3.3.2.3 Tensioning

Procedures used to tension prestressing steel vary widely, but all share the objective of imparting the intended amount of precompression to the concrete at a given location. The following sections describe the procedures and controls used in the tensioning operation, as well as corrections for the influence of external variables. Precast plants compensate for the effects of external influences in each casting line, and should be consulted for specific information. Though the discussion below chiefly addresses pretensioning with strand, many aspects are also applicable to post-tensioning with strand or bars.

3.3.2.4 Pretensioning Configuration

A typical pretensioning set-up is shown in **Figure 3.3.2.4-1**. The end of the bed from which the strands are tensioned or jacked is referred to as the "live" end, while the



$F_v = P_s(\sin \theta)$
$F_h = P_s - P_s(\cos \theta)$

a) Each Point of a Double Harp

$F_v = 2P_s(\sin \theta)$
$F_h = 0$

b) Single-Point Harp

$F_v = P_s(\sin \theta_1) + P_s(\sin \theta_2)$
$F_h = P_s(\cos \theta_2) - P_s(\cos \theta_1)$

c) Harp with Asymmetrical Strand Trajectories

Figure 3.3.2.2.4-1a-1c Calculating Harping Forces opposite end is called the "dead" end. In most cases, the precast member is shorter than the prestressing bed. In order to reduce the amount of strand that is cut off and wasted daily, the member is positioned in the line as close as possible to the dead end. This also reduces the amount of stressed "free" strand which must be dealt with during detensioning (as discussed in Sect. 3.3.2.8). This minimum dimension is normally dictated by the need to deflect the strands from standard holes in the abutments into the endplate at the end of the precast member, while maintaining a shallow slope on the strands (see Sect. 3.3.2.2.2). Positioning the member in this manner normally leaves free strand at the live end. In order to reduce strand waste, most producers use "lead" or "bridle" strands at the live end, which are spliced onto the production strands, and then reused each day of casting.

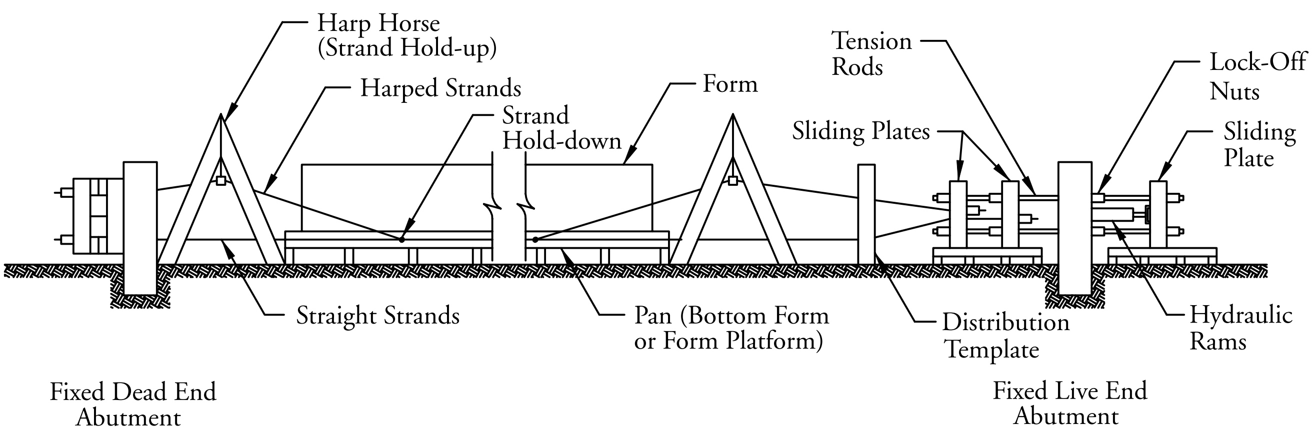


Figure 3.3.2.4-1 Typical Prestensioning Bed Profile Showing Strand Tensioning and Deviation Devices

3.3.2.5
Tensioning Prestressing Steel

Prestressing steel is tensioned to the intended force with hydraulic pumps and rams. The tensioning system is calibrated to correlate the force delivered to the prestressing steel with a gage pressure read by the operator. The single most important control over this operation is a check of the calculated value of steel elongation compared to the actual elongation measured during the tensioning process. These values must agree (within 5 percent for pretensioning, 7 percent for post-tensioning) to validate the procedure. Many variables enter into elongation calculations, all of which must be considered to properly compare the results. Strands may be tensioned individually, or as a group. In pretensioning, procedures differ for these two methods.

3.3.2.5.1
Tensioning Individual Strands

A strand tensioned individually is first jacked to an initial force, or "index" load, somewhere between 5 - 25 percent of its final tensioning force. The reason for this is to take up any slack in the system, which can hamper the reliability of elongation measurements. An initial measurement of the ram extension is taken, and the strand is then tensioned to 100 percent of its intended force. The ram extension is again measured, and the difference between the measured extensions should reasonably match the calculated incremental elongation. This same procedure is also used for post-tensioning single strands or bars. Post-tensioned bars are normally tensioned individually.

3.3.2.5.2
Tensioning Strands as a Group

For strands tensioned as a group (this is sometimes called "gang stressing" or "multiple strand stressing"), the pretensioning procedure is slightly different than that for strands tensioned individually. Prior to jacking the strands to their initial load, each individual strand is "preloaded" to an initial force smaller than the index load (usually about 2000 lbs). This is done to assure that all strands begin the tensioning process with the same amount of force. The tensioning then proceeds in the same manner as for individual strands. Preloading is normally not required in stressing multiple-strand post-tensioning systems, since the strands are confined within a duct, and have about the same amount of slack.

3.3.2.6 Prestressing Strand Elongation

The basic equation for the elongation of prestressing steel is:

$$D = \frac{P_s L}{A_s E} \qquad \text{(Eq. 3.3.2.6-1)}$$

Variations in the steel area and modulus of elasticity are common, though usually quite small. The areas of prestressing strands shown in **Table 2.11.1** are reliable average values, as are the areas for prestressing bars. Average values for the modulus of elasticity are 28,600 ksi for prestressing strand, and 29,000 ksi for prestressing bars. The use of average values for area and modulus of elasticity are normally satisfactory for elongation calculations. If a higher-than-normal degree of precision is necessary, mill certificates available from the steel manufacturers provide the actual area and modulus of elasticity for each heat of material.

Equation 3.3.2.6-1 is based on several idealized assumptions. The prestressing steel:

- has a uniform cross-section of constant area and modulus of elasticity
- is held by infinitely stiff supports at each end
- is maintained at a constant temperature

In reality, strand often has factory splices within its length; prestressing bed components deform to varying degrees under load; and, some movement or "seating" occurs at the anchorage devices. Steel temperatures are rarely constant, particularly when the temperature of the fresh concrete differs substantially from the ambient temperature. Consequently, the basic equation must be modified to account for these unavoidable factors.

3.3.2.7 Variables Affecting Strand Elongation

External variables fall into two categories; those requiring adjustments to the jacking force, and those that merely result in additional elongation. Since the operator is jacking to a pre-determined gage pressure, irrespective of the ram extension, the definitive point in time separating the two categories is when the jacks reach 100% of their intended load, just prior to seating the live end chucks. For multiple-strand jacking, all external influences occurring before live end seating show up as additional elongation. Live end seating, and the effects of any subsequent external influences, are corrected by adjustments to the jacking force, subject to the limitations on the maximum stress in the strand given in AASHTO *Standard Specifications*. PCI's *Quality Control Technician/Inspector Level I & II Training Manual* and PCI manual, MNL-116 provide further discussion on influences external to the prestressing process, as well as examples of elongation calculations.

3.3.2.7.1 Dead End and Splice Chuck Seating

As the strands are tensioned, they move through the chucks as the wedges seat. The additional elongation from this source is the incremental movement that occurs between the index load and final load. This is normally small (on the order of 1/8 in. per chuck), since most of the movement occurs while jacking to the index load. Dead end and splice chuck seating are independent of whether the strands are jacked individually, or as a group. However, where bridles are used with multiple-strand tensioning, the same number of splice chucks should be used on each strand in the system to assure uniform elongation values.

3.3.2.7.2 Elongation of Abutment Anchor Rods

Some multiple-strand stressing systems employ steel blocks and anchor rods for jacking purposes. **Figure 3.3.2.4-1** illustrates this system. The stretching of these rods may add to the elongation of the system, and the apparent elongation of the strands, depending on where the measurements are taken.

3.3.2.7.3 Prestressing Bed Deformations

Prestressing beds are heavily loaded, and will shorten axially during jacking. For abutment-type beds, this is usually minimal. For strutted and self-stressing beds, the relatively small area of the compression members can result in significant shortening. Uprights and cross-heads will also deflect under load. For multiple-strand tensioning, these deformations are of no consequence, other than to add elongation to the system. However, prestressing bed deformations will influence the final load on strands stressed individually. Theoretically, the bed shortens incrementally as each strand is jacked. Strands tensioned early in the sequence will lose force as subsequent strands are tensioned. This is normally corrected by over-tensioning strands jacked early in the sequence. Depending on the number of strands, one or more groups can be over-tensioned to average values. If the earliest strands cannot be jacked high enough to compensate for the total bed shortening (due to specification limits), then re-tensioning is required. Post-tensioning is comparable to pretensioning with self-stressing forms, with the obvious difference being that the "form" is the concrete member itself.

3.3.2.7.4 Live End Chuck Seating

When strand is pretensioned individually, it is normally pulled through the live end chuck by a center-hole ram which bears directly on the chuck. The direction of the pull is opposite that which seats the wedges. Therefore, when the strand is released, it will move through the chuck significantly more than dead end or splice chucks (on the order of 3/8 in.). This is corrected by increasing the jacking load to compensate for the expected seating value.

Multiple-strand pretensioning systems normally are locked-off with abutment anchor rods as described in Section 3.3.2.7.2. Therefore, the live end chucks seat much the same as dead end or splice chucks, with the result being a small net gain in elongation. With most systems, seating of abutment anchor rods is relatively small (about 1/8 in.). Some multiple-strand post-tensioning rams have secondary pistons which seat the live end wedges prior to releasing the load, thereby reducing the amount of seating loss at the live end.

3.3.2.7.5 Temperature Corrections

Strands tensioned at cold temperatures, then exposed to relatively warm concrete (or grout for post-tensioning), will relax and lose some of the force imparted during jacking. The opposite is true of warm temperatures and cool concrete. **Table 3.3.2.7.5-1**

Table 3.3.2.7.5-1 Percentage of Strand Stress Change due to Temperature Differentials

Temperature Variation (Degrees Fahrenheit)

% of Bed In Use	5	10	15	20	25	30	35	40	45	50
5	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2
10	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.4	0.4	0.5
15	0.1	0.1	0.2	0.3	0.4	0.4	0.5	0.6	0.7	0.7
20	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
25	0.1	0.2	0.4	0.5	0.6	0.7	0.9	1.0	1.1	1.2
30	0.1	0.3	0.4	0.6	0.7	0.9	1.0	1.2	1.3	1.5
35	0.2	0.3	0.5	0.7	0.9	1.0	1.2	1.4	1.5	1.7
40	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
45	0.2	0.4	0.7	0.9	1.1	1.3	1.5	1.8	2.0	2.2
50	0.2	0.5	0.7	1.0	1.2	1.5	1.7	2.0	2.2	2.5
55	0.3	0.5	0.8	1.1	1.4	1.6	1.9	2.2	2.4	2.7
60	0.3	0.6	0.9	1.2	1.5	1.8	2.1	2.4	2.7	3.0
65	0.3	0.7	1.0	1.3	1.6	1.9	2.2	2.6	2.9	3.2
70	0.3	0.7	1.0	1.4	1.7	2.1	2.4	2.8	3.1	3.4
75	0.4	0.7	1.1	1.5	1.8	2.2	2.6	3.0	3.3	3.7
80	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.1	3.5	3.9
85	0.4	0.8	1.3	1.7	2.1	2.5	2.9	3.3	3.8	4.2
90	0.4	0.9	1.3	1.8	2.2	2.7	3.1	3.5	4.0	4.4
95	0.5	0.9	1.4	1.9	2.3	2.8	3.3	3.7	4.2	4.7
100	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0

shows the percentage of prestress change as a function of the temperature differential and the percentage of the bed used. Again, corrections for anticipated temperature differentials can be made by adjusting the jacking force. This correction is not applicable to self-stressing forms because the form changes length with temperature change, countering changes in strand force.

3.3.2.7.6 Friction

Friction is another external variable that must be addressed in the prestressing operation. In pretensioning, friction is normally not a problem with straight strands, but can significantly reduce the force in the strands at the dead end if the strands are deflected at several points along the bed. If this is a problem with a particular bed setup, it will be indicated by a reduction in the measured elongations.

Rather than compensating for friction in the jacking load or elongation calculations, most plants have developed tensioning or harping procedures that diminish the effects of friction. For example, some proprietary harping devices feature rollers to decrease friction losses when the strands are tensioned in the draped position (**Fig. 3.3.2.2.3-1a**). These devices are usually expensive and limit the eccentricity that can otherwise be achieved with harped strands. Some plants tension the strands in a straight, or partially deflected profile, then complete harping after the strands are jacked. The resulting change in geometry will increase the force in the strand. With multiple-strand pretensioning systems, the strands can either be under-tensioned to compensate for the expected increase in force, or the rams can be relaxed concurrent with the harping operation to maintain the same force level in the strands. Strands that are jacked individually can be tensioned to lower forces to compensate for the added force due to the change in geometry.

Friction in post-tensioning is unavoidable, and is therefore inherent in elongation calculations. In curved tendons, the strands are in contact with the duct for most of their length, and consequently develop a significant amount of friction. The PTI *Post-Tensioning Manual* provides guidance and example calculations for the amount of friction that can be expected when post-tensioning tendons.

3.3.2.8 Detensioning

Once the concrete has achieved its specified release strength (as determined by cylinder tests or other non-destructive testing methods), the force (tension) in the strands is released from the prestressing bed into the product. This is commonly referred to as detensioning. If the concrete cure has been accelerated by heat, the product should still be warm and moist at the time of detensioning. Otherwise, the unstressed concrete will cool and contract, sometimes resulting in cracking along the length of the member.

Strands can be released hydraulically, by heat cutting, or a combination of both. Hydraulic release is normally used with multiple-strand tensioning systems, while heat cutting is used with both multiple- and single-strand systems. The sequence of detensioning is very important for safety reasons, as well as for avoiding damage to the product. Strands should always be detensioned symmetrically.

3.3.2.8.1 Hydraulic Detensioning

With hydraulic release, the strands are usually relaxed from the live end with the same tensioning system used to jack them. Because the strand is bonded with the concrete, the free strand at the dead end will tend to pull the product toward the dead end as the live end is released. The smaller the amount of free strand, the less the tendency to slide. If the member slides, it can bind in the form. Items projecting through the formwork, such as harping device hold-downs, can damage both the product and the formwork. Sliding can be prevented in two ways:

1) Let the live end down in increments, while heat cutting the appropriate number of strands at the dead end. For example, in a line with 8 strands, the live end can be released in 25 percent increments, with 2 strands being cut at the dead end after each increment.

2) Use short stroke "let-down" rams at the dead end. These rams are released proportionally to the live end rams, allowing the force in the strands to remain the same at either end of the member.

3.3.2.8.2
Detensioning by Heat Cutting

When heat cutting is used without hydraulic release, individual strands must be cut simultaneously at both ends of the member. When strands are indiscriminately cut at one end only, the force in each remaining strand will increase, possibly to the breaking point. The prestressing forces must be kept as equal as possible at each end throughout the entire procedure. For safety reasons, heat cutting should only be applied to relatively short lengths of tensioned free strand, and then only after applying initial detempering heat to allow the strands to yield and relax prior to cutting. This process not only results in improved safety, but also reduces the shock to the precast member.

3.3.2.8.3
Detensioning at Bulkheads

When several members are cast end-to-end in a line, it may be necessary to cut the strands between members simultaneously with the ends, depending on the type of bulkheads used. Bulkheads can be designed to resist the compressive forces developed between members as the ends of the line are released. In this case, the strands between members can be cut after the line has been detensioned. However, "soft" bulkheads, such as those made from wood, will crush and allow the precast members to slide if the strands between members are not cut simultaneously with the ends.

3.3.2.8.4
Harped Strand Considerations at Detensioning

The vertical forces developed by harped strands can cause cracking in the tops of members if the harp hold-downs are released prior to detensioning. These forces can sometimes exceed the weight of the member, and cause the member to lift from the bed. In these cases, enough prestress must be delivered prior to releasing the hold-down devices to reduce the uplift. This partial detensioning must be done symmetrically at both ends of the bed to prevent overstressing the remaining strands.

3.3.2.9
Strand Debonding

In pretensioned members, strands can be debonded for all or part of the member length for three reasons:

- to reduce excessive concrete stresses at the member ends
- to allow the casting of members having differing numbers of strand in the same bed
- to prevent concrete bonding to strands intended to serve for temporary handling and shipping purposes

Various methods are used for debonding, including encapsulating the strand in sheaths (also referred to as "blanketing" or "sleeving"), or applying a bond breaker to the surface of the strand. The effectiveness of these methods vary.

The bond of concrete to tensioned strand develops from several mechanisms as described by Gerwick (1993). These include:

- chemical adhesion
- shrinkage of the concrete surrounding the strand
- mechanical interlock on the deformations between the intertwined wires

• the swelling of the strand after release due to Poisson's ratio, commonly referred to as the "Hoyer" effect.

Bond breakers on strand generally serve to reduce only the chemical adhesion, and therefore, are not as effective as strand encapsulation.

Strands are encapsulated with different materials, some more effective than others. The key properties of encapsulating materials are watertightness, strength and durability to withstand concrete placement, and nonreactivity with concrete or steel. The material must provide enough space between the concrete and strand to mitigate the effects of concrete shrinkage and strand swelling. The sheaths must be properly sealed to avoid intrusion of cement paste during concrete placement.

3.3.3 Nonprestressed Reinforcement and Embedments

In precast concrete fabrication, the placement of mild reinforcement and embedments is generally simpler than in cast-in-place construction, which further improves the quality of plant-cast products. This section describes methods used by precast concrete manufacturers to secure embedments, and provides detailing hints that take full advantage of plant-cast products.

3.3.3.1 Placement and Securing

Precast products are normally cast in an orientation providing the easiest access for placement of embedded items. Although most methods of securing embedments do not differ between precast and cast-in-place construction, the ease of access is critical to the quality of the finished product. For example, vertical members, such as piles, piers and abutment walls, are cast and shipped horizontally by precast plants, and are only tipped to vertical for erection purposes. In members that are not pretensioned, mild steel reinforcement cages are typically set into forms, rather than the forms being placed around them, facilitating the inspection of concrete cover and embedment locations. Placing, tying and inspecting mild reinforcing bars is much more efficient when the member is cast horizontally at ground level, rather than high in the air or below grade.

Tensioned prestressing strands provide an excellent platform for supporting mild reinforcement. Whenever possible, mild reinforcement transverse to the member should be detailed to be tied directly to the strands. This provides excellent control of the bar location, and minimizes the need for "chairs" or "bolsters." Chairs can be used to support the strands if they sag under the weight of the bars. Whenever possible, mild steel reinforcement should be detailed for installation after pretensioned strands are stressed. Sections 3.2.3.1 and 3.2.3.4 provide suggestions on efficient reinforcement configurations.

3.3.3.2 Installation of Lifting Devices

The installation of lifting devices is critical to the safe handling of precast concrete products. Improperly designed or installed devices can fail, with potentially catastrophic consequences. Section 3.2.4.4 describes common configurations. Proprietary devices should be installed in accordance with the manufacturer's recommendations. Generic devices must be properly designed and installed. For any type of lifting device, a very important consideration is proper consolidation of concrete around the device and its anchors.

The most common type of generic lifting device for large bridge products is pretensioning strand lift loops. In many cases, multiple loops are required at each location. When using multiple loops, each must be held at the same height above the concrete surface, and must be engaged by the straight pin of a shackle. It is very important that each strand in the group carry its proportionate share of the load. Curved engagement surfaces, such as a hook or the curved end of a shackle, will load the loops unevenly, potentially creating a progressive failure of individual loops. The legs of each loop should be splayed to allow concrete to envelop them individually. Bundled loop legs can exhibit significantly reduced capacity.

3.3.3.3 Concrete Cover

The amount of concrete cover over reinforcement is important in providing protection of the steel from corrosion. Cover must be sufficient to allow the largest aggregate particles to pass between the reinforcement and the form. Due to superior control of form dimensions, reinforcement placement, concrete quality, curing and inspection procedures, ACI 318 allows the concrete cover requirements for precast products to be reduced when compared to cast-in-place construction. Concrete cover in precast fabrication is normally assured by the use of “chairs” or “bolsters,” by the rigidity of pretensioned strands, or by a combination of both.

3.3.3.4 Steel Spacing Design

The spacing of prestressing steel and mild reinforcement must be sufficient to allow the largest aggregate particles to pass freely between strands or bars. Section 3.2.2.3 discusses the minimum spacing of prestressing strand. For precast concrete, the AASHTO *Standard Specification* requires the minimum clear distance between parallel mild reinforcing bars in a layer to be not less than one bar diameter, 1.33 times the maximum aggregate size, or 1 in. These restrictions are intended to allow concrete to fully envelope the reinforcement. At closer spacings, the reinforcement can act much like a sieve, segregating the larger coarse aggregate particles from the cement paste and smaller aggregate.

Most precast plants use coarse aggregate gradations with a maximum particle size of 3/4 in. For members with reinforcing bar sizes of #8 or less, this means a minimum clear distance between bars of 1 in. At this spacing, it is not practical to effectively use even the smallest of internal vibrators (normally called “stingers,” the smallest of which are 1 in. in diameter) to consolidate the concrete, making external vibration the only reliable method available for consolidation. Limiting possible consolidation methods can increase production costs, particularly if the standard form sides are not stiff enough to withstand external vibration, or have not been previously equipped with external vibrators. When possible, reinforcement spacing should be maximized to allow concrete to be consolidated with either internal or external vibration, to reduce cost and improve quality of the finished product.

Other embedments can also create congestion. Post-tensioning ducts in thin girder webs can obstruct a substantial percentage of the web, making internal vibration difficult for the concrete below the level of the duct. Forcing internal stingers past ducts can dent and possibly puncture the ducts, creating blockages that are difficult to clear. In this case, the webs should be detailed thick enough for easy passage of the stinger, or external vibration should be used to consolidate the concrete below the level of the ducts. Bridge designers should consult local producers for advice on embedment configurations and clearances.

3.3.4 Concrete Batching, Mixing, Delivery and Placement

Procedures used to batch and mix concrete for precast concrete bridge products do not differ substantially from those used in cast-in-place concrete construction. Concrete is normally batched and mixed in a central stationary mixer, though shrink mixing (partial central and partial truck mixing) can also be used. However, in plants that mix their own concrete, the proximity of the mixer and final destination allows a wider range of delivery and placement systems.

The general requirements for equipment and procedures used for batching, mixing, delivering and placing concrete are covered in detail in PCI manual, MNL-116. The required result of all processes, from mixer to final placement, is to provide concrete of a uniform, compacted consistency without segregation of aggregates and paste.

3.3.4.1 Delivery Systems

A wide range of systems are used by precast plants to deliver concrete from the mixer to the forms, including pumps, conveyors, "sidewinders," truck mixers, and short-haul buggies carrying buckets or hoppers. Typically, delivery system capacities are designed to ensure a continuous supply of freshly mixed concrete for the duration of the pour. By necessity, precast concrete products are limited in weight, and do not require placement of large volumes of concrete. Concrete can be batched, mixed, delivered and placed by the plant in relatively small quantities, resulting in improved control of the concrete consistency.

3.3.4.2 Consolidation Techniques

As discussed in Section 3.3.1.2.1, forms for standard precast concrete bridge products are normally of steel construction, and are usually much stiffer than the typical forms used in cast-in-place construction. Part of the reason for this is to allow the use of external form vibration. Areas of the cross-section that are difficult to reach with internal vibrators, such as the bottom flange of deep I-girders, are easily consolidated with external vibration. Concrete consolidated with properly executed external vibration is extremely dense and durable. In many cases, combinations of internal and external vibration are used to enhance the consolidation of the concrete.

3.3.4.3 Normal Weight Concrete

The term "normal weight" concrete is conventionally used to describe mixtures containing naturally occurring igneous, sedimentary, or metamorphic mineral aggregates. Such aggregates are predominantly siliceous or calcareous in composition, with a specific gravity between 2.25 and 2.65. The resulting concrete unit weights are normally between 145 and 160 pcf, with 28-day compressive strengths ranging in excess of 10,000 psi in some parts of the country.

The use of normal weight concrete is predominant in the production of precast concrete bridge products. For most types of bridge members, normal weight concrete provides the best performance for the lowest cost. Efficient, state-of-the-art precast bridge products generally require relatively high concrete strengths in slender sections which are congested with reinforcement. The resulting need for low water-cement ratios and high workability has led to the widespread use of water-reducing admixtures. As discussed in Section 3.2.1.3.1, water-reducing admixtures can reduce the workable life of concrete. However, since the interval between mixing and placing is short when precast plants mix and deliver their own concrete, optimum workability is usually maintained throughout the duration of the pour.

3.3.4.4 Lightweight Concrete

Lightweight and semi-lightweight concretes can be produced with unit weights ranging from approximately 100 pcf up to the unit weight of normal weight concrete. This is done by replacing varying quantities of normal weight aggregate with lightweight aggregate. For example, replacement of normal weight coarse aggregates with some of the ESCS (expanded shale, clay and slate) lightweight coarse aggregates can result in structural concretes with unit weights as low as 110 pcf. Further reduction of the concrete unit weight is achieved by also replacing the normal weight sand with lightweight sand. Additional information about structural lightweight concrete is found in ACI 213R. (Also, see Sects. 2.4.7.2 and 2.4.7.3.)

Members made with lightweight concrete are easier to handle and ship, and reduce the superstructure weight, with resulting economies in substructure and seismic design. However, depending on the type of aggregate, lightweight concrete can exhibit lower compressive strength though always has a lower modulus of elasticity than comparable normal weight concrete, resulting in increased deformations (camber, deflections, elastic shortening). Creep deformation is independent of modulus of elasticity and, with some lightweight aggregates, can be less than comparable normal weight concrete. The ultimate shrinkage of lightweight concrete is also generally higher than normal weight concrete. Creep, shrinkage and splitting tensile strength (which affects shear strength) values should be provided by the aggregate supplier.

Procedures for batching, mixing, transporting and placing lightweight concrete are essentially the same as for normal weight concrete, although special handling of the lightweight aggregate concrete must be considered. Lightweight aggregate suppliers should be consulted for recommendations. For the “softer” lightweight aggregates, overmixing should be avoided to prevent grinding of the aggregate. When using a lightweight mixture for the first time, verification should be provided that standard handling and placing techniques will result in concrete of uniform consistency.

3.3.4.5 High Performance Concrete

High performance concrete is a mixture exhibiting one or more specific properties in its hardened form, such as high strength, low permeability, low shrinkage or abrasion resistance. Some of these properties occur naturally when striving to achieve others. For example, the density needed for concrete to exhibit low permeability normally also results in high strength. There are those who believe that to achieve high performance concrete, mineral admixtures must be added to conventional, normal weight concrete. This is not necessarily the case. As reported by Pfeifer, et al (1996), the low water-cementitious materials ratio and accelerated curing required to achieve overnight release strength results in concrete of comparable durability to moist-cured concrete with silica fume. In some parts of the country, materials and fabrication procedures are of such high quality that precast plants routinely produce high performance concrete with standard normal weight mixes.

Batching and mixing procedures for high performance concretes containing chemical or mineral admixtures are essentially the same as for standard concretes, with the exception of the addition of the admixture. Admixtures should be charged into the mixer in accordance with the manufacturer’s recommendations. Water-reducing admixtures usually provide better performance if added after the cement and water have reacted for several minutes. Depending on the type of high performance concrete, some delivery systems are better than others. For example, concretes with relatively high dosages of silica fume tend to be overly cohesive, and are difficult to pump. Section 3.2.1.3 discusses the effects that different types of admixtures have on concrete placement and consolidation. Sophisticated techniques, such as external form vibration, are generally required to successfully place high performance concrete in typical precast concrete bridge members.

3.3.5 Concrete Curing

The economic viability of precast concrete depends on the ability of the plant to fabricate precast products on a daily basis. In certain applications, forms are used—“turned over”—twice each day. For some precast products, the required concrete strength at stripping may be low enough to allow normal curing practices for the relatively short duration between pouring and stripping. However, most pretensioned products require relatively high concrete strengths at the release of prestress, which cannot be achieved without accelerating the strength gain of the concrete. This section describes methods used by precast plants to accelerate concrete curing, and the beneficial effects these techniques have on the properties of the concrete. Section 3.3.5.5 addresses both the quality control aspects and optimization of accelerated curing.

Apart from the use of Type III cement and accelerating admixtures, the primary method of accelerating the strength gain of concrete is with the application of heat. This process, along with prevention of moisture loss from the hardening concrete, is called accelerated curing.

3.3.5.1 Benefits of Accelerated Curing

In a typical precast plant, stripping of the prior day's casting and set-up of a new member are normally scheduled to be accomplished in a standard 8-hour shift. Assuming concrete placement occurs within the subsequent 4 hours, that leaves 12-16 hours to cure the concrete and achieve the required stripping or release strength prior to the start of the next cycle. For most bridge products, accelerated curing is the only way to achieve these strengths in the available curing period. Optimum application of modern cements, admixtures, and accelerated curing systems can result in concrete strengths at release of prestress of 6,500 psi or higher, facilitating a wide variety of sophisticated, long span products. The ability to achieve high levels of overnight strength is not uniform throughout the country, nor is it consistent from plant to plant. Bridge designers should consult with local producers. For economy, the specified release and stripping strengths should always be the minimum required by design, subject to the AASHTO *Standard Specifications* minimum values of 4000 psi for pretensioned members (except piles), or 3500 psi for post-tensioned members and pretensioned piles.

Accelerated curing is also beneficial to concrete durability. Studies by Klieger (1960) and Pfeifer, et al (1987, 1996) have shown that accelerating the early strength development of concrete by heat curing improves freeze-thaw durability and chloride permeability, as well as decreasing absorption and the volume of permeable voids within the concrete. This is particularly important in areas where de-icing chemicals are common, and in coastal areas.

3.3.5.2 Preventing Moisture Loss

Moisture loss from exposed surfaces must be prevented during the entire curing cycle. Several methods are used to achieve this:

- covering the exposed surface with wet burlap
- covering with a polyethylene sheet vapor barrier
- covering with impermeable curing blankets
- applying a curing compound

Moisture loss varies with the geographic location of the plant, the ambient conditions, and whether the bed is inside or out. In cool climates with relatively high humidity, covering the product with an impervious sheet during the curing cycle is generally all that is necessary. In hot climates with low humidity, additional means of moisture retention, such as wet burlap or other absorbent material, may be necessary. Failure to take precautions can allow rapid evaporation of mix water from the concrete, resulting in plastic shrinkage cracking and, in severe cases, a loss of strength development in the affected area. Bridge designers should consult with local manufacturers for applicable moisture loss prevention techniques.

3.3.5.3 Methods of Accelerated Curing

Accelerated curing begins only after the concrete takes its initial set, which is generally 3-5 hours after batching. Once heat is applied, the temperature of the concrete may increase at a rate of up to 80F per hour to a maximum concrete temperature of 190F, where it may be held for the remainder of the curing period. Section 3.3.5.5 provides further discussion on optimizing the accelerated curing cycle.

When heating the air surrounding the forms, uniform concrete curing temperatures are sometimes difficult to control in members of variable or complex shape. Differential expansion between portions of a member with varying volume-to-surface ratios can create thermal stresses at the interface, possibly causing cracks. Thermostatic control is also difficult in some cases, since many heaters are not adjustable (they are either on or off), and the temperature of the air in the enclosure will not be the same as the temperature of the concrete. For these reasons, it is important to monitor the internal temperature of the concrete and not the temperature of the enclosure.

All accelerated curing methods perform substantially better when used with metal forms as compared to wooden forms. Wooden forms have inherent insulating properties that restrict heat from reaching the concrete. Steel forms and concrete have similar thermal expansion properties, but the coefficient of thermal expansion for wood is only about half of that for concrete, resulting in increased wear on wooden forms during repeated heating cycles. Care must be taken when placing heaters around wooden forms, which have been known to catch fire during the curing process.

With accelerated curing, the heat of hydration of the cementitious materials in the concrete must be considered when determining the amount of heat to apply to the member. Massive members with large volume-to-surface ratios generate large amounts of heat during the hydration process. High performance concrete combining portland cement with mineral admixtures also exhibits increased heat of hydration. Internal concrete temperatures of the first members cast under these circumstances should be closely monitored to assure they remain below the maximum allowable temperature.

3.3.5.3.1 Accelerated Curing by Convection

The most common method of accelerated curing is provided by elevating the temperature of the air surrounding the form. A typical convection process involves "tenting" the form with a frame and a polyethylene sheet or insulated tarp, and placing gas-fired forced air heaters under the tent. **Figure 3.3.5.3.1-1** shows an insulated tarpaulin on a track-mounted reel. Depending upon the member size, heaters from 50,000 - 500,000 Btu/hr are common. A rule of thumb is that one cubic yard of concrete will require about 2,000,000 - 4,000,000 Btu-hrs to raise the concrete temperature approximately 100F in a 12-hour curing period.

Figure 3.3.5.3.1-1 Track-Mounted Insulated Tarpaulin Reel

3.3.5.3.2 Accelerated Curing with Radiant Heat

Heat can also be provided by electric or gas radiant heaters. Producers have successfully cured small products by using common hardware-store-variety heaters under the enclosure. For larger products, such as stemmed members or box beams, finned tubes installed under the form can be used to circulate hot water or hot oil from a localized or central boiler. In the case of hot water, a typical 2-in. dia. finned tube operating at 200F will produce about 1,200 Btu/ft/hr. Elaborate piping schemes are employed in an attempt to maintain uniform heat throughout the concrete cross section. Anti-freeze is added to the water to prevent the line from freezing when the system is shut down.

3.3.5.3.3 Accelerated Curing with Steam

Another method used to provide accelerated curing is steam. Steam may be provided in a closed circulating system or as "live" steam which is allowed to enter the enclosure, or a combination of both. Live steam exhibits the same thermal characteristics as convection or radiant heat, with the added benefit of providing a water-saturated environment. This largely eliminates the potential for moisture loss from exposed concrete surfaces. The ability to pipe the steam to the most advantageous locations, and to control the boiler temperature and flow, allows good uniformity in the curing process.

Two significant disadvantages of live steam curing are the high cost of energy required to generate the steam, and the deleterious effect steam and condensed hot water have on the plant, tooling and forms. Even the best curing covers and energy-efficient boilers result in relatively high curing costs. In addition, collection of the condensate runoff is costly and messy, and steam causes metal tooling and forms to corrode at an accelerated rate. This can be particularly detrimental to strand anchorage devices and harping hardware.

3.3.5.3.4 Accelerated Curing with Electric Heating Elements

The heat for accelerated curing can also be provided by electrical heating elements attached to the skin of metal forms. These elements are firmly secured to the form skin, which is then covered with 2 - 3 in. of sprayed-on foam insulation. The heat is provided by conduction through the insulated form skin to the concrete.

Several advantages exist with this technology. First, the time-temperature curve can be programmed to deliver the optimum curing cycle. This can also be done with other curing systems, but with less direct control of the concrete temperature. Further discussion of the optimum curing cycle can be found in Section 3.3.5.5.

The second advantage is that electric curing is energy efficient. The forms are heated directly, rather than energy being wasted by heating the surrounding environment. Exposed areas of concrete are covered with impervious curing blankets that are relatively light and easily removed. Typical installations demonstrate energy consumption significantly less than with other systems. Though there are initial costs associated with the elements, power distribution and computer controls, the long-term energy savings and superior curing control can provide a rapid pay-back when compared to other systems.

A third advantage of electric curing is that by planning the spacing and control of the electric heating elements, different parts of a member can be cured with varying energy outputs. Thin flanges can be treated differently than bulky webs, resulting in a more uniform cure of the entire cross section, with less potential for cracking due to thermal stresses. This degree of control is not available with any other curing system. Besides the relatively high initial cost, one disadvantage of electric curing is that it can only be used with metal forms.

3.3.5.4 Curing Following Stripping

Specifications sometimes require an additional period of moist curing following the accelerated curing cycle. Studies by Klieger (1960) have shown that this additional moist curing period is not necessary, and may in fact be detrimental to the freeze-thaw durability of the concrete. Accelerated curing by the application of heat is equivalent or superior to the moist cure period specified for cast-in-place concrete. Further hydration of the cement under moist conditions is not necessary after the accelerated cycle.

Specifications sometimes require controlled cooling rates for heat-cured members during cold winter months due to a perceived potential for “thermal shock.” Many years of experience with accelerated curing have shown that thermal shock does not occur and is not a problem. In addition, recent studies by the Massachusetts Highway

Department have shown that precast bridge products, removed from the casting bed at 160°F in -5 to +10°F ambient conditions, took 24 to 96 hours to cool to ambient temperatures and continued significant strength gain during the cooling period. Additional discussions of extended moist curing and stripping to storage in cold temperatures can be found in PCI Publication TR1 (1981).

3.3.5.5 Optimizing Concrete Curing

The methods used to accelerate the initial cure of concrete in precast bridge members, as well as the benefits of accelerated curing, are discussed in Section 3.3.5. Section 3.3.5.3 introduces the concept of an optimum curing cycle, which is made up of three critical stages:

- the preset or initial set period
- the period during which the temperature of the concrete is increased
- the period during which the maximum curing temperature is maintained

Regardless of curing method, plants monitor concrete temperature with thermocouples embedded in the product. Because of heat due to hydration of the cementitious materials, it is important that the temperature of the concrete, not the air under the enclosure, be monitored during this process. This is discussed in Section 3.3.5.3. In more sophisticated systems, a computer monitors the thermocouples and automatically adjusts the heat applied to the product by activating switches or valves. Some plants use night watchmen to control the heat application. In either case, the goal is to add heat energy to augment the heat of hydration and achieve a temperature that follows a predetermined optimum cycle.

The following sections describe the quality control aspects of this process.

3.3.5.5.1 Determination of Preset Time

As introduced in Section 3.3.5.3, application of heat should begin only after the concrete has taken its initial set. Temperatures as low as 125F have been shown by Hanson (1963) to significantly decrease the 28-day strength of the concrete when applied with an insufficient preset period. Concrete placed in warm or cold temperatures should be maintained at the placement temperature until the preset period is complete. This preset period is currently established by AASHTO T197 (ASTM C403) for each mix design in use. Unfortunately, this test procedure is difficult and time consuming to perform in the plant.

In recent years, an alternate test method has been developed that is easier to perform. In lieu of initial set, it is now recognized that the optimum time to start the application of heat may correspond more closely to the initial development of the cement's heat of hydration. This point can be determined by a hydration chamber, which is an enclosure in which freshly mixed concrete is placed and maintained in nearly adiabatic conditions. Using commercially available chambers, it is possible to determine the onset of hydration, and hence determine the optimum preset period. Electric curing equipment suppliers are currently working toward next-generation curing systems in which the computer controller, with the aid of a hydration chamber, automatically determines the optimum preset time and programs the curing cycle.

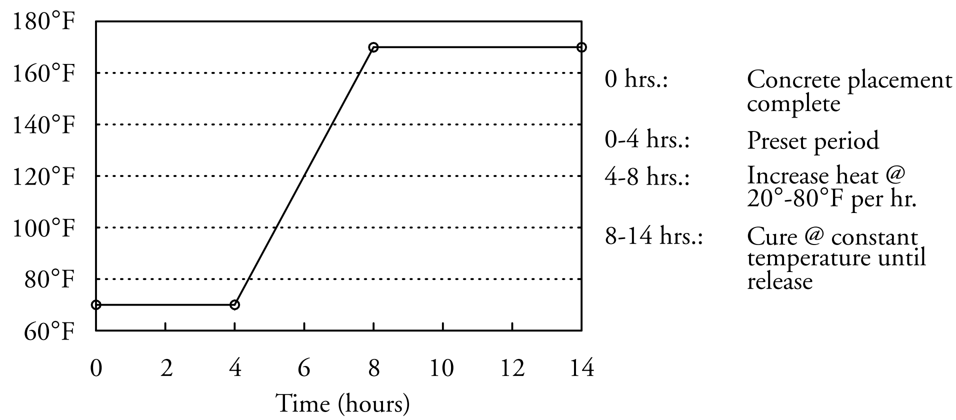
Excessive delay periods reduce the effectiveness of accelerated curing.

3.3.5.5.2 Rate of Heat Application

Once heat is applied, the rate of heat gain must be controlled to prevent damage to the concrete. The AASHTO *Standard* and *LRFD Specifications* limit the temperature rise to a maximum of 40F per hour. However, assuming the proper preset period is observed, Pfeifer and Landgren (1982) has shown that increases of up to 80°F per hour do not significantly reduce the 28-day strength of the concrete when compared to lower temperature rate gains. In reality, properly optimized heat gain is normally dictated by parameters other than the maximum rate allowable.

The optimum rate of heat application can be determined by balancing the concepts of concrete "maturity," the thermodynamic law of heat exchange, and the power requirements of the curing system. Maturity is defined as the area under the time-temperature curve. A typical time-temperature curve is illustrated in **Figure 3.3.5.5.2-1**. For a given concrete mix, equal maturities theoretically result in equal concrete strengths, and can be obtained with different rates of heat application by varying the length of time the heat is applied. The desirability of long preset periods, combined with the need for a minimum level of maturity to achieve the required concrete release strength, can lead to the conclusion that the concrete temperature should be raised rapidly. However, the law of heat exchange requires a larger amount of heat energy for rapid increases in temperature than for more gradual increases.

Figure 3.3.5.5.2-1
Typical Time-Temperature Curing Cycle Graph



These considerations have implications for both the initial and operating cost of the curing system. For example, with an electric curing system, the watt density of the heating elements on the form would need to be high, resulting in a close spacing of the elements. The system would also require greater peak power capacity. The high initial cost of the elements, as well as a larger power supply, is usually not justified when the peak power demand will be required for less than 25 percent of the curing cycle. Economic analysis of the installation and operating costs show that the optimum solution is to install a system that under 100 percent power raises the concrete temperature at a slower rate, usually between 15F - 20F per hour. Curing system equipment suppliers can assist with this determination.

3.3.5.5.3 Maximum Curing Temperature

Both AASHTO *Specifications* limit the maximum concrete temperature during accelerated curing to 160F, while the PCI manual, MNL-116, allows a maximum concrete temperature of 190F. This remains the subject of debate, as some producers maintain that field experience has shown no detrimental effect from curing temperatures as high as 200F. Given the proper preset period, laboratory tests by Pfeifer (1982) have shown that curing temperatures as high as 180F do not significantly decrease the 28-day strength of the concrete when compared to concrete cured at lower temperatures.

3.3.6 Removing Products from Forms

Irrespective of whether a product is cured normally or by accelerated methods, common procedures and precautions must be followed to safely remove the member from the form without damage. This procedure is referred to as “stripping” the products or, sometimes, “stripping the beds.” The sequence of tasks are generally performed in the following order:

1. Verify that the strength of the concrete in the product is at or above the specified stripping or release strength. Concrete strength testing is discussed in Section 3.4.5.

2. For accelerated curing systems, cease heating. In some of the more sophisticated systems, the heat can be stopped before Step 1 based on the “maturity” calculated from the time-temperature curve. See Section 3.3.5.5.2.

3. Remove curing blankets, tarpaulins, and where necessary, form sides. For pretensioned products which have been heat cured, the members must still be warm and moist at the time of detensioning. If not, the unstressed concrete, still restrained by the tensioned strands, will cool and contract, possibly resulting in transverse cracking through the member. See Section 3.3.1.5 for a discussion on the removal of side forms.

4. Remove all remaining ties, inserts and other devices that will prevent lifting the product free of the form, with the exception of the strand hold-down devices. Strand hold-downs are to be released at the appropriate time in the detensioning sequence. Section 3.3.2.8.4 provides discussion on releasing strand hold-downs.

5. For pretensioned products, detension the prestressing bed using the procedures and precautions outlined in Section 3.3.2.8. Cut all strands at both ends of a member if the line has been detensioned hydraulically.

6. Connect proper rigging to the lifting devices embedded in the member, and install lateral stability hardware, if required. See Sections 3.2.4.4 and 3.3.3.2 for information on lifting devices. Section 3.3.7.4 discusses lateral stability issues for long slender members.

3.3.6.1 Form Suction

After performing the steps in the previous section, the member is now ready to be stripped from the form and transported to the yard for storage. When lifting the product from the form, the cranes, rigging, and lifting devices should be sized considering the amount of suction expected from the specific form. Concrete stresses should also be determined considering such effects. Suction on pretensioned members whose sides have been removed is normally minimal, since elastic shortening and camber that result at the release of prestress will usually break the bond between the remaining forms and the concrete. Pretensioned members should not have transverse monolithic ribs or diaphragms unless provisions are made in the formwork to prevent the member from locking itself into the form as it shortens. Conventionally reinforced members picked from fixed forms with numerous drafted vertical surfaces can experience significant suction. For purposes of analysis, increasing the member dead load by 50% is normally sufficient to account for form suction.

3.3.7 In-Plant Handling

Precast plants are normally designed in “linear” fashion in order to facilitate the most efficient movement of products from the casting bed to yard storage. **Figure 3.3.7-1** shows the linear pattern of a typical precast plant. Usually, products just stripped are first moved from the casting bed to a designated finishing area. The finish area is set up to provide ready access to all portions of the member that need post-stripping finish work. For deep members, this area may include scaffolding or platforms with railings that meet OSHA fall protection standards. Many of the finishing tasks described in Section 3.2.5 are performed in this area. Once the member is moved into storage, access is normally limited due to stacking and stored adjacent members.

Figure 3.3.7-1
Typical Precasting Plant Showing "Linear" Plan

Precast products should be handled only by properly designed and installed lifting devices. The lifting devices used in the plant may or may not be the same as those used for erection in the field, since the product orientation in the completed structure may not be the same as that in which it is cast, stored and shipped. Erection considerations are sometimes significantly different than handling and storage considerations in the plant.

3.3.7.1 Handling Equipment

Precast products are moved around the yard by a variety of equipment, ranging from forklifts for smaller products to large gantry cranes on tracks. Rubber-tired gantry cranes, normally referred to as travel lifts, or straddle-carriers, are probably the most common choice by precast producers. See **Figure 3.3.7.1-1**. This equipment is designed to lift and transport heavy loads without the need for shuttle trucks or other equipment, and are not confined to movement on tracks. Travel lifts can use relatively narrow aisles to pass between stored products, allowing the producer to maximize

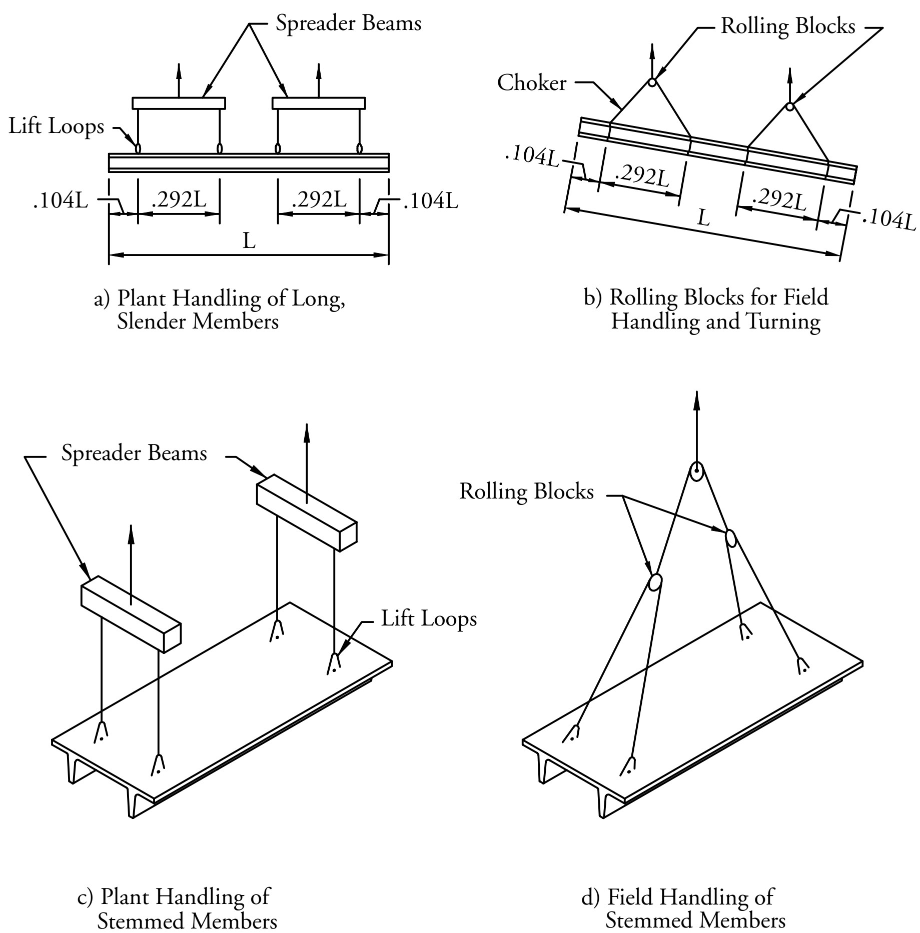
Figure 3.3.7.1-1
Straddle-Carrier

yard storage. Travel lifts are widely available in capacities that accommodate the heaviest practical precast members. Maximum piece weights can be limited by lifting equipment available in the plant, or by the maximum weight that can be shipped by truck. Bridge designers should consult local producers for plant handling capability.

3.3.7.2
Rigging

When multiple lifting points are used, techniques for equalizing the load on each lifting device are necessary to assure that the rigging is statically determinate. This is usually done with rolling blocks, spreader beams or lifting trusses. **Figure 3.3.7.2-1** shows typical rigging arrangements for multiple point lifts.

Figure 3.3.7.2-1a-1d
Rigging for Multiple Point Lifting



3.3.7.3
Handling Stresses

The most critical time in handling a precast member in the plant is when it is initially lifted from the form. The concrete strength is lower and, in pretensioned members, the prestressing force is higher than at any other time in the life of the member. To minimize concrete stresses due to the eccentricity of prestress, pretensioned flexural members are handled with lifting devices as close as practical to the location where the member will be supported in the structure. With the exception of members with pretensioned cantilevers, lifting devices are located near the ends.

Concentrically pretensioned or conventionally reinforced members are handled at two or more points in order to restrict the concrete tensile stresses below the cracking limit. Normally, a factor of safety of 1.5 is applied to the concrete modulus of rupture (see Section 2.5.4), resulting in an allowable tensile stress of $5\sqrt{f'_c}$ for normal weight concrete. In addition, an impact factor is applied to the dead weight of the member if form suction is expected to be significant, as discussed in Section 3.3.6.1. Optimum lifting locations equalize positive and negative moments in members of constant cross-section where the section modulus is the same at the top and bottom. For example, members lifted at two points will have equal positive and negative moments if the lifting points are located 0.207 times the member length from the ends. The use of optimum lifting locations is not always necessary, as long as the concrete stresses are within allowable limits. In many cases, available plant equipment determines the lifting locations. Phillips and Sheppard (1980) and the PCI Design Handbook provide useful information on handling precast concrete products.

3.3.7.4 Lateral Stability During Handling

Long, slender sections can become unstable when handled with lifting devices located near the ends. Studies by Mast (1989 and 1993) conclude that the most important parameter for lateral stability during handling is the lateral bending stiffness of the member. The simplest method to improve lateral stiffness is to move the lifting devices in from the ends. However, doing so normally increases the concrete stresses at lifting and, sometimes, the required concrete release strength. Other methods of improving lateral stiffness are available, as discussed by Imper and Laszlo (1987), but add to the cost of the product.

3.3.8 In-Plant Storage

Precast products must be stored so they do not touch the ground and in a manner that minimizes the potential for damage. Foundations should be of sufficient size and strength to resist crushing or excessive settlement. Properly designed storage is normally governed by consideration of the control of permanent concrete deformations rather than control of concrete stresses. Although improper storage can lead to cracking, spalling, or other damage, supports which cause no apparent initial damage can result in undesirable permanent deformations caused by creep of the concrete.

Storage techniques depend on the product type, and whether the members are eccentrically prestressed, concentrically prestressed or conventionally reinforced.

3.3.8.1 Storage of Eccentrically Prestressed Products

Eccentrically prestressed flexural members (without pretensioned cantilevers) should be supported as close to the ends as possible. Storing members on supports a significant distance from the ends may result in undesirable camber growth. Deep members, such as I-beams or bulb-tees, should always be stored plumb. The dead load of an out-of-plumb member induces moments about its weak axis, which can lead to a permanent horizontal sweep. Long, slender members may require temporary bracing for stability during long-term storage.

3.3.8.2 Storage of Concentrically Prestressed or Conventionally Reinforced Products

Concentrically prestressed piles are supported in storage at relatively short intervals along their length (approximately 20 ft). Piles are normally long and slender, with a relatively high level of prestress. Although they can be handled and shipped with relatively large spaces between supports, storing them in this manner for more than a few days can result in permanent deformations. Conventionally reinforced beams and columns are normally stored with supports under the lifting locations. Wall panels, which are usually cast flat, can be stored in this orientation for a short period, but generally are turned on edge for long-term storage to prevent permanent bowing or warping.

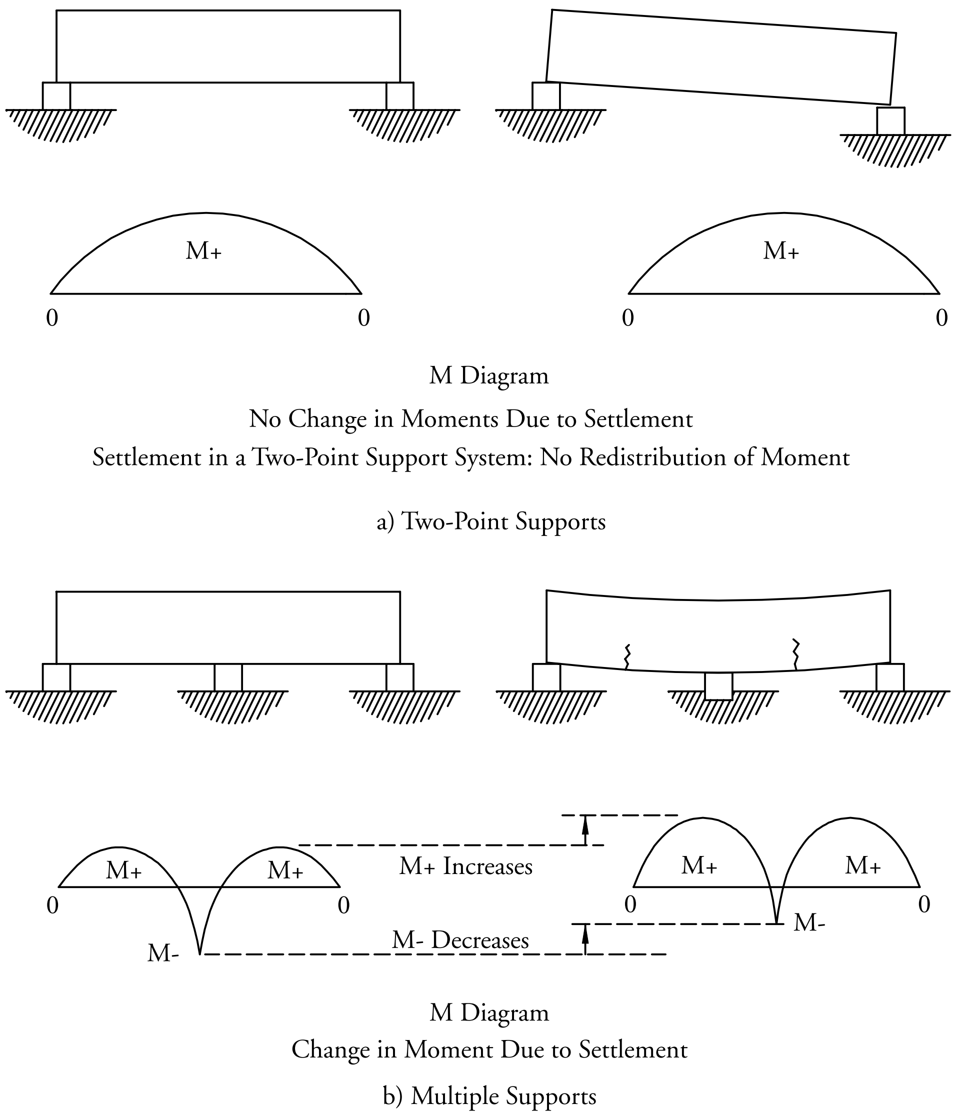


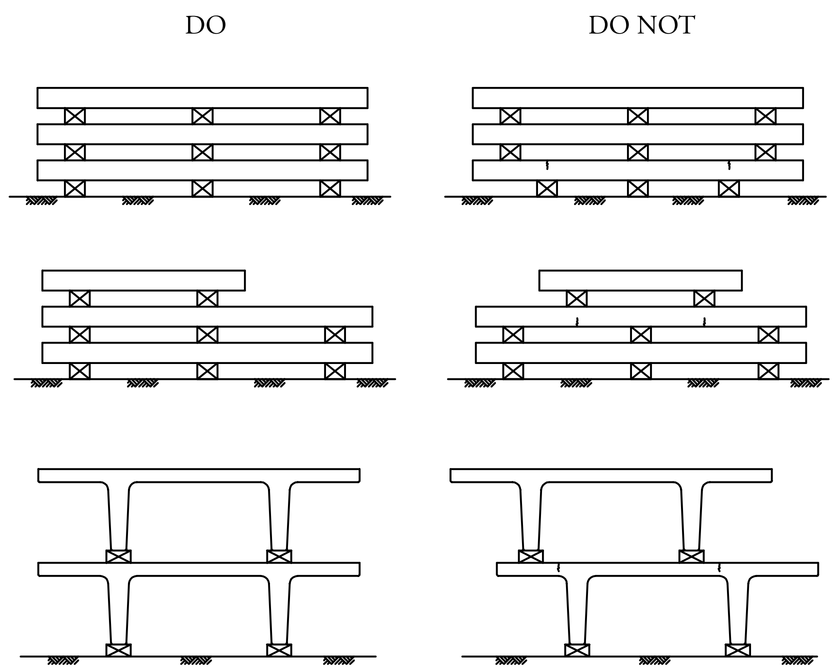
Figure 3.3.8.2-1a-1b Product Storage Points

Multiple supports must be set and maintained at the proper elevation to provide uniform support to the member. This is not as critical for two-point supports, because differential settlement between supports has no detrimental effect on concrete stresses. However, misplaced or differential settlement of multiple supports can have a substantial effect on both concrete stresses and permanent deformations. **Figure 3.3.8.2-1** illustrates this condition.

3.3.8.3 Stacking

In most precast plants, yard storage is limited. Deep flexural members, such as I-beams or some box beams, are generally placed close to one another to conserve space. Shallow members, such as deck panels, stemmed members, or piles, are normally stacked. When stacking products, foundations and supports between levels, must be of sufficient size and strength to support the increased weight. Foundations and intermediate supports must align vertically, providing a direct load path to the foundation. Short members should not be stacked on longer members, unless the supports can be aligned vertically or analysis shows that the lower members will not be damaged or otherwise compromised. **Figure 3.3.8.3-1** illustrates some "dos and donts" for stacking. Steel projecting from the tops of members, such as stirrups or lifting devices, can hamper stacking. Supports between levels must be of sufficient height to prevent damage to the projecting steel, or marring of the upper-level concrete soffits.

Figure 3.3.8.3-1
Some "Dos" and "Do Nots" When Stacking Precast Products



3.3.8.4
Weathering

For long-term storage, consideration should be given to the effects of weathering. It is not practical to expect precast concrete products to be stored indoors, or to be effectively protected from the environment. Section 3.2.5.6 discusses measures which may be taken to prevent corrosion of exposed steel, and the resulting unsightly staining of the concrete surface. When a "like new" appearance is desired in the finished structure, the most cost-effective choice is to clean the concrete surfaces at completion of construction.

3.3.9
Roughened Surfaces

Many precast concrete bridge products are designed to behave compositely with cast-in-place concrete. That is, the two separate concrete placements are intended to act as a unit when resisting externally applied loads. In order for this to occur, shear must be transferred across the interface between the two concrete layers. Typical designs use the "shear friction" concept at the interface. Design advantages are realized when the surface of the precast member which will interface with cast-in-place concrete is intentionally roughened to a full amplitude of approximately 1/4 in., although the shear friction concept does not require roughening. Roughening of surfaces is very common in the precast industry. Methods used depend upon whether the surface to be roughened is exposed or formed.

A requirement common to both exposed and formed roughened surfaces is that they must be clean and free of laitance prior to placing the cast-in-place concrete. It is also generally desirable to moisten the precast surface prior to the second placement.

3.3.9.1 Roughening Exposed Surfaces

The standard method of roughening exposed surfaces is to "rake" or "broom" the concrete while it is still in its plastic state. After the concrete has been struck level, a workman rakes the surface with a tool that creates grooves at a specified spacing and depth. These grooves normally run transverse to the direction of the anticipated shear force, and must be deep enough to produce the desired roughness, but not so deep so as to dislodge individual aggregate particles near the surface. **Figure 3.3.9.1-1** shows a typical raked surface. This type of surface is common on the tops of I-beams, bulb-tees, and box beams that are subsequently made composite with a cast-in-place concrete bridge deck.

Figure 3.3.9.1-1
Roughened Composite Surface

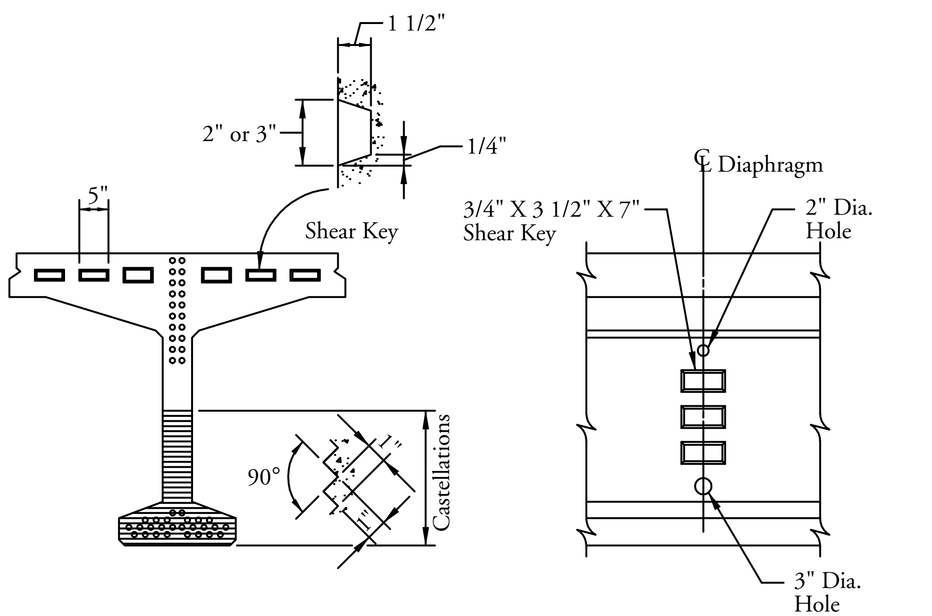
3.3.9.2 Roughening Formed Surfaces

Obviously, formed surfaces cannot be roughened in the same manner as exposed surfaces. Several methods are used to roughen formed surfaces:

- chemical surface set retarders
- bush-hammering
- castellations
- deep sandblasting
- shear keys

Surface set retarders, which locally retard the setting of cement, are painted onto the form in the desired location prior to casting the concrete. After form removal, the retarder is pressure washed from the concrete surface, resulting in a roughened, exposed-aggregate finish. Set retarders are formulated with different strengths to result in varying depths of retardation. Normally, the strongest formulation is required to achieve the roughness desired for composite action. Both sandblasting and bush-hammering are done manually after the product is stripped. They are labor intensive. Shear keys and castellations are formed into the concrete surface. **Figure 3.3.9.2-1** shows typical shear key and castellation configurations. Roughened formed surfaces are normally used at the interface with cast-in-place concrete diaphragms, or at girder ends which frame into cast-in-place piers.

Figure 3.3.9.2-1a-1b
Typical Castellations and Shear Keys in Formed Surfaces



a) Beam End for End Diaphragm Cast
b) Beam Web for Intermediate Diaphragm Cast

3.3.10 Match-Cast Members

Match-cast precast products are typically used in segmental construction to ensure the proper fit-up of mating surfaces between precast segments while providing for the profile grade and horizontal alignment required by design. Segment sizes are most often determined by available handling and shipping equipment capacities, but may also be dictated by the amount of form the manufacturer has on hand. Although practically any type of precast product can be match-cast, this construction method applies primarily to long-span construction using flanged box beam or deep I-beam segments.

3.3.10.1 Match-Casting Techniques

Two basic techniques are used to match-cast precast bridge segments, one employing a stationary form, the other involving a form that is moved for every casting. With the stationary form, the first segment is cast with endplates at both ends of the form. After this segment has been cured to a concrete strength adequate for stripping, it is lifted out of the form and positioned adjacent to the form so that one of its ends serves as the endplate for the match-cast end of the second segment. The other end of the second segment is formed with one of the original endplates.

The positioning of the first segment relative to the form is critical, since it dictates the alignment of the two segments in the completed structure. Sophisticated surveying techniques, together with adjustable screw jacks and stops, are normally used to accurately position the segment. Prior to casting, the match-cast end of this segment is coated with a debonding agent to allow separation of the segments after casting.

After the second segment achieves stripping strength, both segments can be stripped from the form. The first segment is moved to storage, while the conventionally formed end of second segment assumes the role of the endplate for the third segment to be cast. This process continues until all segments are cast.

The “moving form” technique begins in a similar manner, however, after the first segment is cast and cured, it is left stationary on the form pallet. The form is stripped, moved longitudinally, and positioned adjacent to the first segment. The second segment is then match-cast against the first in the same manner as described above. This approach has the advantage of decreasing segment handling, but requires multiple form pallets and significantly more space.

3.3.10.2 Joining Match-Cast Members with Epoxy

A common method of joining match-cast segments is by “cementing” them together with a thin (approximately 0.02 to 0.04 in.) layer of epoxy bonder. Because the epoxy coat is thin, it is essential that the member ends be properly matched. The normal construction sequence begins with the application of a slow-setting epoxy to the mating ends. The epoxy should be applied in accordance with the manufacturer’s recommendations. The ends are then assembled, and an initial post-tensioning force is applied across the interface. Gerwick (1993) notes that the best results are obtained when the epoxy cures under a stress of about 40 psi. This is done progressively for each pair of match-cast segments. Once a predetermined number of segments have been joined, and the epoxy in all joints has cured, a final post-tensioning force is applied to the superstructure (or portion of the superstructure). In I-beam bridges, final post-tensioning is usually done after the cast-in-place deck has been placed.

3.4 PLANT QUALITY CONTROL AND QUALITY ASSURANCE

Plant-cast concrete bridge products benefit substantially from the controlled conditions under which they are fabricated. Section 3.3 offers many examples of fabrication procedures that are easier to control and inspect that is the case with field construction. PCI Certified Plants are required to maintain rigorous quality control programs that satisfy the requirements of the project specifications, or PCI manual, MNL-116, whichever are more stringent. Twice each year, certified plants receive unannounced audits for compliance with these requirements by an independent engineering firm that is employed by and reports to PCI. The following section discusses plant quality control procedures, and the resulting benefits to the purchaser.

3.4.1 Plant and Inspection Agency Interaction

The production process for precast, prestressed concrete differs substantially from common field construction. Consequently, it is important that quality control personnel be qualified to inspect all phases of fabrication. PCI currently offers three levels of training and certification for quality control personnel, with a fourth level under development. Since the evaluation criteria for plant certification includes personnel qualifications, PCI Certified Plants must employ in-house quality control personnel who have been suitably trained in the inspection of precast, prestressed concrete products. This is not necessarily the case with outside inspection sources. However, some agencies have taken advantage of PCI training seminars, and require that their agency personnel are appropriately certified.* In addition, the production process frequently begins before sunrise with the testing of release cylinders, and ends after sunset with the covering of the product for accelerated curing. This time span complicates the inspection of all phases of fabrication by an individual inspector. Precast plants efficiently schedule their team of in-house inspectors to cover all phases of production.

In order to make the best use of available personnel, several agencies have developed Quality Control/Quality Assurance programs that shift the accountability and responsibility for product quality to the manufacturer. Under these programs, the manufacturer is responsible for performing day-to-day quality control functions, while the agency assumes the role of review and acceptance. PCI Plant Certification provides the basis for these programs, which are then expanded to cover any specific needs of the agency. These industry/agency partnerships are part of the National Quality Initiative (NQI), which has been endorsed by the American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), American Road and Transportation Builders Association (ARTBA), American Consulting Engineers Council (ACEC), Associated General Contractors of America (AGCA), American Public Works Association (APWA), and the concrete and asphalt industries. For more information on these programs, consult the AASHTO reports titled *Quality Assurance Guide Specifications* (1996) and *Implementation Manual For Quality Assurance* (1996).

3.4.2 Product Evaluation and Repair

As with any manufacturing process, non-conformances can occur in precast concrete bridge products. Examples may include voids or cracks in the concrete, missing or improperly located inserts or holes, and incorrect projection of reinforcement. Non-conformances fall into one of three categories, including:

- those that can be accepted in spite of the non-conformance
- those that can be repaired satisfactorily
- those requiring rejection of the member

3.4.2.1 Surface Voids

In spite of the finest placing and consolidation techniques, surface voids or "bug-holes" resulting from water and air bubbles trapped against the side forms should be

*For further details contact the PCI Director of Certification Programs anticipated in hardened concrete surfaces. These minor imperfections are usually not structurally detrimental. Additional finishing requirements should be based on the end use of the product, and should be established in the contract documents. It is generally not practical to specify an acceptable level of imperfections, such as an allowable percentage of void area within a given square foot of concrete surface, since these judgments are highly subjective. Maximum acceptable void sizes (diameter and depth) can be specified, although the evaluation of these criteria is tedious. The most cost-effective choice is to accept the surface "as-is." Beyond this, it is questionable which is more economical: to identify and patch individual voids larger than specified; or to simply finish the entire surface as described in Section 3.2.5.3. The PCI manual, MNL-116, provides a description of different grades of surface finishes.

3.4.2.2 Honeycomb and Spalls

Larger imperfections, such as honeycombed surfaces or spalls, require mortar patching. This type of repair, while being relatively simple to execute, is difficult to control from the perspective of long-term durability. The relatively shallow nature of the patch creates differential shrinkage between the patching and parent materials, potentially resulting in cracking or failure of the patch. The following techniques are recommended and will mitigate problems with durability:

- proper preparation of the void
- application of a bonding agent
- a patching mortar that exhibits low shrinkage properties
- careful curing of the patch

Most producers have proven patching materials and established procedures with proven performance histories. For honeycombed areas, it is important to remove all loose material to expose sound concrete prior to applying the patch. See Section 3.2.5 for further information on patching materials and procedures.

Like conventional structural concrete, it is important that patches be properly cured after application, because their durability depends on the ultimate strength of the material and control of shrinkage. Application of a non-weathering, non-staining curing compound to a patch surface is recommended.

3.4.2.3 Repairing Large Voids

Very large voids, including those in pretensioned bridge products, can often be repaired by the concrete replacement method. By necessity, this repair procedure is performed prior to transferring the prestressing force to the member. First, the defective concrete is carefully chipped out to expose sound concrete. Care must be taken to avoid damaging reinforcing bars or strands. An epoxy-bond coat is then applied, and new concrete is consolidated into the void using internal vibration. This new concrete is specified to be the same or better than the concrete used in the original placement. The cure of the patch is carefully controlled and accelerated until it reaches the strength required for transfer of the prestressing force.

The key to the quality of concrete replacement is the ability to fully consolidate the new concrete into all portions of the void. From this perspective, the orientation of the void is important. For example, replacement of concrete in the top of an I-beam bottom flange is relatively easy to achieve. It is more difficult on vertical surfaces, such as I-beam webs.

3.4.2.4 Cracks

Cracks develop in conventionally reinforced precast members when the tensile stresses exceed the tensile strength of the concrete. In prestressed members, cracks occur when the tensile stresses exceed the tensile strength of the concrete combined with the internal stresses imparted by the prestressing. Tensile stresses develop in several ways:

- restraint of volume changes
- internal forces from prestressing
- externally applied loads

Precast concrete bridge products are designed to be furnished crack-free. However, cracks should not be considered a reason for rejection unless the product is structurally or aesthetically impaired beyond repair. The following sections discuss cracks related to fabrication, common fabrication procedures used to minimize such cracking, and methods of repairing cracks that occur. Section 3.3.7.3 discusses control of cracks during plant handling. Gerwick (1993) provides a comprehensive discussion of cracking. Also, see PCI *Fabrication and Shipment Cracks in Precast or Prestressed Beams and Columns* (1985).

3.4.2.4.1 Plastic Shrinkage Cracks

A common cause of cracking is shrinkage of the cement paste while the concrete is in its plastic state. During this period, the concrete has developed little or no tensile strength. Excessive evaporation of moisture from the surface will cause the paste to shrink, resulting in cracks that are jagged, discontinuous, and multidirectional in appearance, not unlike a crack pattern observed in a dried mud puddle. The shallow nature of these cracks (usually less than 1/2 in.) means they normally are not of structural concern, and can easily be repaired by rubbing full with mortar. However, they are unsightly, and often raise questions about the acceptability of the product. The best solution is to prevent these cracks from occurring altogether by providing a saturated atmosphere over all exposed surfaces during the curing process.

3.4.2.4.2 Cracks Due to Restraint of Volume Change

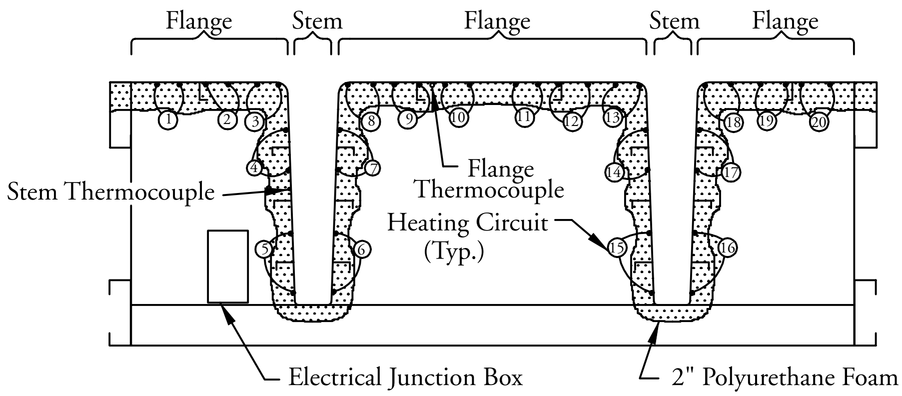
Volume changes are most pronounced along the longitudinal axis of a member, and can result from several sources, including:

- temperature changes
- drying shrinkage
- elastic shortening upon release of prestress
- creep of the concrete

If these movements occur before the member is stripped, certain forms or attachments may restrain the change in volume, possibly resulting in tensile stresses and cracking. Cracks of this nature are normally continuous, narrow and relatively straight. To mitigate this potential for cracking, forms should be of a constant cross section, with no appreciable offset at joints, and attachments restraining the longitudinal movement of the member should be removed as soon as possible after accelerated curing is discontinued. Attachments transverse to the longitudinal axis of the member, such as monolithically cast diaphragms, should not be used unless provisions are made in the formwork to accommodate the anticipated volume changes.

Prestressing strands can also restrain longitudinal volume changes. It is not uncommon for an I-beam to develop vertical cracks at intervals along its length when it is allowed to cool with the strands still tensioned. One way to prevent this is to keep the member warm until it is ready to be detensioned, although this is not always practical for beams that remain in forms over the weekend. Zia and Caner (1993) found this potential for cracking to diminish with increased length of free strand in the casting bed.

Figure 3.4.2.4.3-1
Form Cross-Section Showing Electric Heat Element Layout and Insulation



3.4.2.4.3
Differential Curing Cracks

Differential curing can also contribute to volume change cracking. Some products, such as stemmed members, have cross-sectional regions with varying volume-to-surface ratios. Depending upon the type of accelerated curing used, some regions can be warmer than others, causing a thermal differential which can result in cracking. This phenomena contributes to a relatively common crack at the interface between the web and flange of stemmed members. One advantage of electric curing is that by varying the spacing and control of the heating elements for areas of different volume-to-surface ratios, relative heat gain can be better balanced. **Figure 3.4.2.4.3-1** shows a common electric curing configuration for stemmed members. The elements are more closely spaced in the flange than in the web, since the large open top results in significant heat loss from the flange with less heat of hydration. The flange and web elements are controlled separately by flange and web thermocouples to provide uniform heat gain in the different segments.

3.4.2.4.4
Accidental Impact Cracks

Another source of cracks during fabrication is from accidental impact. This type of cracking comes in all shapes and forms, and must be evaluated on an individual basis. A common example is cracking of the top flange of I-beams or bulb-tees during form removal. These cracks are not considered structurally significant unless they show signs that the reinforcement crossing the crack has yielded, such as for large crack widths or displacement of the adjacent surfaces.

3.4.2.5
Crack Repair

3.4.2.5.1
Autogenous Healing

Under certain conditions, cracks in precast concrete members can literally repair themselves by autogenous healing. This process can be best described as unhydrated portland cement crystals growing together across the crack in the presence of moisture and under a clamping force. Zia and Caner (1993) recommends the region of the crack be kept moist a minimum of 7 days.

3.4.2.5.2
Crack Repair by Epoxy Injection

The most common method of repairing relatively narrow structural cracks is by epoxy injection. Modern epoxy injection methods using equipment that automatically meters, mixes and injects the two-component epoxy are very convenient and give excellent results. Follow manufacturer recommendations to seal cracks and install epoxy injection ports. Cracks as small as 0.002 in. have been successfully injected in the field with full penetration. Cracks, however, should be evaluated with regard to location and effect on serviceability. Small cracks in compression zones in service and not exposed to severe environments, may be best left alone. The ACI

Committee 224 report, *Control of Cracking in Concrete Structures,* states that tolerable crack widths are 0.006 in. for concrete exposed to seawater and seawater spray, wetting and drying; 0.007 in. for concrete exposed to deicing chemicals; and 0.012 in. for concrete exposed to humidity, moist air or soil. It is recommended that the agency and precast producers establish limits for acceptable crack widths as well as repair procedures for those cracks that are determined to need repair. This type of repair is not always aesthetically acceptable, but most producers have developed cosmetic procedures to improve the appearance of the repair.

3.4.2.5.3 Crack Repair by Concrete Replacement

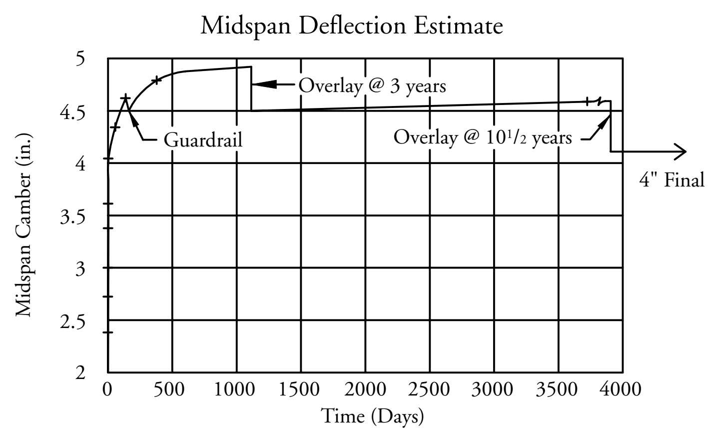
Large cracks which suggest yielding of the reinforcement generally are not repaired by epoxy injection. If the damage is localized, an appropriate repair procedure is to remove the damaged concrete and replace it in the manner described in Section 3.4.2.3. In cases where this repair is not applicable, judgment must be exercised as to the structural severity of the crack.

3.4.2.6 Camber

Camber is defined as the net upward deflection of an eccentrically prestressed member due to the combined member dead load moment and eccentricity of the prestress force. Camber can increase or decrease with time, depending on the level of prestress and sustained loads. A typical camber versus time graph is shown in **Figure 3.4.2.6-1**. Camber can be predicted with relative accuracy at the time of initial prestress, but the prediction of long-term camber should be considered an approximation.

Measuring and recording actual initial camber, and comparing results to the theoretically computed value, is valuable in quantifying the consistency of production and quality control. Small variations in initial camber indicate good consistency in stressing and concreting procedures, while large camber variations may represent poor consistency. Camber that is significantly lower than expected can indicate inadequate tensioning, improper quantity or placement of strands, or loss of bond between concrete and strand (excessive strand slip). Low camber can also result from concrete release strength that is higher than anticipated, such as in members that remain in the form over a weekend prior to initial prestress. Camber significantly higher than expected can result from low concrete strength, excessive force in the strands, or improper quantity or placement of strands.

Figure 3.4.2.6-1
Typical Time-Camber Graph (Deck Bulb-Tee)



3.4.2.6.1 Measuring Camber

The PCI manual, MNL-116, requires measurement of camber to be taken on all members produced from the first cast on a new or unusual bed layout, and on no less than 25 percent of all other members produced each day. This measurement is to be taken as soon as possible after initial prestress, but not to exceed 72 hours after transfer of the prestressing force. The elapsed time to measurement of camber after release should remain consistent for a plant.

Several methods are used to measure initial camber. The simplest is to measure the upward deflection at midspan immediately after initial prestress, but before the member is lifted from the form, using the form soffit as the point of reference. Some products, such as stemmed members, are not easily accessible for this measurement. Once a product is stripped and moved to the yard, camber can be measured with a string-line, laser level, or a surveying level and rod. Camber measurements should be taken to a well defined point on the member, such as the top corner of a bottom flange, and not to an inconsistent surface, such as an intentionally roughened top flange.

3.4.2.6.2 Thermal Influences on Camber

Camber measurements should not be taken when the member is influenced by temporary differences in surface temperature. On a sunny day, the top of the top flange can be significantly warmer than the rest of the member, leading to a temporary increase in camber. Camber readings under these conditions will be misleading.

3.4.2.6.3 Mitigation of Camber Growth

Practical methods for mitigating camber growth are limited. As discussed in Section 3.3.8.1, eccentrically pretensioned flexural members should be stored on dunnage located as close to the ends as possible (or final support locations for members with cantilevers). Moving the dunnage away from the ends toward midspan reduces the dead load deflection, and can lead to increased permanent upward deflection. Adding a load to a member in storage to reduce long-term creep and camber is generally not feasible. Control is best accomplished by scheduling production closer to erection or, if not possible, by allowing for increased camber in the design and detailing of the structure. In an unusual situation where camber is not adequate, it can be increased by moving the dunnage in from the ends during storage.

3.4.2.7 Sweep

Sweep is defined as horizontal bowing of a member, and can result from one of the following:

- misaligned forms
- lateral offset of the pretensioning strands
- improper tensioning
- thermal effects (sun on one surface)
- improper storage

3.4.2.7.1 Mitigation of Sweep

Sweep is usually measured with a string-line after the first day of production in a new form set-up. Once the initial casting is found to be acceptable, it is generally satisfactory to estimate the straightness of subsequent members, measuring only when the eye indicates a potential problem. As with camber, sweep should not be measured when the member is influenced by temporary differences in surface temperature from face-to-face.

The most obvious methods to control straightness are to assure that the forms are installed straight and true and that the prestressing strands are properly located. Also, as discussed in Section 3.3.8.1, precast members which are within tolerance for sweep must be stored plumb. Excessive sweep can sometimes be corrected by leaning the member in the direction opposite the sweep during storage. In this case, the effects of creep will work to straighten the member. In other cases, long slender members can be pulled laterally into alignment prior to final attachment in the structure.

3.4.3 Water-Cementitious Materials Ratio

The definition of water-cementitious materials ratio and its relationship to mix design is discussed in Chapter 2 Section 2.4.6. In addition to portland cement, certain mineral admixtures are cementitious and contribute to the strength of concrete. These are discussed in Section 2.3.4. Additional information may be found in the PCI manual, TM-103.

3.4.3.1 Mineral Admixtures and Workability

The high water demand of mineral admixtures has a significant impact on concrete workability. The influence of water-cementitious materials ratio on workability depends on the proportions of the different cementitious materials. For example, if a mix uses 225 pcy of water, 500 pcy of AASHTO M85 Type III cement, and 50 pcy of silica fume, its water-cementitious materials ratio is 0.41 (225/500+50). Assume this mix has a slump of 4 in. with a certain dosage of a water-reducing admixture. If the silica fume content is increased to 100 lb, and the cement content is reduced to 450 lb, the water-cementitious materials ratio is still 0.41, but the concrete will have a slump less than 4 in. if the same dosage of admixture is used. Trial mixes that investigate the relationship between mix constituents, plastic concrete properties, and hardened properties are essential in effective use of these admixtures and various mix designs. A discussion on calculating the relative equivalency of cementitious materials in mix designs is given in ACI 211.1.

3.4.3.2 Water-Cementitious Materials Ratio and Durability

It has been well documented that the primary variable affecting concrete durability is the water-cementitious materials ratio. Studies by Pfeifer, et al (1987, 1996) have shown that lowering the water-cementitious materials ratio of a given mix reduces the chloride permeability of the concrete. Neville (1981) provides extensive discussion on the benefits of low water-cementitious materials ratios in improving concrete's resistance to abrasion, freeze-thaw deterioration, chemical attack and deterioration in sea water.

A basic tenet is that the strength of concrete, be it compressive, tensile, or flexural, is inversely proportional to the water-cementitious materials ratio. Furthermore, the values of modulus of elasticity, shrinkage, creep and permeability also have inverse relationships to the water-cementitious materials ratio. Hence, in producing high quality concrete, the goal is to keep the water-cementitious materials ratio to a minimum, and to maintain consistency throughout the concrete placement.

3.4.3.3 Water-Cementitious Materials Ratio without Water-Reducing Admixtures

Before the advent of water-reducing admixtures, the only means of obtaining a low water-cementitious materials ratio was to use minimal water in the mix. In order to achieve the strength necessary to make precast concrete bridge products feasible, concretes used high cement contents, very low slumps, and water-cementitious materials ratios in the range of 0.45 to 0.50. Good placement and consolidation were difficult to achieve with the relatively unsophisticated equipment available at the time.

3.4.3.4 Water-Cementitious Materials Ratio with Water-Reducing Admixtures

Water-reducing admixtures have made it possible to produce workable concrete while simultaneously decreasing the water content. The effects of water-reducing admixtures on the workability of concrete are discussed in Section 3.2.1.3.1. Normal water-reducing admixtures can generally produce workable concrete with water-cementitious ratios as low as 0.40. High range water-reducing admixtures can further reduce the ratio to about 0.30 or slightly lower. Water-cementitious materials ratios at this low level can be handled in precast plants because of the short duration between mixing and placing, as well as the use of sophisticated consolidation techniques. This is generally not true of cast-in-place construction.

Water-reducing admixtures can also be viewed as "cement-reducers." Because the strength of concrete increases as the water-cementitious materials ratio decreases, in many cases the cement content can be reduced while maintaining concrete strength.

3.4.3.5 Controlling Water-Cementitious Materials Ratio

The PCI manual, MNL-116, requires water to be added to the mixer within a tolerance of ±1.5% or one gallon, whichever is greater, from that which is specified in the mix design. This quantity of water includes free moisture in the aggregates, as discussed in Section 2.4.6. Most plants use some form of moisture meter that allows for continuous adjustment of water, based on the free moisture contained in the aggregates. Batching scales are accurately calibrated to assure that materials delivered to the mixer are within the specified tolerances.

3.4.3.6 Testing Water-Cementitious Materials Ratio

Concrete slump measured in accordance with AASHTO T119 provides a good measure of batching consistency for all types of concrete. For concretes without water-reducing admixtures, it can also provide an indication of water-cementitious materials ratios. However, this is generally not true of concretes with water-reducing admixtures. Concretes with very low water-cementitious materials ratios can exhibit high slumps when dosed with high-range water-reducers, yet are superior for use with precast products. As discussed in Section 3.2.1.3, concrete with water-reducing admixtures is less likely to segregate during placement than conventional concrete. Consequently, slump is not an appropriate indicator of water-cementitious materials ratio, and hence long-term performance, in mixes using water-reducing admixtures. The actual water-cementitious materials ratio of water-reduced concrete is best determined by calculation using the recorded quantity of each constituent added to the mixer, plus the free moisture in the aggregates.

3.4.4 Strand Condition

Prestressing strand must be protected from corrosion prior to use. Most strand suppliers provide protective wrappings for this purpose. Once this wrapping is removed, the strand pack should still be protected from extended exposure to the elements. The high tensile strength of strand makes it more susceptible to corrosion than lower strength steels. Storage under cover is preferred as a means of minimizing corrosion, but is not always practical.

Strand in which corrosion has pitted the surface should not be used. However, the presence of light rust on strand is not detrimental to bond, and in fact light rust can increase bond. If no pitting has developed on the strand surface, then there has been no loss of effective strand area. The rule of thumb is that if rust can be removed with a pencil eraser, and the strand shows no pits, then the rust level is not detrimental and the strand is acceptable for use. An article by Sason (1992) provides suggestions and photographs to assist in strand surface evaluation.

Special care must be used to prevent contamination of strand from form release agents, mud, grease or other contaminants. Form release agents should be applied to the form before stringing the strands in the bed. After stringing and tensioning, the strand should be inspected for contamination, and cleaned with an effective solvent if necessary before concrete placement.

Packing bands on strand packs should not be cut with a torch flame as doing so may damage the strand. In addition, welding in the vicinity of strands must be strictly prohibited.

3.4.5 Concrete Strength Testing

There are generally three intervals when it is important to evaluate the compressive strength of the concrete in a prestressed bridge member:

- at the time of transfer of the prestressing force
- at the time of transportation and erection
- at 28 days

If the member is shipped and erected after 28 days, strength tests are normally not required for shipping. Cylinder ages other than 28 days may be specified for members that will receive loads at ages appreciably different from 28 days. Also, recent, higher strength concrete mixes have been used that specify strength testing at 56 days. By far, the most common method of evaluating concrete compressive strength is by making and testing cylinders of the production concrete. This testing is done in accordance with the relevant specifications. PCI manual, MNL-116 provides guidelines used by the industry for concrete strength testing. It includes further discussion on the compressive strength of concrete.

Molds used for forming concrete test cylinders must meet the requirements of AASHTO M205, which describes both reusable and single-use molds. In general, reusable molds are used in precast plant production. When very high strength concretes are being produced, it may be necessary to use very rigid molds, such as reusable steel molds, to ensure that dimensional tolerances of the test cylinders are maintained. Otherwise, precision grinding of the ends may be necessary.

3.4.5.1 Number of Cylinders

PCI manual, MNL-116, requires the strength at any given age to be determined by the average of at least two cylinder tests, with the exception of the release strength or predictive strengths less than 28 days, which can be determined by one cylinder test. Many specifications for bridge products require an average of two cylinder tests each time the concrete strength is to be determined, and still others require three cylinder tests for any age. Testing two cylinders at three separate ages requires a minimum of six cylinders for each product or production line of products cast in a continuous pour. From a producer's perspective, there is a certain level of risk in casting only the minimum number of test specimens. If the first cylinder broken falls below the specified release strength, too few specimens remain for the required testing. Many plants cast extra cylinders to account for this possibility. When sophisticated curing systems are used, the concrete maturity can give a good indication of when the first cylinder should be tested, as discussed in Section 3.3.5.5.2. When the number of cylinders made is not adequate, alternate methods of determining the concrete compressive strength are necessary, as discussed in Sections 3.4.5.5 and 3.4.5.6.

3.4.5.2 Test Cylinder Size

Test cylinders made in a plant are cast in accordance with AASHTO T23, with the exception that PCI manual, MNL-116, permits the use of 4 x 8 in. cylinders in lieu of the otherwise specified 6 x 12 in. cylinders. Because of the high strength of concrete commonly associated with precast bridge products, the smaller cylinders are more compatible with the limitations of more common and less costly testing machines.

Studies by Neville (1966) indicate that 4 x 8 in. test cylinders can result in a slightly higher compressive strength than 6 x 12 in. cylinders. This becomes more pronounced with increasing concrete strength. Accordingly, PCI manual, MNL-116, requires that side-by-side 4 x 8 in. and 6 x 12 in. samples be made and tested to develop a correlation between the two sizes. **Table 3.4.5.2-1** shows a sample correlation of concrete strength for the two cylinder sizes.

Table 3.4.5.2-1 Sample Correlation of Cylinder Compressive Strengths for 4 x 8 in. versus 6 x 12 in. Cylinders

Concrete Strength Range (psi)	$\frac{f'_c \text{ (4" x 8")}}{f'_c \text{ (6" x 12")}}$
2,000 – 3,000	1.00
3,500 – 5,500	1.05
5,500 – 7,500	1.07
7,500 – 11,000	1.12

3.4.5.3 Alternate Cylinder Capping Methods

The ends of cast cylinders or drilled cores are usually not plane, flat and at right angles to the side of the cylinder. PCI manual, MNL-116, requires cylinders to be capped unless their ends are cast or ground to within 0.002 in. of a plane surface.

The capping material used historically has been a fast setting sulfur compound applied in accordance with ASTM C617. This method generates toxic sulfur fumes and involves the hazard of handling very hot molten sulfur. Though this method served the industry well for many years, it is now used much less often. In 1985, AASHTO adopted a method of compression strength testing (AASHTO T22 Annex) using neoprene pads and steel retainer caps. This reusable capping system reduces the cost of sample preparation, since neoprene pads are less expensive than sulfur capping compound and the labor required to prepare a cylinder for testing is reduced. This capping system also produces more consistent test results, and diminishes the effect of the human element in the capping operation. The average compressive strengths obtained are equivalent to, or slightly higher than, cylinders capped with molten sulfur.

3.4.5.4 Cylinder Curing Systems and Procedures

The strength of concrete test cylinders made to evaluate the strength of the concrete in a precast bridge member is only meaningful if the cylinders and the member have been cured under similar time-temperature conditions. The common practice of placing cylinder molds on top, along side, or under product forms may not produce representative test specimens. Cylinders cured in this manner generally do not gain strength as rapidly as the product, and sometimes the reverse can be true. This method is unreliable and can provide misleading results.

3.4.5.4.1 Cylinder Curing Cabinets

Cylinder curing cabinets are essentially insulated enclosures into which standard cylinder molds are placed. There are two basic types of cabinets: a wet system where water is used as the heat transfer medium, and a dry system where air in the cabinet is the heat transfer medium. Both systems usually incorporate an electric heating system with a temperature controller that senses the product temperature and in turn controls the heating system to closely approximate the product temperature.

The water-filled cabinet provides more uniform heat to the test specimen and is easier to control. The test specimen temperature will slightly lag that of the product during the warm-up period, since the water must be heated before the heat can get to the cylinder mold. Temperatures of cylinders in water-filled cabinets will not follow the member if the product temperature begins to fall significantly. The insulated cabinet is incapable of dissipating the heat energy unless the cabinet is opened to the surrounding air.

The dry cabinet consumes less energy than the wet cabinet and is easier to maintain. However, it is susceptible to creating slightly variable temperatures in the cylinders, as temperature is difficult to control with precision. The dry cabinet is easier to cool. Neither cabinet is readily portable and therefore must be set up permanently in one location. The initial cost of both cabinets is quite high – the wet cabinet being the most expensive. Either cabinet is a better solution than placing test specimens with the product.

3.4.5.4.2
Self-Insulated Cylinder Molds

The state-of-the-art method of curing concrete test specimens utilizes metal molds that are self-insulated and have a built-in heater and temperature sensor that work in conjunction with a solid-state temperature controller. A thermocouple located to sense the internal temperature of the precast concrete member being cured is plugged into the controller, along with the thermocouple from the cylinder mold. The controller continuously compares the temperature of the member with the temperature of the test specimen, and toggles the test mold heater on or off depending on whether the temperature of the test specimen is above or below the product temperature. This system is capable of maintaining the temperature of the test specimen within 5F of the product temperature, regardless of whether the temperature of the product is rising or falling.

3.4.5.4.3
Long-Term Cylinder Curing

Typically, all cylinders are initially cured under conditions similar to those of the product. After the release cylinders are tested and the member is stripped, the later-age cylinders are removed from their molds, and placed in moist storage at 73.4°F (± 3°F) in accordance with AASHTO T23.

Some specifications require that the cylinders be stored with the product. Most precast concrete bridge members have much larger volume-to-surface ratios than the cylinders. Consequently, storage under the same conditions would cause the cylinders to both dry and cool much faster that the product they are intended to represent. Experience has shown that cylinders stored in this manner, particularly during the cold winter months, suffer diminished strength development and do not accurately represent the strength of the product. They should never be used for acceptance testing of the concrete mix or ultimate strength of the concrete in the product.

3.4.5.5
Concrete Cores

As mentioned in Section 3.4.5.1, when the number of cast cylinders is inadequate, an alternate means of determining concrete strength is necessary. One of the most common procedures involves drilling and testing cores from the precast member in accordance with AASHTO T24. Cores are usually removed from a "neutral" location in the product, such as near the neutral axis of a flexural member, and must also be located to avoid reinforcement and other embedments. These cores are not evaluated by the same criteria as cast cylinders, since the aggregates are cut at the sides and cannot be compared to a molded specimen. ACI 318 states that concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f'_c and if no single core is less that 75 percent of f'_c. Campbell and Tobin (1967) provides further information on core strengths. Further, the size and shape of the core must be considered when evaluating its strength, as described by Neville (1966). All holes resulting from cores must be filled with a low shrinkage concrete having a compressive strength at least equal to that of the precast member.

3.4.5.6
Non-Destructive Testing

Several alternate procedures can be used to test the concrete strength of products without destroying the product or the area tested. PCI manual, MNL-116, lists the methods currently available. These procedures are normally employed for comparative or qualitative purposes, and are not intended to replace cylinder testing. Nondestructive test methods are acceptable provided the following conditions are met:

- A correlation curve is established for each combination of concrete mix design, curing procedure and age of test.

- A minimum of 30 tests is used for each correlation curve
- Test results fall within the 95 percent confidence limits of the correlation curve
- Correlation curves are established for each test instrument, even of the same type

If properly correlated with cylinder tests, non-destructive tests may be used to evaluate the release strength of products if the number of available cylinders is insufficient. Rebound hammer testing is commonly used to determine concrete strength at all ages for dry-cast products, such as hollow-core slabs.

3.4.6 Tolerances

Good design and detailing practices for precast components and connections always considers allowable tolerances for fabrication, erection, and interfacing field construction. PCI manual, MNL-116, lists industry standard tolerances for typical precast concrete bridge members. Details allowing generous tolerances usually result in economies during construction, while extremely stringent tolerances can be very expensive and in some cases, may not be achievable. Designers should consult local producers when considering tolerances that are tighter than the industry standards.

3.5 TRANSPORTATION

One of the most important aspects of precast component design is the ability to move the member from the precast plant to the jobsite. Three modes of transportation are used in the industry: truck, rail and barge. The following sections describe issues involved in selecting a mode of transportation. The availability of transportation modes, and limitations on member weights and sizes, vary widely depending on the geographical location of the plant and jobsite. Bridge designers should consult with local producers on transportation considerations in their area.

3.5.1 Weight Limitations

The maximum shipping weight of a precast member depends upon the mode of transportation and geographical location of the plant and jobsite. For shipping by truck, restrictions vary from 50-220 kips, depending on state regulations and available equipment. Normally, the maximum weight is determined by the number and minimum spacing of axles that distribute the load to the roadway surface. The minimum spacing requirement is more difficult to achieve with short heavy members than with long heavy members. Single axle loads of 12-16 kips generally do not require “overload” permits, as long as the axle spacing exceeds the specified minimum spacing. Special permits may allow an increase in load per axle, but may require escorts, engineering evaluation costs, or an indirect routing of the load. Maximum axle loads permitted vary from state to state. Some states further limit axle loads after a period of freezing temperatures.

Standard rail cars can usually accommodate larger loads than a standard truck. Rail cars range in capacity from approximately 120-200 kips. However, unless the rail system runs directly from the precast plant to the jobsite, members must be trucked for at least some portion of the route, and the weight of the member may be restricted by the trucking limitations.

The same trucking limitations can be true of barge transportation. However, for marine construction accessible by barge, the weight is only limited by the rated capacity of the loading equipment or barge. Very large precast concrete floating pontoons for bridges (in excess of 5,500 tons) have been successfully delivered by barge.

3.5.2 Size Limitations

The ability to ship a precast member can be limited by its overall dimensions. Dimensional restrictions depend on state regulations, equipment limitations, and physical constraints along the route to the jobsite. Physical constraints include height and width clearances, and required turning radii. Alternate routes can often be selected to alleviate these constraints.

For trucking with an “overdimension” permit, state regulations generally restrict the height of a loaded member to 14 ft above the roadway surface. Without a permit, this may be restricted to 12 ft. For width, either with or without permits, the ranges are 12-16 ft and 8-10 ft, respectively. Most states do not restrict the length of a load, though many require permits for loads over a specified length. Permitted loads may or may not require escort vehicles. Maximum lengths are normally dictated by the smallest turning radius enroute.

Delivery by rail can be significantly more restrictive. Clearances limited by tunnels and other obstructions are often very restrictive. Long precast members, which may span several rail cars, require at least one end support to articulate to accommodate the turning radius of each car. This can further exacerbate clearances at the midpoint of the member. Dimensional limitations for rail delivery are heavily route dependent, and must be closely coordinated with the railroad.

Product dimensions are usually not limited by barge delivery. In most cases, if a product can be made and handled in the plant, it can be shipped by barge. As with weight restrictions, this usually applies only if both the precast plant and jobsite are accessible by barge.

3.5.3 Trucking

The most common mode of transporting precast concrete products is by truck, since most precast plants do not have easy access to rail spurs or waterways. Trucking is accommodated with four basic configurations of trailers:

- standard flat-bed trailers
- "low-boy" trailers
- "pole" trailers
- steerable trailers

Each truck configuration is pulled by a standard tractor, with the differences provided by the trailer arrangement. The following sections describe in general terms the characteristics of the various trailers. As trailer dimensions and hauling capacities vary throughout the country, the dimensions and capacities given in the following sections should be considered approximate.

3.5.3.1 Flat-Bed Trailers

Relatively small precast concrete products are shipped on standard flat-bed trailers, as shown in **Figure 3.5.3.1-1**. The trailers are 8 ft wide and 40-50 ft long, with the top of the bed approximately 4.5 ft above the roadway surface. The beds are typically supported on dual axles at the back of the trailer and on dual axles at the rear of the tractor – a total of four axles. For loads without special permits, hauling capacity is limited to approximately 50-60 kips. This type of trailer is normally used to transport short span flexural members, such as stemmed members or voided slab beams, and other miscellaneous bridge products, such as substructure components or stay-in-place deck panels.

Figure 3.5.3.1-1
Typical Flat-Bed Trailer

Figure 3.5.3.2-1
Low-Boy Trailer

3.5.3.2 "Low-Boy" Trailers

"Low-boy" trailers are used when height restrictions become a problem for flat-bed trailers. Approximately 35 ft of the center section of the trailer is lowered to reduce the top of the bed to within 2 ft of the roadway surface. These trailers are used to haul tall loads, such as wall panels shipped on edge or large segments as shown in **Figure 3.5.3.2-1**. The overall dimensions and hauling capacity of these trailers are similar to standard flat-bed trailers because they are usually supported by four axles.

3.5.3.3 "Pole" Trailers

"Pole" trailers are configurations where the front and rear axle-sets (or "jeeps") are connected with a telescoping pole, as shown in **Figure 3.5.3.3-1**. Therefore, the distance between the front and rear axles is adjustable. Typical pole trailers can extend to approximately 60 ft between supports, and are used to carry precast members longer than can be handled with standard flat-bed trailers. Their hauling capacity depends on the number and spacing of axles, as discussed in Section 3.5.1.

Figure 3.5.3.3-1
Typical "Pole" Trailer with Additional Pole Extending Beyond Rear Axle-Sets

Figure 3.5.3.4-1
Steerable Trailer

3.5.3.4 Steerable Trailers

The trailing end of a very long precast member is usually supported by a detached steerable trailer. Members as long as 177 ft have been delivered with this equipment. There are two basic types of steerable trailers.

One type of trailer is outfitted with a cab and steering wheel, as shown in **Figure 3.5.3.4-1**. The steerable trailer is secured to the member, which in turn is secured to the tractor or front jeep. During delivery, the steerable trailer is operated by a driver who, in conjunction with the tractor driver, maneuver the member to the jobsite.

Another type of steerable trailer is remotely steered by the tractor driver. The driver's controls activate hydraulic cylinders that off-set the rear dollies. This trailer is efficient and highly maneuverable. It is shown in **Figure 3.5.3.4-2**.

Figure 3.5.3.4-2
Remotely-Steered Trailer

3.5.3.5
Truck Loading Considerations

Precast products should be loaded on trucks with supports located as close as possible to the lifting devices. Concrete stresses should be checked considering impact during transportation (generally, an addition or deduction of 20 percent of the member weight is adequate for calculation for impact during truck delivery). For members with multiple lift points, "rocker" assemblies are used to equalize the load at each support location, as shown in **Figure 3.5.3.5-1**.

Figure 3.5.3.5-2
Swivel Support on "Jeep"

Figure 3.5.3.5-1 Rocker Support Assembly on Pole Trailer

Chains, wire rope or nylon straps are used to secure the load to the trailer or jeep. As mentioned in Section 3.2.4.5, some producers provide blockouts in the top flange to prevent damage from the chains, as shown in **Figure 3.5.3.4-1**. When using "pole" or steerable trailers, the front and rear supports are generally designed to swivel to allow for the relative rotation between the front and rear jeeps during turns. Chains must be secured to the top of the swivel assembly to allow the jeep to turn relative to the member, as shown in **Figure 3.5.3.5-2**.

3.5.4
Rail Transportation

Economically, rail transportation is usually only viable for transporting precast members over relatively long distances, or for projects on railroad rights-of-way. Rail cars are constrained to travel on tracks, which normally necessitates moving the member from storage to the rail siding by truck or travel crane. Also, rail cars endure considerably more impact than trucks, and require substantially more longitudinal and transverse lashing, as shown in **Figure 3.5.4-1**. As mentioned in Section 3.5.2, long precast members must straddle several cars, and require swivel supports to accommodate relative rotation, much like "pole" or steerable truck trailers. A rail car swivel support is shown in **Figure 3.5.4-2**. Compared to trucks, rail cars are difficult to obtain on a consistent and reliable basis.

Figure 3.5.4-1 Railcar Lashing Example

Figure 3.5.4-2 Railcar Swivel Support

3.5.5 Barge Transportation

Where available, barge transportation is the most economical mode of transportation for precast concrete products. In local areas, barging is generally limited to marine construction, as shown in **Figure 3.5.5-1**. However, barges are also used to transport precast products over very long distances for land-based projects, with either truck or rail being used from the dock to the jobsite. The large hauling capacity and dimensional flexibility of barge transportation make it the most attractive, and in some cases, the only feasible mode of transportation.

Figure 3.5.5-1
Barge Loaded with Piles

Figure 3.5.5-2
Barge Loaded with Beams

The large hauling capacity of a barge with respect to its deck area, usually requires the members to be stacked on the deck. In this case, the stacking considerations discussed in Section 3.3.8.3 must be observed. The wood dunnage between the stack and the deck generally must align with the internal barge bulkheads. To conserve deck space, wide-flanged members can be nested, as shown in **Figure 3.5.5-2**. The members are blocked and lashed together, and secured to the deck as a unit. This process improves the stability of each individual member during the journey.

For open ocean tows, a significant amount of lashing is required to secure the load. In many cases, vertical uprights, or “stanchions,” are used to prevent the load from shifting. Under storm conditions, impact can be significant, sometimes as high as 100 percent, and members must be supported with this in mind.

3.5.6 Lateral Stability During Shipping

Long, slender members can become unstable when supported near the ends, as discussed in Section 3.3.7.4. Studies by Mast (1993) conclude that, unlike handling, the most important parameter for lateral stability during shipping is the roll stiffness of the trailer or jeep. Methods used for improving the lateral stiffness of long, slender members for handling, as discussed by Imper and Laszlo (1987), do nothing to improve the roll stiffness of the support during transportation. Most producers have extensive experience with shipping long members, and should be consulted on maximum practical shipping lengths. In lieu of experience, the roll stiffness of transportation vehicles should be evaluated according to the method proposed by Mast (1993), particularly when roadway superelevations and cross-slopes will be encountered on the delivery route.

3.6 INSTALLATION

When a bridge member arrives at the jobsite, it must be erected into position for final integration into the structure. The following sections describe the methods used to install typical precast concrete bridge components, and the materials and procedures used in the integration process.

3.6.1 Jobsite Handling

A variety of methods are employed to erect precast concrete bridge members, ranging from single mobile cranes to sophisticated launching trusses. The method chosen depends primarily on member weights and lengths, available crane capacities and access conditions at the site. Erection costs are strongly influenced by the number of cranes required, the crane capacity, and the speed of erection. Additional information can be found in the PCI report, "Recommended Practice for Erection of Precast Concrete," MNL-127.

3.6.1.1 Single-Crane Lifts

The preferred method of erecting long beams is with a single crane located at either bridge beam support or somewhere between supports. Single cranes located at a support are generally limited to short spans of 60 ft or less. I-beams as long as 120 ft have been erected with a single crane placed at midspan. This requires good access for both the crane and the delivery vehicle near midspan to reduce the reach the crane must make to lift the girder. **Figure 3.6.1.1-1** shows a single-crane lift.

Single-crane lifts require a sufficient length of boom to keep the cables at a specified minimum angle from horizontal, generally 60°. Spreader bars or struts can also be used to maintain this minimum angle.

Figure 3.6.1.1-1 One-Crane Lift

Figure 3.6.1.2-1 Two-Crane Lift

3.6.1.2 Dual-Crane Lifts

Erection utilizing two cranes is usually faster than with one crane, but also more expensive. This method is normally used when long beams can be delivered along the bridge span, and cranes positioned at the supports can swing the beams from the delivery vehicle directly to their final position. **Figure 3.6.1.2-1** shows a dual-crane lift. Dual-crane lifts do not require the very long boom lengths of single-crane lifts. This is important in situations where headroom is limited, such as erection under a bridge overpass or near or under power transmission lines.

3.6.1.3 Passing from Crane to Crane

Passing beams from crane to crane is normally required when erecting long beams over waterways and railways, where neither the cranes or the delivery vehicles have access between supports. Using this process, a crane is placed at each of the near and far supports. A truck with steerable trailer, backs the girder to the near crane, which lifts the end of the beam off the steerable trailer. As the tractor, supporting the other end of the girder, backs up, the near crane moves the girder end as far out into the span as allowed by its lifting capacity. The far crane is then hooked onto a separate lifting device at the end out into the span and, provided it has the capacity, picks up the load until the near crane can be released. If neither crane has sufficient load capacity at the transfer point, a triangular load transfer plate is used to spread the load between cranes until the girder is close enough to be carried by the far crane alone. The far crane and tractor continue to move the girder out into the span until the near crane can lift the end of the girder off the tractor. Erection then proceeds in the same manner as for dual crane lifts.

3.6.1.4 Launching Trusses

Launching trusses are used when, due to ecological or practical considerations, the methods described in Sections 3.6.1.1 through 3.6.1.3 are not feasible. This generally occurs with long spans in the range of 160-200 ft. Launching trusses eliminate the need for cranes, delivery vehicle or temporary support towers to be placed near mid-span, and can be used for both single-piece or segmental construction. Launching trusses are usually shipped in pieces and assembled at the jobsite. Methods of installing the trusses vary depending on the type of construction.

3.6.1.4.1 Launching Trusses for Single-Piece Construction

The use of launching trusses for single-piece construction is usually reserved for long, single-span crossings where access is possible only at the ends. In this scenario, a crane is positioned at both the near and far abutments, and the truss is assembled on a runway behind the near abutment. Due to its relatively light weight and strength, the locations where the truss may be picked are flexible, and the crane at the near abutment is able to "pass" the truss to the crane at the far end. **Figure 3.6.1.4.1-1** shows a launching truss in position.

The long, precast beams are shipped from the plant either as single pieces, or in segments that are assembled into full-length girders in a staging area at the jobsite. A truck with steerable trailer backs the beam to the near crane, which lifts the end of the beam off the steerable trailer, and places it on a trolley on top of the truss. The tractor then backs the beam across the span until the crane at the far end can reach the end on the trolley. The crane at the near end picks the beam from the tractor, and both cranes swing the beam to its final position.

Figure 3.6.1.4.1-1
Setting Precast Beam with Launching Truss

Figure 3.6.1.4.2-1
Assembling Precast Segments with Launching Truss

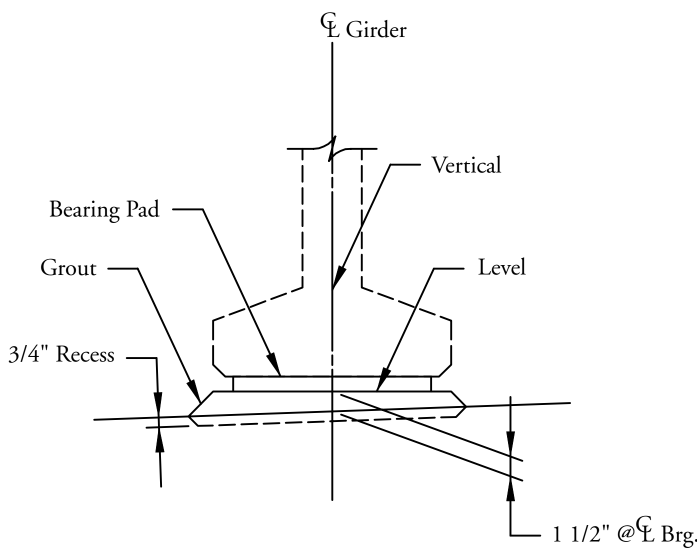
3.6.1.4.2 Launching Trusses for Segmental Construction

Launching trusses for segmental construction are very sophisticated equipment, and are generally reserved for large, multi-span, segmental box beam construction. These trusses are designed to launch themselves from pier to pier, and to lift and hold large box sections in place until the segment is post-tensioned to the structure. **Figure 3.6.1.4.2-1** shows a launching truss used for segmental construction.

3.6.2 Support Surfaces

The construction of supports for precast flexural members is important to provide uniform bearing for the generally high concentrated forces at the beam ends. Elastomeric bearing pads are used predominantly as beam supports. Therefore, the as-cast condition of both the support surface and the beam soffit are critical in providing good bearing. Many designers specify a rectangular grout pad, approximately 1.5 in. thick, to be accurately placed on the pier or abutment as a second stage pour, as shown in **Figure 3.6.2-1**. Support surfaces may be level or sloped to match the roadway profile. When level support surfaces are used with sloped beams, a beveled recess in the beam soffit is used to assure proper slope. For members with two or more support stems, the relative elevation of the multiple support surfaces is critical to prevent warping of the section.

Figure 3.6.2-1 Elastomeric Pad and Grout Pedestal Bearing Detail



3.6.2.1 Inspection of Support Surfaces

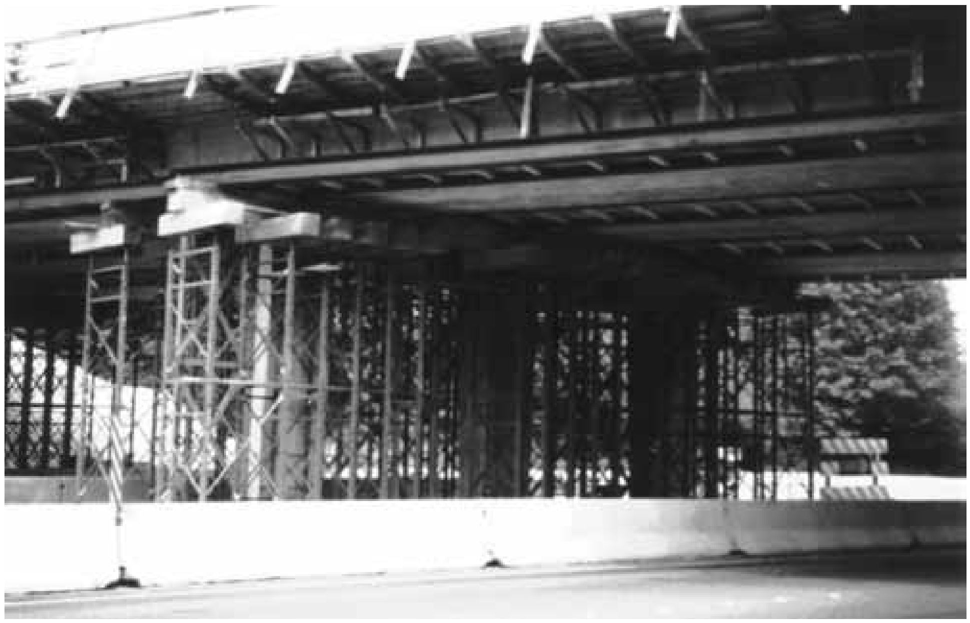
Prior to mobilizing erection equipment, the support surfaces should be checked for horizontal and vertical control, as well as flatness and level or slope. This can be done with standard surveying equipment and a carpenter's level. Flatness is normally limited to 1/16 in. tolerance, and is checked by passing a straightedge over the surface. Any "dishing" of the surface can be detected by light under the straightedge. The same type of check is performed on the beam soffit or bearing recess. Support surfaces and beam soffits which are out of tolerance, normally are corrected by grinding.

3.6.2.2 Temporary Support Towers

When precast concrete beams are too long or too heavy to be shipped as a single piece, they can be cast in segments, erected on temporary support towers, and spliced together in their final position. Support towers usually extend the full width of the bridge to accommodate continuous erection and splicing operations. A solid foundation, usually a compacted crushed gravel base, must be provided for the towers, since very little settlement can be tolerated between the time the splice is completed and the post-tensioning is applied across the joint. Two or more timber mats, placed in perpendicular directions, support the towers and distribute the loads to the base.

The towers themselves are typically heavy-duty aluminum scaffold frames, cross-braced for lateral stability and to reduce the unsupported length of the posts. **Figure 3.6.2.2-1** shows a typical temporary support tower. The top of each post of the frame is supplied with a screw jack, which supports a continuous steel beam across the full width of the bridge. A series of headframes, or interconnected steel beam platforms, are supported on the continuous steel beams and support the girder segments at the splice. Normally, provisions are made for hydraulic jacks to be placed under the girders for final adjustments prior to completing the splice. Abdel-Karim (1992) provides further information on the use of temporary support towers.

Figure 3.6.2.2-1
Temporary Support Tower



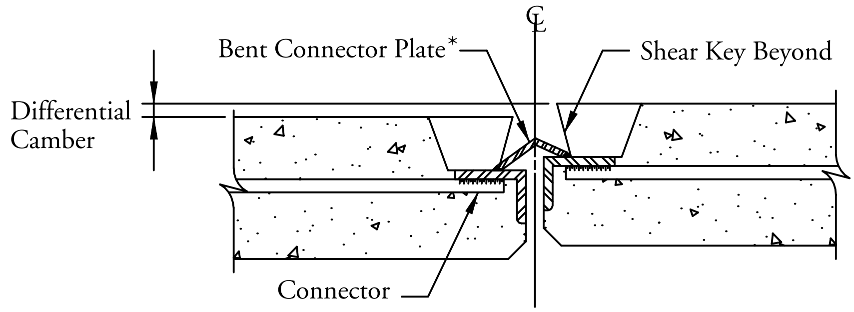
3.6.3 Abutted Members

Precast members are abutted by placing them side-by-side on the supports, and connecting them together so that loads on the bridge deck are shared by adjacent members. AASHTO *Standard Specifications* refer to abutted deck members as precast concrete multi-beam decks. Members that are commonly abutted include solid and voided beams, deck bulb-tees, stemmed members and box beams. Connection details include welding, bolting, grouted shear keys, cast-in-place overlays and transverse post-tensioning. The following sections describe materials and procedures used to connect abutted members.

3.6.3.1 Vertical Alignment

The allowable differential camber between abutted deck members is usually limited to 1/2 in. This is an important consideration since there is often no concrete overlay to compensate for the vertical offset at the joints. However, the stiffer nature of abutted deck members leads to less total camber, and consequently less differential camber, than members that receive a cast-in-place deck. Small amounts of offset between abutted deck members are normally mitigated by feathering grout across the shear key joint. Larger offsets can be minimized by shimming the girder ends to split the offset difference between the ends and midspan, or by leveling the members at midspan with a jack/lever arrangement prior to making the connection between members. Caution must be exercised when leveling thin-flanged abutted members, since weld plates can spall out of the thin flange under the loads imposed by the leveling. Connections should be detailed to accommodate the allowable differential camber, as shown in **Figure 3.6.3.1-1**.

Figure 3.6.3.1-1
Welded Flange Connection Showing Condition with Different Camber

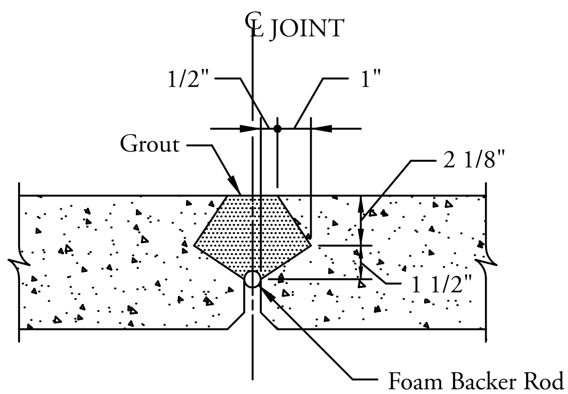


* Note : Plate must be bent prior to installation, never hammered after one edge is welded.

3.6.3.2
Shear Keys

Load sharing between abutted members is normally achieved through shear keys, as shown in **Figure 3.6.3.2-1**, which are filled with grout or concrete. The clamping force required to confine the joint is typically provided by lateral ties consisting of welded connections or transverse post-tensioning. The shear key configuration and joint width vary depending on the type of member and joint filler to be used. Most producers have preferred configurations of shear keys for each standard product, and Stanton and Mattock (1986) provides recommendations for the design and configuration of shear keys. Abutted members which subsequently receive a composite cast-in-place overlay may not require shear keys or lateral ties.

Figure 3.6.3.2-1
Typical Shear Key Connection



3.6.3.2.1
Grout or Concrete in Shear Keys

The choice of grout or concrete to fill shear keys depends primarily on the minimum width of the joint. Concrete can only be used with joint widths approximately 2 in. or greater for two reasons: the joint must accommodate a pencil vibrator (1 in. dia.) for consolidation of the concrete, and the aggregate must be sized 1/5 the minimum joint dimension. Narrower joints are filled with a flowable grout composed primarily of portland cement and fine aggregate, as described in Section 2.6. Both grout and concrete joint fillers must be non-shrink.

3.6.3.2.2
Grouting Procedures for Shear Keys

Thirty minutes prior to grouting shear keys, the joint surfaces must be wetted to achieve a saturated, surface dry condition. The temperature of both the air and concrete should be a minimum of 40F. A volume of grout adequate to fill one or more joints is mixed and poured, preferably with a rolling trough that directs the grout into the joint. The grout is sometimes poured over the joint on the deck, and scraped into the joint with a squeegee, but this tends to stain the deck surface. Consolidation of the grout is accomplished by rodding. The quality control of this operation is important to ensure the soundness and durability of the joint.

3.6.3.3 Welded Connectors

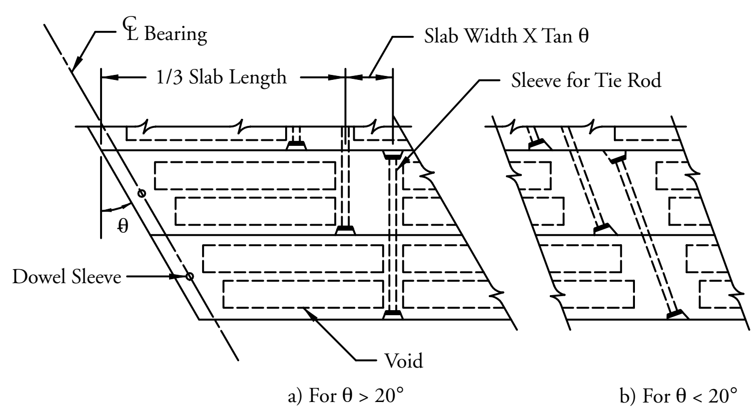
Welded connectors generally consist of plates or angles embedded in the sides of the top flange, and anchored to the concrete with welded reinforcing bars, studs or deformed bar anchors. In some plants, connectors consist of full flange width reinforcing bars welded to plates on both edges of the flange. These connectors are recessed from the top surface of abutted deck members to provide the required cover from the roadway surface, as shown in **Figure 3.6.3.1-1**. This recess is sized to assure adequate access for field welding the connecting plate. Stanton and Matlock (1986) recommends the maximum spacing of welded connectors be the width of the top flange, or 5 ft, whichever is less. Welded connections are most commonly used with deck bulb-tees and stemmed members.

3.6.3.4 Lateral Post-Tensioning

Lateral post-tensioning, located at or above the section's mid-depth, is most often used as the lateral tie system for voided slab beams and box beams, although it can also be used in the flange or concrete diaphragms of stemmed members. Typically, the longitudinal spacing corresponds with diaphragm locations, at the ends and at approximately 40 ft centers. Prestressing bars are most common, although strand systems can also be used. Lateral post-tensioning requires no field welding, and the prestressing steel is fully protected in the completed structure.

One application of lateral post-tensioning for slab beams is shown in **Figure 3.6.3.4-1**. Staggered prestressing bars are placed in ducts normal to the longitudinal axis of the slabs, tying them together two at a time. The bars are placed as erection proceeds, and are stressed using a torque wrench or jack. Enlarged pockets are provided in the shear keys to provide clearance from the bar end to the adjacent slab. This procedure minimizes increase in the bridge width due to dimensional creep, and problems due to misaligned ducts when post-tensioning the full deck width.

Figure 3.6.3.4-1 Lateral Post-Tensioning Connection of Skewed Voided Slab Beams



3.6.3.5 Skewed Bridges

Welded and post-tensioned connections for members abutted on skewed bridges can either follow the skew, or be normal to the longitudinal axis of the member. Connections that follow the skew are normally limited to skews of 20 degrees or less. For skewed, post-tensioned connections, a wedge-shaped pocket is required in the shear key to ensure uniform bearing of the prestressing force on the concrete surface, as shown in **Figure 3.6.3.4-1**.

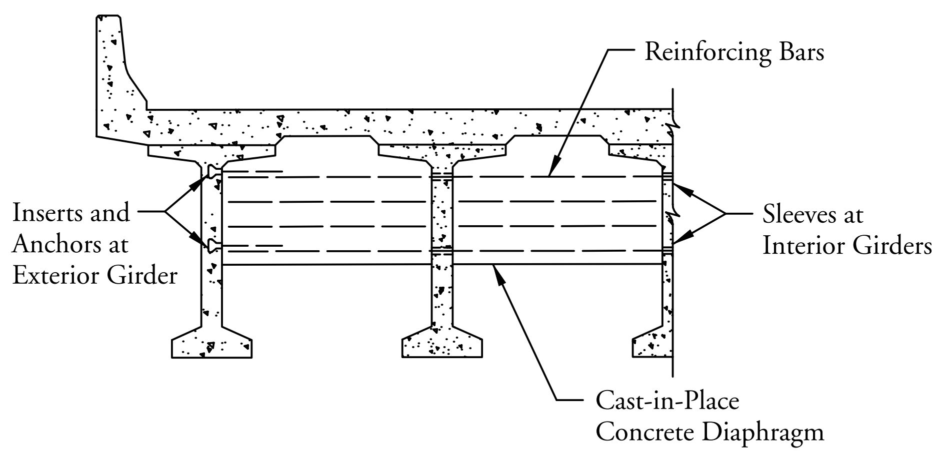
3.7 DIAPHRAGMS

Diaphragms are "stiffeners" that are normal to the longitudinal axis of the bridge and connect precast flexural members to one another. They are generally specified at the bridge ends, and in most regions of the country, at a maximum of 40 ft intervals along the length of the bridge. Rabbat, et al (1982) concludes that end diaphragms ensure uniform reactions at the span ends and provide a smoother ride over the support. In other locations, however, studies by Lin and VanHorn (1969), McCarthy, et al (1979), Sengupta, and Breen (1973), Sithichaikasem and Gamble (1972), and Wong and Gamble (1973) conclude that intermediate diaphragms are not necessary for load distribution and, are in fact, in most cases, detrimental. These studies were performed on bridges with cast-in-place decks, and their conclusions may not be applicable to fully-decked, abutted members. Intermediate diaphragms may also be added above traffic lanes to provide additional strength in the event of impact from overheight vehicles.

3.7.1 Cast-In-Place Concrete Diaphragms

The most common type of diaphragm is cast-in-place, as shown in **Figure 3.7.1-1**. Interior beams are fabricated with holes through the web to allow the top and bottom diaphragm reinforcement to pass through. Exterior beams have threaded inserts embedded in the interior face to accommodate threaded reinforcing steel, bolts or other types of anchors. In lieu of threaded inserts, some exterior beams are cast with holes through the web and a recessed pocket in the exterior face. Threaded reinforcement is passed through the hole, and secured with hand-tightened nut and washer. After the diaphragm concrete has gained some strength, the nut is tightened firmly, and the recess is coated with epoxy and patched with grout. Fully-decked, abutted members, such as decked bulb-tees, are provided with "pour slots," or holes, in the deck to facilitate concrete placement.

Figure 3.7.1-1
Cast-in-Place Concrete Diaphragm Details

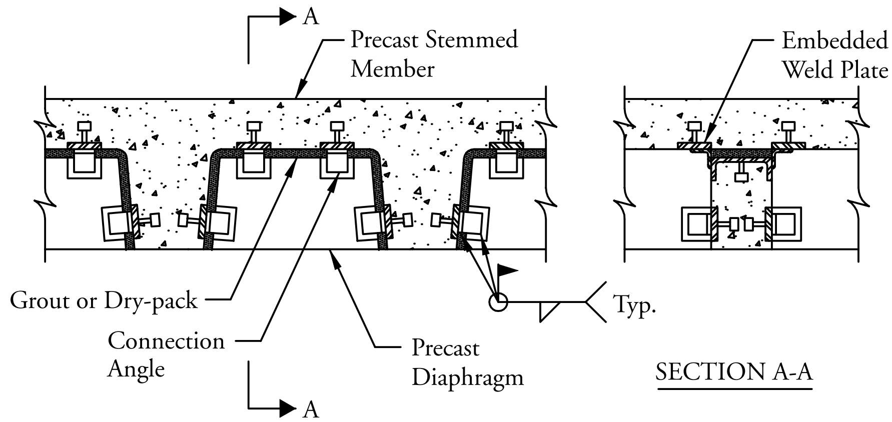


3.7.2 Precast Concrete Diaphragms

3.7.2.1 Individual Precast Concrete Diaphragms

Diaphragms can be fabricated as separate precast pieces and shipped loose to the jobsite for installation into the structure. **Figure 3.7.2.1-1** shows a precast diaphragm detail. These diaphragms must be cast to the shape of the webs and flanges of adjacent beams, and are sensitive to fabrication and erection tolerances. Connections to adjacent beams are usually made by welding. In some cases, tie rods through sleeves in the diaphragms have also been used. The geometry of the structure generally determines the feasibility of precast diaphragms. Among the available diaphragm types, they are the most difficult to properly execute.

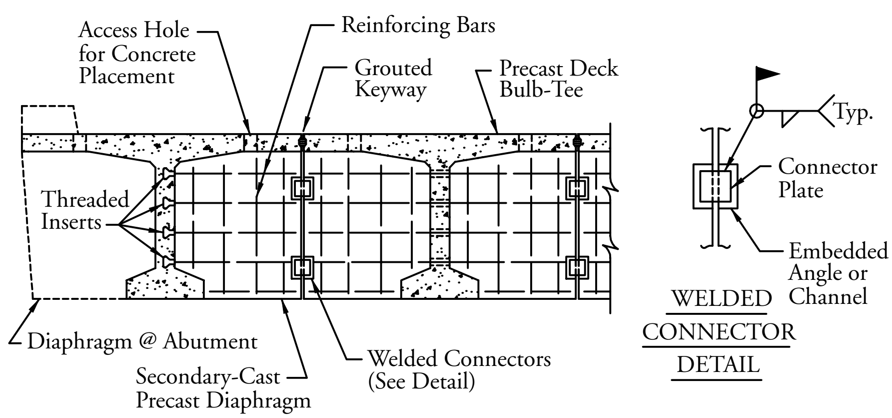
Figure 3.7.2.1-1
Individual, Separate, Precast Concrete Diaphragms



3.7.2.2
Secondary-Cast Precast Concrete Diaphragms

Another option for providing precast diaphragms is to cast the diaphragm directly onto the individual beams in the precast yard, as shown in **Figure 3.7.2.2-1**. The diaphragm reinforcing and connections to the beams are similar to cast-in-place diaphragms. The joint occurs at midpoint between beams, and the connection between diaphragms is usually accomplished by welding or mechanical splicing of exposed reinforcement. The most important aspect of this type of diaphragm is alignment in the field. Proper execution normally requires match-casting of the diaphragms in the precast yard.

Figure 3.7.2.2-1
Secondary-Cast, Precast Concrete Diaphragms



3.7.3
Steel Diaphragms

Steel braces have proven to be an efficient and cost-effective means of providing diaphragms, particularly in remote locations where cast-in-place concrete is not readily available. **Figures 3.7.3-1 and 3.7.3-2** show two types of steel diaphragms used in the industry. The first is normally referred to as a "K" brace. This configuration is not as stiff as most other types of diaphragms, and consequently is used at shorter intervals of approximately 25 ft maximum. The second type is often called a "delta" brace, and has been successfully used at 40 ft intervals. Both types are normally hot-dip galvanized, and connected to the beams by welding. The precautions discussed in Section 3.2.5.6 should be observed when welding galvanized steel. The "K" brace has also been detailed with bolted connections. Due to cumulative fabrication and erection tolerances, predrilled bolt holes are difficult to line-up, so the holes in one of the connecting elements are normally field-drilled.

Figure 3.7.3-1
Steel "K" Brace Diaphragms

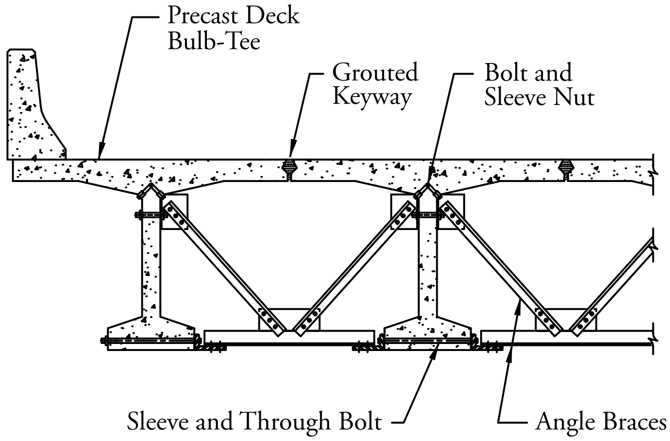
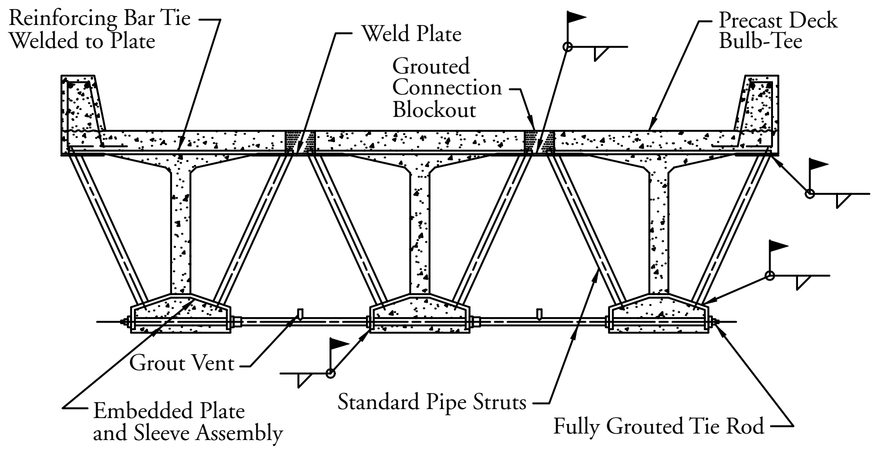


Figure 3.7.3-2
Steel "Delta" Brace Diaphragms



3.7.4
Temporary Diaphragms for Construction

After the beams have been erected, and before they are permanently connected into the structure, they can be subjected to forces of nature that could cause them to fall off the supports. Forces include wind, earthquake or thermally-induced sweep. Temporary braces consisting of steel or timber are used to stabilize the beams. Braces are removed after the final connections are made.

3.7.5
Diaphragms in Skewed Bridges

Diaphragms in skewed bridges can either follow the skew angle or frame normal to the longitudinal axis of the beams. In general, diaphragms perpendicular to the beams are easier to detail and execute, particularly with precast or steel diaphragms. Bridge designers should consult with local manufacturers for the most cost-effective means of providing diaphragms on skewed bridges.

3.8 PRECAST DECK PANELS

Precast, prestressed composite bridge deck panels, combined with a cast-in-place overlay, provide an efficient and cost-effective method of constructing bridge decks. The following sections describe key facets of the fabrication and installation of these panels. Extensive coverage of this subject may be found in PCI's *Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels (1988).*

3.8.1 Deck Panel Systems

Figure 3.8.1-1 Installation of Precast Concrete Deck Panels

Precast composite bridge deck panels are 3-4 in. thick concrete slabs which span between the top flanges of concrete or steel beams. These panels provide a working platform for deck reinforcement placement, and a stay-in-place form for the cast-in-place concrete overlay. **Figure 3.8.1-1** shows panels in place. The panels are fabricated using the materials and procedures discussed in Sections 3.2 and 3.3. Prestressing strands in the panels are oriented perpendicular to the longitudinal axis of the beams and provide all of the positive reinforcement required for the span of the deck between beams. The panels become composite with the cast-in-place overlay to resist superimposed dead and live loads.

Both proprietary and generic panel systems are available to the construction industry. Proprietary systems employ patented methods of erection, temporary support, adjustments, and forming of the gap between the bottom of the panel and the top of the beam. Generic systems use conventional methods to achieve the same results.

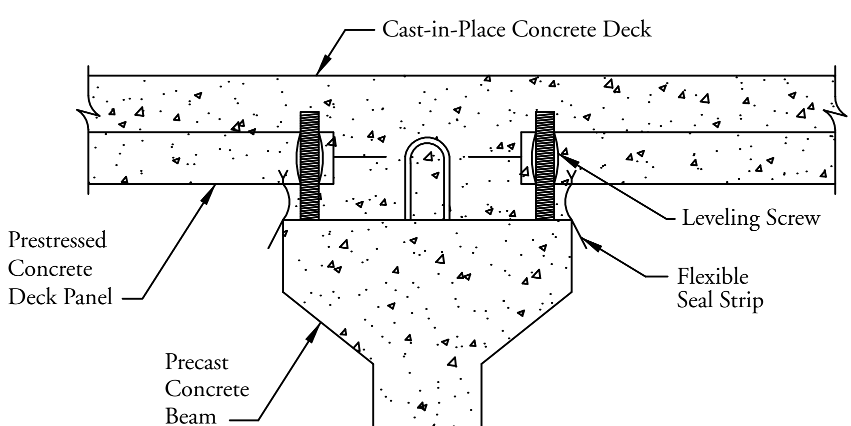
3.8.2 Handling Deck Panels

Precast composite bridge deck panels can be handled with the conventional techniques described in Section 3.3.7, or with proprietary lifting equipment. Proprietary lifting equipment is normally designed to lift the panels along the edge, eliminating the need for embedded lifting devices. This equipment is also designed for quick release to speed erection of the panels.

3.8.3 Installation of Deck Panels

After the panels are erected, they must be temporarily supported until placement of the cast-in-place overlay which also provides concrete under the panel for support. Most systems incorporate a minimum of four screw-jack embedments near the panel corners, which are provided for two purposes. The first is to frame a gap between the bottom of the panel and the top of the beam flange large enough to allow grout or concrete to fully fill the gap, providing uniform bearing for the panel. The minimum gap is nominally 1 in. for grout and 1-1/2 in. for concrete. The screw jacks also allow the panel elevations to be adjusted for the desired profile grade, drainage slope or superelevation, while correcting for beam camber and dead load deflections, maintaining a relatively constant overall deck thickness. **Figure 3.8.3-1** shows a typical detail at the top of the beam. Some proprietary systems offer cast-in baffles to retain grout or concrete in the gap.

Figure 3.8.3-1 Stay-In-Place Composite Deck Panels Bearing Detail (Proprietary System)



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Strategies for Economy

4.0 INTRODUCTION

The use of precast, prestressed concrete products for the construction of bridges results in very economical, high quality structures. This is due to several factors:

- Mass production of standardized, low maintenance sections
- A factory environment that includes quality control
- Speed of erection and construction
- The use of high quality, inexpensive and locally available materials for production

This chapter discusses some considerations for the designer to further enhance the cost effectiveness of precast, prestressed concrete bridge construction.

4.1 GEOMETRY

All bridges must meet the specific geometric constraints for each unique site. The length of the bridge must be sufficient to cross the features below. This can be accomplished by providing fewer longer spans or a larger number of shorter spans. Pier/bent placements may be restricted by roadway or railway systems and their necessary horizontal clearances. Likewise, specific requirements for ships or barges may dictate the placement of piers on either side of a main channel. Existing utilities may limit the locations of foundations. At other locations, such as stream and creek crossings, the designer may have more control over placement of the substructure. The choice of span length can also be affected by the cost of substructure units. Where the foundation conditions are poor or the piers are tall, it could be more economical to use longer spans. The choice of span length should result from the lowest combined cost of the superstructure and substructure. Each site must be evaluated to determine the most appropriate span arrangement to accommodate the necessary horizontal and vertical clearances of the system below the bridge.

4.1.1 Span Length vs. Structure Depth

The depth of the bridge superstructure increases with the span length. As a general rule, this is also true for precast, prestressed concrete. However, the structural efficiency of deeper sections may not always result in cost efficiency.

Raw bridge cost is not the only basis for selecting structure type. Hydraulics and/or gradeline constraints may require very shallow superstructures. Structures that can be constructed rapidly might be justified if detour time, and therefore user costs, can be minimized. Environmental considerations could justify the extra cost of special aesthetic structural designs.

Superstructure depth is frequently controlled by minimum vertical clearance requirements. These are typically established by the functional classification of the highway and the construction classification of the project. As discussed in Chapter 1, a common requirement is that the bridge superstructure be as shallow as possible to satisfy both minimum vertical clearance requirements and to minimize approach grades. Therefore, a high span-to-depth ratio is desired.

4.1.1.1 Shallow Sections

Shallower systems may require more strands and a higher release strength, but, as a rule, are less expensive, since less concrete is required. In addition to the reduced direct material cost, reduced costs can be realized by lower shipping and handling weights. Spans of up to 40 ft can be achieved using solid slabs, voided slab beams or stemmed members placed side-by-side. For a given span length, voided slab beams or stemmed members may use less material and be relatively lightweight. However, solid slabs may be less expensive, since the forms are very inexpensive and the fabrication of the solid slab is relatively uncomplicated.

4.1.1.2 Deeper Sections

As span length increases, there is the need to increase section properties of the superstructure components, while reducing their weight. Deeper sections such as box beams and deeper stemmed sections, placed side-by-side, become advantageous. The greater depth contributes to an increased moment of inertia, while the reduction of the concrete in the voided portion of the beam helps to keep the weight of the section to a minimum. As span length continues to increase, the use of superstructure components not placed side-by-side become the more cost-effective solution. These types of systems, such as spread box beams and I-beams, require the use of a cast-in-place concrete deck to span between beams.

4.1.1.3 Water Crossings

For typical stream or creek crossings where the foundation conditions are good, it may be more economical to use a larger number of shorter spans. The cost of additional substructure units must be weighed against savings associated with the use of smaller cranes which can be used with shorter, lighter beams. Physical constraints on the location of substructures generally are few and are probably restricted only to hydraulic considerations. The balance between the number and costs of substructure units and the size of the superstructure members becomes the primary factor in minimizing construction costs.

4.1.1.3.1 Vertical Profile at Water Crossings

Superstructure depth is important when freeboard of the stream must be maintained, while reducing the impact on the vertical profile of the bridge and approach roadways. Increased structure depth may increase the volume of fill for the approach roadways and have an effect on right-of-way requirements to accommodate roadway fill.

4.1.1.4 Grade Crossings

At grade crossings, span lengths are generally dictated by horizontal clearance requirements and other safety considerations. The span lengths usually are such that the use of spread box beams or I-beams is effective. Depth of structure becomes a consideration in establishing the bridge profile while maintaining the required vertical clearance for the transportation system below. As with water crossings, the structure depth will have a direct impact on the volume of approach roadway fill and the measures necessary to accommodate that fill.

4.1.1.5 Wearing Surface

The use of a wearing surface may be desirable to improve durability and enhance the quality of the ride. A cast-in-place concrete composite topping is a superior wearing surface for high traffic volumes and can also increase the load carrying capacity of the superstructure. On rural bridges with low traffic volumes, especially when deicing salt is not used, the precast concrete surface provides outstanding durability and lowest possible construction cost. In other cases, a waterproof membrane and asphalt surface can be used effectively.

4.1.2 Member Spacing

As span length increases, it becomes necessary to evaluate the use of various beam types, plus evaluating the depth of beams against the number of beams required. For a given span length, a 54 in. deep beam and a 63 in. deep beam may both be acceptable. The number of 54 in. deep beams required in the cross-section will likely be more than 63 in. deep beams. As with shorter span bridges, depth of structure versus the effect on the roadway/bridge profile and the vertical clearance below must be evaluated.

4.1.2.1 Wider Spacings

Generally, the use of fewer beams at a greater spacing will prove to be more economical than more beams at a lesser spacing. The use of fewer members means reduced volume of beam concrete required to be cast and fewer beams to fabricate, ship and erect. Other savings result from the reduction in the number of bearing devices, fewer end diaphragms/edge beams to form and cast, fewer bays between the beams in which to install and remove deck forms and fewer manhours to inspect. Very wide beam spacings (in excess of 12 ft) must be carefully considered, since the cost of the deck and its forming may override the savings of the reduced number of beams. Future deck replacement and staged construction should also be considered in selecting beam spacing.

4.1.3 Maximizing Span Lengths

For a given beam depth, it is often advantageous to use the beam at its maximum span length, even if closer spacings are required.

4.1.3.1 Advantages of Maximum Spans

By using a beam at its maximum span capability, the designer can achieve a longer span without increasing the depth of the structure. This can provide for better horizontal and vertical clearances for the roadway, railway or waterway below. Additionally, for longer bridges, the use of extended spans means fewer substructures must be constructed. Often, longer spans are necessary and consideration of superstructure cost versus substructure cost must be evaluated. For example, when very expensive substructures are required, such as those which must resist ship impact or which require deep or massive foundations, the cost of the superstructure with longer spans becomes more economical.

4.1.3.2 Limitations of Maximum Spans

Designers must be cognizant of the limitations of production facilities, handling, shipping and erection applicable to longer beams. The use of beam sections not available through local producers will usually be more expensive if the forms must be purchased for a small number of beams. Local producers may not have prestressing beds capable of withstanding larger prestressing forces. Longer beams are heavier and may require larger cranes for handling and erection and special truck/trailer systems may be required to transport the beams to the job site. Generally, increased weights are not an issue for erection over water if the beams can be transported to the site by barge.

4.1.4 Splicing beams to Increase Spans

To increase span capabilities of precast, prestressed concrete beams, designers should consider the technique of splicing. Through the use of post-tensioning or other splicing methods, continuity and its inherent benefits relative to moment reduction in the superstructure and a reduction in the number of expansion joints can be achieved. Splicing beams also reduces the size and weight of the segments, allowing easier handling and erection, and lighter weights for shipping. Splicing does, however, have additional costs associated with the time to splice the sections, possible need for temporary supports and the splicing system itself. For more detailed information on the use of spliced beams, see Chapter 11.

4.1.5 Special Geometry Conditions

Overall bridge geometry is very often dictated by the roadway designers. The bridge location within a roadway system frequently establishes the bridge within a horizontal curve, a vertical curve, with skewed substructures, or with flared spans to accommodate ramps.

4.1.5.1 Horizontal Curves

Straight precast, prestressed concrete beams can usually be used for horizontally curved bridges. The beam placement must take into account the degree of curvature and the span length. The primary impact of the curve is on the location of the exterior beams. The overhang of the deck must be evaluated at the beam ends and at midspan to ensure that proper consideration is given to the loading of the beam under both dead and live loads.

4.1.5.2 Vertical Curves

The profile of the deck may include crest or sag vertical curves. The designer must consider the camber of the beam relative to the deck profile to establish the proper build-up of concrete or haunch over the beam (**Figure 4.1.5.2-1**). The volume of concrete in the build-up is larger in wider beams such as bulb-tees (**Figure 4.1.5.2-2**). Horizontal curves also affect the volume of concrete in the build-up due to the superelevation of the roadway. However, this build-up concrete is inexpensive since costs are almost exclusively a function of the concrete material cost. No additional forming, placement or curing costs result from the build-up. In some locations, producers have successfully fabricated beams with a specified top profile and cross slope (within reasonable limits) to accommodate a certain vertical profile and superelevation. This is often done with deck bulb-tees, which are wide, erected with their top flanges touching, and using no cast-in-place topping or asphalt wearing surface.

Figure 4.1.5.2-1
Beam Camber/Deck Relationship

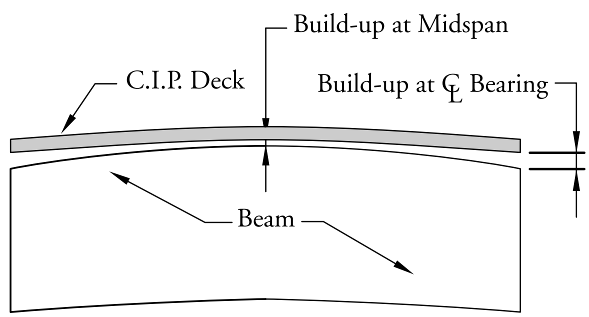
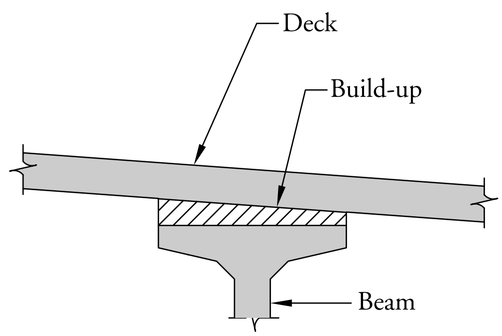


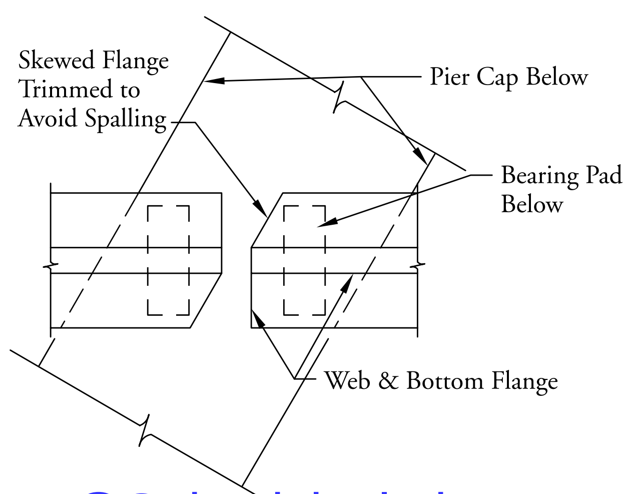
Figure 4.1.5.2-2
Build-up over Beam



4.1.5.3 Skews

Substructures which are skewed to the beam require some consideration. If possible, avoid skewed supports. The *LRFD Specifications* modify the live load distribution factor for skewed superstructures. Additionally, beam ends are usually skewed so that the ends of the beams are parallel to the substructure. Small skews normally will not affect the cost of precast, prestressed concrete beams. Extreme skews usually require the producer to take measures to reduce spalling of the beam end during the strand deten-

Figure 4.1.5.3-1
Beam Ends at Support with Large Skew

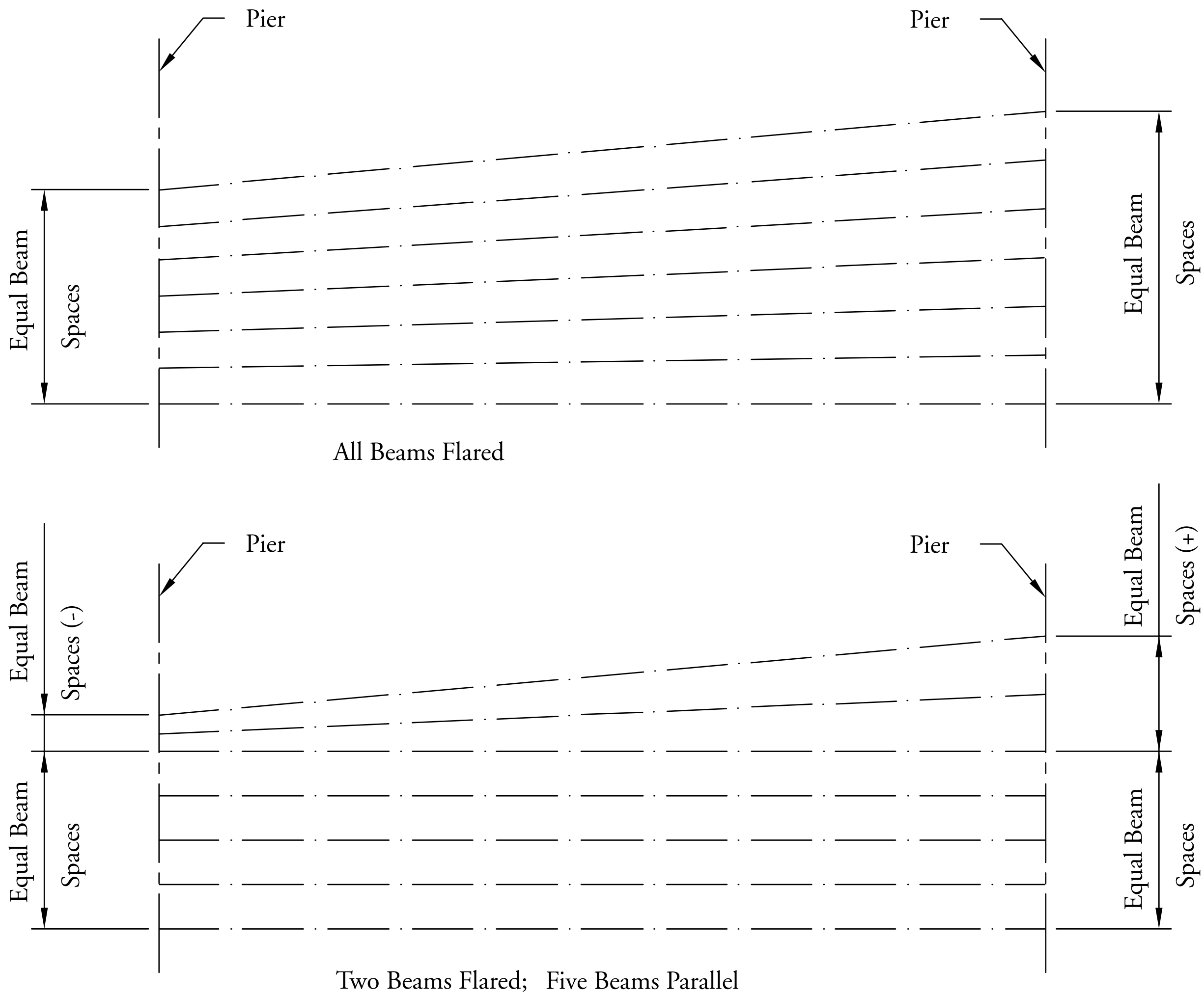


sioning operation. Otherwise, the "point" on the beam end must support the dead weight of the beam when in the prestressing bed. This, combined with elastic shortening, usually results in spalling. One method of reducing the spalling is to trim the point of the skew from the beam as depicted in **Figure 4.1.5.3-1**. If a spall does occur, it is generally minor and can be easily repaired without affecting the integrity of the bearing area.

4.1.5.4 *Flared Structures*

Flared spans are those structures which have one end wider than the other. By using as many parallel beams within the span as possible, the designer can reduce the fabrication and construction costs of the superstructure. This results from maintaining more uniform beam lengths, typical beam end skews and reduced deck forming costs. **Figure 4.1.5.4-1** depicts two beam layouts which could be used for a flared span. Note that with all beams flared, each of the beams is unique. The alternate with five parallel beams has three unique beams and the deck forming will be more uniform.

Figure 4.1.5.4-1
Span Configurations For a Flared Structure



4.1.5.5 *Varying Span Lengths*

When possible, design precast beams with the same cross section and strand pattern. Economy in precasting results from the production of identical sections. If a bridge consists of different span lengths, it may be better to design all of the precast units with the same cross-section rather than to design each span for the minimum depth-span ratio.

4.1.6 Product Availability

Designers must determine the availability of precast products in the local area. If the product selected for the project is not available within 200 to 500 mi, depending on the geographic region, a premium for shipping from a distant precaster or for local form purchase may be added to the project. Designs using local and readily available member types will result in lower bid prices.

4.1.6.1 Economy of Scale

If a single project uses a large quantity of a specific product, or if a new product will be used as a standard for future bridges, the considerable cost of new forms, when amortized over a large volume, becomes far less significant. Designers should consult local producers early in the study phase of a bridge project to determine the available precast products or the costs associated with new products for a specific application. Many times it is possible to create a new section by making small, inexpensive modifications to existing forms, such as casting a 3'-6" deep box beam in a 4'-0" deep form, or placing AASHTO Type II beam side forms on a wider Type IV bottom form.

4.2 DESIGN

Many decisions made during the design of precast, prestressed concrete bridges have a direct economic impact on the bridge construction cost. Some of these decisions are:

- Structural system (simple spans versus continuity)
- Integral caps and/or abutments
- Use of intermediate diaphragms
- Prestressing systems
- Durability systems
- Bearing systems
- Use of lightweight aggregate concrete
- Special construction techniques

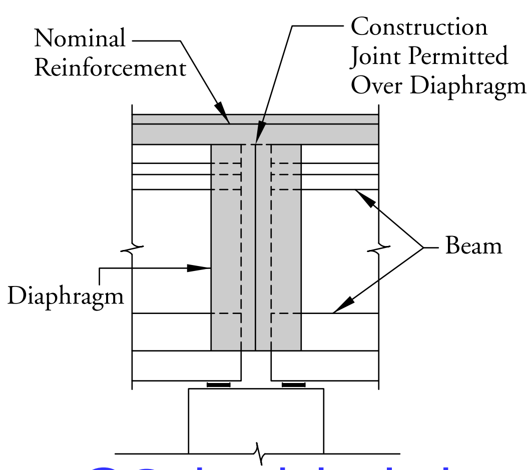
4.2.1 Advantages of Simple Spans

Simple span prestressed concrete superstructures can result in very economical bridges. Many designers rely almost exclusively on simple spans for this very reason. With simple spans, end diaphragms and end connections are greatly simplified. There is a significant reduction in the volume of reinforcement required over interior supports. There are also substantially reduced structural effects of short and long term volume changes due to temperature variations and creep and shrinkage.

4.2.2 Limitations of Simple Spans

Use of simple spans may, however, limit the span length for a particular product or require more beams for a particular span. The use of more prestressing strands may allow for an increased span length, but may create a need for increased concrete strength at release of the prestress force. This may force the cycle time of the prestressed bed to be increased, reducing the efficiency of the plant. There may also be more joints over substructures which can affect deck ride quality. Also, joints must be maintained to reduce premature deterioration of the substructure and bearing devices caused by road salts and deicers. Some designers have successfully eliminated this problem by casting the deck continuous over supports and placing additional nominal reinforcing steel in the deck to reduce deck cracking (**Figure 4.2.2-1**).

Figure 4.2.2-1
Detail to Eliminate Deck Joint



4.2.3 Continuity

In designing continuous superstructures, designers can take advantage of increased span lengths or reduce the number of beams required for a span. The smaller positive moments which occur in continuous systems will reduce the required number of prestressing strands. Continuity will reduce the number of joints in the superstructure and enhance redundancy of the structure.

The continuous superstructure also increases the resistance of the structure to horizontal forces, particularly seismic loads and ship impact forces.

4.2.3.1 Achieving Continuity

Continuity is usually achieved through the use of enhanced, positive beam connections over supports and reinforcing the deck over the substructure to withstand the negative moments due to composite dead and live loads. Longitudinal post-tensioning of the beams may be expensive, but can also be used to achieve continuity. Refer to Chapter 11 for a full discussion of these issues.

4.2.3.2 Limitations of Continuity

Proper detailing of continuous superstructures over the supports should be provided to avoid diaphragm cracking. Some end diaphragms with improper details have resulted in cracks from volumetric changes in the concrete. Use of continuity without post-tensioning requires a significant increase in the amount of mild steel reinforcement in the deck. Some states design beams as simple spans and use continuous slabs over the supports to eliminate joints and reduce the negative effects of the volumetric changes.

4.2.4 Integral Caps and Abutments

Integral caps and abutments have been used successfully in several areas. By creating proper connections between the superstructure and substructure, moments from the superstructure are distributed to the substructure components. More information on integral bridges is found in Chapter 13 and PCI Bridges Committee Report on "Integral Bridges" (1997).

4.2.4.1 Advantages

In addition to the benefits of reduced positive moments in the span, there are also significant increases in the resistance to horizontal forces and redundancy of the structure. Transverse joints and bearing devices are virtually eliminated. Integral abutments are flexible and tolerate a wide range of temperature movements. Integral abutments can be used for precast concrete bridges with lengths up to 1,000 ft. There is also strong potential to reduce the overall construction cost of the substructure.

4.2.4.2 Disadvantages

Design for this type of system is somewhat more difficult than for a continuous superstructure since substructure stiffness must be considered in the distribution of forces. Very stiff substructures make the system sensitive to volumetric changes. Also, connection design and construction requires more attention.

4.2.5 Intermediate Diaphragms

Intermediate diaphragms are a significant cost in the construction of prestressed concrete bridges. When used, intermediate diaphragms may be constructed of either concrete or structural steel. If concrete is used for these diaphragms, it will probably be permanent and its weight must be considered in the design of the beams.

4.2.5.1 Need for Intermediate Diaphragms

Although AASHTO implies that intermediate diaphragms are necessary, several research papers have concluded they are not required. References are cited in Chapter 3, Section 3.7. The cost to construct and install forms and reinforcement for diaphragms is very high, as is the connection to the beams. Several states have eliminated the use of intermediate diaphragms without negative impact on the performance of their prestressed concrete bridges.

4.2.5.2 Steel Diaphragms

Structural steel diaphragms are usually bolted to inserts in the beams, eliminating field forming and casting expense. However, accurate detailing of the steel and placement of the inserts are necessary to ensure proper fit in the field. Connections must allow for fabrication and construction tolerances. Steel diaphragms may also be more susceptible to corrosion, resulting in higher maintenance costs.

4.2.5.3 Precast Concrete Diaphragms

Precast concrete diaphragms have been successfully used. Precast diaphragms reduce the field labor costs associated with forming and placing of cast-in-place concrete. However, as with steel diaphragms, care must be taken in the detailing and fabrication of the precast diaphragms to accommodate fabrication and construction tolerances. Connection schemes for precast diaphragms must also be carefully considered. Recent development and tests in Pennsylvania have resulted in Penn DOT acceptance of a standard for precast diaphragms (Penn DOT, 1996).

4.2.5.4 Temporary Diaphragms

For some longer spans and deeper beams, temporary intermediate diaphragms may be desirable to increase the stability of the beams prior to and during placement of the concrete deck. Typically, these temporary diaphragms are steel.

4.2.6 Prestressing

Selection of either stress-relieved (normal-relaxation) or low-relaxation strands and the size of prestressing strand has a direct impact on the cost of prestressed products. Section 7 of Chapter 2 discusses the various types of prestressing strand materials which are available. Currently, the most common strand used in beams is seven-wire, low-relaxation, Grade 270 strand. The steel used in this strand can be pulled to a higher initial stress and exhibits lower losses than normal relaxation strand.

4.2.6.1 Strand Considerations

The use of fewer strands with larger diameter is generally more cost effective than the use of a larger number of smaller diameter strands. The cost of the strand is usually not directly proportional to the area of the strand (larger strands are proportionately slightly less expensive). But even if it were, the labor to install the larger number of smaller diameter strands will almost always make the use of the larger size strands more cost effective. As concrete design strength increases, the use of larger strands and their associated larger forces becomes more desirable. The use of larger strand enables the designer to place a larger prestressing force at the same eccentricity as the same number of smaller strands. This will increase the capacity of the beam. Using a lesser number of larger strands may also reduce congestion and facilitate concrete placement.

Designers are urged to avoid using more strands or prestressing force than required by design. Excessive strand is costly and can significantly increase camber.

Beams may be designed with strands having either a straight or harped trajectory.

4.2.6.2 Harped Strands

Very often, some of the prestressing strands are placed in a harped (deflected or sometimes draped) profile along the length of the beam. By harping the strands, designers are able to place the strands at the lowest position at midspan where the positive moment is largest, but raise the center of gravity of the prestress force near the end of the beam where the moments are reduced (see **Figure 3.3.2.4-1**). Raising the strands reduces the eccentricity and therefore the negative moment associated with the prestress force. The reduced negative moment results in lower compressive stresses in the bottom of the beam and lower tensile stresses in the top of the beam near its ends. In Chapter 3, detailed information on harping strands is contained in Section 3.3.2.

4.2.6.2.1
Harped Profiles

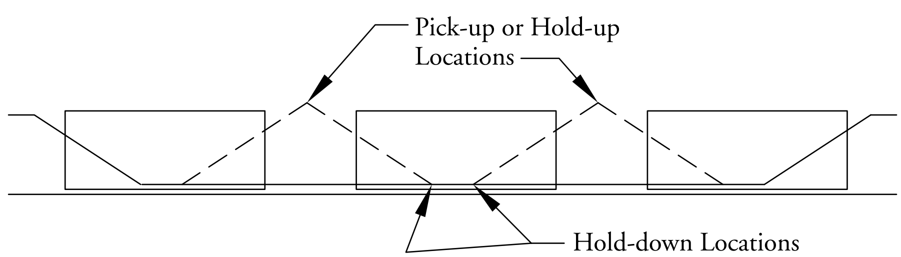
The method of achieving a harped strand profile requires the use of hold-down devices and either hold-up or pick-up devices. The location of the hold-down should be near a point that is approximately 0.4 to 0.45 of the beam length from the ends. Some designers have located the hold-down points as close to the ends as 0.3L; others have used a single point at midspan. Based on the shape of the typical positive moment envelope, the use of the 0.4L to 0.45L location may be the most appropriate choice. Use of a location closer to the end does not appear to provide increased capacity, and increases the forces in the hold-up/hold-down devices. When using a single hold-down at the center of the beam, the load transmitted to the anchorage for the hold-down sometimes becomes excessive.

4.2.6.2.2
Harping Methods

A hold-down device normally consists of rollers attached to a vertical rod, which passes through the bottom form and is anchored to the form substructure or foundation to resist the vertical component of the prestress force. The force which must be resisted by the hold-down device, and therefore its size, depends on the number of harped strands and the trajectory angle of the strands. There is a cost associated with the hold-down devices since they remain in the beam and are not reused. Additionally, when the hold-down locations along the length of the prestress bed are moved to accommodate different beam lengths, the bottom form must be patched.

Frequently, precast concrete producers use hold-up devices to raise the profile of the strand at the ends of beams and then tension the strands in their already harped profile. Others lift the harped strand to the proper elevation after tensioning the strands. Again, the number of harped strands and their angle directly influence the size and cost of the hold-up/pick-up device. **Figure 4.2.6.2.2-1** shows a typical harped strand profile in a prestressing bed. The designer can reduce the cost of the prestressed product by minimizing both the number of harped strands and the heights of the hold-up points.

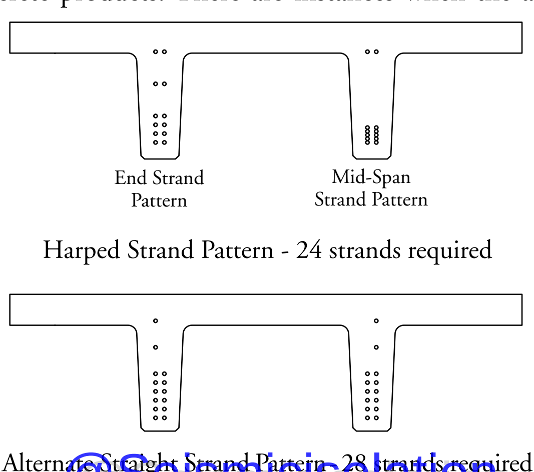
Figure 4.2.6.2.2-1
Harped Strand Profile



4.2.6.3
Straight Strands

The use of straight strand offers some advantages in the fabrication of prestressed concrete products. There are instances when the addition of a few straight strands can eliminate the need for harped strands. This option should be seriously evaluated, since the straight strand option, while using more strands, results in easier fabrication. **Figure 4.2.6.3-1** depicts a harped strand pattern and an alternate straight strand pattern. The increase in stresses due to more strands may be reduced by debonding some of the strands in the ends of the beams (see also Chapter 3, Section 3.3.2.9).

Figure 4.2.6.3-1
Straight vs. Harped Strands



4.2.6.3.1 Advantages of Straight Strands

Use of straight strands is generally less expensive than harped strands for several reasons:

- Hold-down/hold-up devices are not required
- Placement of beams within the bed is less restricted
- The stressing operation is made simpler and safer
- Detensioning operations are also simplified (hold-down/hold-up devices do not have to be released)
- Varying beam lengths will not require moving hold-down locations
- The cost of repairing the bottom form is eliminated

4.2.6.3.2 Debonding Strands

The effect of harping on stresses can be approximated by using straight strands located as required for the maximum positive moment and debonding some of the strands near the ends of the beam. Debonding is achieved by sheathing the strand in plastic tubing. By selectively debonding strands, the designer can effectively control the prestress force and eccentricity, achieving results similar to harping strands.

4.2.6.3.3 Limitations of Straight Strands

When increasing the number of strands, it may become necessary to increase the release strength and/or the final design strength of the concrete in order to resist the larger compressive force. Disadvantages of using debonded strands include the elimination of the vertical components of the prestressing force which may result in a slight increase in shear reinforcement. Design effort may be increased to determine proper debonding patterns, shear reinforcement and camber. Designers should consult precast producers in the project area to determine strand harping capability and debonding preference.

4.2.6.4 Strand Spacing

The *Standard Specifications* currently require that strands be spaced, center-to-center equivalent, no closer than four strand diameters. Several research projects have demonstrated that closer strand spacings do not adversely affect bond between concrete and strand. In fact, some states have successfully used smaller strand spacings for many years. Most plants have constructed stressing headers that provide for a strand grid spacing of 2 inches. Before adjusting strand spacing, it should be determined whether the change will require the producer to adjust plant equipment. Designers should consult producers in the geographic area of the project to determine strand patterns and configurations being used (see Chapter 3, Section 3.2.2.3).

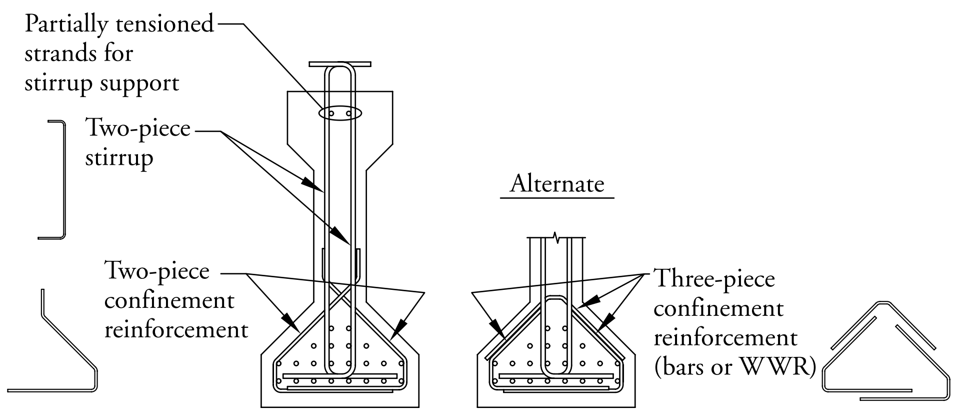
4.2.7 Nonprestressed Reinforcement

Proper detailing of mild reinforcing steel offers the designer an important opportunity to contribute to cost savings. As discussed in Chapter 3, the reinforcing steel is generally placed within the beam after the strands have been tensioned.

4.2.7.1 Detailing for Ease of Fabrication

If the reinforcement is detailed closed around the strands, it requires that the strands be threaded through the closed bars. By using two-piece bars that can be placed after the strand is tensioned, the fabrication process is simplified. **Figure 4.2.7.1-1** illustrates two-piece stirrups and two-piece confinement reinforcement in an I-beam. When specifying concrete cover and spacing of strands and bars, the designer must consider reinforcing bar diameters and bend radii to avoid conflicts. In order to support reinforcing steel located in the tops of some beams and the stirrups in all beams, some producers may prefer to locate one or two strands near the top of the beams (see **Figure 4.2.6.3-1**). Some support could be provided by longitudinal reinforcing bars, but strand is slightly less expensive than mild steel reinforcement and is readily available at precast plants. This strand may be fully tensioned (if considered in the design), or tensioned to a force of 5,000-10,000 lbs. The producer can then tie the reinforcement to the strand which will provide firm support.

Figure 4.2.7.1-1
Multi-Piece Reinforcement



4.2.7.2
Excessive Reinforcement

Minimize the amount of reinforcing steel in prestressed concrete members. There appears to be a tendency to add more reinforcement than is needed "just to be safe." Often, the added reinforcement merely creates congestion making consolidation of the concrete difficult without contributing significantly to the structural strength or behavior.

4.2.7.3
Welded Wire Reinforcement

Welded wire reinforcement (WWR) can be a very cost-effective way to place mild reinforcing steel in precast, prestressed components. WWR is a prefabricated reinforcement consisting of parallel, cold-drawn wires welded together in square or rectangular grids. Each wire intersection is electrically resistance-welded by a continuous automatic welder. The use of WWR is particularly advantageous where large areas have uniform reinforcing spacings, such as flanges of double tees and web shear steel in beams. Although the material cost of the WWR is normally greater than that of reinforcing bars, cost of installation will normally be substantially less. An example of WWR details for a precast concrete I-beam is shown in **Figure 4.2.7.3-1** for the Nebraska University (NU) metric beam section.

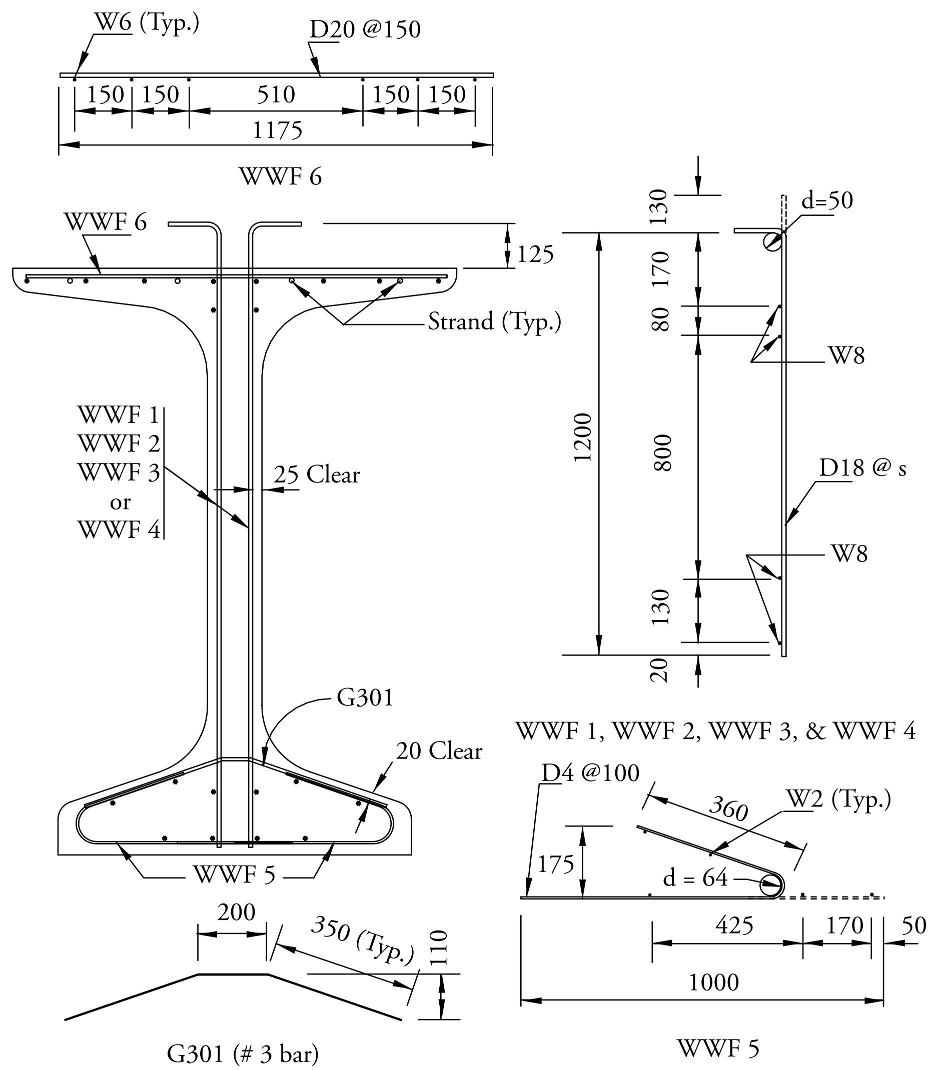
4.2.8
Durability

Prestressed concrete products have an excellent durability record. Review of data in the National Bridge Inventory compiled by the Federal Highway Administration has confirmed the performance of precast, prestressed bridges in all regions of the country. There are several reasons for this excellent record.

4.2.8.1
Benefits of the Fabrication Process

Most prestressed concrete products are fabricated in certified manufacturing plants where strict quality control is maintained. The quality of the concrete is exceptional, and it generally has a higher density and strength than field-placed concrete. Curing procedures, especially those during the first several hours after the concrete is cast, contribute to higher concrete quality. The concrete is almost always maintained in compression due to prestressing, and is therefore essentially crack free. These factors reduce the penetration of water and chloride ions into the concrete, increasing its life. In addition, many precast plants use heat to accelerate curing of the concrete. Recent tests have shown that this further increases the concrete's ability to resist chloride penetration (Pfeifer et al, 1987 and Sherman et al, 1996).

Figure 4.2.7.3-1
Welded Wire Reinforcement Details used by Nebraska Department of Roads



Fabric Designation	WWF1	WWF2	WWF3	WWF4
Spacing, s (mm)	50	100	200	300

Notes:
Dimensions in mm
1 inch = 25.4 mm
D18 designates a deformed wire whose area is 0.18 in.^{2}
W designates a smooth wire

4.2.8.2
Additional Protection

Additional measures can be taken to further enhance the durability of prestressed concrete. Chapter 2 discusses several measures that can be taken to enhance the material properties of the concrete, e.g., using low water/cementitious materials ratios and certain concrete additives. Providing the proper concrete cover over the reinforcement is essential, but excessive cover does little to enhance durability of the product. If the ends of the precast product are not encased in cast-in-place concrete it is important to seal or coat exposed prestressing strand and mild reinforcing steel with an appropriate coating.

4.2.9
Bearing Systems

Bearing systems for precast, prestressed concrete products can be very simple. The bearings need to be designed to transfer the design vertical and horizontal forces to the substructure.

4.2.9.1
Embedded Bearing Plates

In most cases, embedded bearing plates are not needed. If large horizontal forces, such as seismic loads, must be transmitted from the superstructure to the substructure, bearing plates may be necessary on some beams. Beams erected on a steep grade may also need bearing plates. In lieu of costly tapered bearing plates, elastomeric bearing pads placed directly between the precast product and the substructure are commonly used.

4.2.9.2
Bearing Devices

Elastomeric bearing pads are very economical. The bearing pad must be properly designed to accommodate the bearing pressure and the volumetric changes in the superstructure. If necessary, laminated pads can be used, but they cost substantially more than plain pads. Tapered bearing pads have been used in several places to accommodate roadway grades of up to 5%. These pads are more expensive to manufacture than flat pads, but much less expensive than tapered plates. For shallow grades, many states slope the concrete cap at the bearing to provide full contact between the bearing pad and the cap/beam. Pot bearings have been used in conjunction with bearing plates on precast products, but their expense must be carefully considered. They are normally not recommended.

4.2.9.3
Bearing Replacement

Provision for future replacement of bearing devices may be required in some locations. This requires the designer to provide a suitable and practical means for raising the superstructure for removal and replacement of the bearing device. End diaphragms, when used on bridges, can often be designed and detailed to serve this purpose.

4.2.10
Concrete Compressive Strengths

Concrete strength requirements can significantly affect costs. Strength required at release of prestress force is likely to be a predominant concern to the producer. Precast concrete plants rely on daily use of the prestressing beds. Therefore, the concrete strength at release of prestress should be kept to the minimum required to stay within allowable temporary stresses. Local fabricators are the best source of information on details related to optimum concrete strength.

4.2.11
Lightweight Concrete

Lightweight concrete has been successfully used on many bridges in the United States since the early 1950s. Its earliest applications were in lightweight concrete deck slabs. Lighter weight beams can and could allow longer spans or greater beam spacings for the same strand and concrete strength. Lightweight concrete use has become more popular in seismic areas where reductions in weight will reduce seismic forces transmitted to the substructure elements, resulting in substantial savings.

4.2.11.1
Material Properties

Concrete strengths of structural-grade ESCS (expanded shale, clay and slate produced by the rotary kiln method) lightweight aggregate concrete are in the same range as those for normal weight concrete with the same cementitious materials content. Contact a local producer of ESCS aggregate for assistance with mix designs. The modulus of elasticity for a lightweight concrete will be significantly less than that of a normal weight concrete with same strength. For detailed material properties, refer to ASTM STP 169C (1995). Obtaining concrete strengths in lightweight concrete comparable to the commonly used strengths of normal weight concrete is not difficult. Creep, shrinkage and deflection must be appropriately evaluated and accounted for when lightweight concrete is employed.

4.2.11.2
Major Bridges with Lightweight Concrete

There are many notable bridges constructed with lightweight concrete. Some of these include:

- Suwanee River Bridge on U.S. Route 19 at Fanning Springs, Fla. Built in 1964 with Type IV AASHTO I-beams, it uses 5,000 psi lightweight concrete at 120 pcf to achieve six, 121-ft spans. These were constructed in three, 2-span continuous units.
- Chesapeake Bay Bridges near Annapolis, Md.
- Napa River Bridge on State Route 29 near Napa, Calif. This is a segmental, prestressed concrete bridge 2,230-ft long with 250 ft spans. It was constructed in 1978.
- Sebastian Inlet Bridge over the Indian River, Fla. Approach spans are 73 ft long and main spans are 100, 180, 100 ft long. A drop-in I-beam of lightweight concrete, 72 in. deep, is supported by 2 cantilevered pier beams. Built in 1964, the cast-in-place deck, curbs and parapets are also lightweight concrete.
- Full-depth deck panels of lightweight concrete were used on the Woodrow Wilson Bridge in Washington, DC, and the Governor Nice Bridge on Maryland Route 301 over the Potomac River.

4.2.12 Touch Shoring

Touch shoring is a technique that has been used to further extend the capacity of precast, prestressed concrete beams. The process is to provide proper temporary supports during construction to carry a predetermined portion of the weight of the cast-in-place concrete deck when it is cast. After curing of the deck slab concrete, the temporary shoring is removed and the slab weight is transferred to the composite system rather than the prestressed beam alone. This additional capacity of the beams provide for wider beam spacing or longer span lengths compared to a similar unshored system.

4.2.12.1 Example Project

In 1988, touch shoring was used for the main span carrying twin structures of the Florida Turnpike over I-595 in Ft. Lauderdale. For this project, a Type V I-beam, which normally is limited to simple spans of approximately 135 ft, was used for a 150 ft span. This scheme was used in lieu of a spliced beam system and saved over $100,000.

4.2.12.2 Limitations

The drawbacks of the touch shoring system are additional cost of the temporary support and the sensitivity of the system to possible shoring settlements during construction. Touch shoring should be utilized cautiously, with proper attention given to the temporary support design and construction. Subsequent deck replacement will also require specific design and construction provisions; this may be a deterrent to the use of touch shoring in some applications.

4.3 PRODUCTION

Several decisions made by designers can affect production costs adversely. Specific topics include concrete finishes, aesthetic requirements and elements projecting from beams. Refer to Chapter 3 for detailed discussion of precast, prestressed concrete product manufacture.

4.3.1 Beam Top Finish

If the precast product is to be covered with a concrete topping, the top surface of the precast member should be intentionally roughened. This can be done by using a rough float, heavy broom or raked finish to provide a proper bonding surface for the cast-in-place concrete. If this concrete topping is to act compositely with the beam, the designer should provide for the proper volume of mild steel reinforcement extending from the top of the beam into the deck. However, the projection of this steel should be kept to the minimum required since it interferes with the leveling and finishing of the top of the beam. If stay-in-place (SIP) concrete panels are to be used for a deck, a smooth edge of an appropriate width should be provided as a bearing surface for the SIP panel supports.

4.3.2 Side and Bottom Finishes

Precast, prestressed concrete products used as bridge components are normally cast in steel forms. The resulting finish is typically excellent. However, as with all concrete products, there can be minor blemishes or voids which are generally not considered to be defects. Major flaws in the finish should be repaired. Since bridges are usually viewed from some distance, minor surface flaws cannot easily be seen, especially on interior beams. A requirement to eliminate all minor blemishes in these surfaces adds unnecessary cost to the products. It may be desirable to provide special treatment only to products on the exteriors of bridges. Although costly, the aesthetic qualities of bridges have been enhanced through the use of exposed aggregate concrete and special form liners to create distinctive designs or finishes.

4.3.3 Appurtenances

It is sometimes necessary to connect appurtenances to the surfaces of precast units. To reduce the cost, it is necessary to eliminate projections from the beams. Most precast, prestressed concrete members are cast in precision-made steel forms. Projections can be accommodated only by modifying the forms. It is better practice to utilize details that permit attachment through use of threaded inserts, weld plates, or through bolts, as shown in Chapter 3, Section 3.2.4.

4.4 DELIVERY AND ERECTION

Transportation of precast, prestressed concrete bridge products to the bridge site can represent a significant portion of the construction cost. The transportation system from the plant to the site and the means for erecting the product at the bridge must be considered in the design.

4.4.1 Transportation

Construction of bridges over navigable waterways normally makes product delivery by barge possible. Inland bridges will necessitate delivery of components by truck or rail.

4.4.1.1 Water Delivery

Manufacturing plants located on waterways which are also accessible to the project site can load products directly on barges for delivery. When direct delivery by barge from plant to jobsite is possible product weight is a relatively minor concern, since it will be limited only by barge capacity and plant and erection crane sizes. Direct delivery by barge will usually be more economical than overland delivery.

4.4.1.2 Truck Delivery

When shipping overland, several issues will affect the cost. The most dominant consideration is product weight. Smaller products (up to 45 tons) will normally not require special equipment or permits for shipping. Larger components may require special trailers with multiple axles, dual steering systems and load distribution systems to reduce and equalize the loads to the axles. These larger components may also require the shipping agency to obtain special permits for hauling over highways and bridges. Arrangements for lead and trail vehicles and coordination with local traffic control agencies may be required. Evaluation of the highway between the bridge site and precast plant should include horizontal and vertical geometry limitations and capacity of bridges which must be crossed. Additionally, the contractor must provide adequate access to the bridge site by furnishing a suitable haul road. The haul road must be sufficient to support the loaded weight of the truck and be relatively smooth so as not to induce excessive twisting in the precast members.

4.4.1.3 Rail Delivery

Another mode of transportation for finished products is by rail. Rail transport may be especially advantageous for heavy products where rail access is available at both the precast plant and jobsite. Placement limitations of loads on the rail cars, as well as load capacities of the cars themselves may also determine the feasibility of rail shipment.

Short products may be accommodated on one car. Long products may require several cars to be attached into a "set" which will carry a single product. If more than one car is used to carry a product, special attention must be given to the support bolsters on the cars to provide for horizontal rotation. The products must be tied down well in all directions to overcome significant transportation-induced loads. Rail shipment should always be coordinated with precast producers and the railroad.

4.4.2 Handling and Erection

Generally, precast plants have cranes and other equipment for handling products in the plant. At the bridge site, the contractor must have the necessary crane(s) to provide adequate lifting capability at the required working radius. Unstable soil conditions may necessitate the use of mats for crane support. Longer beams may require special handling or a supplemental bracing system to provide proper lateral stability during lifting and shipping. Environmental constraints may require that special techniques be used for erection of precast components. For long or heavy precast products, the designer should discuss shipping and erection methods with both producers and contractors during the design phase.

4.4.2.1 Lifting Devices

For most precast products, the producer will provide means for attaching the precast component to the crane. Usually, the producer will use loops of prestressing strands embedded in the concrete. This is often the most cost effective lifting device. Other specialty lifting devices may be required, but the producer should be allowed to select the means of handling the product.

4.4.2.2 Support and Lift Locations

When prestressed concrete products are resting on supports, it is usually desirable for the supports to be located near the ends of the product. Long prestressed piles may require several points of support and lifting. The location of the lifting points must consider the stability of the product. It may be desirable to locate the lifting device some distance from the ends of long slender members. The bending stresses associated with the resulting cantilevers must be considered when locating lifting points more than 2 to 3 ft from the ends. Chapters 3 and 8 discuss this topic in detail. Designers should consult local fabricators to determine the preferred method of providing stability while maintaining stresses within acceptable limits.

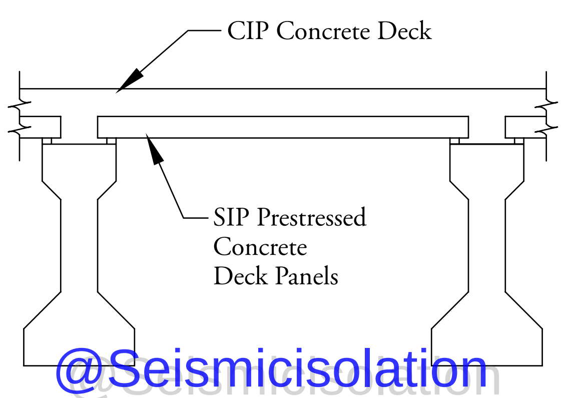
4.5 OTHER PRODUCTS

In addition to using precast, prestressed concrete beams, designers can further increase the cost effectiveness of their designs by considering the use of other manufactured concrete components for bridges. Chapter 16 contains more detailed descriptions of these products and their applications.

4.5.1 Stay-in-Place Panels

Cast-in-place (CIP) concrete bridge decks are used on a variety of bridge superstructures and usually require the use of forms. Stay-in-place (SIP) composite concrete deck panels, schematically shown in **Figure 4.5.1-1,** offer several advantages over the use of traditional removable form systems or SIP metal forms. Since the lower portion of the deck (SIP panel) is prestressed, all of the advantages associated with plant-cast concrete are made available to the deck. The deck durability is enhanced

Figure 4.5.1-1
Typical Deck Configuration with SIP Concrete Panels



since the SIP panel is virtually crack free. The SIP concrete panel is not subject to the corrosion susceptibility of the metal SIP form. Cost advantages result from the elimination of the bottom mat of reinforcing in the deck and a reduction in the volume of concrete which must be field cast. Field labor is not required to remove the forms after the deck cures. For further information, refer to "Precast Prestressed Concrete Bridge Deck Panels" (1988) published by PCI.

4.5.2 Full Depth Precast Decks

In addition to using precast concrete as deck forms, full depth precast bridge decks have been used successfully on many projects. The main advantages associated with this type of construction are the speed with which the deck is placed, and the previously enumerated benefits that are associated with plant-cast concrete. Connection of this type of deck to the beams and connections between the individual deck units must be properly designed to include bearing of the slab on the beams as well as proper shear transfer if composite action is desired. A special report, "Precast Bridge Deck Design Systems" by Biswas (1986), discusses the use of this product in detail.

4.5.3 Precast Substructures

Economic designs of bridge substructures can be achieved using precast components, especially when there is the possibility of form reuse. The precast components are generally simple to form and fabricate. Precast substructures have been successfully used on both large and small bridges.

4.5.3.1 Advantages of Precast Substructures

Increased speed of construction can decrease costs through reduced traffic maintenance requirements, enhanced safety and reduced overhead for the contractor. For construction over water, using smaller crews working less time not only reduces labor costs, but can significantly decrease workman's compensation expenses. Plant-cast concrete will normally be of high quality, and, hence, very durable.

4.5.3.2 Components

Precast substructure components include prestressed concrete piles, abutment walls, caps for pile bents, pier columns and caps. Piles are normally prestressed to resist the stresses that result from driving. The other components listed are normally reinforced with mild steel. Pile bents with prestressed piles and concrete caps have been used in lieu of piers, especially for short span bridges. Precast bent caps are very simple to fabricate and have been used widely. For grade crossings, precast pier caps eliminate the need for erecting and removing expensive form work, installing the reinforcing cage and curing the cap at an elevation above grade. Major bridges successfully built using precast columns and caps include the Sunshine Skyway Bridge in Tampa Bay, Fla, and the Edison Bridge in Ft. Myers, Fla, shown in **Figure 4.5.3.2-1**.

Figure 4.5.3.2-1
Edison BridgeFt. Meyers, Fla, showing precast concrete columns and caps

4.5.3.3 Connections

A primary concern for designers of economical precast substructures is to provide effective and durable, yet reasonably simple means of connecting precast components to other precast and CIP components. The connections between precast elements must be designed and detailed for full transfer of all applicable forces. Bent caps normally provide a socket in the cap into which the piles are set and subsequently grouted. Other connection schemes use reinforcing bar splices such as mechanical splices, or grouted sleeves, and post-tensioning.

4.5.4 Barriers

Precast concrete railings or barriers are frequently used as bridge components. Cast-in-place railings are normally cast independent of the bridge deck requiring separate delivery of concrete. Precasting the railing or barrier eliminates this requirement and speeds the construction process. Barriers have been attached to bridges by bolted connections or with the use of bar splicing devices and mechanical anchors.

4.6 ADDITIONAL CONSIDERATIONS

When compared to other bridge systems, the direct cost of precast concrete components alone can be significantly less. There are other benefits that can be achieved with the use of specific products or materials.

4.6.1 Wide Beams

Over the past several years, the use of precast, prestressed concrete beams with wide top flanges has grown in use. The increased width provides a smaller area requiring deck forming, probable reduction in the amount of deck reinforcing steel, improved lateral stability for handling and shipping longer beams and a wider work surface for construction crews prior to installation of deck forms. Excessive width may, however, increase the volume of haunch concrete over the beam and, for very thin flanges, increase the difficulty of deck replacement.

4.6.2 Adjacent Members

By placing precast concrete beams side-by-side, the need for a CIP concrete deck may be eliminated, further reducing the cost of construction. This is especially beneficial at remote construction sites where transporting concrete to the site is difficult or too time consuming. Cost savings related to the deck include forming, placing, finishing, curing, form stripping, and the material and delivery expense. By eliminating the deck, total construction can be completed in significantly less time.

4.6.3 High Strength Concrete

The use of higher concrete strengths has been increasing. With the higher strength, comes the ability to increase the span length for given beam depths and the associated economy of longer spans. These longer spans are accompanied by increases in the amount of prestressing force in the products. Designers must take into account the potential increase in beam camber and also increased concrete release strengths that could preclude casting on a daily cycle. The ability of prestressing beds to withstand the larger prestress force should also be investigated. The stability of long, slender members during handling and shipping must be considered as part of the member design. Precast producers in most areas are familiar with these parameters and can provide assistance.

4.6.4 Contract Considerations

During the planning phase of projects, agencies should evaluate contract procedures and use one that gives the best opportunity to save money. When a number of small bridges are to be constructed or replaced in one area, significant savings can be realized by grouping several bridges in one contract.

4.7 SUMMARY AND REFERENCES

4.7.1 Summary

There are several keys to the economical use of prestressed concrete for bridges. These include proper design and detailing, local availability of products, repetitive use of products and open communications between designers, contractors and manufacturers starting with the concept of the design through final construction. As noted several times in this chapter, designers should contact local precast, prestressed concrete fabricators to obtain information vital to the design of a cost-effective structure.

4.7.2 Cited References

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PRELIMINARY DESIGN

f'_c = compressive strength of concrete at service

f'_{ci} = compressive strength of concrete at time of initial prestress

@Seismicisolation

Preliminary Design

6.0 SCOPE

Preliminary design is usually the first step in designing an economical precast, prestressed concrete bridge. This chapter discusses the preliminary plan, superstructure and substructure considerations, foundations, and member selection criteria with design aids and examples. Additional information is given in Chapter 4, "Strategies for Economy."

6.1 PRELIMINARY PLAN

6.1.1 General

The preliminary planning process consists of collecting and analyzing site information, applying established policies and practices, and considering alternatives including cost evaluations, for the purpose of providing the bridge that is the most cost-effective and the most functionally, structurally and aesthetically appropriate. The preliminary plan lays the groundwork for the final bridge design. It specifies the structure type and is the basis for the design schedule estimate and construction cost estimate.

6.1.2 Development

The preliminary planning process begins with bridge site data. Preliminary studies (such as type, size and location (TS&L) studies), geometric data, foundation data and hydraulic data are reviewed. Preliminary geometric approval is received. Struc-ture alternatives are evaluated considering such details as length, type, geometric constraints such as vertical and horizontal clearances, span arrangement, staging, falsework, substructure requirements, environmental and community issues, and costs. Plan, elevation and section views are developed and approved. Cost estimates are prepared. The preliminary plan and cost estimate are approved prior to beginning final design.

6.1.3 Factors for Consideration

A number of factors should be addressed at the preliminary design stage.

6.1.3.1 General

Funding classification (for example, state funds, federal and state funds, local funds) and available funding level should be determined. Environmental concerns include site conditions (for example, wetlands or environmentally sensitive areas) and mitigating measures.

6.1.3.2 Site

Site requirements that should be determined include topography, horizontal alignment (curves and skews), required clearances, vertical alignment and limits, superelevation, and existing and proposed utilities. Safety considerations include sight distances, horizontal clearance to piers, and hazards to pedestrians.

End slopes are controlled by soil conditions and stability, right-of-way availability, fill height or depth of cut, roadway alignment and functional classification, and existing site conditions.

6.1.3.3 Structure

Structural considerations include foundation and groundwater conditions, requirements for future widening, and anticipated settlement. Aesthetics, including general appearance, level of visibility and compatibility with surroundings and adjacent structures should be evaluated. Railroad separations may require negotiations with the railroad company concerning clearances, geometry, utilities, drainage and provision for maintenance roads.

The total length of the bridge is based on horizontal and vertical clearances to roadway(s) or rail(s) below or above, or hydraulic studies if over water, and/or environmental concerns or other restrictions as set by the owner agency. The bridge width is typically controlled by the geometry of the approaching roadway. The span arrangement is controlled by such factors as:

- Allowable girder depth due to clearance requirements
- Placement of piers in waterways
- Horizontal clearance between supports and rights-of-way below
- Economic ratio of end span to interior span

Considering the ratios of spans, the following have been found to produce a balanced design, where the reinforcement requirements for end spans are comparable to those for interior spans:

End span/interior span	Condition
0.95	Simple span for girder and deck weight, continuous span for all other loads
0.80	Simple span for girder weight, continuous span for all other loads

As previously discussed, bridge details are largely dictated by obstructions above and below ground, maximum span limitations, and required abutment locations. However, to the extent possible, large skews, steep profile grades, sharp horizontal curves and differing span lengths should be avoided. Slightly lengthening the bridge may be preferable to using an extreme skew angle that tightly fits the bridge site.

6.1.3.4 Hydraulics

Hydraulic considerations include bridge deck drainage, stream flow conditions and channel drift, passage of flood debris, scour and the effect of the pier as an obstruction (for example, the pier's shape, width, skew, number of columns), banks and pier protection, permit requirements for navigation, and stream work limitations. After piers have been located, specific information on scour and backwater is obtained.

Vertical clearances for water crossings should satisfy floodway clearance requirements. In accordance with the flood history, nature of the site, character of drift, and other factors, the minimum vertical clearance (for the 100-year flood, for example) is determined. The roadway profile and the bridge superstructure depth should accommodate this clearance requirement. Bridges over navigable waters should also comply with any clearance requirements of the U.S. Coast Guard.

6.1.3.5 Construction

Construction considerations include falsework and other construction clearances, working space requirements, hauling and erection details, access to the site, construction season, and construction scheduling limitations. Safety considerations such as traffic flow, staging, detours and falsework requirements should be addressed.

Access routes should be checked and sites reviewed to ensure that the precast concrete beams can be transported to the site. Possible routes to the site should be adequate to handle the truck and trailer which are hauling the beams. Generally, the designer is not responsible for construction of the bridge. However, prudent designers always consider constructability issues. Therefore, it is recommended that both size and weight of the beams be checked and hauling permit requirements determined. The details related to erecting the beams once they reach the site also need to be assessed. The site should be reviewed for adequate space for the contractor to set the cranes and equipment necessary to lift and place the beams.

6.1.3.6 Utilities

Often, electric, water, telephone and other utility conduits are required to be supported by the bridge. Most loads imposed by these utilities, except perhaps those of large water pipes, do not have significant impact on structural design. However, aesthetics and accessibility to utility lines, as well as relocation of existing utilities, may affect the selection of the superstructure system.

6.1.4 Required Details

The preliminary plan should include, as a minimum, the following details (see **Figure 6.1.4-1**):

- Location, including highway identification, name of city or county, and major features crossed
- Total length
- Total width
- Span arrangement with expansion joint locations
- Abutment and pier type with dimensions
- Foundation type with dimensions
- End slopes, with type and rate
- Profile grade and superelevation diagram
- Horizontal alignment
- Hydraulic data
- Cross-section, including barrier type and wearing surface type
- Beam type, number and spacing
- Deck thickness and build-up dimensions, if applicable
- Minimum vertical and horizontal clearances, with dimensions
- Utilities
- Borings
- Superstructure bearing types (expansion, fixed, guided, ... etc.)
- Design method (or specification)
- Design loads

6.2 SUPERSTRUCTURE

6.2.1 Beam Layout

Redundant supporting elements minimize the risk of catastrophic collapse. A typical guideline would recommend a minimum of four beams (webs). This number allows the bridge to be repaired in phases under traffic. For roadways less than 30 ft wide, a minimum of three beams (webs) may sometimes be justified.

When establishing beam layout, deck overhangs should be limited to 0.50 times the beam spacing. In some cases, this ratio has been increased to 0.625. However, large overhangs may require more costly form erection brackets and provisions to prevent overturning of the exterior beams.

Design aids are provided at the end of this chapter to assist with superstructure system selection for preliminary design.

6.2.2 Jointless Bridges

By using integral abutments at bridge ends, long continuous jointless bridge construction is possible with prestressed concrete beams. Some proponents believe that lengths on the order of 1,000 ft are realistic with this construction method. The elimination of joints minimizes beam end deterioration from inadequate protection from leaking joints and deleterious materials, such as deicing chemicals applied to the deck. Chapter 13 has more information on integral bridges.

6.3 SUBSTRUCTURES

6.3.1 Piers

In selecting the pier type, preliminary designs should be made for various configurations to evaluate costs. The most economical pier may not be the one with the least material, but instead, the one that is easiest to form and that maximizes repetitive use of forms. This is especially true on large bridge projects.

The most commonly used pier types are illustrated in **Figure 6.3.1-1** and discussed below.

6.3.1.1 Open Pile Bents

Open pile bents are used on low-volume roads and stream crossings where the possibility of debris entrapment between piles is not likely. Open pile bents are extremely economical. This type can be readily combined with precast concrete pile caps to permit very rapid construction.

6.3.1.2 Encased Pile Bents

Encased pile bents are used in water crossings where the channel carries debris or where protection against ice is desired. This pier type is usually preferred when scour is a concern and spans are of medium length.

6.3.1.3 Hammerhead Piers

With increasing pier height, the hammerhead pier becomes more economical, since this type offers a reduction in material and forming. Hammerhead piers are sometimes used as crash walls when constructed adjacent to railroad tracks. Other types of piers may also be used next to railroads as long as sufficient crash wall requirements are provided.

Figure 6.1.4-1 Example Preliminary Plan

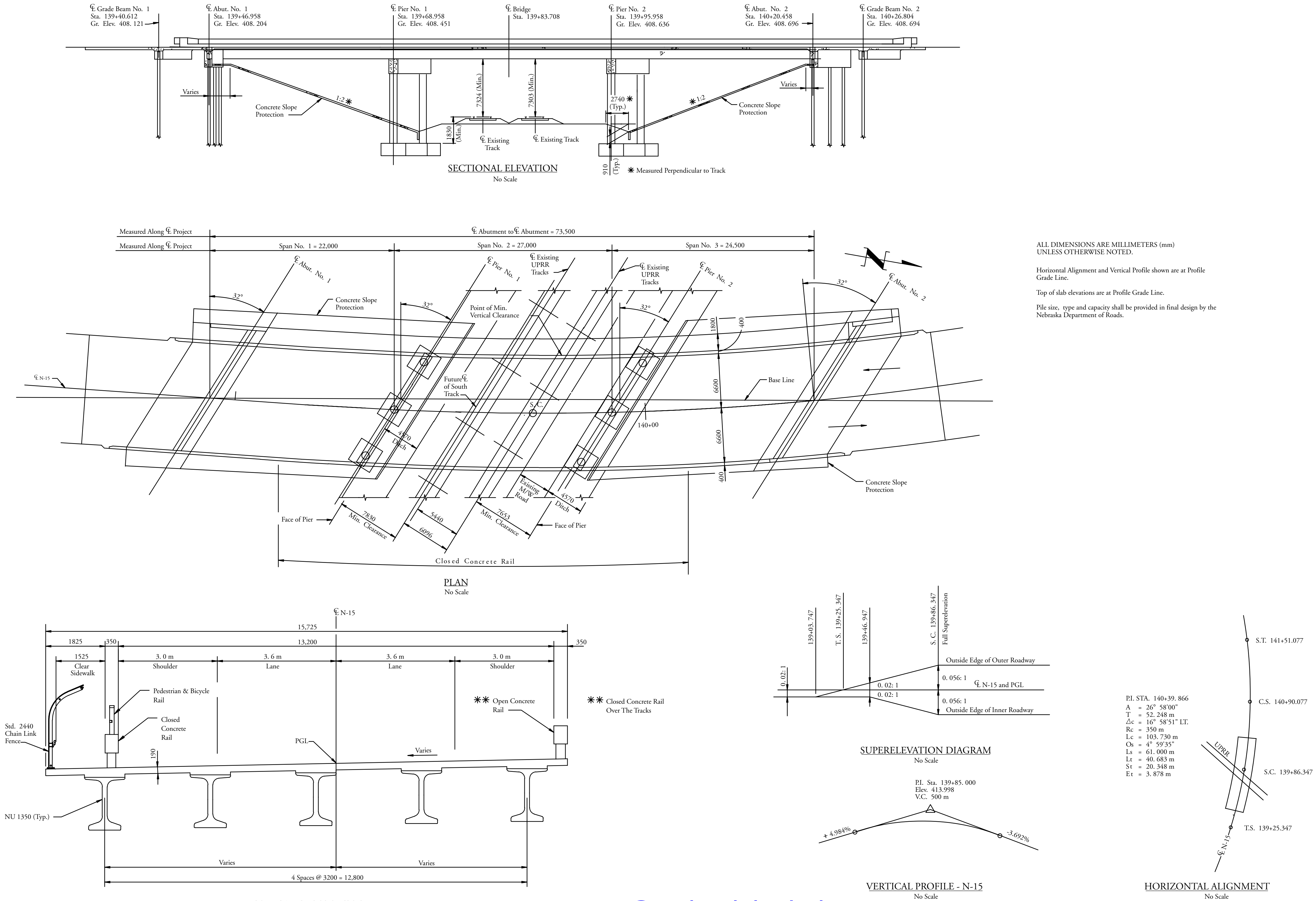
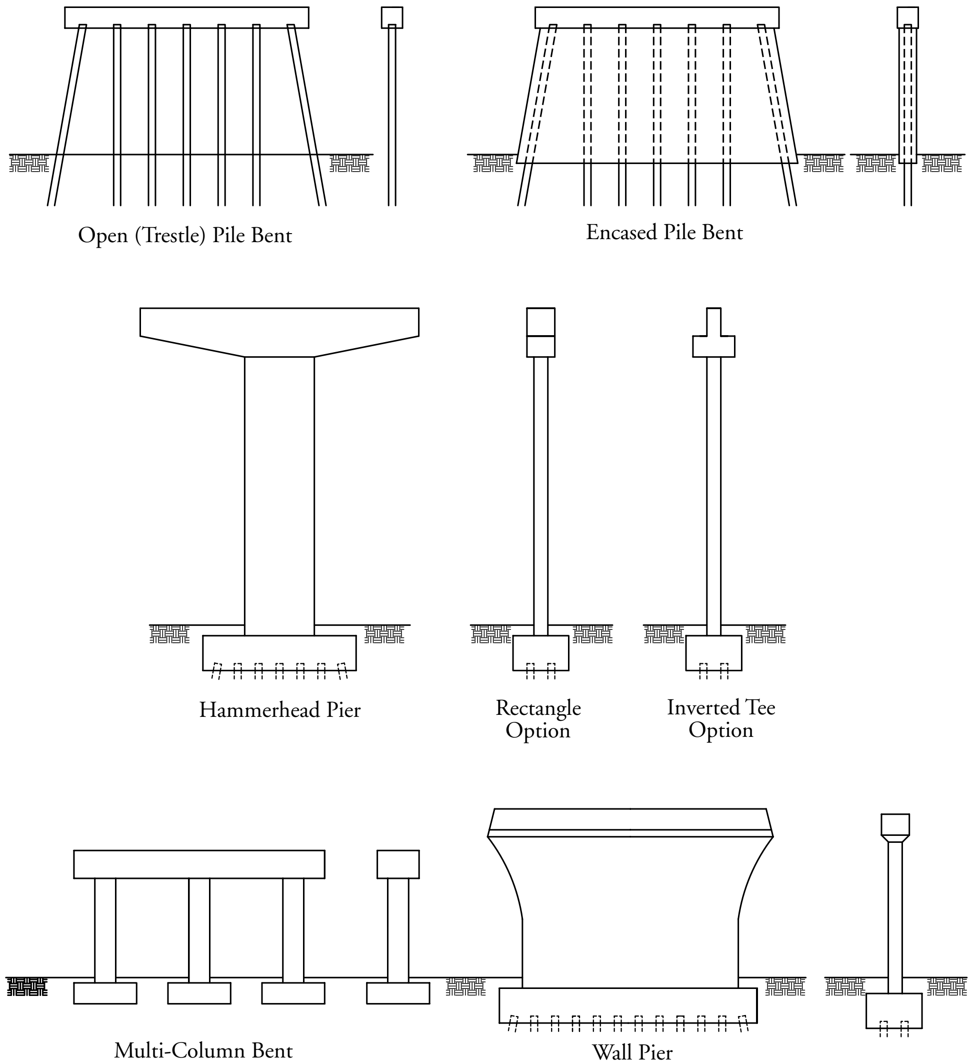


Figure 6.3.1-1
Types of Commonly Used Piers



6.3.1.4
Multi-Column Bents

Multi-column bents are sometimes referred to as rigid frame piers. Basically, this pier type is a concrete beam supported on at least two columns. It is used for wide superstructures and longer spans. Generally, a round column is the simplest and the most economical shape since forms are commercially available and require no form ties. This reduces labor considerably. Forms for this type of pier are most likely found in a typical contractor's inventory. Columns may be extensions of piles or drilled shafts.

In situations where vertical clearance is a concern, a cap shaped like an inverted tee may be used to reduce the depth of cap beneath the superstructure.

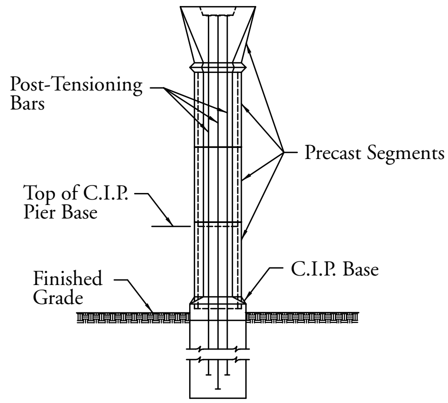
6.3.1.5
Wall Piers

Used primarily for river crossings, a wall pier is typically constructed as a combination of a solid shaft and hammerhead pier.

6.3.1.6
Segmental Precast Piers

Precast concrete segmental piers can be thin-walled hollow segments, match-cast or mass-produced with a thin mortar bed between segments. Shims can be used to maintain proper vertical alignment. The mortar bed should be designed to resist the anticipated loads, provide a thorough closure of the joint, and be designed with permissible creep and shrinkage characteristics.

Figure 6.3.1.6-1
Segmental Concrete Pier Column



Post-tensioned threaded bars are generally inserted in ducts cast in the segments and stressed. Later, ducts are grouted solid. Another alternative is the use of splice sleeves that couple reinforcing bars to provide full bar capacity. **Figure 6.3.1.6-1** shows a column designed and built with precast segments.

6.3.2 Abutments

Unlike piers, abutment types do not vary widely. The most common types of abutments are the backwall type and the integral type. For more information on integral abutments, see Chapter 13. Among the advantages of the integral type is the elimination of the deck joint, which often leaks and causes deterioration, and is therefore a maintenance problem. Integral abutments are flexible and tolerate movement caused by expansion and contraction of the superstructure due to temperature changes. It may be necessary, however, to use a backwall abutment if bridge length or skew dictate.

For precast abutment walls, full capacity may be accomplished by means of field welding of connecting steel plates, followed by corrosion protection of exposed steel.

Location of the abutments is a function of the profile grade of the bridge, the minimum vertical and horizontal clearances required, and the type and rate of end slope.

6.3.3 Hydraulics

Pier shapes that streamline flow and reduce scour are recommended. Consideration is based on the anticipated depth of scour at the bridge piers. Measures to protect the piers from scour activity (for example, riprap and pier alignment to stream flow) are recommended.

For bridges over navigable channels, piers adjacent to the channel may require pier protection as determined by the U.S. Coast Guard. The requirement is based on the horizontal clearance provided for the navigation channel and the type of navigation traffic using the channel. In many cases, piers over navigable waterways should be designed to resist vessel impact in accordance with AASHTO requirements.

6.3.4 Safety

Due to safety concerns, fixed objects should be placed as far from the edge of the roadway as economically feasible, maintaining minimum horizontal clearances to bridge piers and retaining walls.

Redundant supporting elements minimize the risk of catastrophic collapse. A typical guideline would recommend two columns minimum for roadways from 30 to 40 ft wide and three columns minimum for roadways over 40 to 60 ft wide. Also recommended is collision protection or design for collision loads on piers with one or two columns.

6.3.5 Aesthetics

The principal direction of view of the piers should be considered when determining their size, shape and spacing. The piers should be correctly sized to handle the structural loads required by the design and shaped to enhance the aesthetics of the overall structure. Column spacing should not be so small as to create the appearance of a "forest of columns." Chapter 5 discusses aesthetics in greater detail.

6.4 FOUNDATIONS

Typical foundation types include:

- Spread footings
- Drilled shafts
- Steel pipe piles
- Prestressed concrete piles
- Steel H-piles
- Timber piles

Round or square columns of multi-column bents, usually rest on single drilled shafts or on footings that cap multiple piles. Single columns usually rest on footings that cap multiple piles or drilled shafts.

Prestressed concrete piles are used extensively in the coastal regions, as well as other locations. For short bents on stream crossings, a line of piles may be extended into the cap, forming a trestle pile bent. These are economically competitive even when the soil is suitable for drilled shafts.

Prestressed piles can double as foundations and piers, thus reducing the amount of on-site forming and concreting. Precast, prestresseed concrete piles come in different sizes and shapes, ranging from 10 x 10-in.-square piles to 66-in.-diameter hollow cylinder piles.

6.5 PRELIMINARY MEMBER SELECTION

6.5.1 Product Types

The standard AASHTO-PCI sections given in Appendix B were used for development of the charts. It should be noted that neither the deck bulb-tee nor the double stemmed (double tee) sections are AASHTO standard products. However, they are included due to common use of these products in several states and due to their cost-effectiveness, particularly for bridges on secondary roads.

A number of states have their own standard products. Designers should check with their local precast concrete producers on product availability before they begin design. This manual may include similar design charts for local products which have been supplied by local or regional organizations. Until these design aids are available, the charts given here may be used as a convenient means of assessing the appropriateness of various product shapes, depths and spacings for a given span arrangement. This should, of course, be followed by detailed design incorporating locally available products.

6.5.2 Design Criteria

The design charts provided in this chapter were developed to satisfy flexure according to AASHTO *Standard Specifications.* However, use for preliminary member selection according to AASHTO *LRFD Specifications* should still be valid.

6.5.2.1 Live Loads

The live load considered for the charts is the HS25 truck which is 1.25 times the standard HS20 truck. This relatively heavy load is consistent with the practices of several states and generally produces designs that are similar to those produced by the AASHTO *LRFD Specifications.* The live load distribution factor is taken here as (spacing/5.5), the factor used in AASHTO *Standard Specifications* for I-beam systems. This factor is generally reasonable for the purpose of establishing the preliminary charts for all products. This assumption was made to simplify development of the charts for adjacent members such as box beams, deck bulb-tees, double-stemmed beams and voided slab beams.

6.5.2.2 Dead Loads

In many areas, adjacent members are constructed without a composite topping. In developing the charts, these members are assumed to have an additional dead load of 50 psf representing an overlay plus barriers and railing. Members with a cast-in-place composite topping are assumed to carry their own weight plus the topping weight as non-composite members, and 40 psf superimposed dead load as composite members. The 40 psf includes allowance for barriers, railing and 25 psf for a future wearing surface.

6.5.2.3 Composite Deck

In all cases of spread sections, an 8-in. thick composite topping (deck) is assumed to be used. Also for these spread sections, a 1/2 in. haunch is assumed placed directly over the top flange. It is accounted for in computing loads and section properties.

For adjacent sections which are recommended to have a composite topping, the topping thickness is assumed equal to 6 in. The topping weight is based on the indicated thickness. However, composite section properties were determined with the assumption that long-term wear reduces the thickness by 0.5 in. The 8-in. slab thickness should be adequate except for beam spacings larger than about 12 ft. The charts should still be valid for slightly wider spacings, say up to 14 ft, as the increased section properties should offset the increased weight.

6.5.2.4 Concrete Strength and Allowable Stresses

Except for the charts that include information on high strength concrete, concrete strength was kept the same for all charts. The precast concrete products are assumed to have $f'_{ci} = 5{,}500$ psi and $f'_c = 7{,}000$ psi, and the cast-in-place topping is assumed to have $f'_c = 4{,}000$ psi.

The allowable concrete tensile stresses are taken as $7.5\sqrt{f'_{ci}}$ at release and $6\sqrt{f'_c}$ at service. The allowable compression is taken as $0.6f'_{ci}$ at release and $0.6f'_c$ at service.

6.5.2.5 Strands and Spacing

One-half inch diameter, seven-wire, 270 ksi low-relaxation strands are used in all applications except where high strength concrete is indicated. The center-to-center strand spacing is assumed to be 2 in., although that spacing can be reduced to 1-3/4 in., according to Federal Highway Administration recommendations (1996).

All strands are assumed to have an initial tension of 202.5 ksi prior to release. Member end stresses are assumed to be controlled through debonding (shielding) and/or harping of some of the strands as needed. Prestress losses are calculated assuming 75% relative humidity. When the relative humidity is less as in several regions, prestress losses would be higher and member capacity reduced. Therefore, points on the charts above the dashed line ($f'_{ci} = 5{,}500$ psi) must be used with caution as values are highly sensitive to design assumptions.

Strand patterns used by producers vary. For the box beams in the charts in Section 6.9, two layers of strands are assumed in the bottom flanges.

6.5.2.6 Design Limits

The charts indicate whether tensile stress at service or strength controls the maximum span limit. A third controlling criterion may be compressive stress at release. If this is the case, the curves are continued until either tension at service or strength controls, with the ending point labeled with the minimum value of f'_{ci} required to allow its use. In no case was the end point allowed to have f'_{ci} greater than f'_c.

For the longer spans, camber and stability of the beams should also be evaluated.

6.5.3 High Strength Concrete

According to recent surveys, little difficulty is encountered anywhere in the country in obtaining 7,000 psi concrete on a consistent basis. Some owners and designers still specify 5,000 psi concrete, a practice which unnecessarily penalizes precast concrete bridge products.

6.5.3.1 Attainable Strengths

In recent years, higher strength concretes have been commercially achieved. The strength ranges from 10,000 to 15,000 psi. Use of such strengths is expected to increase in the future. Therefore, design charts for the AASHTO I-beams and bulb-tee beams include span capacities with concrete having f'_{ci} = 8,000 psi and f'_c = 12,000 psi. This limited coverage is intended to introduce the significant impact of using higher strength concrete on two common products. The increased span capacity should be weighed against the possible cost increase associated with producing higher strength concrete. Chapter 4 discusses many of these considerations.

6.5.3.2 Limiting Stresses

Where high strength concrete is used, the allowable tensile stress is increased to $10\sqrt{f'_{ci}}$ at release and $8\sqrt{f'_c}$ at service. Both of these limits have been justified by several recent studies. This 33 percent increase in allowable tensile stress, from 7.5 to 10 and from 6 to 8, has not been recognized by AASHTO specifications. Its impact on design is much less significant than that of the allowable concrete compressive stress, especially at release. The allowable compressive stresses are $0.6\,f'_{ci}$ and $0.6\,f'_c$ as assumed for normal strength concrete.

6.5.3.3 Larger Strands

In order to utilize the full potential of high strength concrete, it is sometimes necessary to use 0.6 in. diameter strands. These larger strands provide about 40 percent higher tensile capacity than for 1/2 in. diameter strands at only about 20 percent increase in diameter. The curves for high strength concrete were developed with 0.6 in. strands at 2 in. spacing. This spacing is in accordance with the provisions of a 1996 FHWA memorandum.

6.6 DESCRIPTION OF DESIGN CHARTS

6.6.1 Product Groups

The design charts provide preliminary design information for different products grouped into several types. These include:

CHARTS	PRODUCTS
Charts BB-1 through BB-10	AASHTO Box beams
BT-1 through BT-4	AASHTO-PCI Standard bulb-tees

DBT-1 through DBT-5	Deck bulb-tees
IB-1 through IB-7	AASHTO Standard I-beams
DT-1 and DT-2	Double-stemmed beams (double tees)
SB-1 through SB-3	AASHTO Voided slab beams

(Geometric properties for all products are given in Appendix B.)

6.6.2 Maximum Spans Versus Spacings

Within each group, the first chart, e.g. BB-1, BT-1,… etc., depicts the maximum attainable span versus member spacing for all member depths within the group. This type of chart is convenient to use in the early stages of design to identify product types, spacings and approximate depths for the span length being considered.

6.6.3 Number of Strands

The remainder of the charts within each group give the number of strands needed for specified span lengths and beam spacings. This type of information is needed to: (1) develop an estimate of the final design requirements, and (2) to determine if the number of strands needed is within the prestressing bed capacity of local producers. Otherwise, the member depth, or spacing if applicable, must be adjusted.

In developing the charts, no attempt was made to judge whether or not the number of strands given is feasible for local production. For example, in Chart IB-7 for AASHTO Type VI I-beams, only a very limited number of precast producers in the country are likely to have a prestressing bed capable of resisting 90 tensioned strands. The number of strands was strictly based on flexural stress requirements. In some cases, e.g. shallow I-beams at wide spacing, shear capacity may not meet maximum limits of the *Standard Specifications*. A complete check should be made during final design.

It should be noted that all charts were based on providing the lowest possible center of gravity of strands in the midspan section. This is accomplished by filling the first (bottom) row to capacity before any strands can be placed in the second row, and so on.

6.6.4 Controls

In certain situations, compressive strength at release controls the maximum span capacity. This is indicated in the charts by a crossing line labeled with the specified value of f'_{ci}. However, the line representing number of strands versus span is continued as a thinner line until another design criterion (usually tension at service) controls or until the required value of f'_{ci} reaches the assumed value of strength at service, f'_c.

Because of the recent increase in use of higher strength concrete, the charts of I-beams and bulb-tee beams include span capacities and number of strands required when the precast product strength, f'_c equals 12,000 psi. As can be seen, significant improvement in span capacity can be realized. Part of the improvement is due to the use of 0.6 in. diameter strands.

6.7 PRELIMINARY DESIGN EXAMPLES

6.7.1 Preliminary Design Example No. 1

Design a simple span for HS25 loading with a 95 ft design span. The total width of the bridge is 28'-0". The conditions do not allow for field forming of the concrete deck.

Referring to the preliminary design charts, the only applicable products would be adjacent box beams or deck bulb-tees in order to avoid deck forming. Using the charts, all possible solutions are summarized in **Table 6.7.1-1.**

Table 6.7.1-1 Product Options for Example No. 1[1]

Product		Depth, in.	Spacing, in.	Topping (Deck)	Number of strands	Design Chart
		35	48	No	36	DBT-2
Deck Bulb-Tee		53	48	No	20	DBT-3
		65	48	No	15	DBT-4
AASHTO	BII-36	33	36	No	27	BB-8
Box		33	36	Yes	28	BB-8
Beam	BIII-36	39	36	No	22	BB-9
		39	36	Yes	22	BB-9
	BIV-36	42	36	No	25	BB-10
		42	36	Yes	27	BB-10
	BI-48	27	48	Yes	50	BB-2
	BII-48	33	48	No	34	BB-3
		33	48	Yes	35	BB-3
	BIII-48	39	48	No	28	BB-4
		39	48	Yes	28	BB-4
	BIV-48	42	48	No	26	BB-5
		42	48	Yes	26	BB-5

Note 1. Refer to Section 6.5 for design assumptions.

From the table above, the deck bulb-tee generally requires more depth, but fewer strands. Please note that the product may not be available in all regions.

While the 36-in.-wide box beam is an option, the bridge width is not divisible by 3 ft. Further, unless weight of a single beam is a factor, wider units allow casting, transporting and installing fewer pieces. This usually results in lower cost.

Detailed Design Example 9.1, Chapter 9, has similar span, width and loading requirements. In that example, an AASHTO BIII-48 box beam was used. Considering **Table 6.7.1-1**, it is clear that a shallower section could be used, or, it could be reasoned, a lower concrete strength.

6.7.2 Preliminary Design Example No. 2

Design a simple span for HS20 loading with 120 ft design span. The total width of the bridge is 51'-0" with a cast-in-place deck slab 8-in. thick. **Table 6.7.2-1** shows the product options and the number of strands required for each product.

It is generally most beneficial to use the widest possible spacing to minimize the number of beam lines. Clearance requirements may dictate the structure depth. Assuming no maximum depth limitations, the most economical products will be the deepest in order to minimize the number of strands required. Accordingly, an AASHTO Type VI I-beam or 72 in. deep bulb-tee (BT-72) at a 9 ft spacing are recommended. However, since the bulb-tee is a lighter section and the number of strands required (47 strands) is about the same, a BT-72 at a 9 ft spacing will be a more efficient solution.

Table 6.7.2-1 Product Options for Example No. 2[1]

Products		Depth, in.	Spacing, ft	Topping (Deck)	Concrete strength, psi	Number of strands	Design Chart
AASHTO I-Beam	IV	54	6-8	Yes	12,000	34-45	IB-5
		54	6	Yes	7,000	53	IB-5
	V	63	6-12	Yes	12,000	22-52	IB-6
		63	6-10	Yes	7,000	40-68	IB-6
	VI	72	6-12	Yes	12,000	22-40	IB-7
		72	6-12	Yes	7,000	32-65	IB-7
AASHTO Bulb-Tee	BT-54	54	6-8	Yes	12,000	29-47	BT-2
	BT-63	63	6-10	Yes	12,000	22-45	BT-3
		63	6	Yes	7,000	38	BT-3
	BT-72	72	6-12	Yes	12,000	20-45	BT-4
		72	6-9.5	Yes	7,000	30-55	BT-4
Deck Bulb-Tee		53	4-6	Yes	7,000	29-48	DBT-3
		65	4-8	Yes	7,000	24-46	DBT-4
AASHTO Box Beam	BIII-36	39	3	No	7,000	36	BB-9
		39	3	Yes	7,000	43	BB-9
	BIV-36	42	3	No	7,000	40	BB-10
	BIII-48	39	4	No	7,000	45	BB-4
		39	4	Yes	7,000	50	BB-4
	BIV-48	42	4	No	7,000	41	BB-5
		42	4	Yes	7,000	45	BB-5

Note 1: Refer to Section 6.5 for design assumptions.

A deck bulb-tee can be utilized for this bridge if the product is locally available. An AASHTO box beam is also suitable if the superstructure depth needs to be relatively shallow.

Detailed Design Example 9.3, Chapter 9, has a 120 ft simple span, concrete strength of 6,500 psi and HS20 loading conditions. Referring to the above table, the BT-72 was chosen with 9 ft spacing.

6.8 REFERENCES

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6.9 PRELIMINARY DESIGN CHARTS

Chart BB-1
AASHTO Box Beams-48 in. Wide

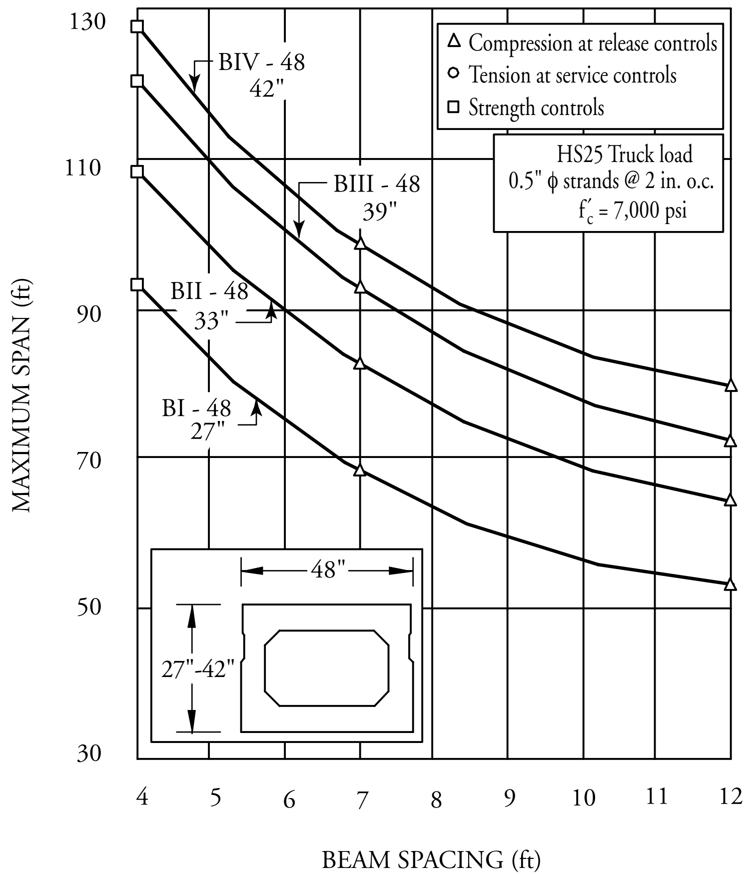


Chart BB-2
AASHTO Box Beams-BI-48

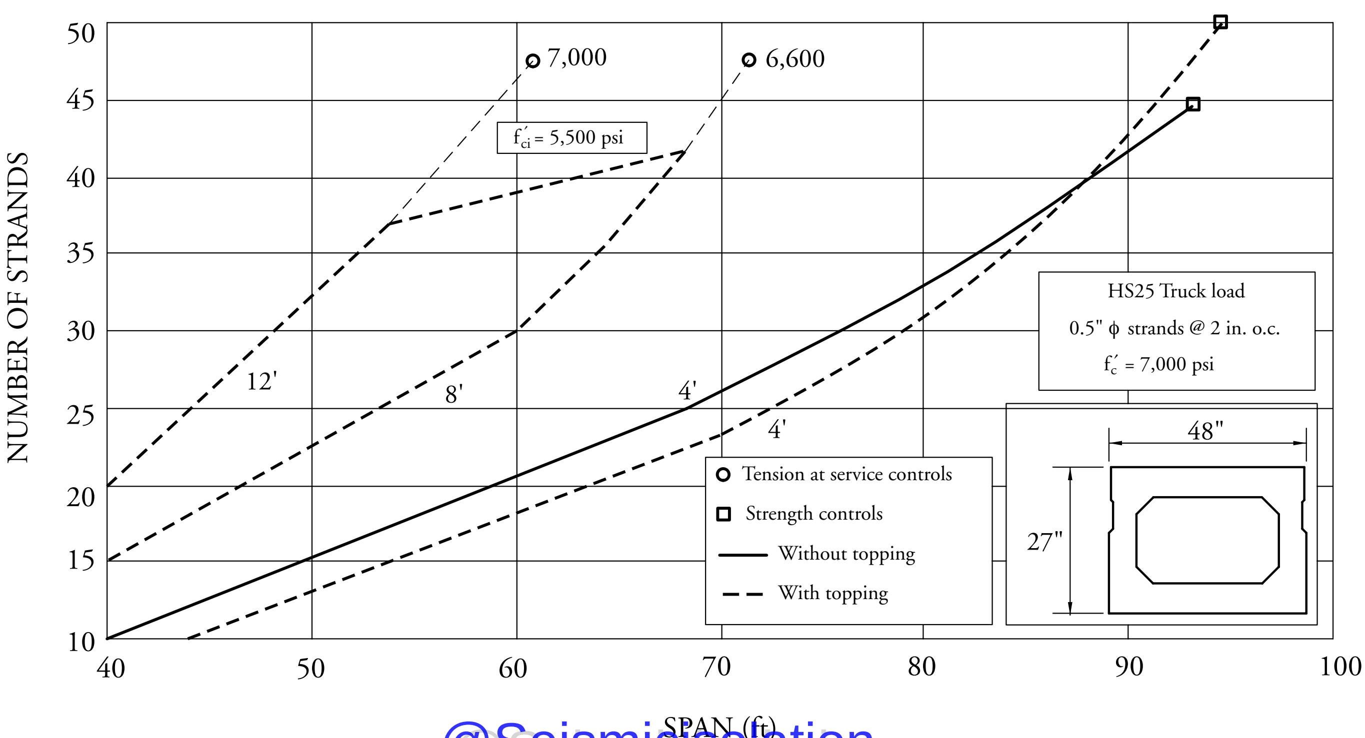


Chart BB-3
AASHTO Box Beams-BII-48

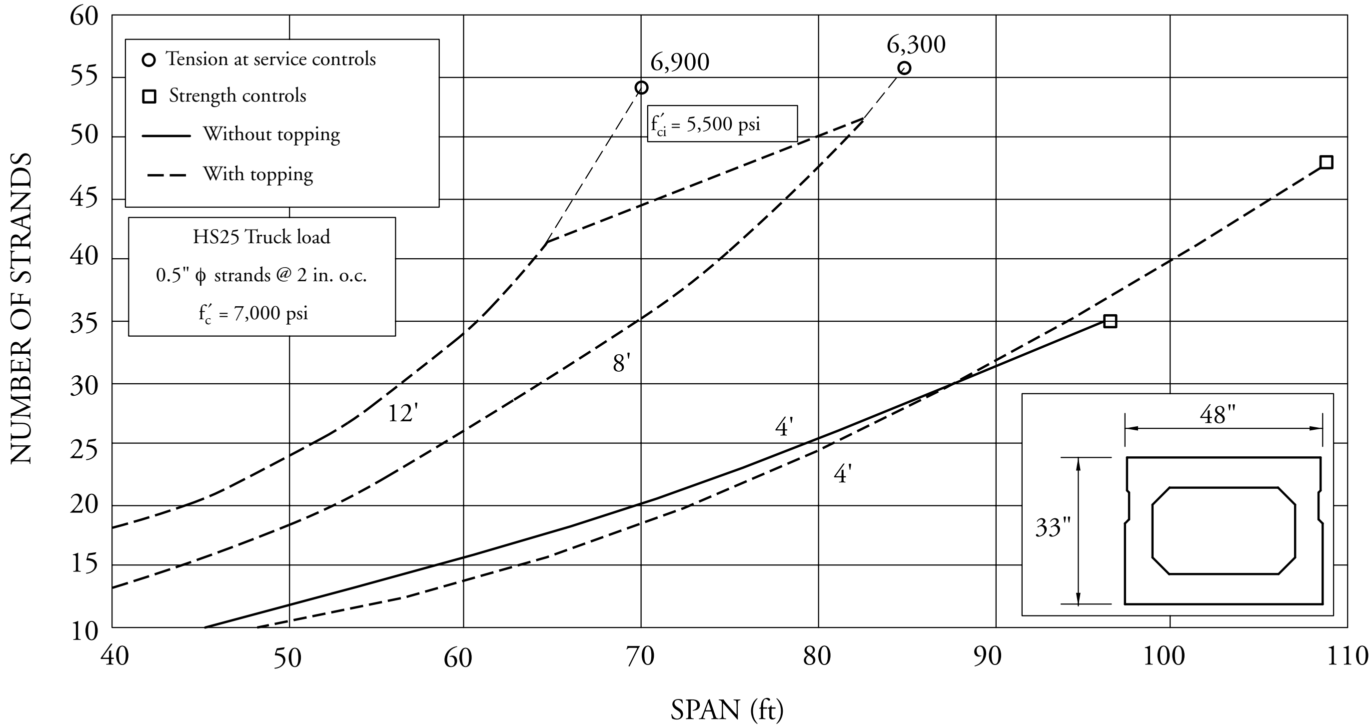


Chart BB-4
AASHTO Box Beams-BIII-48

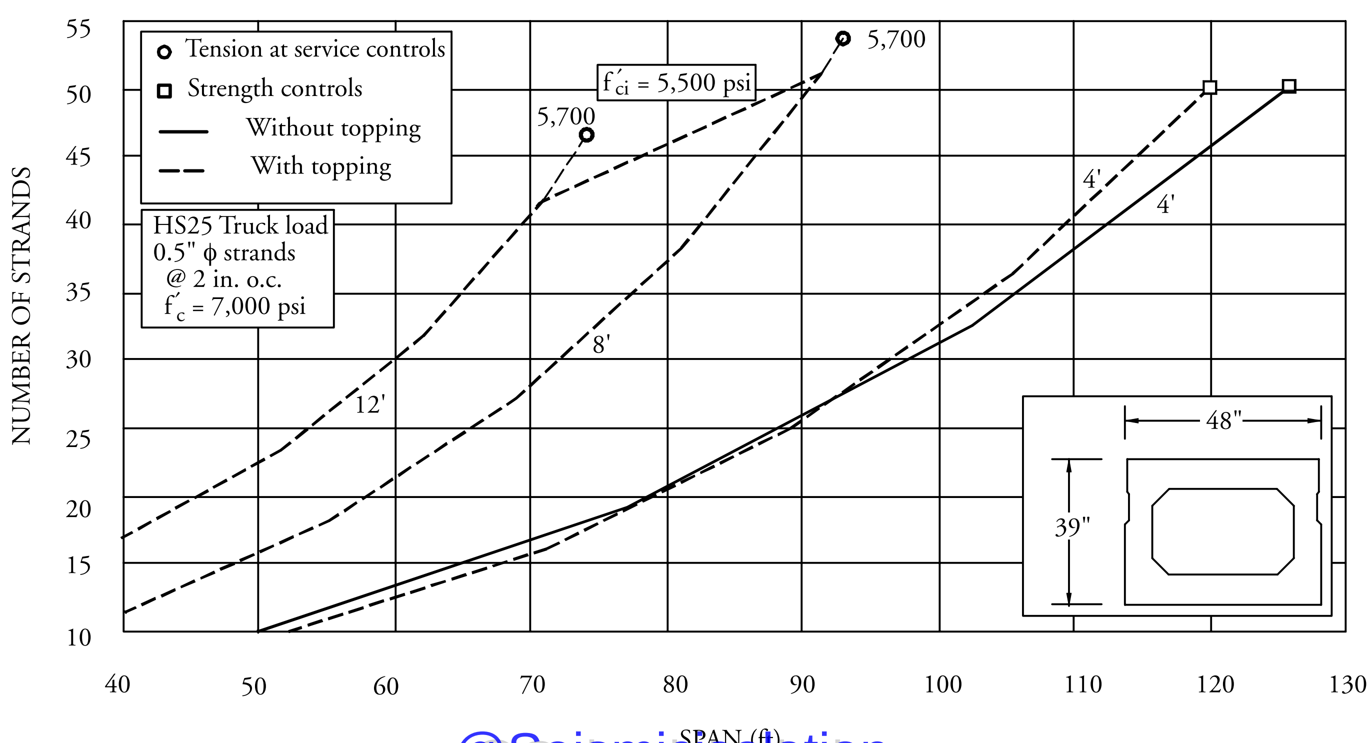


Chart BB-5
AASHTO Box Beams-BIV-48

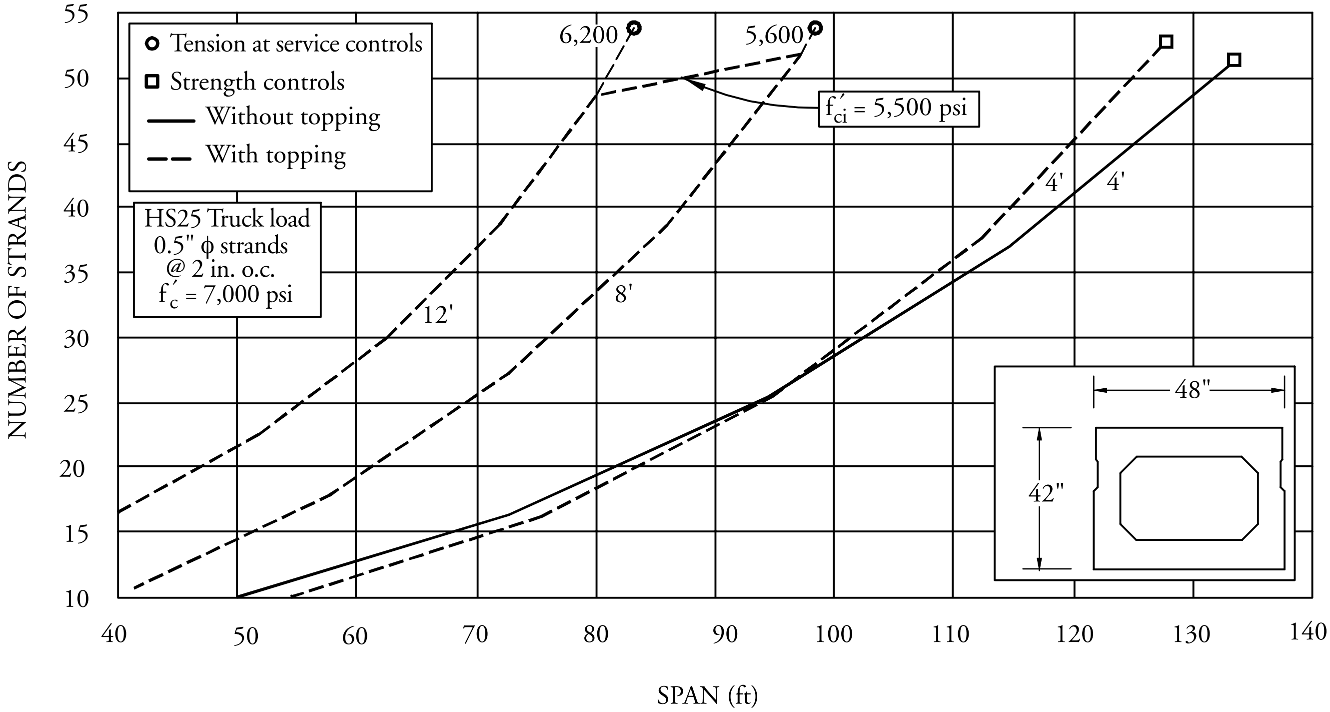


Chart BB-6
AASHTO Box Beams - 36 in. Wide

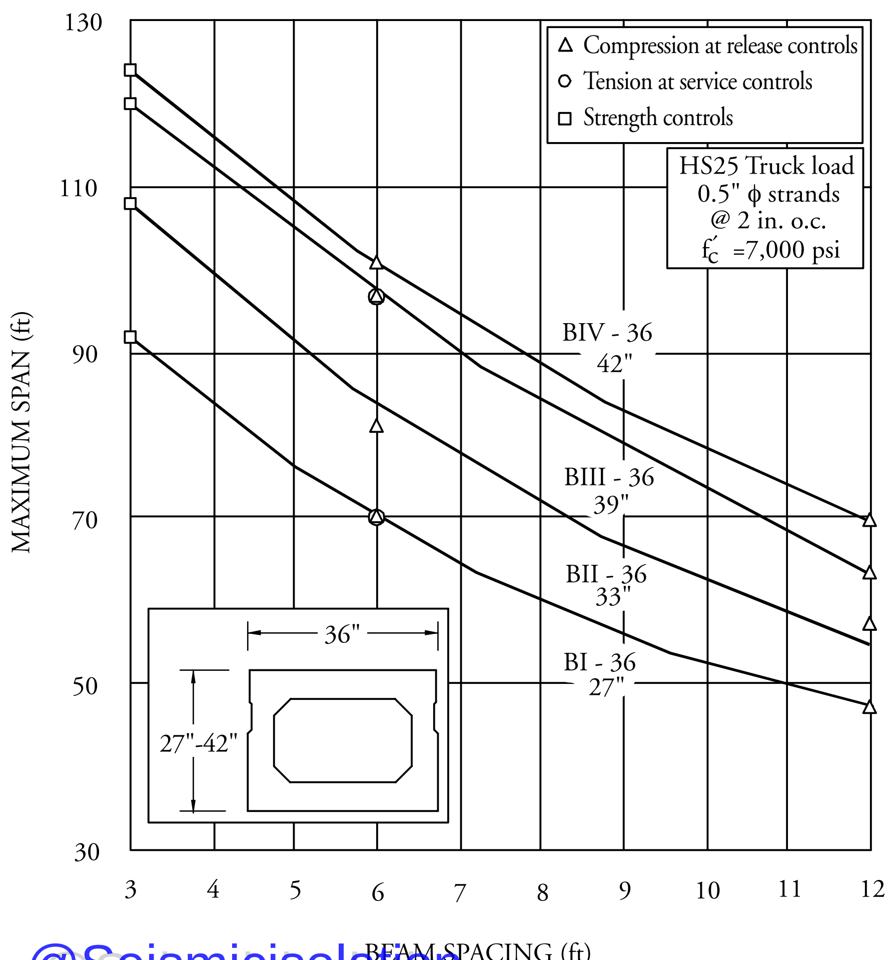


Chart BB-7
AASHTO Box Beams-BI-36

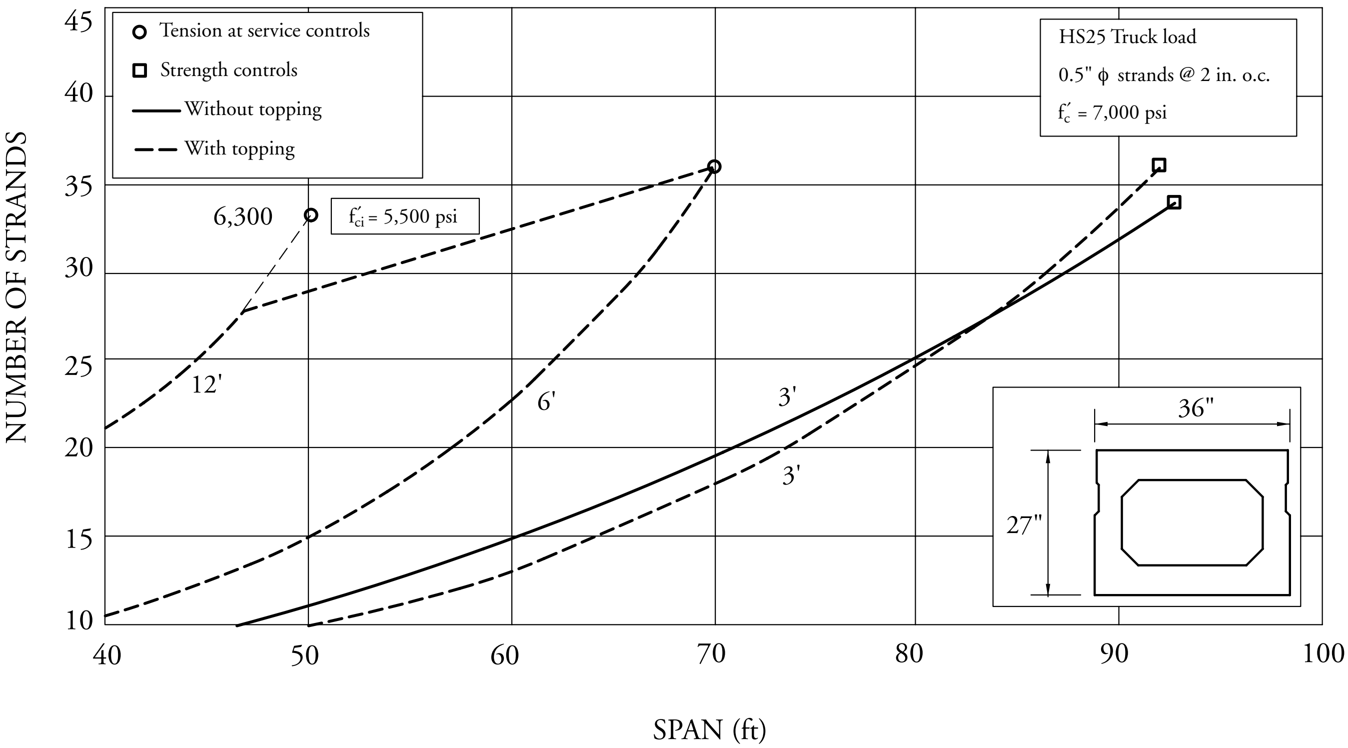


Chart BB-8
AASHTO Box Beams-BII-36

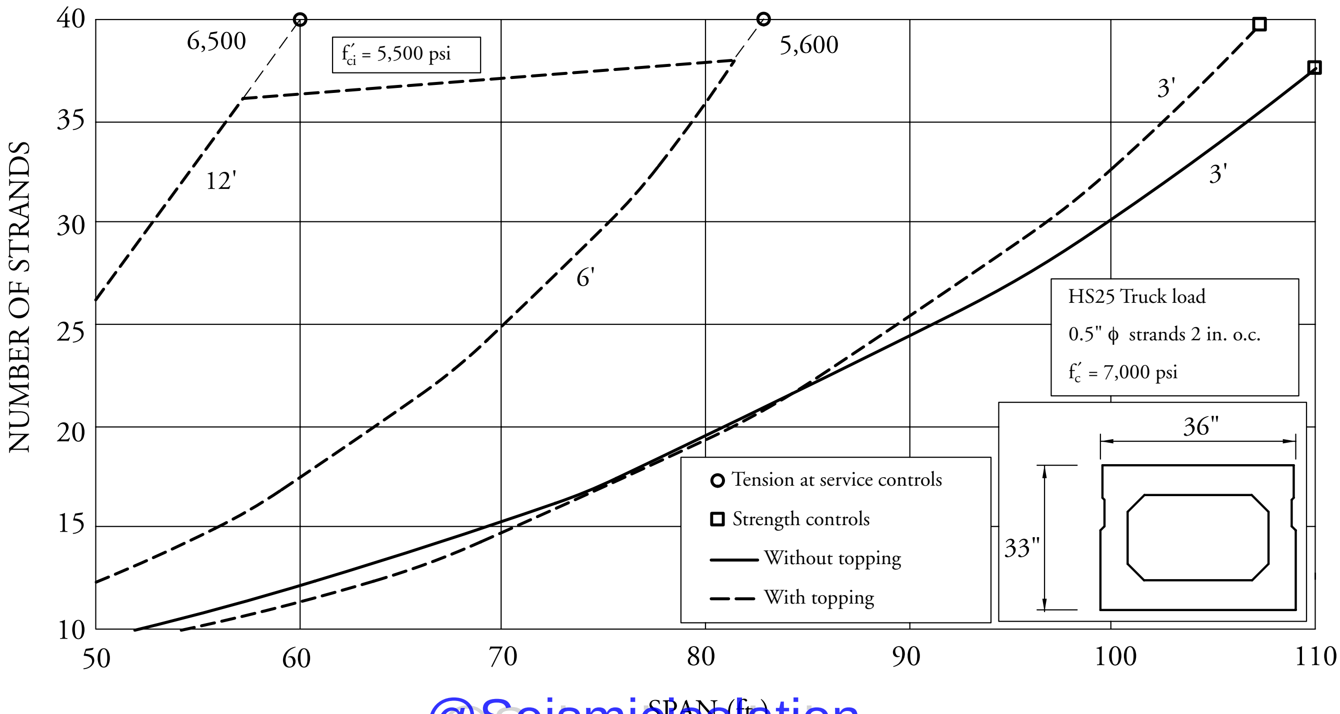


Chart BB-9
AASHTO Box Beams-BIII-36

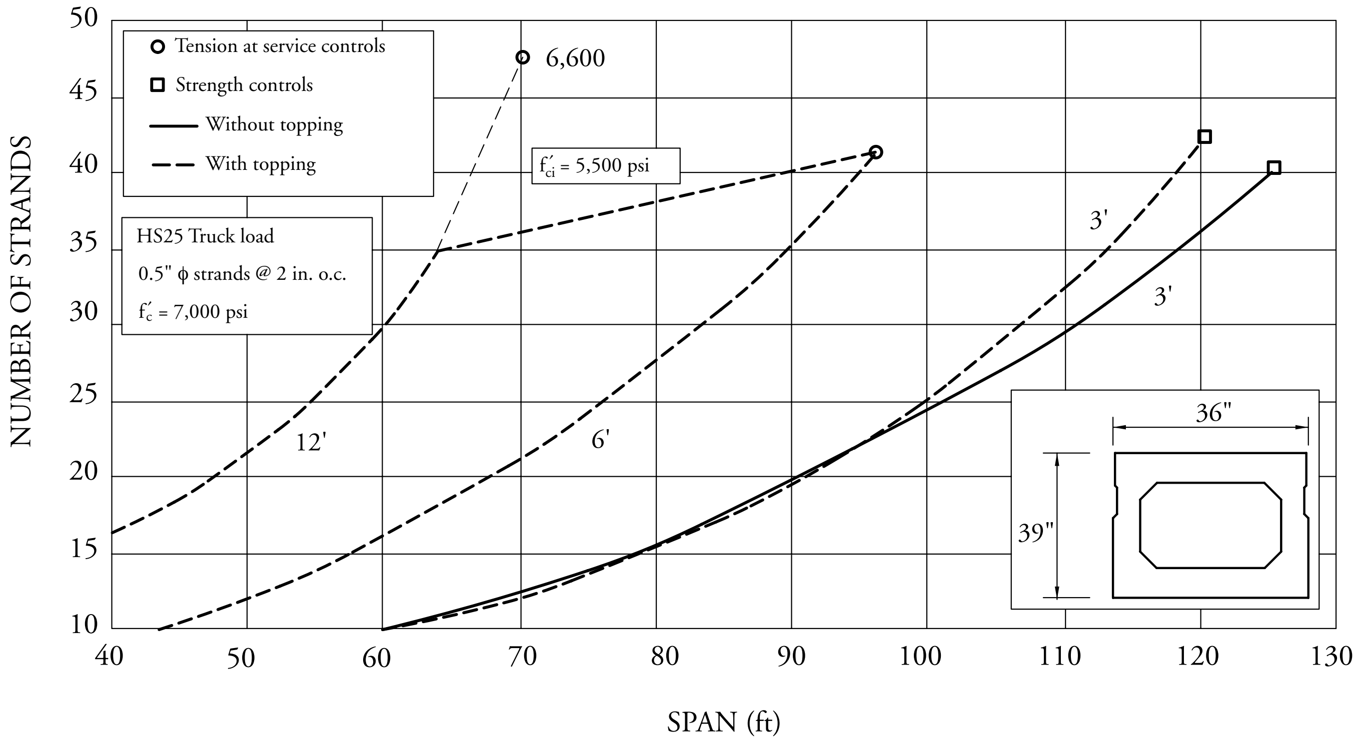


Chart BB-10
AASHTO Box Beams-BIV-36

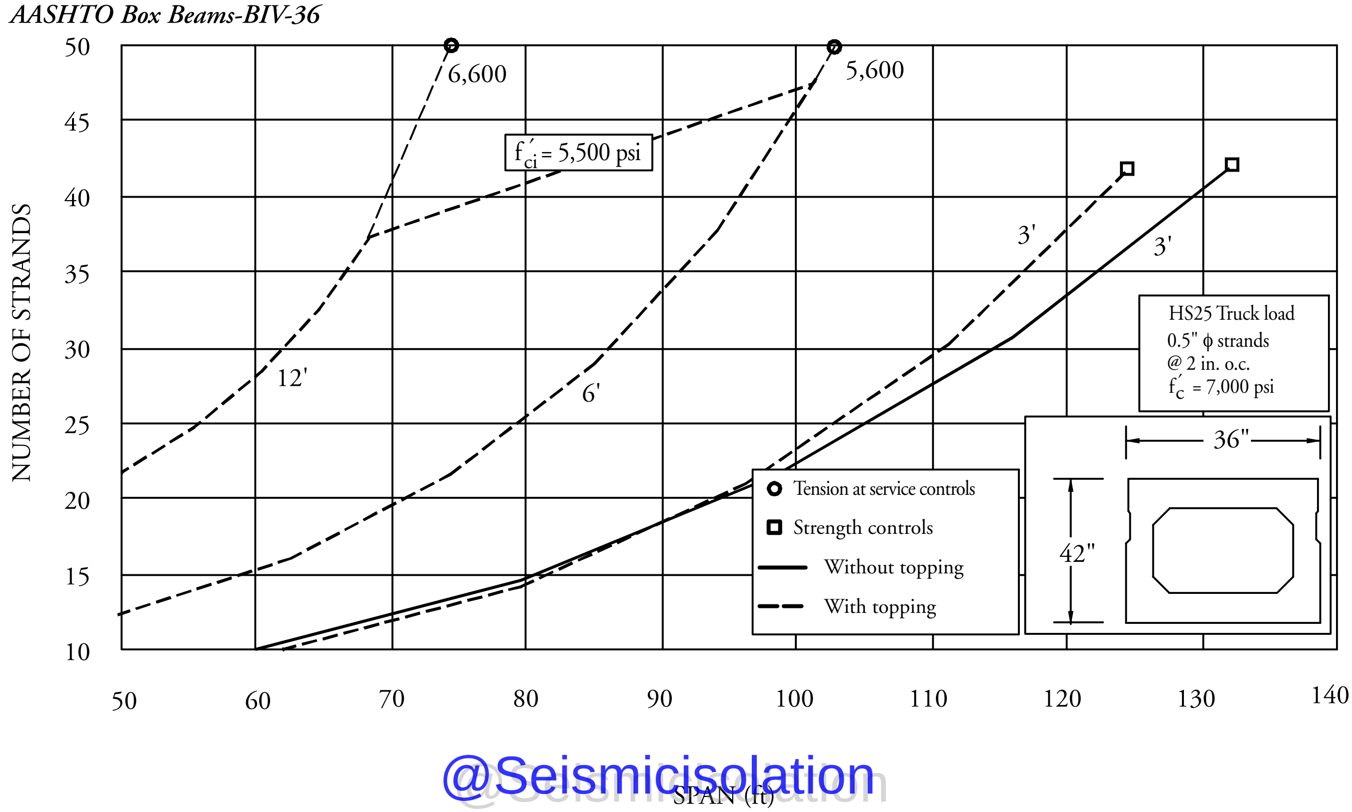


Chart BT-1
AASHTO-PCI Bulb-Tees

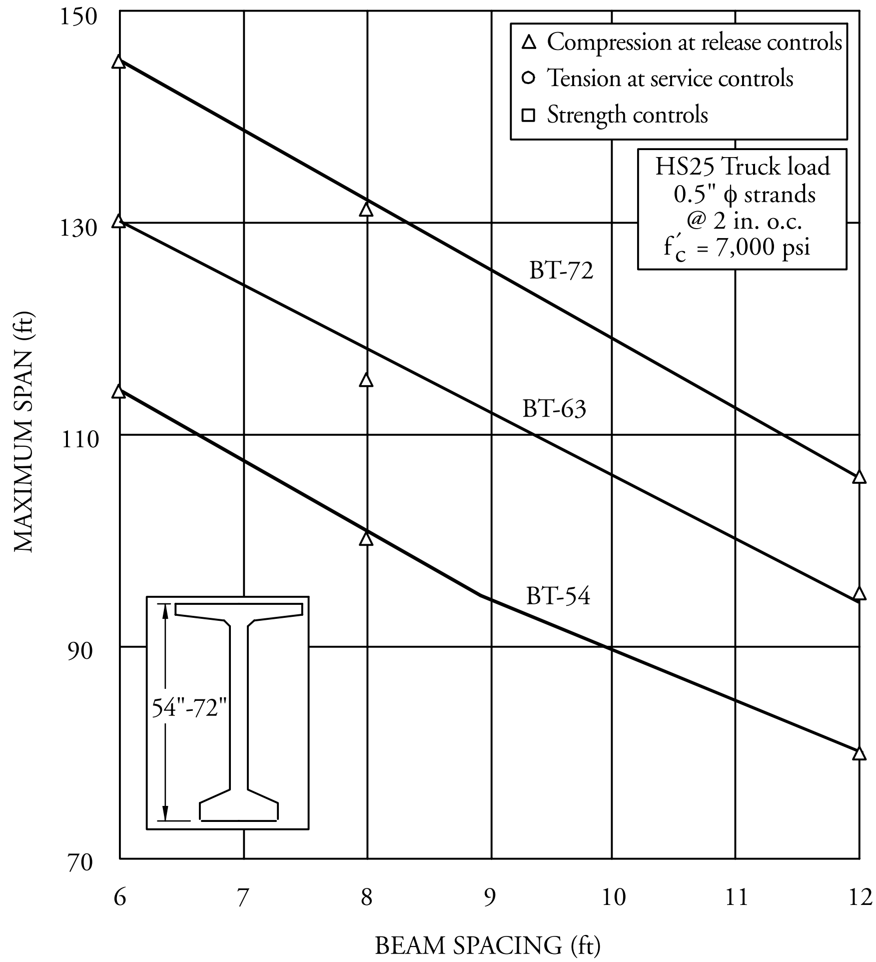


Chart BT-2
AASHTO-PCI Bulb-Tee – BT-54

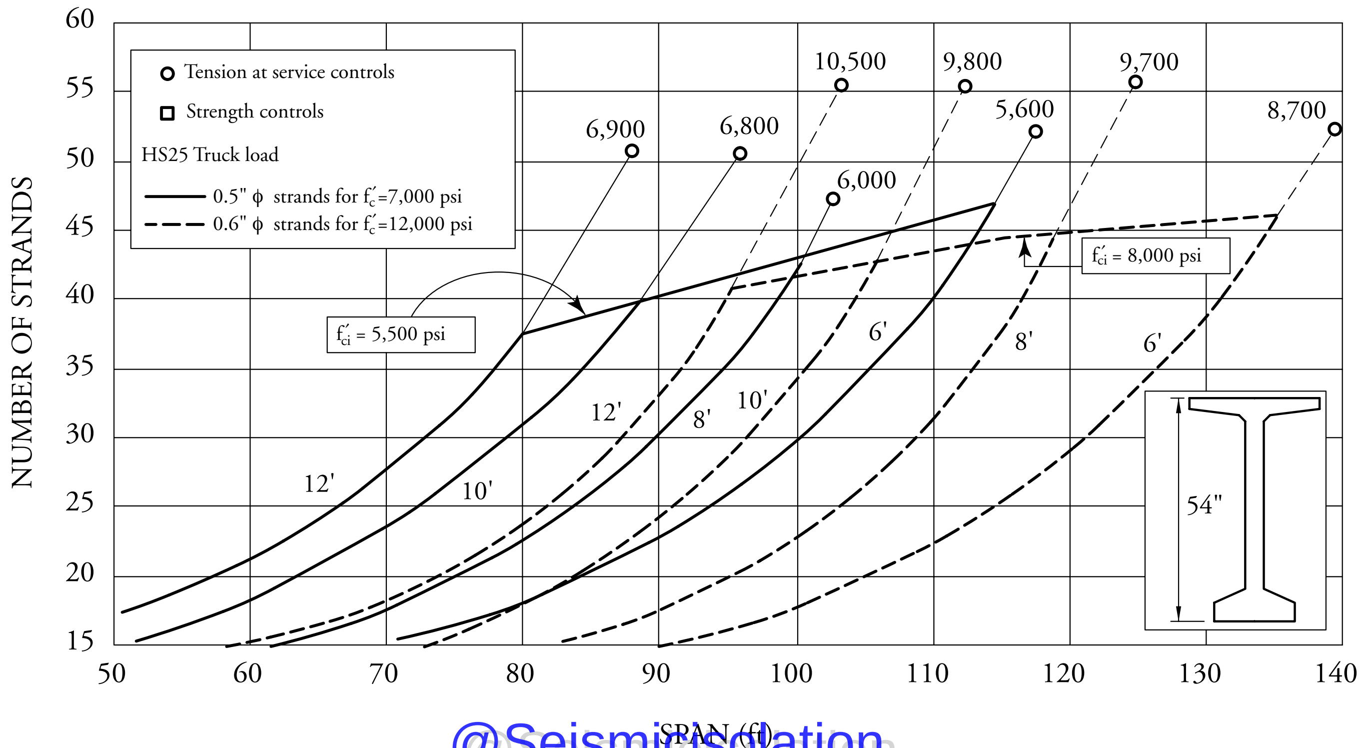


Chart BT-3
AASHTO-PCI Bulb-Tee – BT-63

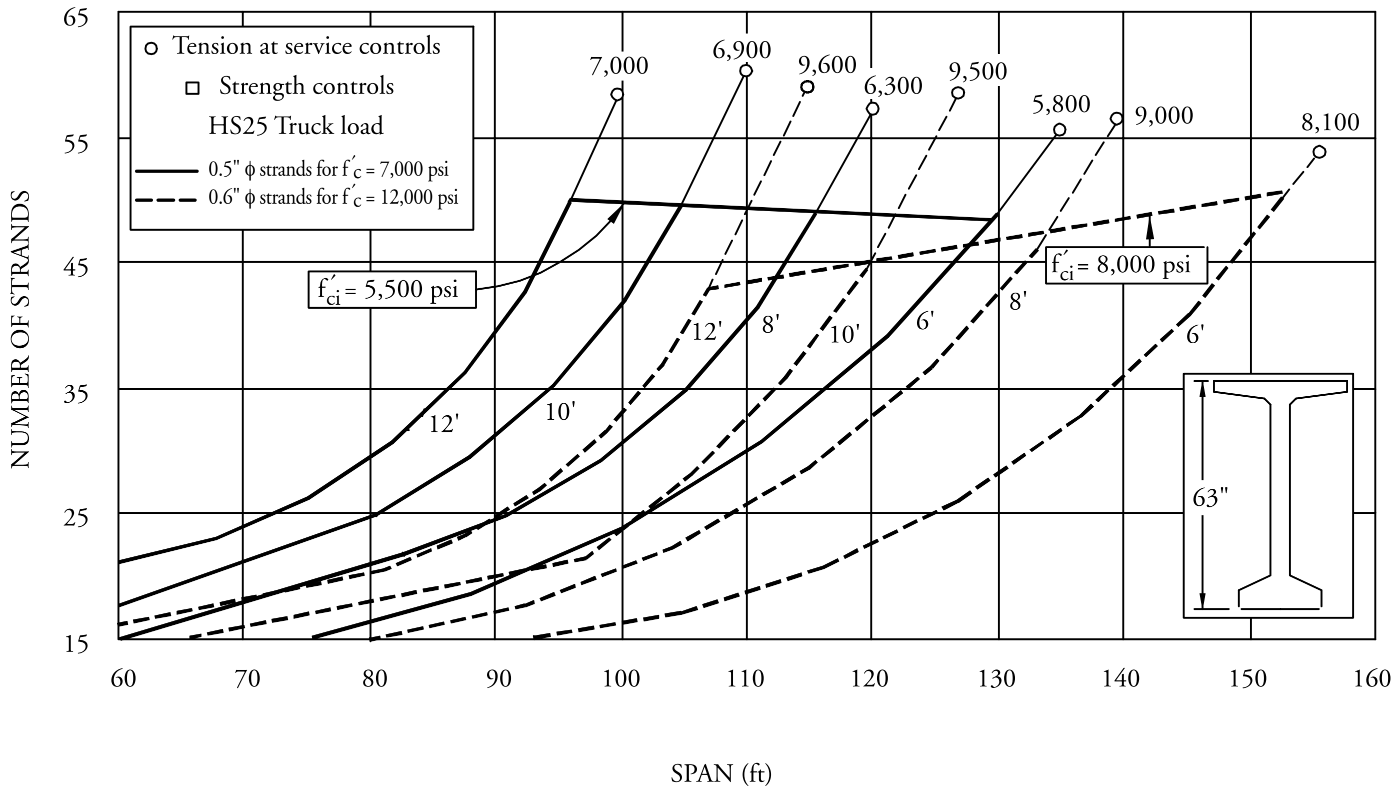


Chart BT-4
AASHTO-PCI Bulb-Tee – BT-72

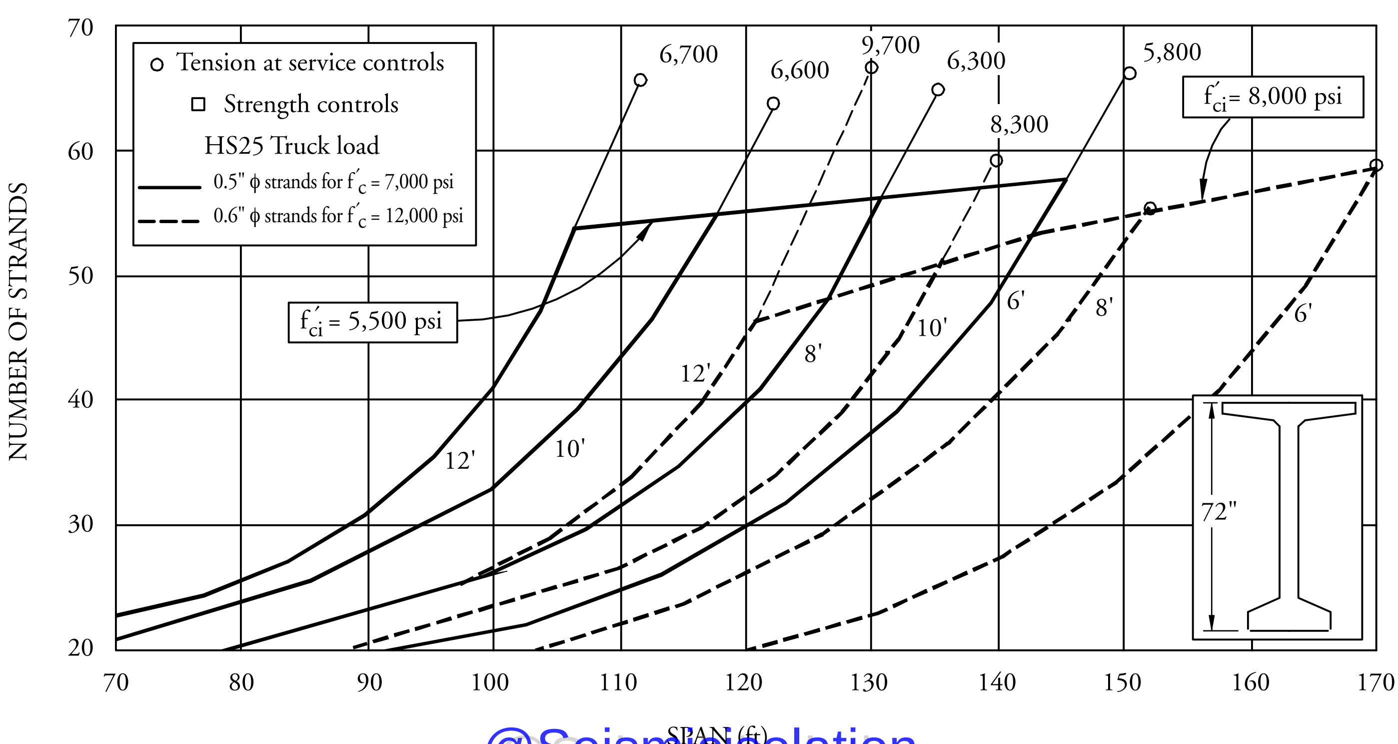


Chart DBT-1
Deck Bulb-Tees

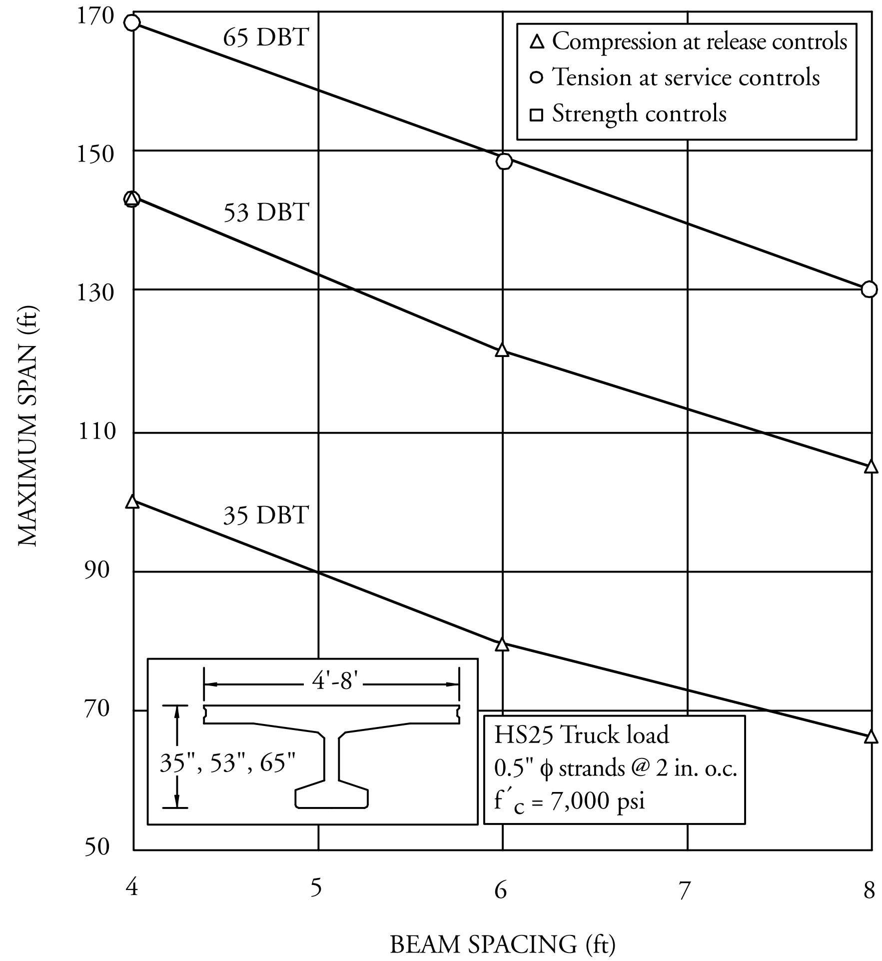


Chart DBT-2
35" Deck Bulb-Tees

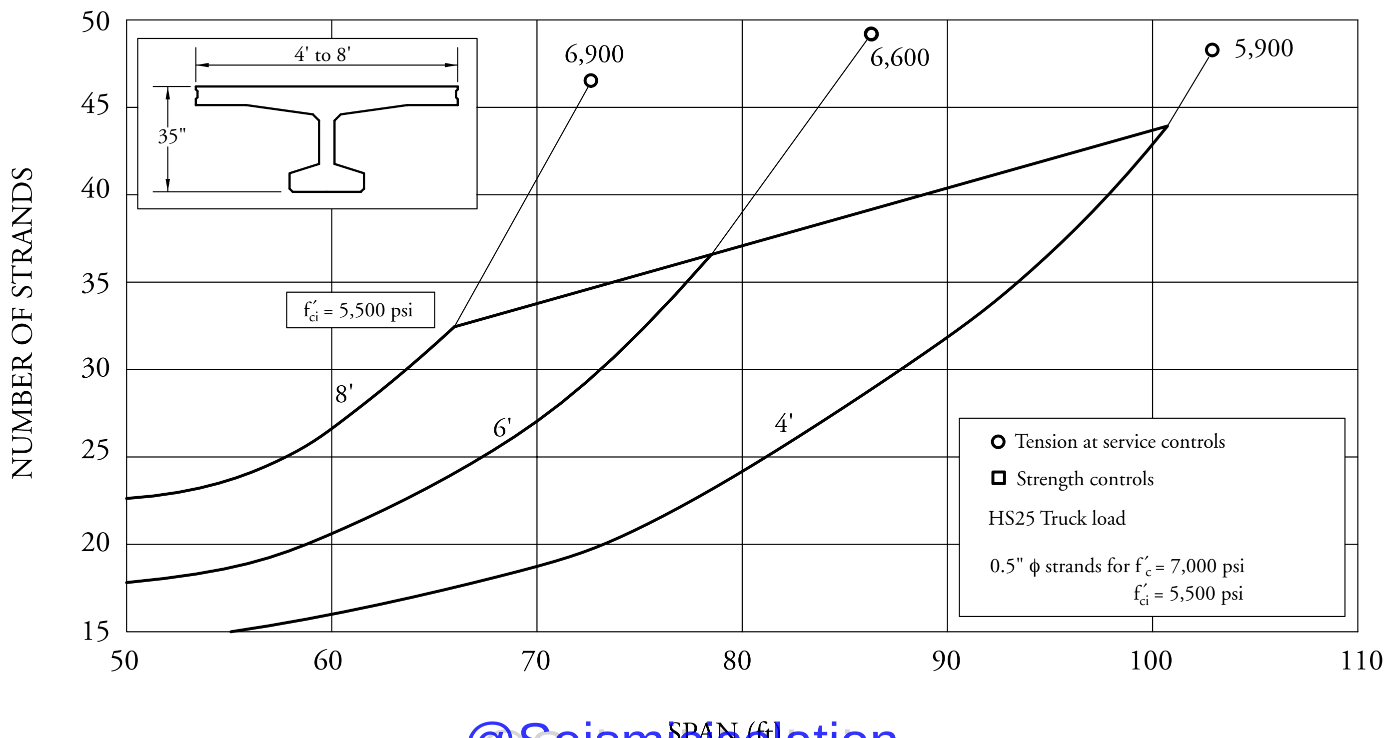


Chart DBT-3
53" Deck Bulb-Tees

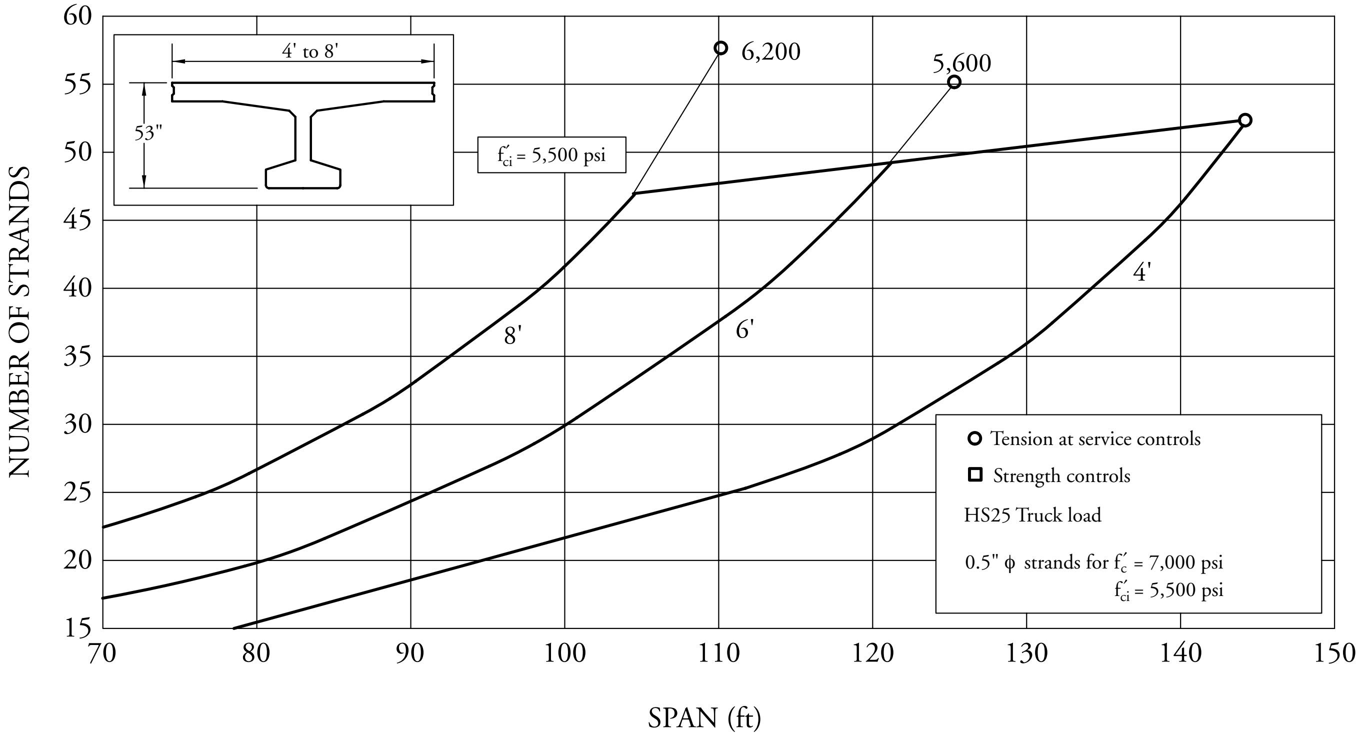


Chart DBT-4
65" Deck Bulb-Tees

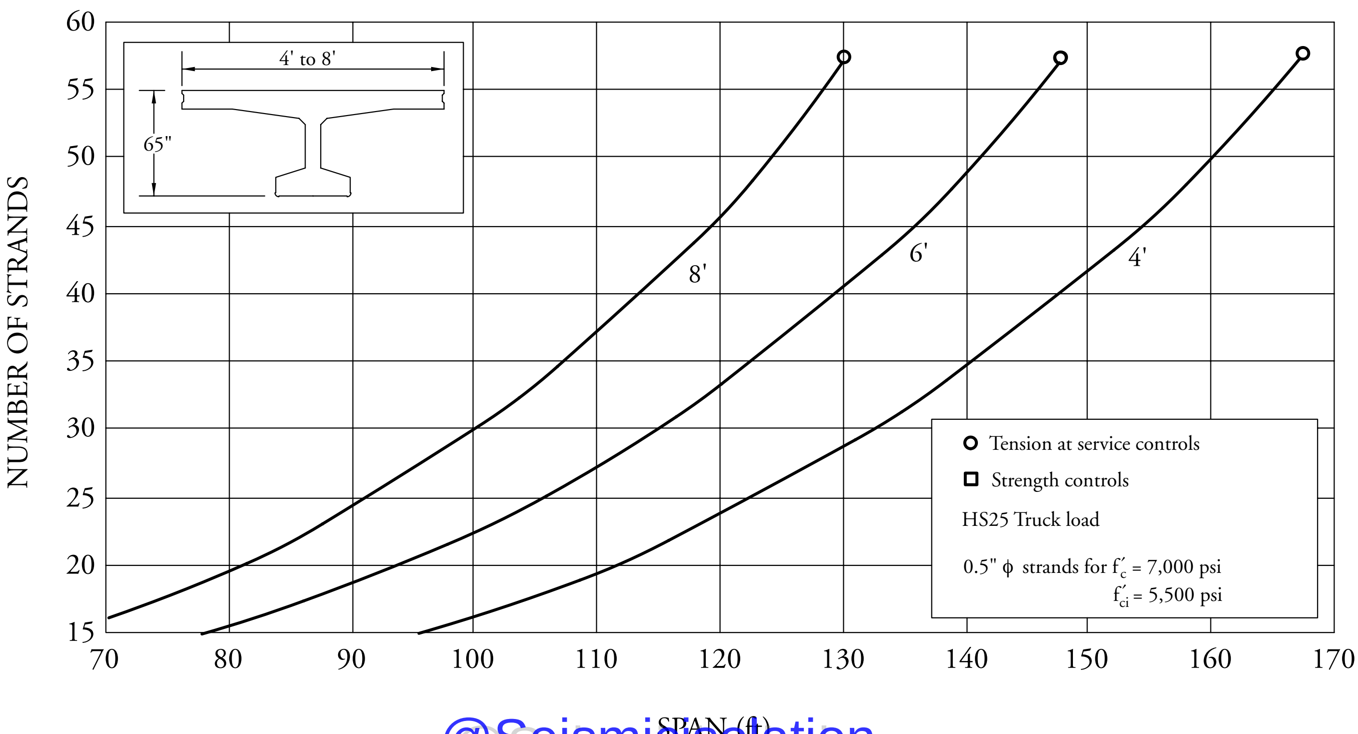


Chart DBT-5
Depth vs. Maximum Span

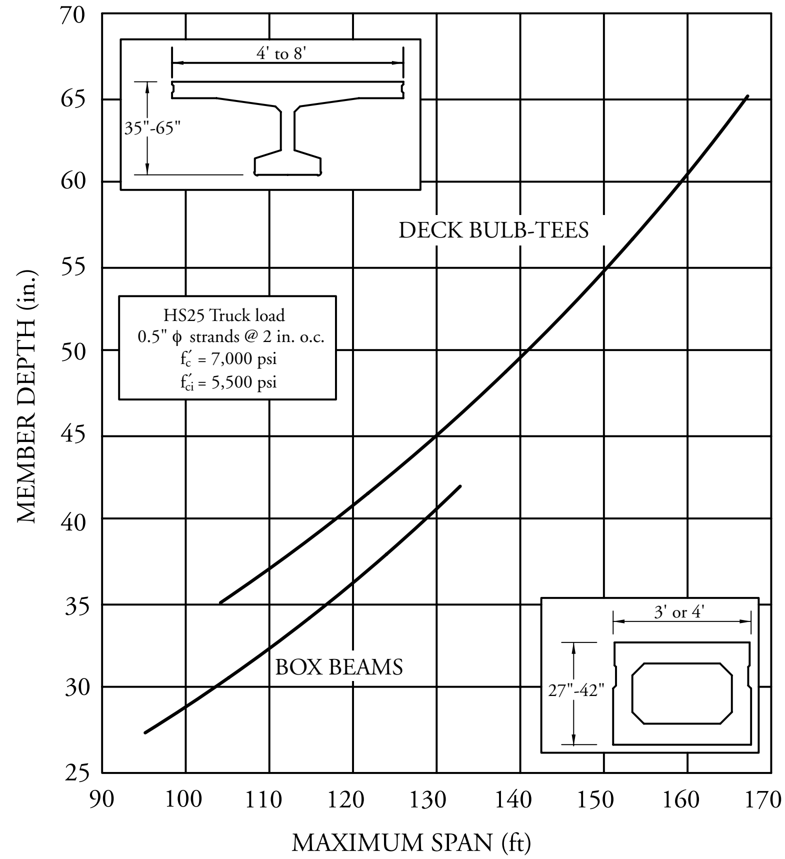


Chart IB-1
AASHTO I-Beams

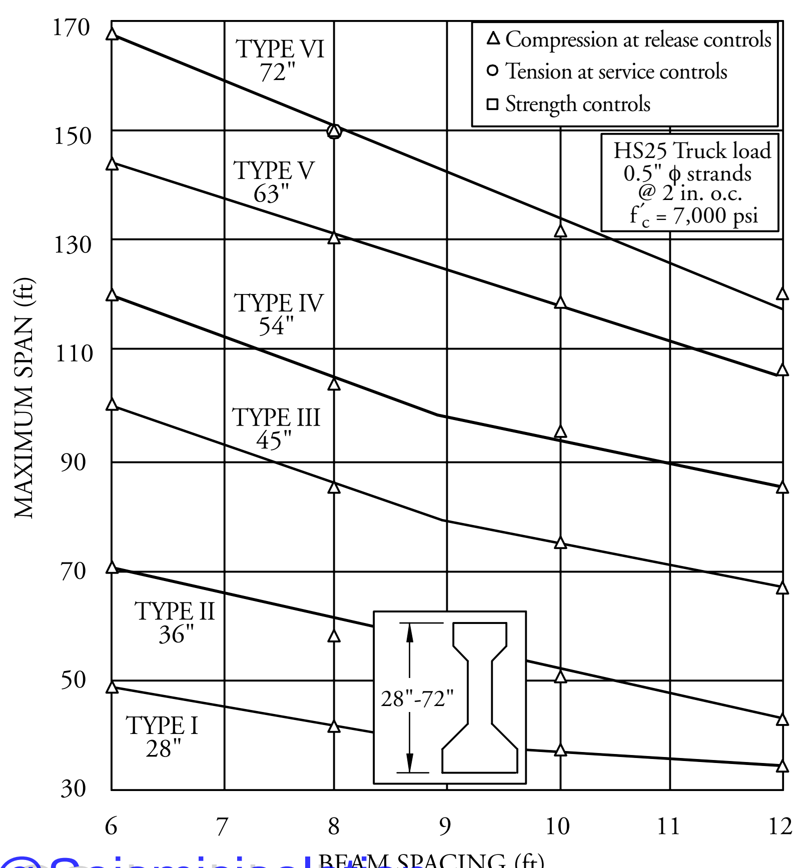


Chart IB-2
AASHTO I-Beams – Type I

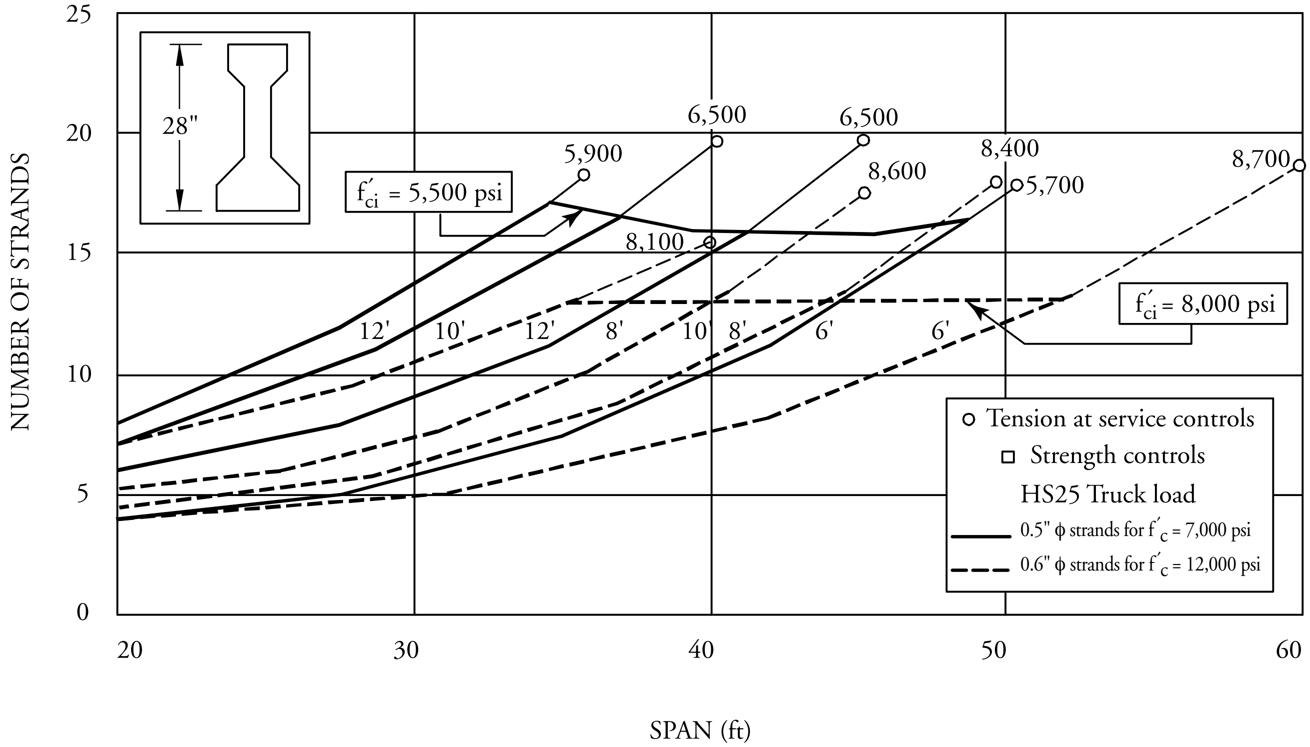


Chart IB-3
AASHTO I-Beams – Type II

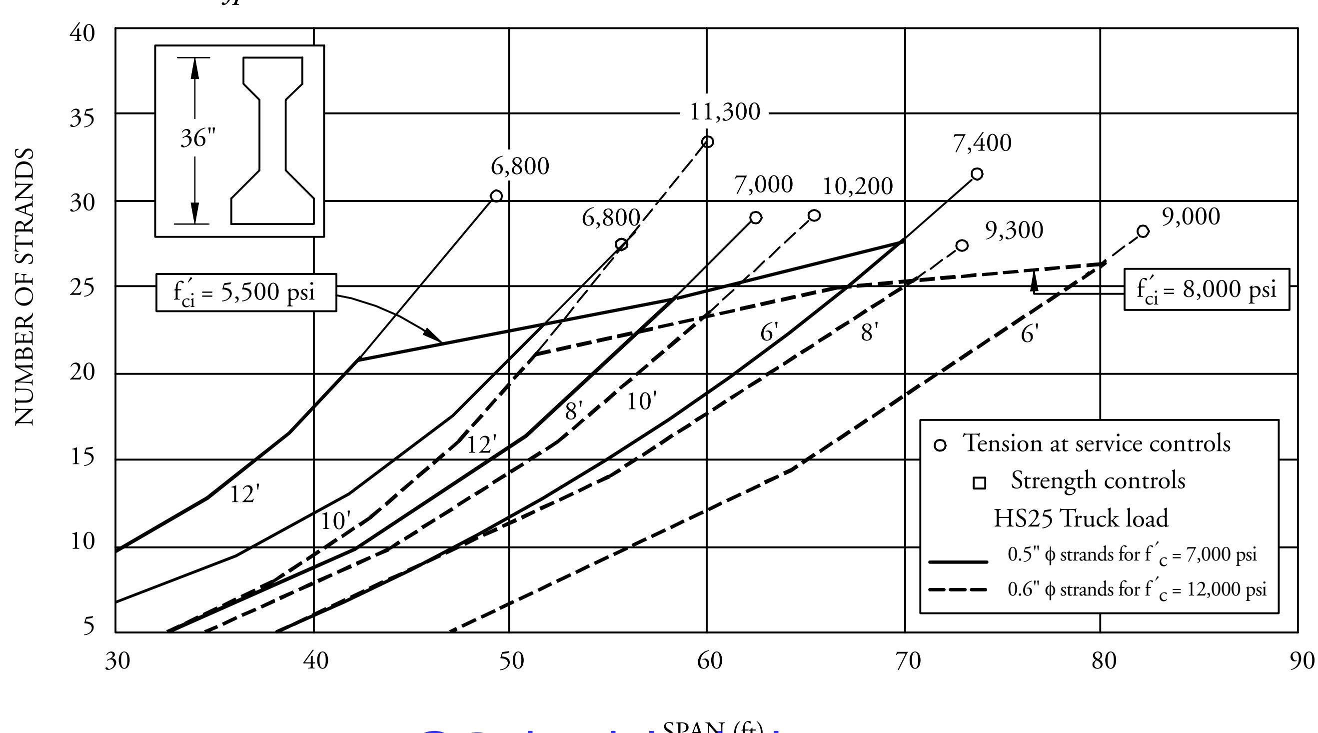


Chart IB-4
AASHTO I-Beams – Type III

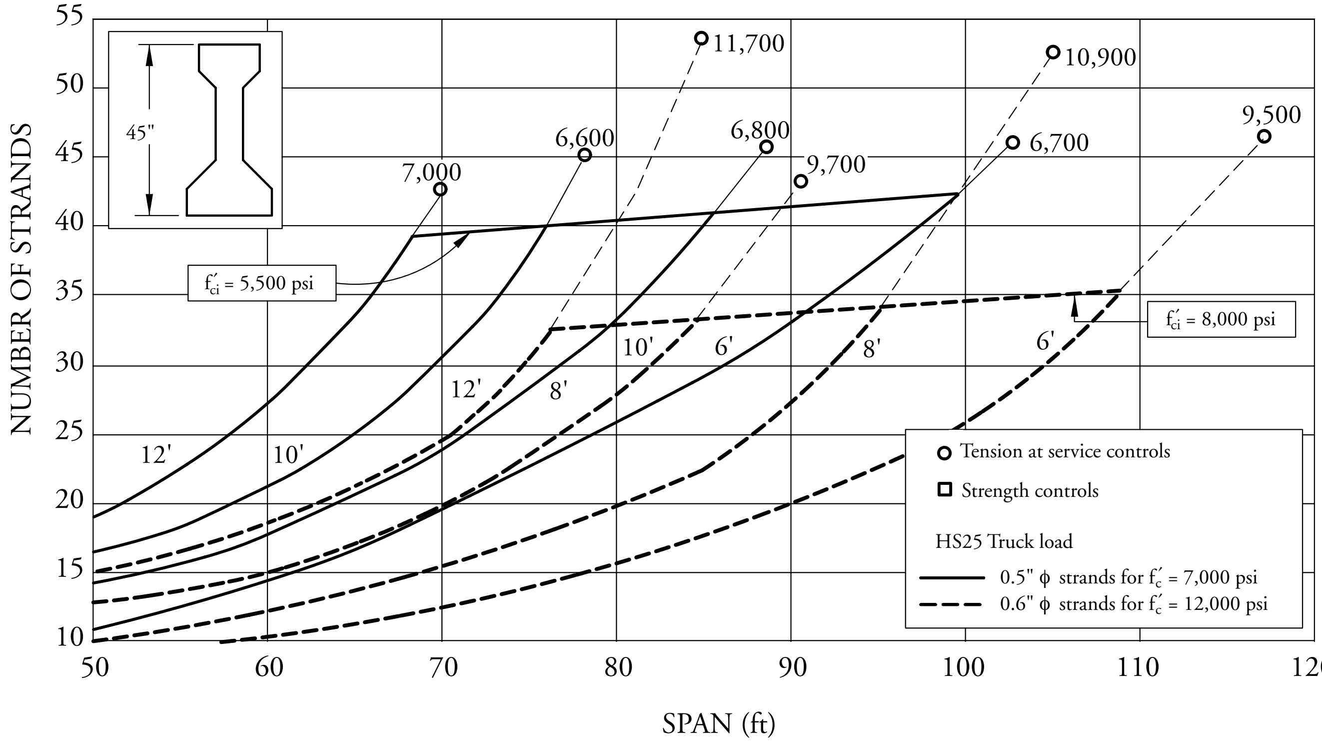
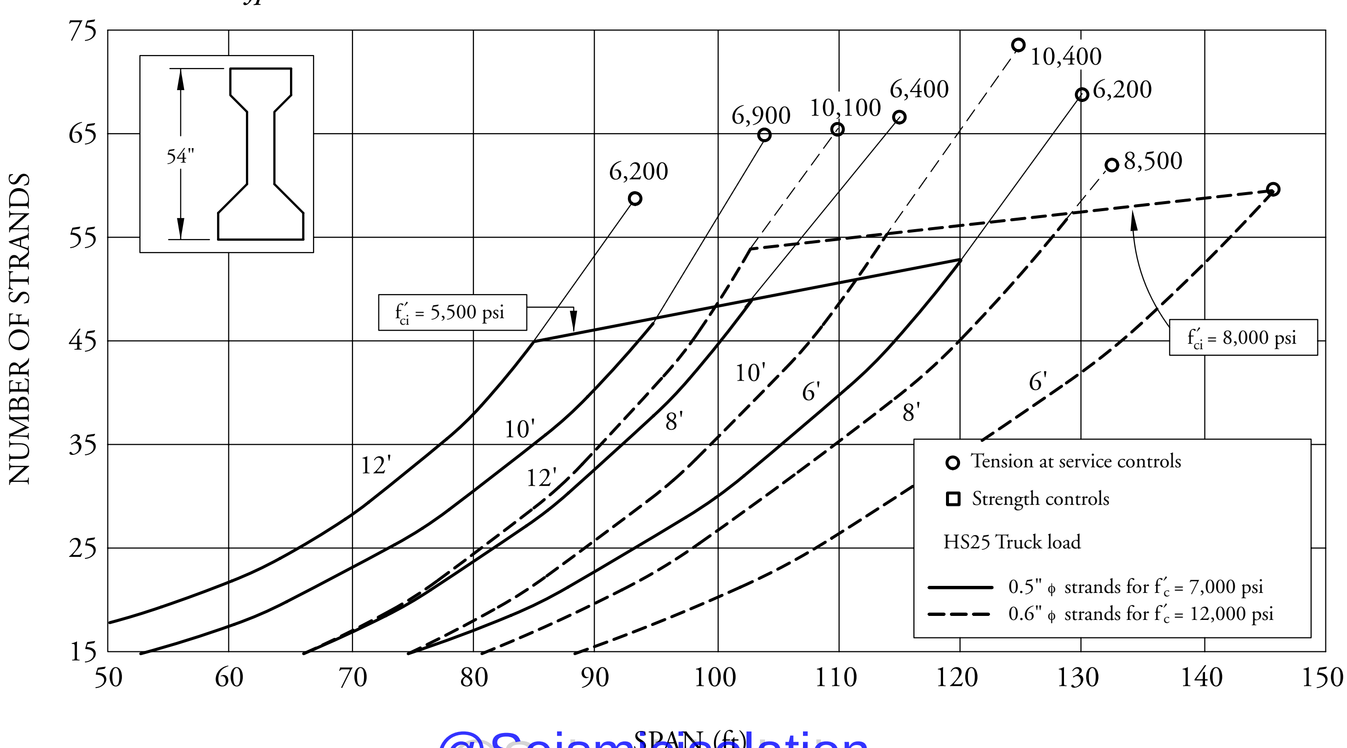


Chart IB-5
AASHTO I-Beams – Type IV



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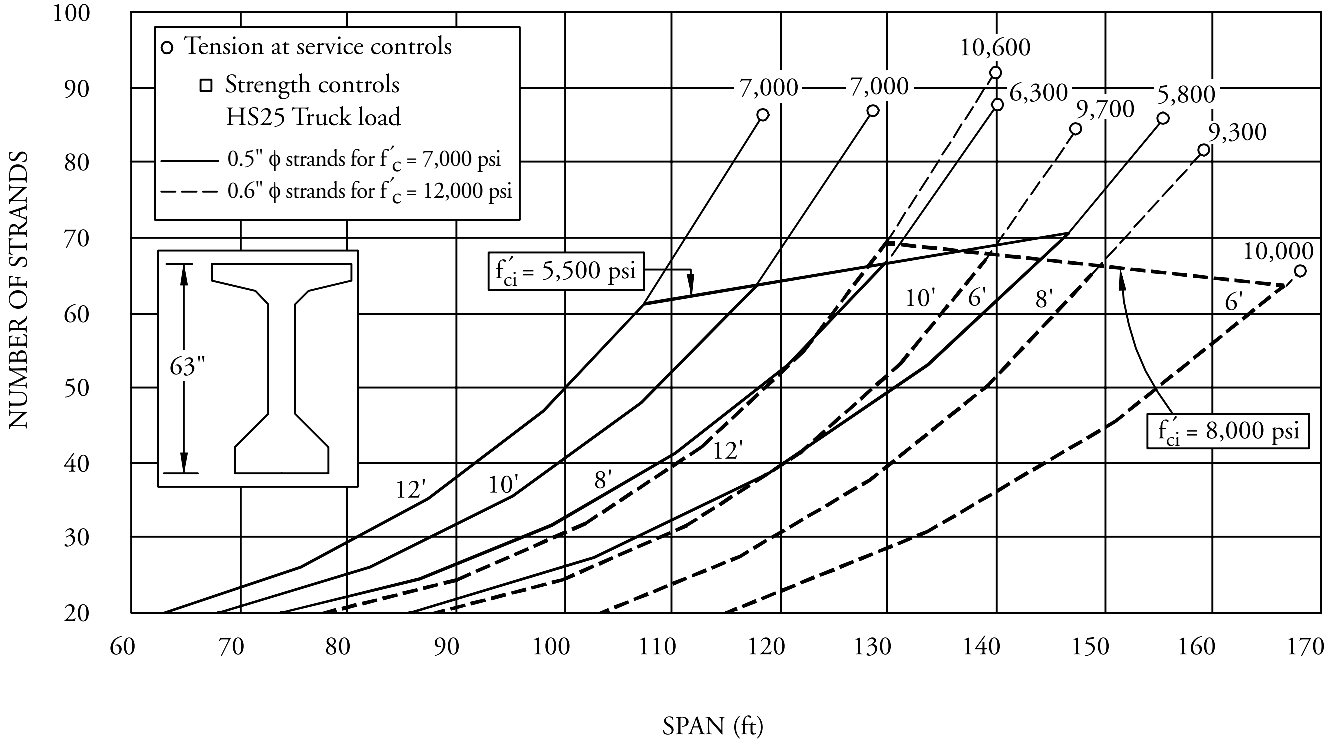


Chart IB-7
AASHTO I-Beams – Type VI

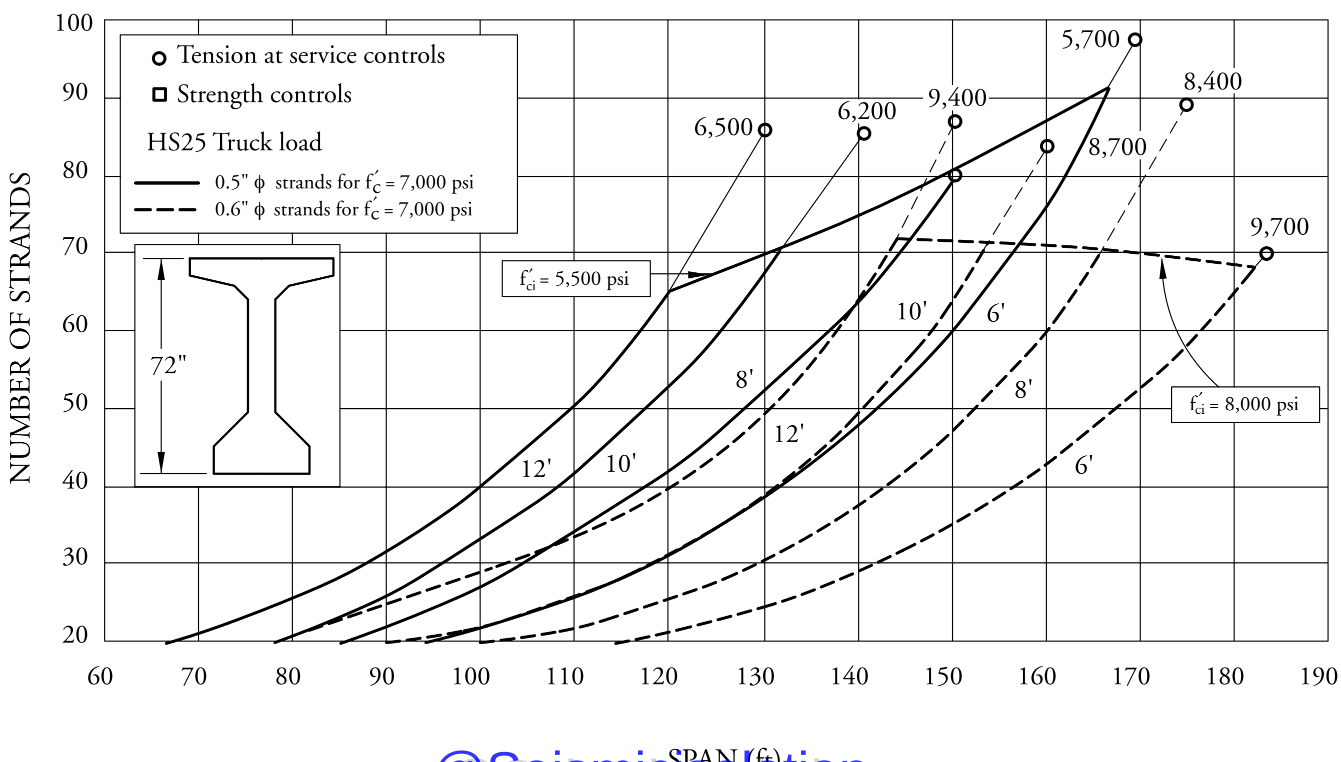


Chart DT-1
Untopped Double-Stemmed Beams

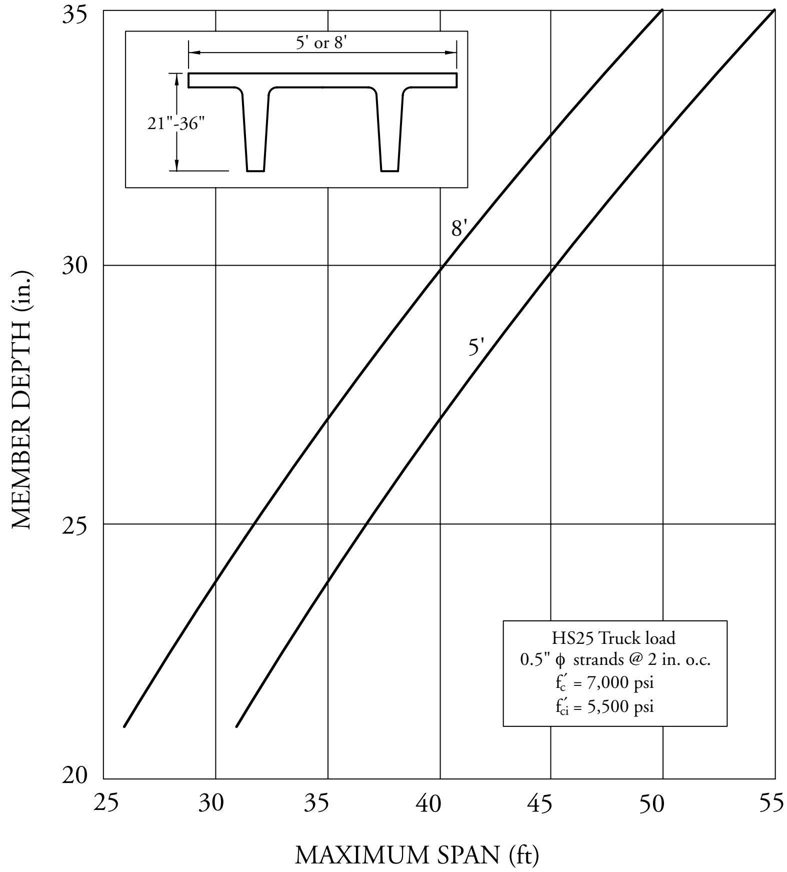


Chart DT-2
Topped Double-Stemmed Beams

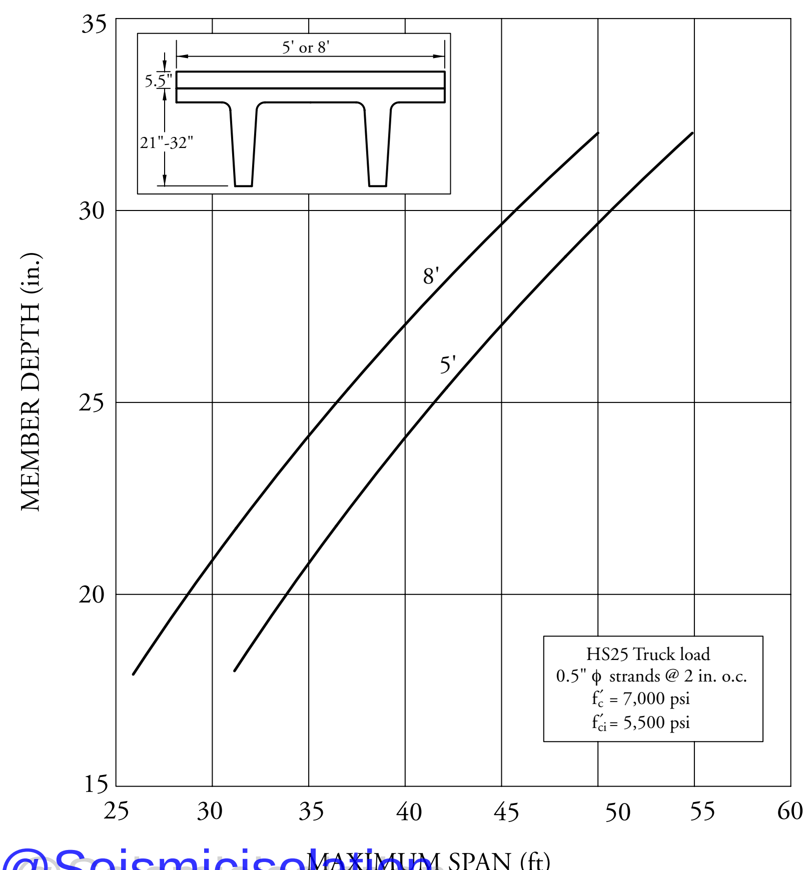


Chart SB-1
Voided Slab Beams

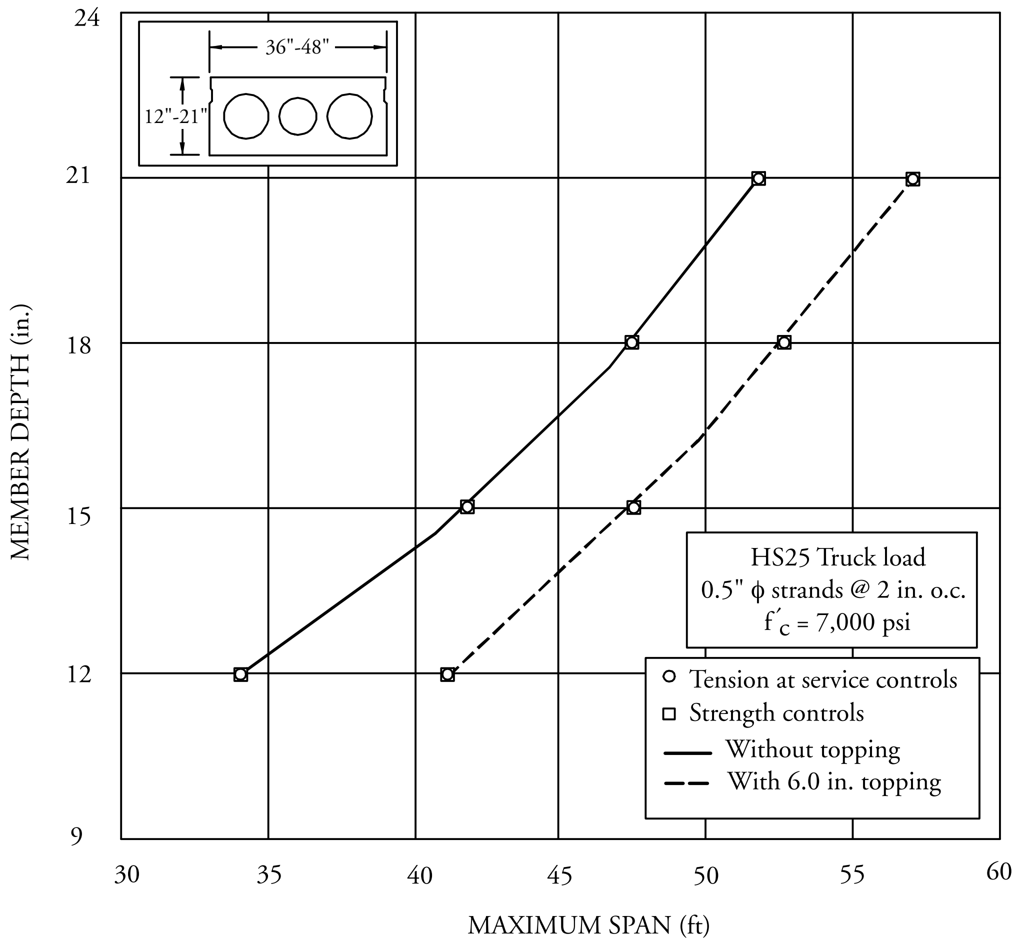


Chart SB-2
Voided Slab Beams – 48 in. Wide

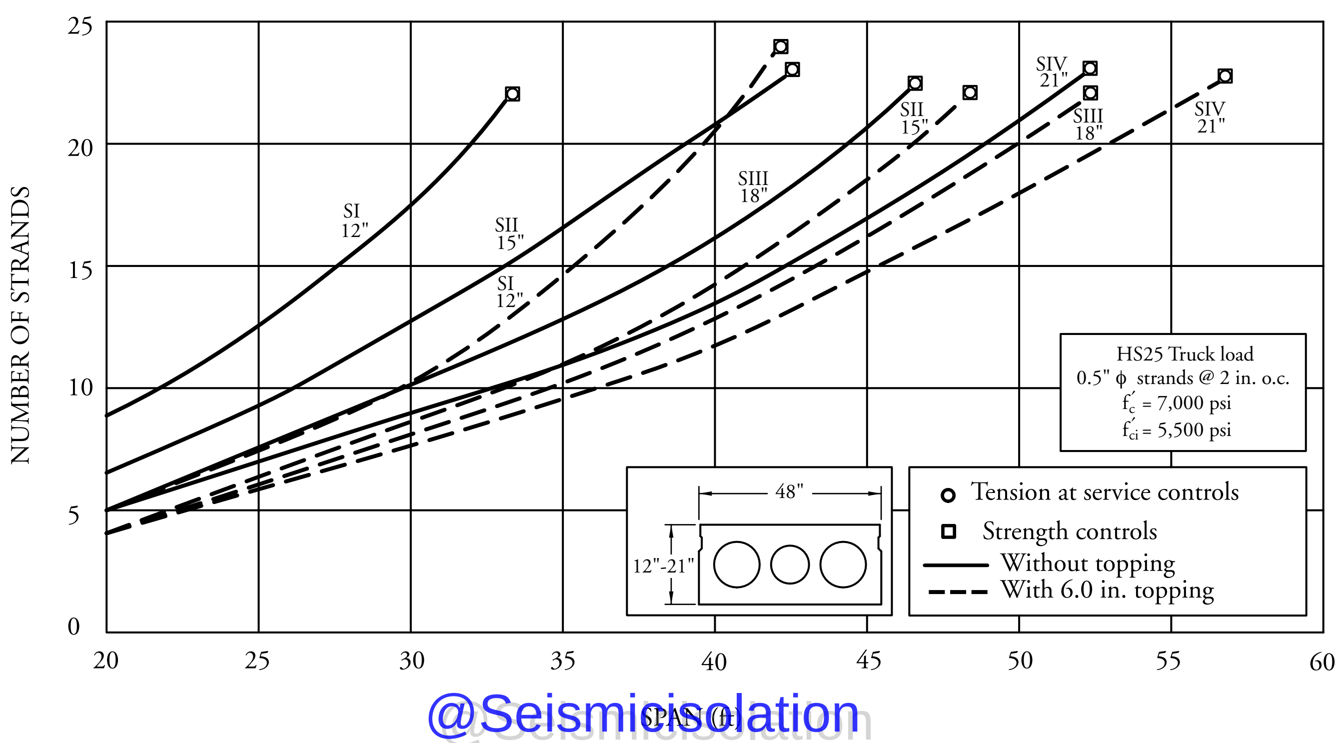


Chart SB-3
Voided Slab Beams – 36 in. Wide

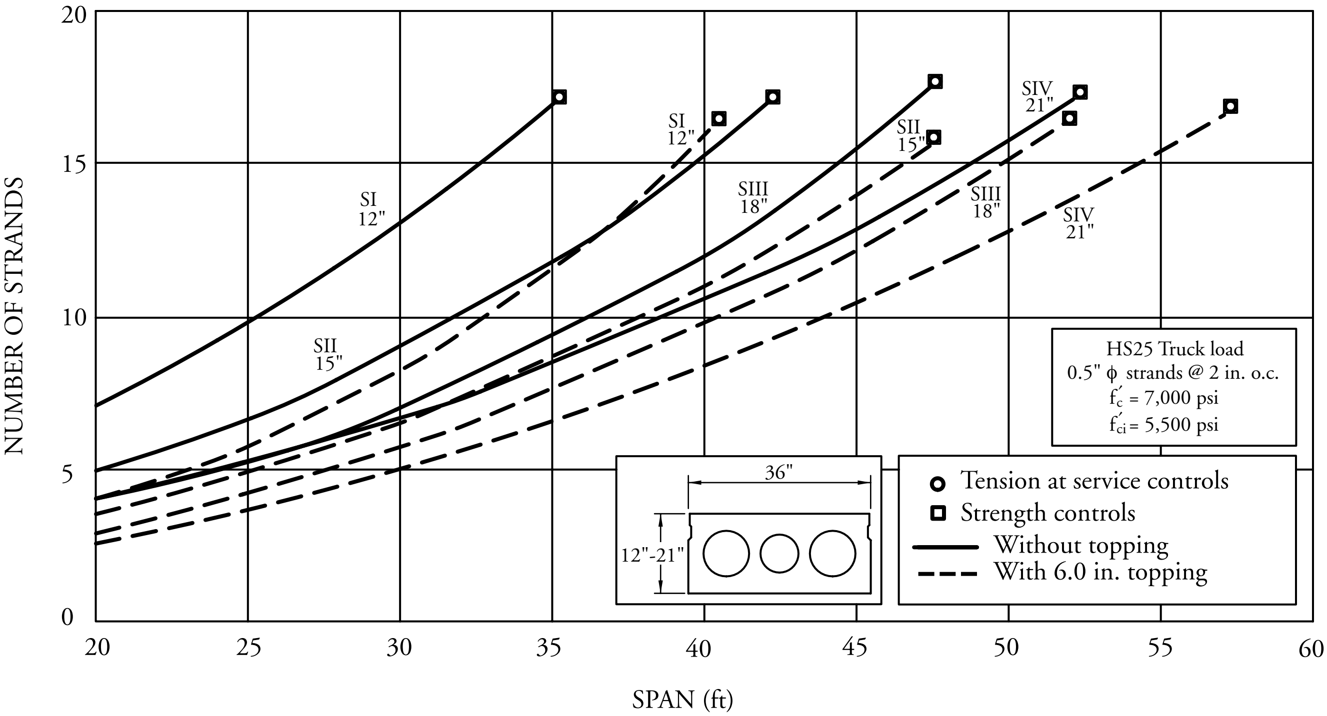


TABLE OF CONTENTS
LOADS AND LOAD DISTRIBUTION

NOTATION

LOADS AND LOAD DISTRIBUTION

A = area of stringer or beam
A_o = area enclosed by centerlines of elements (walls)
B = buoyancy
BR = vehicular braking force
b = width of beam
c_1 = constant related to skew factor
C = stiffness parameter
CE = vehicular centrifugal force
CF = centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
D = a constant that varies with bridge type and geometry
D = width of distribution per lane
D = dead load
DC = dead load of structural components and nonstructural attachments
DD = downdrag
DW = dead load of wearing surfaces and utilities
d = depth of beam
d = precast beam depth
d_e = distance between the center of exterior beam and interior edge of curb or traffic barrier
E = earth pressure
EH = horizontal earth pressure load
EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
EQ = earthquake
ES = earth surcharge load
EV = vertical pressure from deal load of earth fill
e = correction factor
e = eccentricity of a lane from the center of gravity of the pattern of beams
e_g = distance between the centers of gravity of the beam and deck
FR = friction
$f_{(L+I)}$ = live load plus impact bending stress
f_D = the sum of dead load bending stresses
g = a factor used to multiply the total longitudinal response of the bridge due to a single longitudinal line of wheel loads in order to determine the maximum response of a single beam
g = distribution factor
I = impact fraction
I = live load impact
I = moment of inertia
I = moment of inertia of beam

IC = ice load
ICE = ice pressure
IM = vehicular dynamic load allowance
J = St. Venant torsional constant
K = a non-dimensional constant
K_g = longitudinal stiffness parameter
L = live load
L = span of beam
L = simple span length (except cantilevers) when computing truck load moments
L = length of the loaded portion of span from section under consideration to the far reaction when computing shear impact due to truck loads
LF = longitudinal force from live load
LL = vehicular live load
LS = live load surcharge
m = multiple presence factor
N = group number
N_b = number of beams
N_B = number of beams
N_L = number of design lanes
N_L = number of loaded lanes under consideration
N_L = number of traffic lanes
n = modular ratio between beam and deck material
PL = pedestrian live load
Q = total factored load
Q_i = force effect
q_i = specified loads
R = reaction on exterior beam in terms of lanes
R = rib shortening
R_n = nominal resistance
S = beam spacing
S = shrinkage
S = center-to-center beam spacing
S = width of precast member
s = length of a side element
SE = settlement
SF = stream flow pressure
SH = shrinkage
T = temperature
TG = temperature gradient
TU = uniform temperature
t = thickness of an element
t_s = depth of concrete slab

V = distance between axles

W = edge-to-edge width of bridge

W = combined weight on first two truck axles

W = roadway width between curbs

W = overall (edge-to-edge) width of bridge measured perpendicular to the longitudinal beams

W = wind load on structure

WA = water load and stream pressure

WL = wind load on live load

WS = wind load on structure

X_{ext} = horizontal distance from the center of gravity of the pattern of beams to the exterior beam

x = horizontal distance from the center of gravity of the pattern of beams to each beam

β = coefficient, **Table 7.3.1-1**

γ = load factor, **Table 7.3.1-1**

γ_i = load factors specified in **Tables 7.3.2-1** and **7.3.2-2**

η = variable load modifier which depends on ductility, redundancy and operational importance

ϕ = capacity reduction or resistance factor

μ = Poisson's ratio, usually assumed equal to 0.20

θ = skew angle

Loads and Load Distribution

7.1 SCOPE

One main task in bridge design is to collect information on the various permanent and transient loads that may act on a bridge, as well as on how these forces are distributed to the various structural components. This chapter will introduce engineers to the general types of loads to which a bridge is subjected. It presents the load provisions of both the AASHTO *Standard Specifications for Highway Bridges* (referred to as *"Standard Specifications"* in the following) and *AASHTO LRFD Bridge Design Specifications ("LRFD Specifications").* The in-depth discussions will be limited to live load and its distribution to precast, prestressed concrete superstructure systems. Detailed discussion of other load effects, such as seismic forces and soil pressures, are covered in other chapters of the manual. Although both specifications form a consistent set of guidelines for bridge design, the engineer should be aware that many state DOTs have additional requirements for loads, load distribution or load combinations. Such requirements are not discussed in this chapter.

This chapter is based on the provisions of the Standard Specifications, 17th Edition, 2002, and the LRFD Specifications, 2nd Edition, 1998, with all of the Interim Revisions through and including the 2003 Interim Revisions.

7.2 LOAD TYPES

In the design of bridge structure components, the engineer should consider all loads which the component must resist. These forces may vary depending on duration (permanent or transient), direction (vertical, transverse, longitudinal, etc.) and deformation (thermal, shrinkage and creep). Furthermore, the type of effect (bending, shear, axial, etc.) will sometimes influence the magnitude of such forces. A brief description of these forces is detailed below.

7.2.1 Permanent Loads

These loads are sustained by the bridge throughout its life. In general, permanent loads may be subdivided into the following categories.

7.2.1.1 Dead Loads

One of the first tasks in superstructure design is to identify all elements contributing to loads on the beams before composite deck concrete, if any, has cured (some concrete decks are designed to remain noncomposite). These noncomposite dead loads include the beams, weight of the deck slab, haunch, stay-in-place forms and diaphragms.

7.2.1.2 Superimposed Dead Loads

All permanent loads placed on the superstructure after deck curing is completed are usually designated superimposed dead loads. These include the wearing surface, parapets, railings, sidewalk, utilities and signage. In the *LRFD Specifications,* the load factors for wearing surface and utilities are higher than for other dead loads to recognize the increased variability of these loads.

7.2.1.3 Earth Pressures

These forces, which primarily affect substructure elements, are usually considered permanent loads. However, they may occasionally affect the superstructure elements at locations where substructure and superstructure interface (abutment backwall, etc.). Detailed equations are listed in both AASHTO specifications. Generally, these pressures do not affect superstructure design.

7.2.2 Live Loads

7.2.2.1 Gravity Vehicular Live Load

7.2.2.1.1 Number of Design Lanes

Unless otherwise specified, the number of design lanes should be determined by taking the integer part of: roadway width in ft between barriers or curbs divided by 12.0. The loads are assumed to occupy 10.0 ft transversely within a design lane.

7.2.2.1.2 Multiple Presence of Live Load

In view of the improbability of coincident maximum loading in all lanes, the following percentages of live loads are allowed in the STD Article 3.12, when using refined methods of analysis:

One or two loaded lanes	100%
Three lanes	90%
Four (or more) lanes	75%

LRFD Specifications Article 3.6.1.1.2 provides a multiple presence factor, m, which applies when using the refined method [LRFD Articles 4.4 and 4.6.3] or the lever rule for distribution of live load. When considering one loaded lane, the multiple presence factor must be used. For three or more loaded lanes, the multiple presence factor is optional. The extreme live load force effect is determined by considering each possible combination of number of loaded lanes multiplied by the corresponding factor given below. The multiple presence factors are not to be used with the approximate load assignment methods of LRFD Articles 4.6.2.2 and 4.6.2.3 because these factors are already incorporated in the distribution factors for both single and multiple lanes loaded.

One loaded lane	m = 1.20
Two loaded lanes	m = 1.00
Three loaded lanes	m = 0.85
Four (or more) loaded lanes	m = 0.65

7.2.2.1.3 Highway Live Loading - Standard Specifications

[STD Article 3.7]

There are four classes of notional truck or lane loadings to be used in the design of medium- or long-span superstructures. The majority of bridges are designed for the

Figure 7.2.2.1.3-1
Standard HS Truck

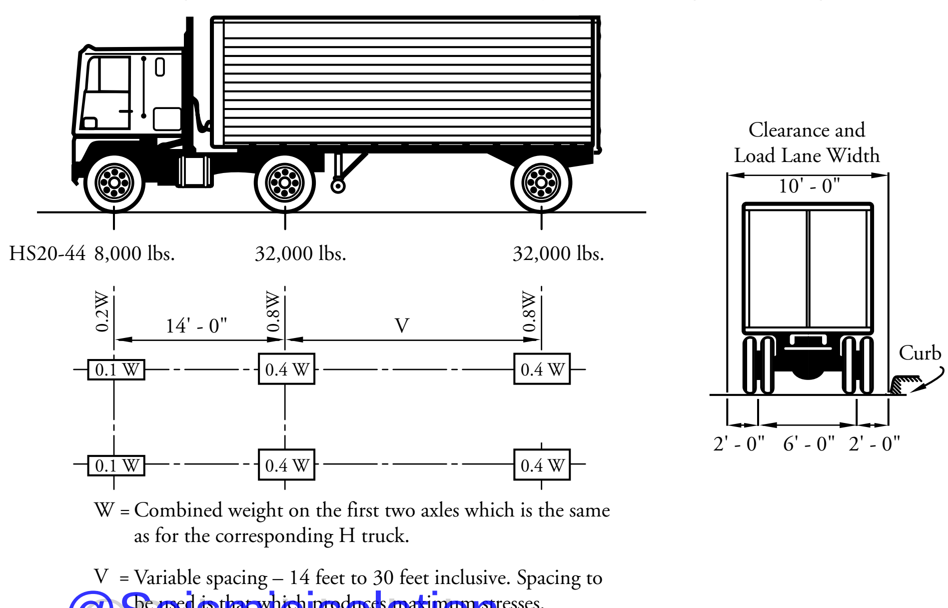


Figure 7.2.2.1.3-2
Standard HS Lane Load

*** FOR CONTINUOUS** span bridges an additional concentrated load should be used in determining maximum negative moment only (AASHTO 3.11.3). The second load should be placed in another span of the series. For simple span bridges and for the computation of maximum positive moment in continuous span bridges, a single concentrated load is used as shown.

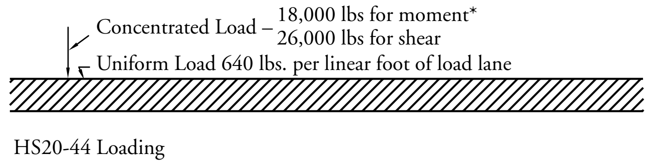
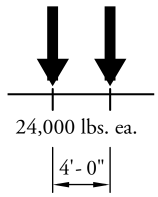


Figure 7.2.2.1.3-3
Tandem Loading
(Alternate Military)



HS20-44 loading shown in **Figure 7.2.2.1.3-1** and **Figure 7.2.2.1.3-2**. The lane loading usually controls beam design for spans longer than approximately 140 ft. For simple spans, the variable distance between rear axles, V, should be set at the 14 ft minimum. In continuous spans, the distance V is varied to create the maximum negative moment. In checking for lane loading in continuous spans, two concentrated loads are used to maximize negative moment (STD Article 3.11.3).

A tandem load, known as the Alternate Military Loading, **Figure 7.2.2.1.3-3**, is also required in the design of U.S. Interstate System bridges. This loading simulates heavy military vehicles and may control beam design in the case of spans shorter than approximately 40 ft.

Some states have begun using the HS25 design loading which represents a 25 percent increase over the standard HS20 truck and lane loadings. Furthermore, in order to provide for potential overweight trucks, some states have developed additional live load configurations known as permit design loadings. These loadings may control the design of prestressed beams and slab design.

7.2.2.1.4 Design Vehicular Live Load - LRFD Specifications

[LRFD Art. 3.6]

The vehicular live loading on bridges, designated as HL-93, consists of a combination of the:

Design truck OR Design tandem
AND
Design lane load

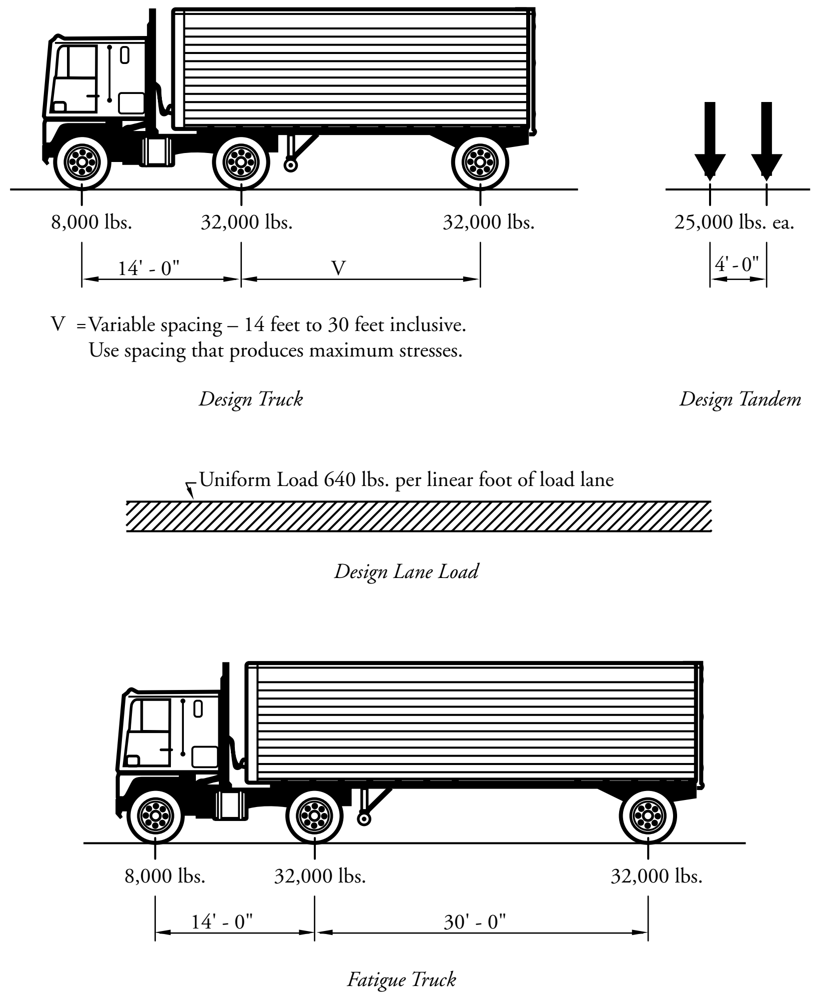
The design truck is the HS20 vehicle used in the *Standard Specifications,* **Figure 7.2.2.1.4-1**. The design tandem consists of a pair of 25.0 kip axles spaced 4.0 ft apart. In either case, the transverse spacing of wheels is taken as 6.0 ft. The design lane load consists of a uniform load of 0.64 klf in the longitudinal direction. It is distributed transversely over a 10.0 ft width.

The extreme force effect for the vehicular live load is the larger of the following:

- The combined effect of the design tandem with the design lane load, or
- The combined effect of one design truck with the variable axle spacing with the design lane load, and
- For continuous members, for both negative moment between points of dead load contraflexure and reaction at interior piers only: the combination of 90% of the effect of two design trucks (spaced a minimum of 50.0 ft between the lead axle of

one and the rear axle of the other truck) with 90% of the effect of the design lane load. The distance between the 32.0 kip axles of each truck shall be taken as 14.0 ft.

Figure 7.2.2.1.4-1 LRFD Design Vehicular Live Loads (HL-93) and Fatigue Load



Axles which do not contribute to the extreme force effect under consideration shall be neglected. Both the design lanes and the position of the 10.0 ft loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely so that the center of any wheel load is not closer than 2.0 ft from the edge of the design lane when designing beams.

Unless otherwise specified, the lengths of design lanes, or parts thereof, which contribute to the extreme force effect under consideration shall be loaded with the design lane load. Only those portions of the span which contribute to maximizing the force effect should be loaded. Influence lines can be used to determine those portions of the span which should be loaded for maximum effect.

7.2.2.1.5 Impact or Dynamic Load Allowance

In STD Article 3.8, the amount of the impact allowance or increment is expressed as a fraction of the live load and is determined using the formula:

$$I = 50/(L + 125) \qquad \text{[STD Eq. 3-1]}$$

where

I = impact fraction (maximum 0.30)

L = simple span length (except cantilevers) when computing truck load moments

= for shear due to truck loads: the length of the loaded portion of span from section under consideration to the far reaction. Note: In practice, the use of variable impact to calculate shear for simple and continuous spans is not used, rather the span length of the section under investigation is used.

In *LRFD Specifications* Article 3.6.2, the static effects of the design truck or tandem are multiplied by (1 + IM/100), where IM is the Dynamic Load Allowance as given for different bridge components below: [LRFD Table 3.6.2.1-1]

Deck joints: All limit states	75%
All other components:	
Fatigue and Fracture Limit State	15%
All Other Limit States	33%

This dynamic allowance is not applied to the design lane load or to pedestrian loads.

7.2.2.1.6 Fatigue Load

In the *Standard Specifications,* there are no provisions for any special fatigue loading in the case of prestressed beams. In the *LRFD Specifications*, there is a new provision for a single fatigue truck, **Figure** 7.2.2.1-4, but with a constant spacing of 30.0 ft between the 32.0-kip axles. The applicable dynamic load allowance is 15%. When the bridge is analyzed using approximate methods, the distribution factor for one traffic lane is to be used and the force effect is to be divided by 1.20 (except if the lever rule is used).

7.2.2.2 Other Vehicular Forces

7.2.2.2.1 Longitudinal (Braking) Forces

These forces result from vehicles accelerating or braking while traveling over a bridge. Forces are transferred from the wheels to the deck surface.

In the *Standard Specifications,* provision is to be made for a longitudinal force of 5% of the live load (without impact) in all lanes carrying traffic headed in the same direction. The center of gravity of such force is assumed to be located 6 ft above the slab and is transmitted to the substructure through the superstructure. Usually, the effect of braking forces on superstructures is inconsequential.

In the *LRFD Specifications,* the braking forces are taken as the greater of:

- 25% of the axle weights of the truck or tandem
- 5% of the truck plus lane load
- 5% of the tandem plus lane load

This braking force is placed in all lanes carrying traffic headed in the same direction. The multiple presence factor, m, is applicable here.

7.2.2.2.2 Centrifugal Forces

This effect must be considered for bridge structures on horizontal curves. The ratio of this force to the truck (or tandem) axle loads is proportional to the square of the design speed and inversely proportional to the curve radius. This force is applied at 6.0 ft above the roadway surface. Usually, concrete decks resist centrifugal forces within their own plane, and transmit them to the substructure through end diaphragms.

7.2.2.2.3 Vehicular Collision Forces

These forces need to be considered whenever piers or abutments are not adequately protected to prevent vehicle or railway collisions.

7.2.2.3 Pedestrian Loads

In the *Standard Specifications,* the sidewalk area is loaded with a variable uniform load which decreases with beam span. For spans larger than 25 ft, the maximum load is 60 psf.

In LRFD Article 3.6.1.6, a load of 0.075 ksf is applied to all sidewalks wider than 2.0 ft and must be considered with the vehicular live load. For bridges carrying only pedestrian and/or bicycle traffic, the load is set at 0.085 ksf.

The above provisions may be excessive where a significant sidewalk loading is unlikely.

7.2.3 Water and Stream Loads

These forces primarily affect substructure elements and are due to water course-related characteristics. Static water pressure is assumed perpendicular to the surface which is retaining the water, while buoyancy is an uplift force acting on all submerged components.

7.2.3.1 Stream Forces

Stream flow pressure affects the design of piers or supports located in water courses. The average pressure of flowing water on a pier is proportional to the square of water velocity, to the drag coefficient for a specific pier geometry and to the projected pier surface exposed to the design flood.

7.2.3.2 Ice Forces

Floating ice sheets and ice floes on streams cause major dynamic (and static) forces to act on piers in cold weather climates. If clearance is low, the superstructure may also be affected, often with severe damage. Usually, the dynamic force on a pier is a function of ice thickness, ice strength, pier width and inclination of the nose to vertical. Both the *Standard Specifications* and the *LRFD Specifications* contain detailed equations and factors for calculation of stream flow and floating ice loads on piers and supports.

7.2.4 Wind Loads

Wind is a dynamic load. However, it is generally approximated as a uniformly distributed static load on the exposed area of a bridge. This area is taken as the combined surfaces of both superstructure and substructure as seen in elevation (orthogonal to the assumed wind direction). AASHTO loads are based on an assumed "base wind velocity" of 100 mph.

7.2.4.1 Wind Forces - Standard Specifications

Wind forces are applied in a transverse and longitudinal direction at the center of gravity of the exposed region of the superstructure. The specifications provide wind loading values for beam bridges based on the angle of attack (skew angle) of wind forces. Conventional slab-on-stringer bridges with span lengths less than or equal to 125 ft can utilize the following basic loading:

Wind Load on Structure	
Transverse Loading	50 psf
Longitudinal Loading	12 psf
Wind Load on Live Load (Vehicle)	
Transverse Loading	100 plf, based on a long row of passenger cars exposed to a 55 mph wind
Longitudinal Loading	40 plf

The transverse and longitudinal loads are applied simultaneously to both the structure and live load. Also, an upward force acting on the deck must be considered.

7.2.4.2 Wind Forces - LRFD Specifications

A more refined analysis is required, although it follows the same general pattern of "wind pressure on structures" and "wind pressure on vehicles." The specifications also require varying the wind load direction to determine extreme force effects, and the consideration of a vertical upward force acting on the deck (especially when checking overturning of the bridge).

7.2.5 Earthquake Loads and Effects

7.2.5.1 Introduction

These temporary natural forces are assumed to act in the horizontal direction and are dependent on the geographic location of the bridge, the structure dead weight (mass), the ground motion (duration and acceleration), the period of the structural system and type of soil. In some cases, a vertical component of acceleration may have to be considered. These factors enter into the seismic analysis which is a simplification of the actual effects of an earthquake. The bridge response assumes the form of an equivalent static load which is applied to the structure to calculate forces and deformations of bridge elements.

For most pretensioned structures, where the superstructure is not integral with the substructure, earthquake forces do not affect beam design, see Chapter 15 for additional information about seismic design of prestressed beam bridges.

7.2.6 Forces Due to Imposed Deformations

These effects include temperature, creep, differential shrinkage and differential settlement. Some general guidelines are offered in the *LRFD Specifications.* Normally, the difference between the base construction temperature and the temperature range limits in a region is used to calculate thermal deformation effects. Nearly all engineers neglect the effect of temperature gradient in pretensioned multi-beam bridges. This practice has been used for over 40 years with good performance. For other types of bridges, judgment and experience should be used in deciding to consider the effects of temperature gradient. Where appropriate, the effects of creep, differential shrinkage and differential settlements should be considered.

7.3 LOAD COMBINATIONS AND DESIGN METHODS

Vehicle live loads may act on a bridge simultaneously with other live loads. The design engineer is responsible to size and reinforce the structural components to safely resist the possible combinations of loads which may act on a bridge. Therefore, the *Standard Specifications* and *LRFD Specifications* contain load combinations, subdivided into various groups, which represent probable simultaneous loadings on the structure. In theory, all structural elements should be designed to resist all groups of loads. In practice, though, many of the groups do not control the design and may be disregarded.

There are two principal methods of design:

1. Service Load Design (Allowable Stress Design)
   In this method, the allowable stress is defined as the material strength (stress) reduced by a suitable factor of safety. The total stress caused by load effects must not exceed this allowable stress. This is expressed in the following relationship:
   $$f_{total} \leq f_{allowable} \qquad \text{(Eq. 7.3-1)}$$
2. Strength Design (Load Factor Design)
   In this method, the general relationship is defined as follows:

Provided Strength ≥ Required Strength
OR
Factored Resistance ≥ Factored Moment, Shear or Axial Force (Eq. 7.3-2)

The nominal resistance of a member, R_n, is computed using procedures given in the specifications. This value is then modified by a resistance factor, ϕ, appropriate for the specific conditions of design to obtain the provided strength. The load effects, Q_i, are usually calculated using conventional elastic analysis procedures. These are then modified by the specified load factors, γ_i, to obtain the required strength. In a concise form, Equation 7.3-2 can be expressed as follows:

$$\phi R_n \geq \sum \gamma_i Q_i \qquad \text{(Eq. 7.3-3)}$$

where Q_i is the load effect.

7.3.1 Standard Specifications

Group loading combinations for Service Load Design and Load Factor Design are given by: [STD Art 3.22.1]

$$\begin{aligned}\text{Group}(N) = \gamma\,[&\beta_D D + \beta_L (L + I) + \beta_C CF + \beta_E E \\ &+ \beta_B B + \beta_S SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_L LF + \beta_R (R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned} \qquad \text{[STD Eq. 3-10]}$$

where

N = group number
γ = load factor, see **Table 7.3.1-1**
β = coefficient, see **Table 7.3.1-1**
D = dead load
L = live load
I = live load impact
E = earth pressure
B = buoyancy
W = wind load on structure
LF = longitudinal force from live load
CF = centrifugal force
R = rib shortening
S = shrinkage
T = temperature
EQ = earthquake
SF = stream flow pressure
ICE = ice pressure
WL = wind load on live load

For Service Load Design, the percentage of the basic unit stress for the various groups is given in **Table 7.3.1-1**. In the design of pretensioned flexural elements in the superstructure, such as stringers or beams, the design is governed by the Group I loading combination which may be stated as:

$$f_D + f_{(L+I)} \leq f_{allowable} \quad \text{(Eq. 7.3.1-2)}$$

where

f_D = the sum of dead load bending stresses

$f_{(L+I)}$ = live load plus impact bending stress

Table 7.3.1-1
Table of Coefficients γ and β–Standard Specifications

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
GROUP		γ	β FACTORS													%	
			D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE		
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		Culvert

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L+I)_p$ - Live load plus impact consistent with overload criteria of the operation agency

* and ** - Refer to *Standard Specifications* for explanation

For Load Factor Design of pretensioned stringers or beams, the section design is also governed by Group I requirements:

Provided Strength ≥ 1.3[D + 1.67(L+I)] (Eq. 7.3.1-3)

One exception is the case of an outside roadway beam when the combination of sidewalk live load and traffic live load (plus impact) may govern the design. Then the load factor 1.67 may be replaced by 1.25, provided the section capacity is not less than that required for traffic live load only using β_L = 1.67.

In many states, structures are required to be analyzed for an overload that is selected by the particular transportation department. This load is then applied in Group IB as defined in **Table 7.3.1-1**, which may or may not control the design.

7.3.2 LRFD Specifications

The total factored load, Q, is given by:

$$Q = \eta \, \Sigma \, \gamma_i q_i \qquad \text{(Eq. 7.3.2-1)}$$

where

η = variable load modifier which depends on ductility, redundancy and operational importance. Its value is often set by state DOTs

q_i = specified loads

γ_i = load factors specified in **Tables 7.3.2-1** and **7.3.2-2**

Table 7.3.2-1
Load Combinations and Load Factors, LRFD Specifications [LRFD Table 3.4.1-1]

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time: EQ	IC	CT	CV
STRENGTH-I	γ_p	1.75	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-II	γ_p	1.35	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-III	γ_p	–	1.00	1.40	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
STRENGTH-IV EH, EV, ES, DW	γ_p	–	1.00	–	–	1.00	0.50/1.20	–	–	–	–	–	–
DC ONLY	1.5												
STRENGTH-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
EXTREME EVENT-I	γ_p	γ_{EQ}	1.00	–	–	1.00	–	–	–	1.00	–	–	–
EXTREME EVENT-II	γ_p	0.50	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-II	1.00	1.30	1.00	–	–	1.00	1.00/1.20	–	–	–	–	–	–
SERVICE-III	1.00	0.80	1.00	–	–	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
SERVICE-IV	1.00	–	1.00	0.70	–	1.00	1.00	–	1.00	–	–	–	–
FATIGUE-LL, IM & CE ONLY	–	0.75	–	–	–	–	–	–	–	–	–	–	–

For notes on γ_p, γ_{EQ}, γ_{TG} and γ_{SE}, refer to *LRFD Specifications*.

Table 7.3.2-2
Load Factors for Permanent Loads, γ_p***, LRFD Specifications***
[LRFD Table 3.4.1-2]

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
• Overall Stability	1.00	N/A
• Retaining Walls and Abutments	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

Components (and connections) of a bridge structure must satisfy the applicable combinations of factored extreme force effects as specified at each of the limit states. The following load designations are used:

- Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EL = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
EH = horizontal earth pressure load
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

- Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = settlement
SH = shrinkage
TG = temperature gradient
TU = uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

As has always been the case, the owner or designer may determine that not all of the loads in a given load combination apply to the situation being investigated. The various applicable load factors are in **Tables 7.3.2-1** and **7.3.2-2**. The minimum load factors are especially important in the negative moment regions of continuous beams.

The factors must be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes must be investigated. In load combinations where one force effect decreases the effect of another, the minimum value is applied to the load reducing the force effect. For permanent force effects, the load factor (maximum or minimum) which produces the more critical combination is selected from **Table 7.3.2-2**.

The design of pretensioned superstructure beams using the *LRFD Specifications* usually consists of satisfying the requirements of Service I, Service III and Strength I load combinations. Use of the new larger vehicular live load for working stress design of prestressed concrete members would result in over-conservative designs. Also, since no significant cracking has been observed in existing bridges that were designed for the relatively lower loads of the *Standard Specifications,* the Service III load combination was introduced. Service III specifies a load factor of 0.80 to reduce the effect of live load at the service limit state. This combination is only applicable when checking allowable tensile stresses in prestressed concrete superstructure members. Service I is used when checking compressive stresses only. The load combination Strength I is used for design at the strength limit state. Other load combinations for the strength and extreme event limit states are not considered here, but may be required by specific agencies or DOTs—such as Strength II combination for permit vehicles.

The various load combinations applicable to prestressed beams and substructures (Service IV) and shown in **Table 7.3.2-1** are described below.

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

STRENGTH II - Load combination relating to the use of the bridge by permit vehicles without wind. If a permit vehicle is traveling unescorted, or if control is not provided by the escorts, the other lanes may be assumed to be occupied by the vehicular live load herein specified. For bridges longer than the permit vehicle, addition of the lane load, preceding and following the permit load in its lane, should be considered.

SERVICE I - Load combination relating to the normal operational use of the bridge with 55 mph wind. All loads are taken at their nominal values and extreme load conditions are excluded. Compression in prestressed concrete components is investigated using this load combination.

SERVICE III - Load combination relating only to prestressed concrete superstructures with the primary objective of crack control. Tensile stress in prestressed concrete superstructure members is investigated using this load combination.

SERVICE IV - Load combination relating only to tension in prestressed concrete substructures with the primary objective of crack control. Tensile stress in prestressed concrete substructure members is investigated using this load combination.

FATIGUE - Fatigue and fracture load combination relating to gravitational vehicular live load and dynamic response. Consequently BR, LS and PL loads need not be considered. The load factor is applied to a single design truck.

7.4 LIVE LOAD DISTRIBUTION - STANDARD SPECIFICATIONS

7.4.1 Introduction and Background

The following sections present several approximate formulas for live load distribution factors taken from the *Standard Specifications.* A wheel load is defined as one half of a full lane (or truck) load. These procedures may be used in lieu of refined methods, such as the finite element or grillage analysis (see Section 7.6). They utilize the concept of a wheel load distribution factor, g, for bending moment and shear in interior beams given by:

$g = S/D$, or (Eq. 7.4.1-1)

g = function of: number of lanes and beams and S/L (Eq. 7.4.1-2)

where

- g = a factor used to multiply the total longitudinal response of the bridge due to a single longitudinal line of wheel loads in order to determine the maximum response of a single beam.
- S = center-to-center beam spacing, ft
- D = a constant that varies with bridge type and geometry, ft
- L = span length, ft

The live load bending moment for each interior beam, is determined by applying to the beam, the fraction of a wheel load as determined from the applicable equation. No longitudinal distribution of loads is assumed. Except for the case of multi-beam decks, the live load moment for exterior beams is determined by applying to the beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between the beams.

The approximate equations described below are suitable for the design of normal (non-skewed) bridge decks. There are no guidelines for adjustments in the case of skews. Designers should be aware that a major shortcoming of the current specifications is that the piecemeal changes that have taken place over the last four decades have led to inconsistencies and general conservatism in the load distribution criteria.

7.4.2 Distribution Factors for I-Beams and Bulb-Tees

[STD Arts. 3.23.2.2 and 3.23.2.3.1.2]

When a bridge is designed for two or more traffic lanes and the beam spacing, $S \leq 14$ ft, the distribution factor for interior beams is determined by:

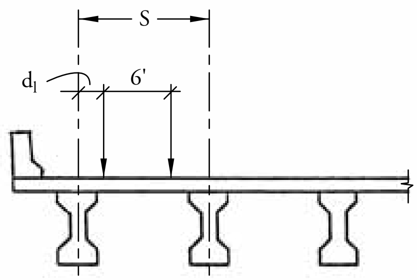
$g = S/5.5$ (Eq. 7.4.2-1)

If a bridge is narrow and designed for only one traffic lane then:

$g = S/7.0$ (Eq. 7.4.2-2)

Eq. (7.4.2-1) is credited to Newmark and has not changed until the introduction of the *LRFD Specifications.* Although composite double tee decks are not specifically listed in STD Table 3.23.1, it has been a common practice to use this equation with S equal to the stem or web spacing.

Figure 7.4.2-1
Distribution Factor for Exterior Beam



For exterior beams, the distribution factor is determined using the requirements of STD Article 3.23.2.3.1.2. These provisions, which are often called the "Lever Rule," are best explained by an example, as shown in **Figure 7.4.2-1**. The bridge deck is modeled as a simple span with an overhang. The fraction of a wheel load carried by exterior beams is determined by summing moments about the center of the first interior beam.

$$g = \{(S - d_1)/S\} + \{(S - d_1 - 6)/S\} \qquad \text{(Eq. 7.4.2-3)}$$

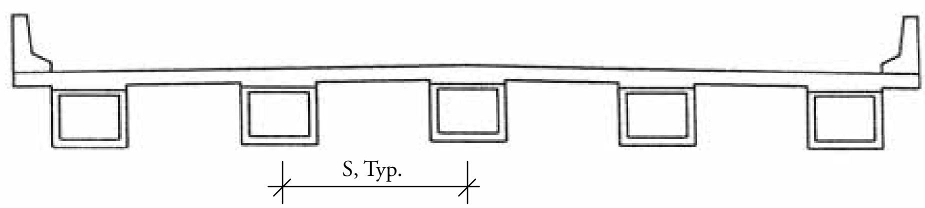
If the overhang is wide enough to accommodate a wheel position outside of the center of the exterior beam, then d_1 is negative in Eq. (7.4.2-3).

7.4.3 Distribution Factors for Spread Box Beams

[STD Art. 3.28]

The live load bending moment for each interior beam in a spread box beam superstructure is computed by applying to the beam the fraction of a wheel load **Figure 7.4.3-1** determined by the following equation:

Figure 7.4.3-1
Typical Cross-Section of a Spread Box Beam Bridge Deck



$$g = \frac{2N_L}{N_B} + k\left(\frac{S}{L}\right) \qquad \text{[STD Eq. 3-33]}$$

where

$k = 0.07\,W - N_L(0.10N_L - 0.26) - 0.20N_B - 0.12$ [STD Eq. 3-34]

N_L = number of design traffic lanes

N_B = number of beams ($4 \le N_B \le 10$)

S = beam spacing, ft ($6.57 \le S \le 11.00$)

L = span length, ft

W = roadway width between curbs, ft ($32 \le W \le 66$)

These two equations are based on a statistical correlation with the results of finite element analyses covering some 300 cases (Motarjemi, 1969). However, no multi-lane reduction factor was considered. If a spread box beam bridge is designed for a two-lane roadway, there is probably little advantage in using a refined analysis method.

For exterior beams, the lever rule discussed in the preceding section is used.

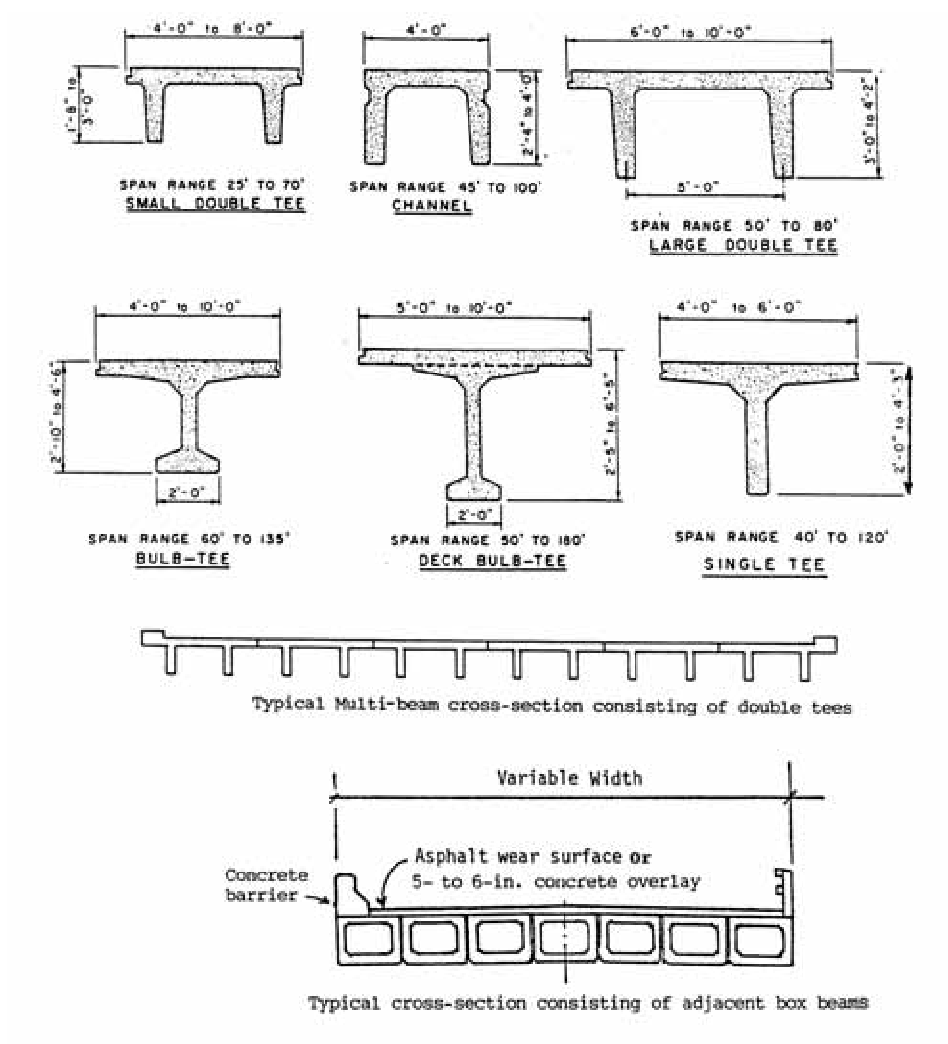
7.4.4 Distribution Factors for Adjacent Box Beams and Multi-Beam Decks

[STD Art. 3.23.4]

A multi-beam bridge deck consists of precast or prestressed concrete beams that are placed side-by-side on the supports. Adjacent box beams, channels, double tees, deck bulb-tees and solid or hollow slabs (**Figure 7.4.4-1**) fall under this category. A structural concrete or asphalt overlay may be required by state or local practice.

In general, the interaction between beams is developed by continuous longitudinal shear keys used in combination with metal tie plates or lateral bolting or prestressing. Full-depth rigid end diaphragms for channel, single tee, or multi-stem beams are required by the specifications. However, midspan diaphragms often are not required by local practice. It has been traditional in some states to use steel cross frames or K-braces in lieu of cast-in-place end diaphragms.

Figure 7.4.4-1
Adjacent Box Beam and Multi-Beam Stemmed Sections with Approximate Geometries and Span Ranges



The live load distribution factor for interior or exterior beams is given by:

$$g = S/D \qquad \text{[STD Eq. 3-11]}$$

where

S = width of precast member, ft

$$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2 \qquad \text{[STD Eq. 3-12]}$$

where

N_L = number of traffic lanes

C = K(W/L) for W/L < 1

= K for W/L ≥ 1 [STD Eq. 3-13]

where

W = overall (edge-to-edge) width of bridge measured perpendicular to the longitudinal beams, ft

L = span length measured parallel to longitudinal beams, ft; for beams with cast-in-place end diaphragms, use the length between diaphragms

$$K = [(1 + \mu)I/J]^{0.5} \quad \text{(Eq. 7.4.4-1)}$$

where

I = moment of inertia

J = St. Venant torsional constant

μ = Poisson's ratio for beams

For preliminary design, approximate values of K are provided in the LRFD Specification.

7.5 SIMPLIFIED DISTRIBUTION METHODS—LRFD SPECIFICATIONS

The following sections will focus on precast, prestressed concrete bridges using box, I-, bulb-tee or multi-stem beam cross sections. The majority of the live load distribution formulas in the *LRFD Specifications* are entirely new and are based on an NCHRP project (Zokaie, 1991). However, as with any new technology, revisions and clarifications are inevitable.

7.5.1 Background

Advanced computer technology and refined procedures of analysis—such as the finite element method—constitute the basis for development of the approximate formulas given in the *LRFD Specifications.* First, a large database of more than 800 actual bridges was randomly compiled from various states to achieve national representation. Then average bridges were obtained for each slab and beam category. Finally, refined analyses were implemented on selected bridges from each group.

Approximate formulas were developed to capture the variation of load distribution factors with each of the dominant geometric and material parameters. It was assumed that the effect of each parameter could be modeled by an exponential function of the form ax^b, where 'x' is the value of the given parameter (span, spacing, box depth, etc.) and 'b' is an exponent to be defined. The final distribution factor is given in the following general format which is based on a multiple regression analysis:

$$D.F. = A + B(x)^b(y)^c(z)^d \ldots \quad \text{(Eq. 7.5.1-1)}$$

Although the multiple exponential procedure worked well in many cases, it is inherently conservative in general because of several assumptions made during its development, such as:

- midspan diaphragms were disregarded thereby increasing moments in interior beams and reducing moments in exterior beams
- multi-lane presence factors were higher than the final factors used in the *LRFD Specifications* (See Sec. 7.2.2.1.2)
- the width of the concrete parapet (1'-6" or 1'-9") was often neglected, thereby increasing the load factors for the first two beams.

Furthermore, in order to assure conservative results, the constants in the formulas were adjusted so that the ratio of the average value computed using the approximate method to the accurate distribution factor was always larger than 1.0.

7.5.1.1 Introduction

LRFD Article 4.6.2.2 presents approximate live load distribution factors that may be used when a refined method is not used. Different structure types are identified descriptively and graphically in **LRFD Table 4.6.2.2.1-1** to assist the designer in using the correct distribution factor for the structure being designed. There are 12 structure types included in the table, eight of which utilize precast concrete.

Longitudinal joints connecting adjacent members are shown for five of the types of structures. If adjacent beams are "sufficiently connected to act as a unit," they may be considered to act monolithically. Those types without composite structural concrete topping may require transverse post-tensioning. (See Section 7.5.5.)

The live load distribution factors for beam-slab bridges presented in the *LRFD Specifications* are significantly different from those used in the *Standard Specifications.* The differences between the two specifications include:

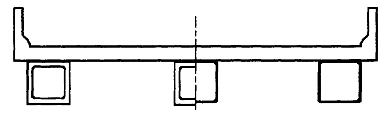
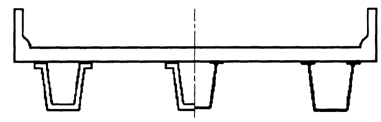
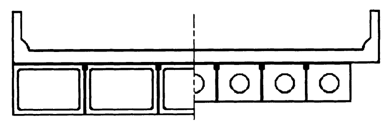
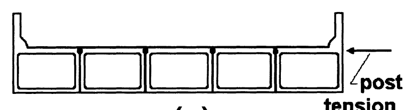
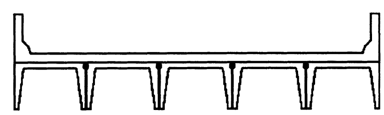
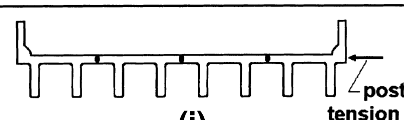
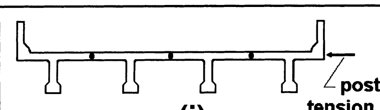
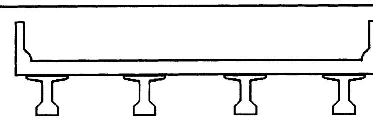
- There are now eight types of distribution factors for different types of structures and connections between beams, four of which apply to precast concrete sections.
- Separate distribution factors are provided for moment and shear in interior beams.
- Distribution factors for moment and shear in exterior beams are computed either by modifying the distribution factor for interior beams or by using the lever rule.
- Where rigid intermediate diaphragms are provided, the load on the exterior beams must also be checked assuming that the cross section remains straight, deflecting and rotating as a rigid body.
- The effect of multiple lane loading is included in the distribution factors. Therefore, multiple presence factors should not be used unless a refined analysis method is used or the lever arm procedure is required.
- For skewed bridges, distribution factors for moment and shear are adjusted using factors given for different structure types in appropriate tables.

The following general conditions must be satisfied for the approximate distribution factor equations to be used:

- the width of deck is constant
- the number of beams is not less than three, four or five depending on the case
- beams are parallel and have approximately the same stiffness
- unless otherwise specified, the roadway part of the overhang, d_e, does not exceed 3.0 ft
- curvature in plan is less than the specified limit
- the cross-section is consistent with one of the cross-sections shown in **Figure 7.5.1-1**
- for beams, other than box beams, used in multi-beam decks with shear keys:
  - deep, rigid end diaphragms are required
  - if the stem spacing of stemmed beams is less than 4.0 ft or more than 10.0 ft, a refined analysis is to be used

All formulas in the tables in the *LRFD Specifications* provide the live load distribution per lane. Where roadway width is larger than 20 ft, the formulas for "Two or More Design Lanes Loaded" must be used for the following limit states: Strength I, Service I and Service III. For the Strength II limit state, the same distribution factor may be used. However, results can be overly conservative if the permit load is heavy. To circumvent this

Figure 7.5.1-1
Common Deck Superstructures
[LRFD Table 4.6.2.2.1-1]

SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	 (b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	 (c)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	 (f)
Precast Solid, Voided or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (g)
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	 (h)
Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (i)
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	 (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	 (k)

situation, where it controls the design, the engineer can use a refined method as discussed in Section 7.6. Finally, when checking for fatigue, the formulas for "One Design Lane Loaded" must be used. In the following sections, two loaded lanes will be assumed.

Specific limitations for each equation are given in the tables. These must also be satisfied before the equations can be used.

Where bridges meet the specified conditions, permanent superimposed loads, such as parapets and wearing surface, may be distributed equally between all beams in the bridge.

The live load distribution factors specified herein may also be used for permit and rating vehicles whose overall width is comparable to the width of the design truck.

7.5.2 Approximate Distribution Formulas for Moments (Two Lanes Loaded)

[LRFD Art. 4.6.2.2]
[LRFD Table 4.6.2.2.2b-1]
[LRFD Table 4.6.2.2.2d-1]

The following notation is used in the distribution factor equations:

A = area of stringer, or beam, in.^{2}

b = width of beam, in.

C = stiffness parameter = $K(W/L)$

d = depth of beam, in.

d_e = distance between the center of exterior beam and interior edge of curb or traffic barrier, ft

D = width of distribution per lane, ft

e = correction factor

g = distribution factor

J = St. Venant torsional constant, in.^{4}

K = a non-dimensional constant

K_g = longitudinal stiffness parameter, in.^{4}

L = span of beam, ft

N_b = number of beams

N_L = number of design lanes

S = spacing of beams or webs, ft

t_s = depth of concrete slab, in.

W = edge-to-edge width of bridge, ft

θ = skew angle, deg

μ = Poisson's ratio, usually assumed equal to 0.20

The longitudinal stiffness parameter, K_g, is taken as:

$$K_g = n(I + Ae_g^2) \qquad \text{[LRFD Eq. 4.6.2.2.1-1]}$$

where

n = modular ratio between beam and deck materials, generally ≥ 1

I = moment of inertia of beam, in.^{4}

e_g = distance between the centers of gravity of the beam and deck, in.

7.5.2.1 I-Beam, Bulb-Tee, or Single or Double Tee Beams with Transverse Post-Tensioning

The applicable live load distribution factor equation for interior beams [**Figure 7.5.1-1, types (i)**, **(j)** and **(k)**] is:

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1} \qquad \text{(Eq. 7.5.2.1-1)}$$

The only practical conditions affecting applicability of this equation are that N_b must be equal to or larger than 4 and $10{,}000 \le K_g \le 7{,}000{,}000$. The latter limit may be exceeded in the case of I-beams that are 96 in. deep or more. For preliminary design, the engineer may assume that $(K_g/12.0Lt_s^3)^{0.1} \doteq 1.10$, which is an average value obtained from a large database.

The equation for exterior beams without midspan diaphragms is:

$g = eg_{interior}$ (Eq. 7.5.2.1-2)

where $e = 0.77 + (d_e/9.1) \ge 1.0$ (Eq. 7.5.2.1-2a)

If rigid midspan diaphragms are used in the cross-section, an additional check is required using an interim, conservative procedure for I- and bulb-tee beam sections and applying the related multiple presence factor, m:

$$g \ge R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{(Eq. 7.5.2.1-3) [LRFD Eq. C4.6.2.2.2d-1]}$$

where

R = reaction on exterior beam in terms of lanes

N_L = number of loaded lanes under consideration

N_b = number of beams

e = eccentricity of a lane from the center of gravity of the pattern of beams, ft

x = horizontal distance from the center of gravity of the pattern of beams to each beam, ft

X_{ext} = horizontal distance from the center of gravity of the pattern of beams to the exterior beam, ft

7.5.2.2 Open or Closed Precast Spread Box Beams with Cast-In-Place Deck

The live load flexural moment for interior beams [**Figure 7.5.1-1, types (b)** and **(c)**] may be determined by applying the following lane fraction:

$$g = \left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125} \qquad \text{(Eq. 7.5.2.2-1)}$$

where d = precast beam depth.

This formula is subject to two practical limitations: $N_b \ge 3$ and $6.0 \le S \le 18.0$ ft. The other geometric conditions are usually met.

The corresponding formula for exterior beams is:

$g = eg_{interior}$ (Eq. 7.5.2.2-2)

where $e = 0.97 + (d_e/28.5)$ (Eq. 7.5.2.2-2a)

Equation (7.5.2.1-3) must also be checked in the case of rigid midspan diaphragms.

7.5.2.3
Adjacent Box Beams with Cast-In-Place Overlay or Transverse Post-Tensioning

The applicable distribution factor equation for interior beams [**Figure 7.5.1-1, types (f)** and **(g)**], is given by:

$$g = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06} \qquad \text{(Eq. 7.5.2.3-1)}$$

where $k = 2.5(N_b)^{-0.2} \geq 1.5$ (Eq. 7.5.2.3-1a)

In a preliminary design situation one may assume $(I/J)^{0.06} = 1.0$. These equations are limited to box beam widths not exceeding 5.0 ft and to span lengths $L \leq 120$ ft.

The bending moment for exterior beams is determined by applying the following lane fraction:

$$g = eg_{interior} \qquad \text{(Eq. 7.5.2.3-2)}$$

where $e = 1.04 + (d_e/25)$, $d_e \leq 2.0$ (Eq. 7.5.2.3-2a)

7.5.2.4
Channel Sections, or Box or Tee Sections Connected by "Hinges" at Interface

For interior beams, [**Figure 7.5.1-1, types (g)**, **(h)**, **(i)** and **(j)**], the applicable formula for the distribution factor, regardless of the number of loaded lanes, is:

$$g = S/D \qquad \text{(Eq. 7.5.2.4-1)}$$

where

$D = 11.5 - N_L + 1.4N_L(1 - 0.2C)^2$ when $C \leq 5$ (Eq. 7.5.2.4-1a)

$D = 11.5 - N_L$ when $C > 5$ (Eq. 7.5.2.4-1b)

where

$C = K(W/L) \leq K$ (Eq. 7.5.2.4-1c)

where $K = [(1 + \mu)I/J]^{0.5}$ (Eq. 7.5.2.4-1d)

LRFD Table 4.6.2.2.2b-1 suggests values of K for preliminary design.

The specified procedure for exterior beams is simply the 'Lever Rule' in conjunction with the multiple presence factor, m (see Section 7.2.2.1.2). However, this presents some interpretation problems regarding how many lanes should be loaded (say 2, 3 or 4 lanes if roadway width is 48 ft or more). Until this question is resolved, it is prudent to at least assign the same live load distribution factor for exterior beams as for interior beams, which is the approach used in the *Standard Specifications*. Furthermore, LRFD Article 2.5.2.7 requires that, in general, the load carrying capacity of an exterior beam be not less than the one for an interior beam.

7.5.3
Approximate Distribution Formulas for Shear (Two Lanes Loaded)

The live load shear for interior and exterior beams is determined by applying the lane fractions specified for the categories below. The shear distribution factors are normally higher than the moment factors for the same cross-section and span.

7.5.3.1
I-Beam, Bulb-Tee, or Single or Double Tee Beams with Transverse Post-Tensioning

The applicable live load distribution factor equation for interior beams, [**Figure 7.5.1-1, types (i)**, **(j)** and **(k)**], is:

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} \qquad \text{(Eq. 7.5.3.1-1)}$$

The only practical limitation on its applicability is $N_b \geq 4$.

The corresponding equation for exterior beams without midspan diaphragm is:

$g = eg_{interior}$ (Eq. 7.5.3.1-2)

where $e = 0.6 + (d_e/10)$ (Eq. 7.5.3.1-2a)

If rigid midspan diaphragms are present, then the conservative approach in Eq. (7.5.2.1-3) must be used.

7.5.3.2
Open or Closed Spread Box Beams with Cast-In-Place Deck

The live load shear for interior beams [**Figure 7.5.1-1, types (b)** and **(c)**], may be determined by applying the following lane fraction:

$$g = \left(\frac{S}{7.4}\right)^{0.8}\left(\frac{d}{12.0L}\right)^{0.1} \quad \text{(Eq. 7.5.3.2-1)}$$

The formula is subject to two practical limits: $N_b \geq 3$ and $6.0 \leq S \leq 18.0$ ft. The other conditions are generally satisfied.

The related equation for exterior beams is:

$g = eg_{interior}$ (Eq. 7.5.3.2-2)

where $e = 0.8 + (d_e/10)$ (Eq. 7.5.3.2-2a)

Equation (7.5.2.1-3) must also be checked in case of rigid midspan diaphragms.

7.5.3.3
Adjacent Box Beams in Multi-Beam Decks

The applicable distribution factor equation for interior beams [**Figure 7.5.1-1, types (f)** and **(g)**], is:

$$g = \left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12.0L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05} \quad \text{(Eq. 7.5.3.3-1)}$$

These equations are limited to box widths not exceeding 5.0 ft, to span lengths $L \leq 120$ ft and to I or $J \leq 610{,}000$ in^4. The latter value may be exceeded if depth exceeds 66 in.

The shear for exterior beams is determined by applying the following lane fraction:

$g = eg_{interior}$ (Eq. 7.5.3.3-2)

where $e = 1.02 + (d_e/50)$, $d_e \leq 2.0$ (Eq. 7.5.3.3-2a)

7.5.3.4
Channel Sections or Tee Sections Connected by "Hinges" at Interface

For interior or exterior beams [**Figure 7.5.1-1, types (h)**, **(i)** and **(j)**], the 'Lever Rule' in conjunction with the multiple presence factor, m, is specified.

7.5.4
Correction Factors For Skews

Skewed beam layout is generally dictated by complex highway intersections and/or by the lack of space in urban areas. When the skew angle of a bridge is small, say, less than 20°, it is often considered safe to ignore the angle of skew and to analyze the bridge as a zero-skew bridge whose span is equal to the skew span. This approach is generally conservative for moments in the beams, and slightly unsafe (<5%) for slab-on-beam decks for longitudinal shears.

LRFD Table 4.6.2.2.2e-1, lists reduction multipliers for moments in longitudinal beams. Also listed in LRFD Table 4.6.2.2.3c-1 are correction factors (> 1.0) applicable to the distribution factors for support shears at the obtuse corner of exterior beams. The commentary reminds the designer to check the possibility of uplift at the acute corners of large skews. Unfortunately, reliable multipliers and correction factors are missing for some bridge cross-sections.

7.5.4.1 Multipliers for Moments in Longitudinal Beams

Bending moments in interior and exterior beams on skewed supports may be reduced using the following multipliers: [LRFD Table 4.6.2.2.2e-1]

a) I-Beam, Bulb-Tee, Single or Double Tee Beams with Transverse Post-Tensioning [**Figure 7.5.1-1, types** (**i**), (**j**) and (**k**)]:

Use: $1 - c_1 (\tan\theta)^{1.5}$ (Eq. 7.5.4.1-1)

where $c_1 = 0.25\left(\dfrac{K_g}{12.0Lt_s^{\,3}}\right)^{0.25}\left(\dfrac{S}{L}\right)^{0.5}$ (Eq. 7.5.4.1-1a)

Set $c_1 = 0$ when $\theta < 30°$

Set $\theta = 60°$ when $\theta > 60°$

b) Spread Box Beams, Adjacent Box Beams with Concrete Overlays or Transverse Post-Tensioning, and Double Tees in Multi-Beam Decks [**Figure 7.5.1-1, types** (**b**), (**c**), (**f**) and (**g**)]:

Use: $1.05 - 0.25 \tan\theta \leq 1.0$ (Eq. 7.5.4.1-2)

Set $\theta = 60°$ if $\theta > 60°$

7.5.4.2 Multipliers for Support Shear at Obtuse Corners of Exterior Beams

Shears in exterior beams on the obtuse corner of the bridge may be reduced using the following multipliers: [LRFD Table 4.6.2.2.3c-1]

a) I-Beam, Bulb-Tee, Single or Double Tee Beams with Transverse Post-Tensioning [**Figure 7.5.1-1, types** (**i**), (**j**) and (**k**)]:

Use: $1.0 + 0.20\left(\dfrac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$ (Eq. 7.5.4.2-1)

This formula is valid for $\theta < 60°$.

b) Spread Box Beams [**Figure 7.5.1-1, types** (**b**) and (**c**)]:

Use: $1.0 + \left\{\left(\dfrac{Ld}{12.0}\right)^{0.5}\left(\dfrac{\tan\theta}{6S}\right)\right\}$ (Eq. 7.5.4.2-2)

Two practical limits apply, $\theta < 60°$ and $N_b \geq 3$.

c) Adjacent Box Beams with Cast-In-Place Overlay or Transverse Post-Tensioning [**Figure 7.5.1-1, types** (**f**) and (**g**)]:

Use: $1.0 + \left\{\dfrac{12.0L\,(\tan\theta)^{0.5}}{90\,d}\right\}$ (Eq. 7.5.4.2-3)

7.5.5 Lateral Bolting or Post-Tensioning Requirements

The following discussion concerns apparent inconsistencies in provisions of the *LRFD Specifications* related to the transverse connection between adjacent members.

7.5.5.1 Monolithic Behavior

As noted earlier, the *LRFD Specifications* indicate that adjacent beams connected by longitudinal joints may be considered to act monolithically if they are "sufficiently connected to act as a unit." The *LRFD Specifications* also note that transverse post-tensioning provides the best connection between adjacent beams to achieve monolithic behavior but that a reinforced structural concrete overlay may also be used.

7.5.5.2 Minimum Post-Tensioning Requirement

LRFD Commentary Article C4.6.2.2.1 *recommends* a minimum transverse post-tensioning stress of 0.250 ksi to make the beams act as a unit. However, in LRFD Article 5.14.1.2.8, this same level of effective stress is *required* for the connection between adjacent members if transverse post-tensioning is used. Excessively large post-tensioning forces will be required to achieve this level of prestress across the depth of typical shear keys. There is no support in the literature or current practice for requiring this high level of prestress.

7.5.5.3 Concrete Overlay Alternative

LRFD Article 5.14.4.3.3.f gives requirements for a structural concrete topping that can also be used to achieve monolithic action, according to LRFD Commentary Article C4.6.2.2.1.

7.6 REFINED ANALYSIS METHODS

7.6.1 Introduction and Background

LRFD Article 4.6.3 allows the use of refined methods of analysis for lateral load distribution in lieu of the tabulated simplified equations. Although the simplified equations are based on a statistical approach, they are often quite conservative.

7.6.2 The Economic Perspective

The refined methods most often used to study the behavior of bridges are the grillage analysis and the finite element methods. The finite element analysis (FEA) requires the fewest simplifying assumptions in accounting for the greatest number of variables which govern the structural response of the bridge deck. However, input preparation time, and derivation of overall forces for the composite beam are usually quite tedious. On the other hand, data preparation for the grillage method is simpler and integration of stresses is not needed.

7.6.2.1 Moment Reductions

Analyses by Aswad and Chen (1994) have shown that using the FEA may result in a reduction of the lateral load distribution factor for moments by at least 18% for interior I-beams when compared to the simplified LRFD approach. The analysis for exterior I-beams and spread box beams showed a smaller but non-negligible reduction.

7.6.2.2 Stretching Span Capability

Detailed prestress designs by Aswad (1994) have shown that the percentage reduction in strands and release strength for interior beams is roughly one-half of the reduction in the distribution factor. For instance, a 22% reduction of midspan moment will result in about 11% less strands and less required release strength, or may allow a 4 to 5% increase in span length without having to use a deeper section. Clearly, there is a significant incentive for both the owner and the industry to use refined methods in many future projects. This is especially significant for beams with higher span-to-depth ratios.

7.6.3 St. Venant Torsional Constant, J

An important step in the FEA method is the computation of the torsional constant, J, for the basic precast beam. The torsional constant of a thin-walled, hollow box section, is given by the familiar formula from standard textbooks (Hambly, 1976):

$$J = 4A_0^2/\Sigma(s/t) \qquad \text{(Eq. 7.6.3-1)}$$

where

A_0 = the area enclosed by centerlines of elements (walls)

s = the length of a side element

t = the thickness of that element

Table 7.6.3-1 Torsional Constant J for AASHTO I-Beams

Shape	J value, in.^{4}
Type I	4,745
Type II	7,793
Type III	17,044
Type IV	32,924
Type V	35,433
Type VI	36,071

For I-beams, the engineer should use rational methods such as those given in the report by Eby (1973). The use of formulas for open, thin sections is not appropriate. A list of St. Venant torsional constants for AASHTO I-beams is shown in **Table 7.6.3-1**.

7.6.4 Related Publications

The following reports by Lehigh University are recommended:

- For I-beams — Reports by Wegmuller (1973) and Zellin (1976)
- For spread box beams — Reports by Lin (1968), Guilford (1968), VanHorn (1969), Motarjemi (1969) and Chen (1970).

7.6.5 Modeling Guidelines

The following guidelines are suggested for refined analysis methods:

- A minimum of 9 nodes per beam span is preferred
- Aspect ratio of finite elements and grid panels should not exceed 5.0 (Note: this ratio should be reduced to 2.0 ± for better accuracy)
- Nodal loads shall be statically equivalent to the actual point load being applied
- For FEA, relative vertical distances should be maintained between various elements
- For grillage analysis, composite properties should be used
- St. Venant torsional constant, J, is to be determined rationally
- For grillage analysis, only one-half of the effective flange width of the flexural section, before transformation, should be used in computing J. In finite element analysis, an element should have membrane capability with sufficient discretization. Therefore, a shell element is ideal for modeling the cast-in-place slab.

7.6.6 Finite Element Study for Moment Distribution Factors

A parametric study for distribution factors was conducted by Chen and Aswad (1996) using FEA and the ADINA (1991) software. The number of beam elements per span was 16. There were two 4-noded shell elements between adjacent beam lines.

The study covered 10 different I-beam superstructures with spans, L, varying between 90 and 140 ft, and spacings, S, between 8 and 10 ft. The number of beam lines was 5, 6 or 7 while the total slab width (out-to-out) was either 48 or 60 ft. The midspan diaphragm is separated from the cast-in-place deck slab by a 6-in. gap.

The investigation also covered six various superstructures with a spacing, S, of either 8'-3" or 10'-6" and spans, L, varying between 60 and 100 ft. There were either 4 or 5 beam lines. The total slab width was either 39'-6" or 41'-0" which corresponds to 3 design lanes.

The following paragraphs summarize the findings of the study:

1. Refined methods of analysis may reduce the midspan moment by 18 to 23% in the case of interior I-beams, and by 4% to 12% for exterior I-beams when compared to the LRFD simplified method.

2. The same FEA may reduce the midspan moment by 6 to 12% for spread box beams. However, the reduction may reach 30% for exterior beams when midspan diaphragms are used. This is so because the *LRFD Specifications* have an interim formula that may result in an exaggerated midspan moment due to the assumption of infinitely rigid diaphragm.
3. The approximate equations for computing distribution factors are generally quite conservative when the span-to-depth ratios approach the upper limits of the span capability.

Based on this study, it is recommended that finite element or grillage analysis be used for the design of bridges with high span-to-depth ratios because they allow a significant reduction in the required release strength or, alternatively, a stretching of the span capability.

7.7 REFERENCES

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied [LRFD]

Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]

Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]

Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]

Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]

Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]

Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT} = total loss in pretensioning steel stress [LRFD]

Δf_s = total loss of prestress —

ε = strain —

ε_c = strain in concrete beam —

ε_{cr} = the time dependent creep strain —

ε_f = the immediate strain due to the applied stress f —

ε_{fc} = elastic strain in concrete —

ε_{fk} = element strain —

ε_{fs} = elastic strain in steel —

ε_k = strain in element k —

ε_p = strain in prestressing steel —

ε_s = strain in mild steel —

ε_s = tensile strain in cracked concrete in direction of tensile tie [LRFD]

ε_{sh} = free shrinkage strain —

$\varepsilon_{shb}(t,t_2)$ = shrinkage strain of the beam from time t_2 to time t —

$\varepsilon_{shb}(t_3,t_2)$ = shrinkage strain of the beam from time t_2 to time t_3 —

$\varepsilon_{shd}(t,t_3)$ = shrinkage strain of the deck from time t_3 to time t —

ε_{shu} = ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity —

ε_{si} = strain in tendons corresponding to initial effective pretension stress —

ε_x = longitudinal strain in the web reinforcement on the flexural tension side of the member [LRFD]

ε_{0c} = initial strain in concrete —

ε_1 = principal tensile strain in cracked concrete due to factored loads [LRFD]

γ^* = factor for type of pretensioning reinforcement [STD]

ϕ = strength reduction factor [STD]

ϕ = resistance factor [LRFD]

ϕ = curvature —

ϕ_c = curvature at midspan —

ϕ_{cr} = curvature due to creep —

ϕ_{fk} = element curvature —

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

Design Theory And Procedure

8.0 AASHTO SPECIFICATION REFERENCES

The references to the AASHTO Specifications in this chapter are based on the provisions of the *Standard Specifications*, 17th Edition, 2002, and the *LRFD Specifications*, 2nd Edition, 1998, with revisions through and including the 2003 Interim Revisions.

8.1 PRINCIPLES AND ADVANTAGES OF PRESTRESSING

8.1.1 History

The principles of prestressing have been used for centuries. For example, wooden barrels have always been made by tightening metal straps around barrel staves. In the making of early wheels, the wooden spokes and rim were first held together by a hot metal tire which, upon cooling, became tensioned. This induced radial compression on the rim and spokes. Other forms of mechanical, chemical and thermal prestressing have been attempted or used with varying degrees of success.

The use of prestressing for concrete bridge members has been employed with great success for over five decades. Concrete is strong in compression but relatively weak in tension. Therefore, prestressing is used to control tensile stresses and to precompress the concrete. This is analogous to providing the concrete with a "storage" capacity to resist loads which would otherwise produce tension and cracking in the concrete.

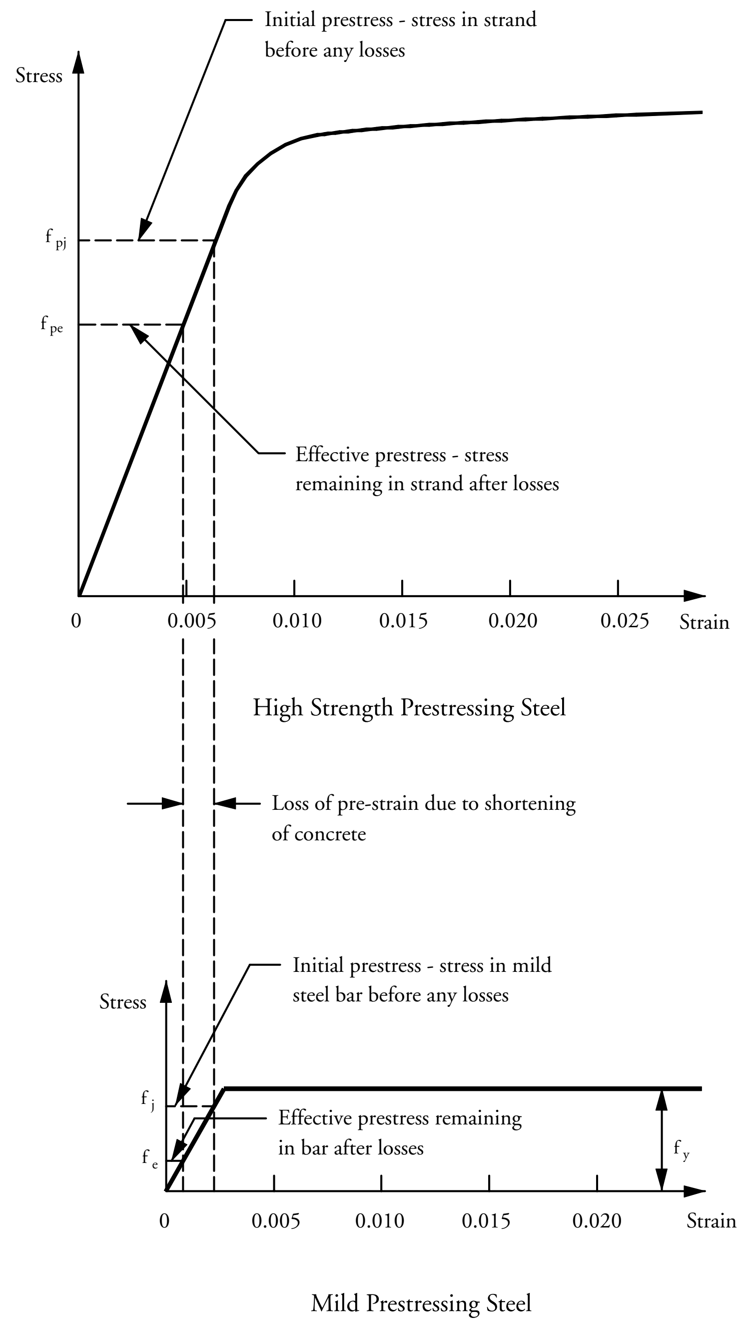
The prestressing of precast concrete bridge members is accomplished by stretching high strength steel strands, then casting concrete around them. As the concrete hardens, it bonds to the strands. When the clamps holding the tensioned strands are released, the force in the strands is applied to (or resisted by) the concrete. This puts the concrete into compression. This technique of prestressing, through the placing of concrete around prestretched strands, is called pretensioning. The high strength steel strands used for pretensioning typically have an ultimate strength, f_{pu}, of 270 ksi and a yield strength, f_{py}, of 243 ksi.

8.1.2 High Strength Steel

High strength steel is necessary for achieving prestressed concrete. Strands are typically tensioned initially to $0.75f_{pu}$ = 202.5 ksi. Although high, this stress is still safely less than yield (f_{py} = 243 ksi). Any loss of stress from this level will be elastic, related to strains by the modulus of elasticity. With time, creep and shrinkage cause shortening of the member, and, since they are bonded, shortening of the strands. The shortening of the strands relieves some of the prestrain in the strands, so the prestress is also reduced.

To illustrate why high strength steel is necessary, consider a concrete member pretensioned with high strength strand versus mild steel reinforcement (see **Figure 8.1.2-1**). Assume that the shortening of the member produces a corresponding loss of prestress of 40 ksi. The stress remaining in the strand after losses, which is called the "effective prestress," would therefore be 202.5 − 40 = 162.5 ksi. While the 40 ksi loss is significant, over 80% of the initial prestress remains. Compare this with the same member being prestressed using mild reinforcement (f_y = 60 ksi). In this case, the

Figure 8.1.2-1
Use of High Strength Versus Mild Steel for Prestressing Concrete



initial stress can only be about 50 ksi in order to remain safely below the yield stress and within the elastic range. Since the member and the prestress force are the same, the losses would also be the same, i.e., 40 ksi. However, in this case, the final conditions are much different, with the effective prestress dropping to 10 ksi, which leaves only 20% of the prestress remaining. So much of the prestress is lost using mild reinforcement for prestressing that it becomes ineffective and unreliable. The high level of prestrain in the strand due to the initial prestress is what makes high strength strand an effective method of prestressing. The large prestrain reduces the significance of losses.

Another outstanding benefit of high strength (Grade 270) strand is relative cost. While strand may cost nearly twice as much as mild reinforcement per pound, it provides over four times the strength of mild reinforcement. Furthermore, prestressing provides a significant enhancement in the behavior of reinforced concrete members. Thus, the combination of high-quality, plant-cast concrete with prestressing using high strength steel, results in the most economical bridge for most situations.

8.1.3 Prestressing Versus Conventional Reinforcing

The behavior of flexural members is illustrated using **Figures 8.1.3-1, 2** and **3**. **Figure 8.1.3-1** shows the conditions in a reinforced concrete member that has mild reinforcement and no prestressing. Under service load conditions, concrete on the tension side of the neutral axis is assumed to be cracked. Only concrete on the compression side is effective in resisting loads. In comparison, a prestressed concrete member is normally designed to remain uncracked under service loads (see **Figure 8.1.3-2**). Since the full cross-section is effective, the prestressed member is much stiffer than a conventionally reinforced concrete member resulting in reduced deflection (see **Figure 8.1.3-3**). No unsightly cracks are expected to be seen. Reinforcement is better protected against corrosion. Fatigue of strand due to repeated truck loading is generally not a design issue when the concrete surrounding the strands is not allowed to crack.

Figure 8.1.3-1
Behavior of Conventionally Reinforced Concrete Members

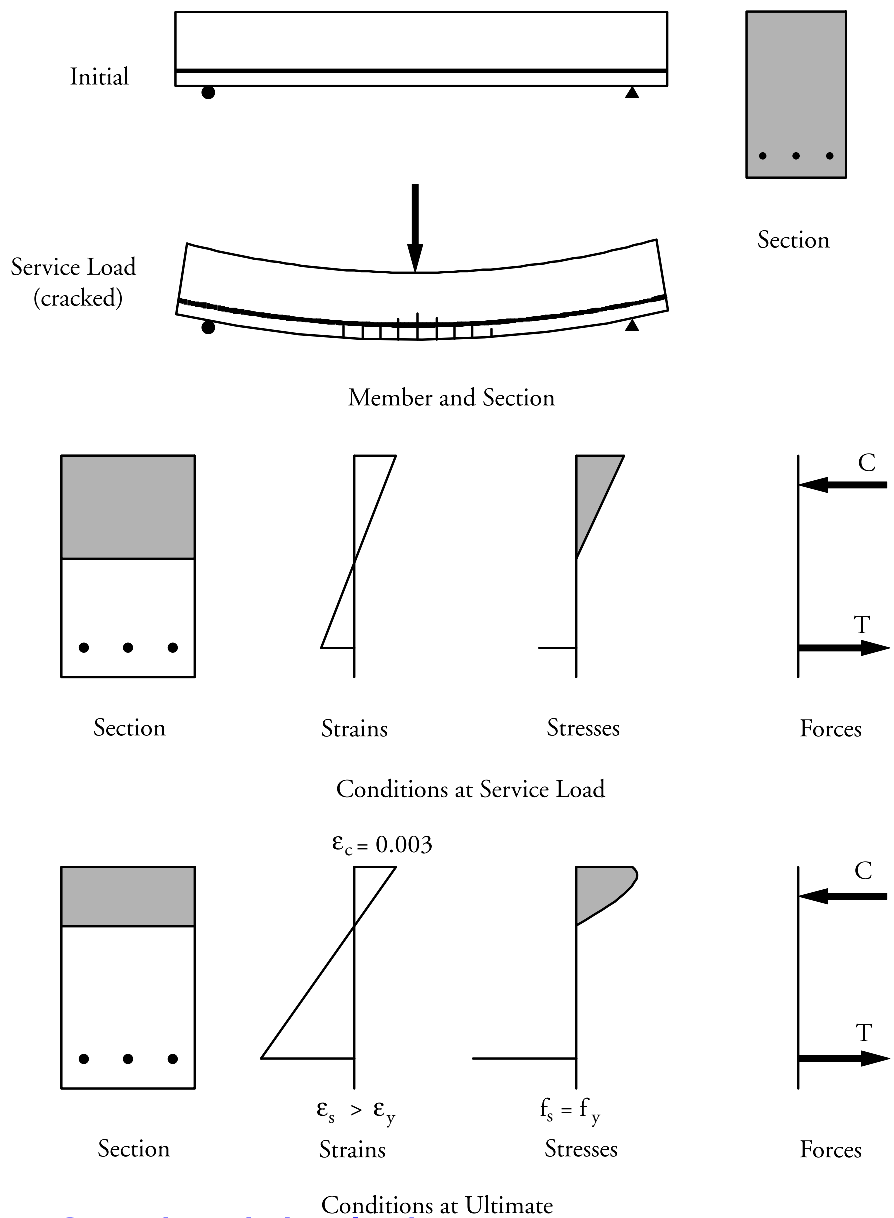
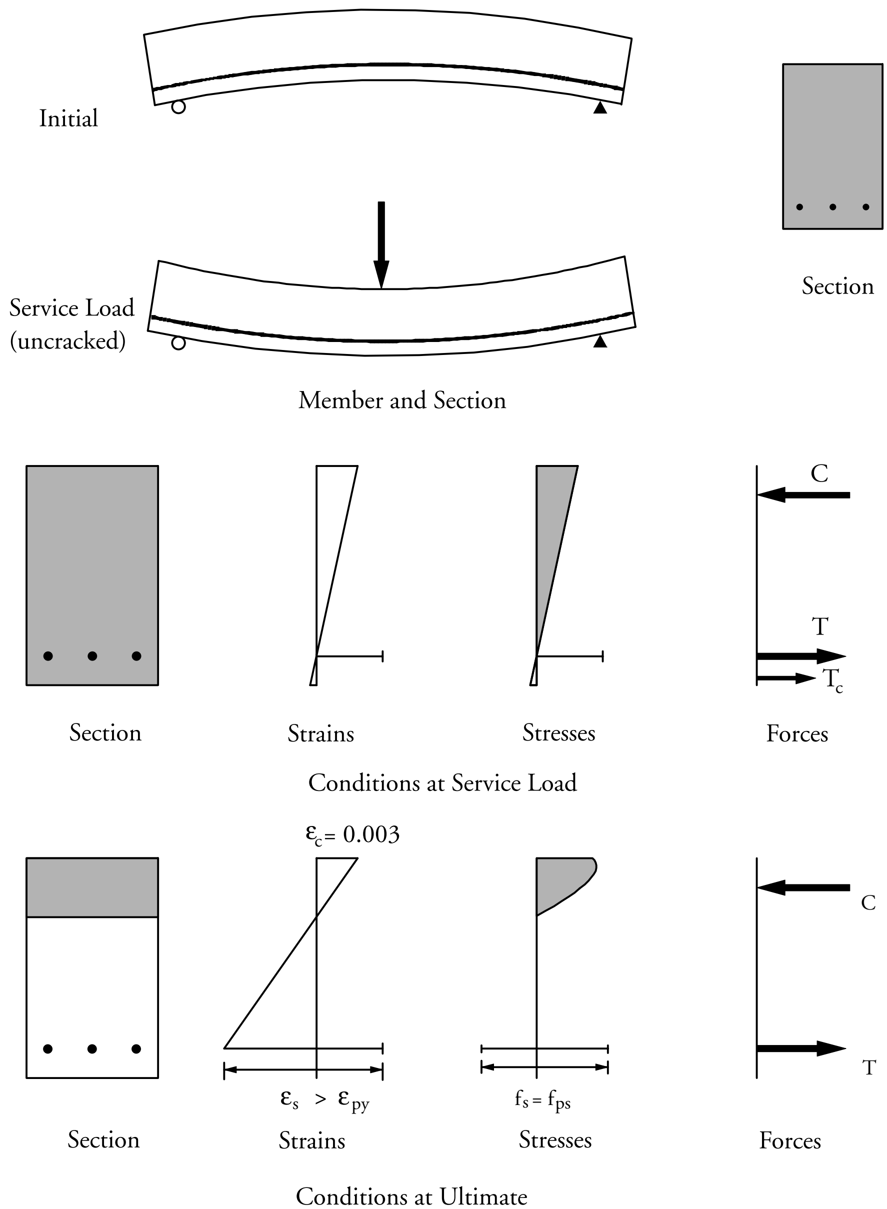
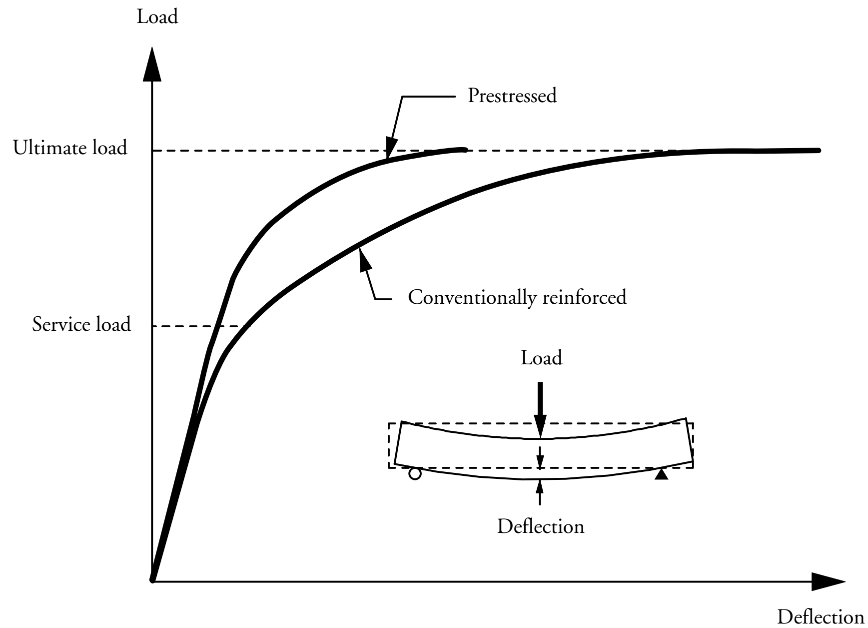


Figure 8.1.3-2
Behavior of Prestressed Concrete Members



At ultimate load conditions, conventionally reinforced concrete and prestressed concrete behave similarly. However, due to the lower strength of mild bars, a larger steel quantity is needed to achieve the same strength as a prestressed member. This increases the member material costs for a conventionally reinforced member. It should be noted, however, that strand has a lower ultimate elongation at rupture (about 4 to 6%) than that of Grade 60 reinforcement (about 10 to 15%). This lower strain capacity or material ductility may lead one to expect that prestressed concrete members may lack ductility or the capacity to deflect adequately prior to failure. However, prestressed concrete members have been shown both analytically and experimentally to have more than adequate deflection capability prior to failure. It is not unusual in laboratory experiments to observe 10 to 15 in. deflection in a 40 ft-long prestressed concrete member before it fails. This deflection easily exceeds minimum ductility requirements.

*Figure 8.1.3-3
Typical Load-Deflection Behavior of Conventional Reinforced and Prestressed Concrete Beams*



Another major advantage of prestressing is the improvement in the member's ability to resist shear forces. As a result of the concrete being precompressed, prestressed concrete members have a higher shear capacity, V_c, than conventionally reinforced concrete. This is why thin-webbed I-beam and box-beam bridges have been used very successfully without shear problems. In addition, harped strand, when used, provides a vertical force component that tends to balance part of the gravity load shear force.

8.1.4 Concrete to Steel Bond

Because of the high strength of prestressing strand and the absence of deformations like those found on the surface of reinforcing bars, anchorage of strand in concrete must be carefully assessed. For example, while a Grade 60 #4 bar has a typical development length of 12 to 15 in, the development length of a 1/2-in. diameter strand is about 72 to 100 in. Development length can be a limiting design factor in short members such as precast stay-in-place deck panels. It may also be significant for piles that are subjected to moment near the top end. However, the design and performance of most precast bridge beams are not significantly affected by strand development length.

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DESIGN THEORY AND PROCEDURE

A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.10 LATERAL STABILITY OF SLENDER MEMBERS

Prestressed concrete members are generally stiff enough to prevent lateral buckling. However, during handling and transportation, support conditions may result in lateral displacements of the beam, thus producing lateral bending about the weak axis.

8.10.1 Introduction

There are two important cases: that of a beam hanging from lifting devices and that of a beam supported on flexible supports. For hanging beams, the tendency to roll is governed primarily by the properties of the beam. For supported beams, the tendency to roll is significantly influenced by the conditions of the supports and the roadway geometry (cross-slope). Detailed explanations of these two cases are given in Mast (1989, 1993).

8.10.1.1 Hanging Beams

The equilibrium conditions for a hanging beam are shown in **Figures 8.10.1.1-1** and **8.10.1.1-2a-2b.** When a beam hangs from lifting points, it may roll about an axis through the lifting points. The safety and stability of long beams subject to roll are dependent upon:

e_i = the initial lateral eccentricity of the center of gravity with respect to the roll axis

y_r = the height of the roll axis above the center of gravity of the beam

$\bar{z}_o$ = the theoretical lateral deflection of the center of gravity of the beam, computed with the full weight applied as a lateral load, measured to the center of gravity of the deflected arc of the beam

θ_{max} = tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture

Figure 8.10.1.1-1 Perspective of a Beam Free to Roll and Deflect Laterally

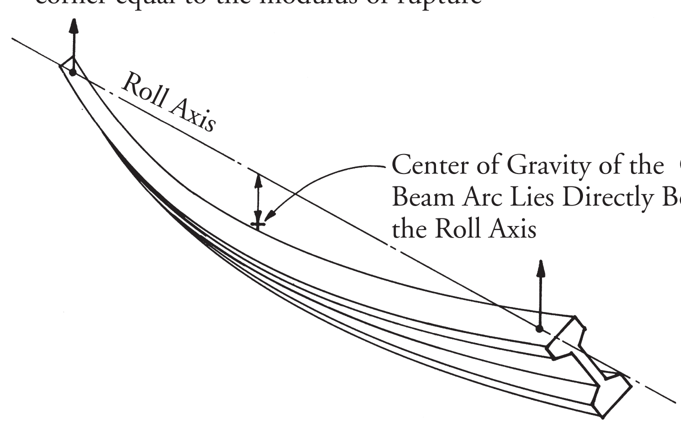
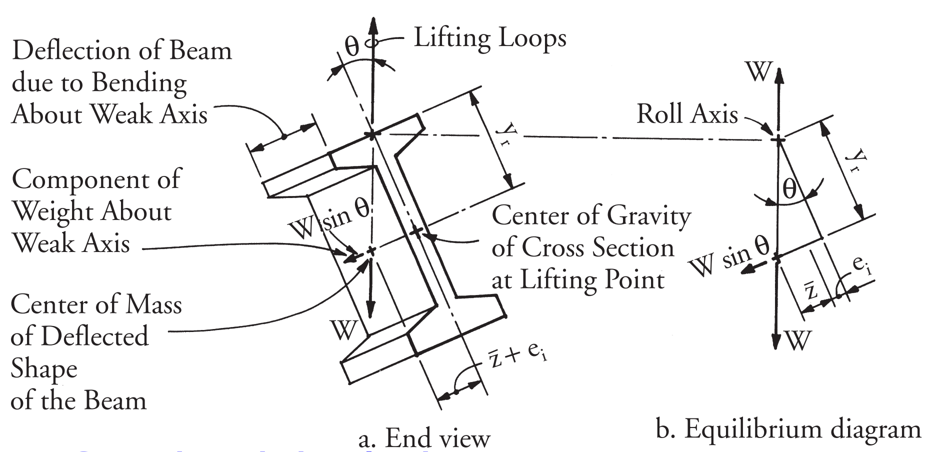


Figure 8.10.1.1-2a-2b Equilibrium of Beam in Tilted Position



For a beam with overall length, ℓ, and equal overhangs of length, a, at each end:

$$\bar{z}_o = \frac{w}{12EI_g \ell}\left[0.1(\ell_1)^5 - a^2(\ell_1)^3 + 3a^4(\ell_1) + 1.2(a^5)\right] \qquad \text{(Eq. 8.10.1.1-1)}$$

where

ℓ_1 = $\ell - 2a$

I_g = moment of inertia of beam about weak axis

For a beam with no overhangs, ($a = 0$, $\ell_1 = \ell$), and:

$$\bar{z}_o = \frac{w(\ell)^4}{120EI_g} \qquad \text{(Eq. 8.10.1.1-2)}$$

The factor of safety against cracking, FS_c, is given by:

$$FS_c = \frac{1}{\dfrac{\bar{z}_o}{y_r} + \dfrac{\theta_i}{\theta_{max}}} \qquad \text{(Eq. 8.10.1.1-3)}$$

where θ_i = the initial roll angle of a rigid beam = $\dfrac{e_i}{y_r}$

It is recommended that e_i be based, as a minimum, on 1/4 in. plus one-half the PCI tolerance for sweep. The PCI sweep tolerance is 1/8 in. per 10 ft of member length.

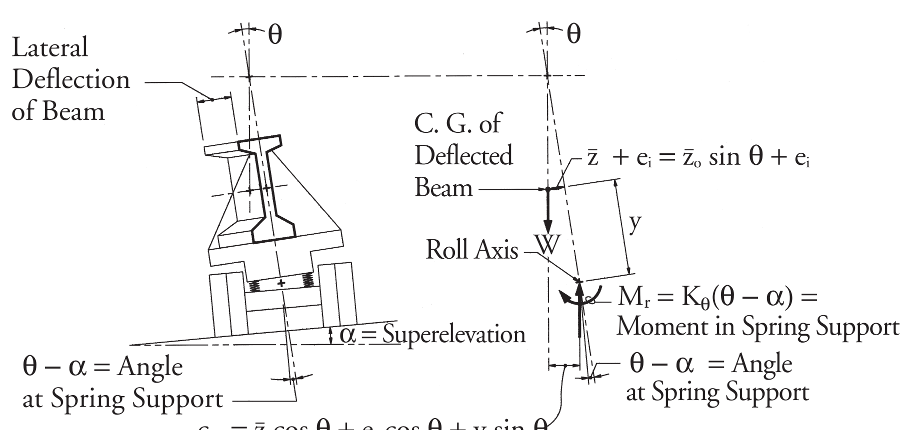
When cracking occurs, the lateral stiffness decreases and $\bar{z}_o$ increases. Thus, failure may occur shortly after cracking as the tilt angle increases rapidly due to the loss of stiffness. Consequently, the factor of safety against failure, FS_f, is conservatively taken equal to FS_c. See Section 8.10.2 for suggested factors of safety.

8.10.1.2 Beams Supported from Beneath

When a beam is supported on flexible supports such as bearing pads or truck and trailer, there is a tendency for the beam to roll about the roll center below the beam (**Figure 8.10.1.2-1**). Because the roll axis is beneath the center of gravity of the beam, the support must be capable of providing resistance to rotation. This resistance is expressed as an elastic rotational spring constant, K_θ.

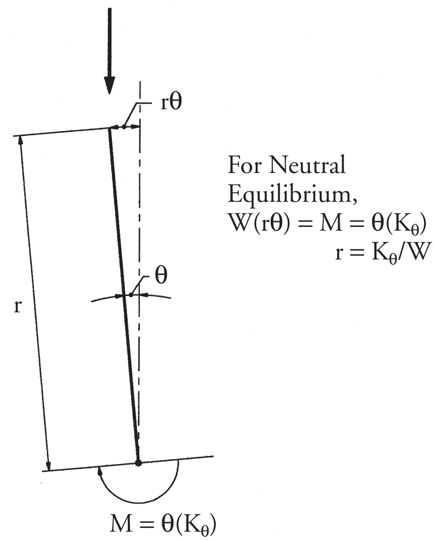
The rotational spring constant of an elastic support is found by applying a moment and measuring the rotation. The quantity, K_θ, is equal to the moment divided by the rotation angle with units of moment per radian.

Figure 8.10.1.2-1 Equilibrium of Beam on Elastic Support



It is convenient to define a quantity, $r = \frac{K_\theta}{W}$, where W is the weight of the beam, as shown in **Figure 8.10.1.2-2**. The quantity, r, has a physical interpretation: it is the height at which the beam weight would be placed in neutral equilibrium with the spring for a given small angle.

Figure 8.10.1.2-2
Definition of radius of stability, r



The equilibrium tilt angle, θ, of the major axis of the beam is given by:

$$\theta = \frac{\alpha r + e_i}{r - y - \bar{z}_o} \qquad \text{(Eq. 8.10.1.2-1)}$$

where

α = superelevation angle or tilt angle of supports in radians

y = height of center of gravity of beam above roll axis (beam supported from beneath)

$= h_{cg} - h_r$

where

h_{cg} = height of center of gravity of beam above road

h_r = height of roll center above road

When r is very large (i.e., the support is very stiff), θ approaches α.

The factor of safety against cracking, FS_c, is:

$$FS_c = \frac{r(\theta_{max} - \alpha)}{\bar{z}_o\theta_{max} + e_i + y\theta_{max}} \qquad \text{(Eq. 8.10.1.2-2)}$$

where θ_{max} = the tilt angle at which cracking begins, based on tension in the top corner equal to the modulus of rupture

For shipping, sweep may be larger (due to creep) and tolerances on location of the support may be larger. Therefore, it is recommended that, e_i, for shipping, be based on 1 in. plus the PCI tolerance for sweep.

Prestressed concrete I-beams possess significant post-cracking strength. After cracking, the beams resist lateral bending by a lateral shift in the centroid of the internal compressive force.

A simplified relationship for the strength and effective stiffness of long prestressed concrete I-beams of ordinary proportions, such as the PCI BT-72, is given by Mast (1993).

- For tilt angles that produce top flange tensile stresses less than the modulus of rupture, $7.5\sqrt{f'_c}$, use the gross moment of inertia, I_g about the weak axis
- For tilt angles that produce top flange tensile stresses in excess of $7.5\sqrt{f'_c}$, , use an effective stiffness:

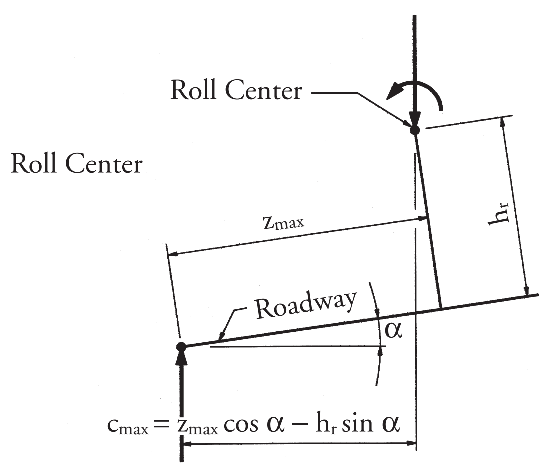
$$I_{eff} = \frac{I_g}{(1+2.5\theta)} \qquad \text{(Eq. 8.10.1.2-3)}$$

- Assume the maximum θ at failure, θ'_{max} , to be 0.4 radians (or 23°)

The maximum tilt angle at failure, θ'_{max} , may be limited by rollover of the transport rig, not by the strength of the beam.

The resisting moment arm is limited by the geometry of the hauling rig. Assuming a height of roll center, h_r (normally about 24 in.), and a transverse distance from centerline of the beam to the center of dual tires, z_{max} (normally about 36 in.), the maximum resisting moment arm may be found **(see Figure 8.10.1.2-3)**:

Figure 8.10.1.2-3 Maximum Resisting Moment Arm for a Beam on Truck and Trailer



Using the usual small angle approximations:

$$\theta'_{max} = \frac{z_{max} - h_r\alpha}{r} + \alpha \qquad \text{(Eq. 8.10.1.2-4)}$$

To find the factor of safety, FS_f, against rollover (overturning) failure, Eq. 8.10.1.2-2 may be modified by substituting the cracked section $\bar{z}'_o$ for $\bar{z}_o$ and θ'_{max} for θ_{max}.

$$FS_f = \frac{r(\theta'_{max} - \alpha)}{\bar{z}'_o\theta'_{max} + e_i + y\theta'_{max}} \qquad \text{(Eq. 8.10.1.2-5)}$$

For the angle θ'_{max}, the quantity, $\bar{z}'_o$, is computed using I_{eff} from Eq. 8.10.1.2-3 for that angle. The calculation is:

$$\bar{z}'_o = \bar{z}_o(1 + 2.5\,\theta'_{max}) \qquad \text{(Eq. 8.10.1.2-6)}$$

8.10.2 Suggested Factors of Safety

The necessary factor of safety cannot be determined from scientific laws; it must be determined from experience. It is suggested to use a factor of safety of 1.0 against cracking, FS_c, and 1.5 against failure, FS_f. This applies to both hanging and supported beams.

8.10.2.1 Conditions Affecting FS_c

For supported beams, the major unknowns are the roll stiffness of the supporting vehicles and the transverse slope of the roadway. It should be noted that transverse slopes occur for reasons other than superelevation. On soft ground (on a shoulder or at the construction site) one side of the hauling rig may sink into the ground, creating a large transverse slope.

These unknowns primarily affect the factor of safety against cracking. It is believed that many beams have been successfully shipped with a theoretical factor of safety against cracking of less than unity. But until the factors of safety used in the past can be better documented, a minimum factor of safety against cracking of 1.0 is recommended. The factor of safety against failure is essentially the factor of safety against overturning of the hauling rig. A factor of safety against overturning of 1.5 is believed to be adequate.

8.10.2.2 Effects of Creep and Impact

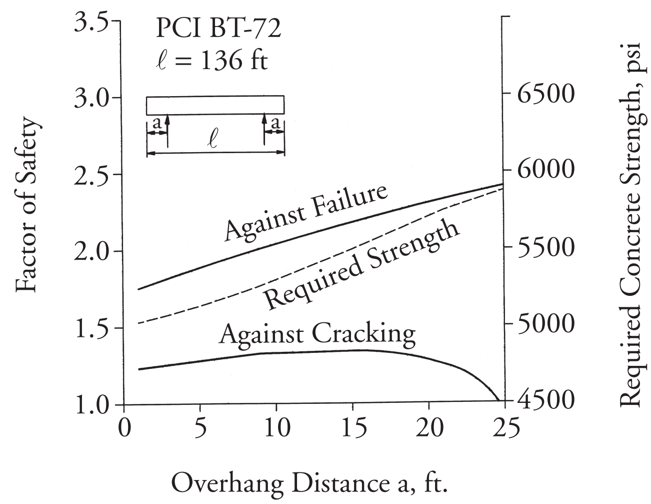
The recommended minimum factors of safety apply to calculations that do not account for creep and impact. Impact is normally of significance during hauling, but stability is primarily a problem when traveling along sections of high superelevation at low speeds. It is recommended that the effects of impact and superelevation be accounted for separately. This was also recommended by Imper and Laszlo (1987).

8.10.2.3 Effects of Overhangs

Figure 8.10.2.3-1 shows the factors of safety for a supported beam on a 6% slope and with supports having a K_θ of 40,500 in.-kip per radian. The factors of safety for supported beams are much less sensitive to overhang distance. For supported beams, the stability of the beam is more a function of the stiffness of the support than the stiffness of the beam. However, the factor of safety against cracking is determined by the top fiber stresses in the beam. Past practice has been to support the beam on the truck at the lifting points. **Figure 8.10.2.3-1** would indicate that some deviation in location of support points on the truck and trailer is permissible.

Unequal overhangs are sometimes necessary during shipping. It is sufficiently accurate to use the average overhang in stability calculations, but the stress at the support should be checked using the actual overhang.

Figure 8.10.2.3-1
Effect of Overhangs for Beam on Truck and Trailer

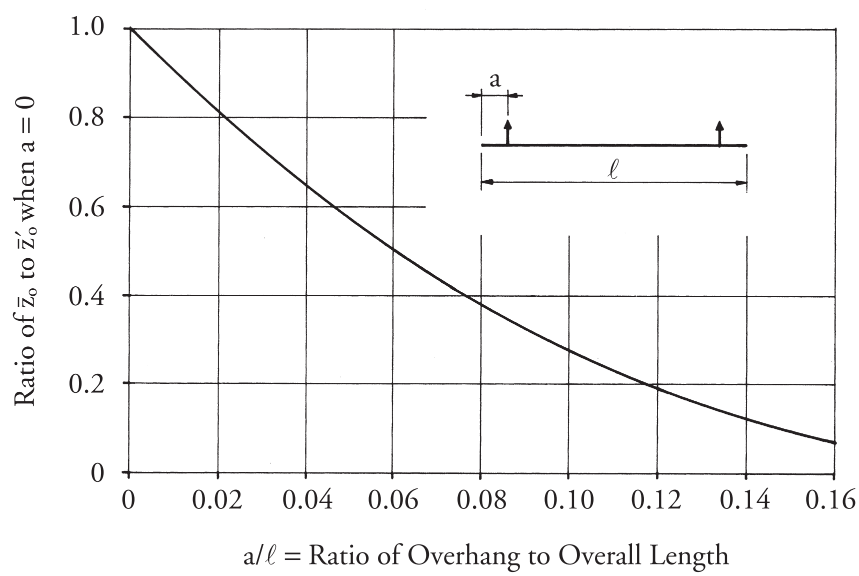


8.10.2.4 Increasing the Factor of Safety

For safe handling of long members, resistance can be improved by several methods. These are listed below in order of effectiveness and relative ease of accomplishment, with the easiest and most effective first:

1. Move the lifting points inward **(see Figure 8.10.2.4-1)**. Decreasing the distance between lifting points by just a small amount can significantly increase the safety factor. Stresses must be checked; temporary post-tensioning can be introduced to control stresses
2. Increase the distance between the center of mass and the lifting point, y_r, by use of a rigid lifting yoke
3. Provide temporary lateral bracing, in the form of a stiffening truss, composed of structural steel shapes
4. Change the shape of the cross-section of the member
5. Increase the stiffness of the member by increasing the concrete modulus of elasticity, E_c, and tensile strength

Figure 8.10.2.4-1
Reduction of $\bar{z}_o$ with Overhangs



8.10.3 Measuring Roll Stiffness of Vehicles

The roll stiffness (rotational spring stiffness) of transport vehicles is a very important parameter in evaluating the safety of slender beams during transportation. Information on the roll stiffness of transport vehicles is normally not available, however, roll stiffness may be determined by placing a weight on the vehicle at various eccentricities to the vehicle longitudinal centerline. The weight should be of the same order of magnitude as the beam, a convenient weight is the beam itself. One end of the beam may be secured, and the other end placed on the vehicle at eccentricities of, say, 10 and 20 in. either side of the centerline.

Rotations may be determined by measuring the vertical movement at either end of the bolster or cross-member used for chaining the beam. The roll stiffness is the average of the values obtained by dividing the eccentric moment by the rotation in radians. Because the bolster tilts under eccentric loads, it is necessary to use a narrow bearing strip of hard material between the beam and the cross-member, in order to know the eccentricity of the load on the trailer.

A very limited number of measurements indicates that the rotational spring constant, K_θ, may be expected to be in the range of 3,000 to 6,000 in.-kip per radian per dual-tire axle. The higher values apply to rigs without leaf springs, in which the spring is primarily in the tires. For instance, a steel trailer with four dual axles plus a single axle might be expected to have a roll stiffness of 4.5 times (3,000 to 6,000) = 13,500 to 27,000 in.-kip per radian. The total, K_θ, is the sum of that for the tractor and the trailer.

The above figures are based on very limited data and must be used with caution until more data are available. Also, these values apply to axles for which the load is balanced mechanically. Axles with air suspension may contribute little, if any, to roll stiffness. For critical shipments, measuring the roll stiffness of the vehicles that will be used is strongly recommended. Rigs with independent axle systems may give inconsistent results.

8.10.4 Bearing Pads

Elastomeric bearing pads also provide a resilient support for prestressed concrete beams. The rotational spring constant, K_θ, is determined by the dimensions and properties of the pad. When the load is outside the kern of the bearing pad, the rotational spring constant becomes highly nonlinear. Test results for this situation are lacking since the objective is generally to keep the load within the kern. In addition, there may be stability problems with thick plain (unreinforced) pads; laminated pads provide more stability.

8.10.5 Wind Loads

Wind forces on beams produce applied moments that must be added to other moments. This causes an additional initial eccentricity due to the deflection caused by the wind load. Additionally, the wind load itself causes an overturning moment about the bottom of the bearing pads which, divided by the beam weight, produces a moment arm. The total moment arm at zero tilt angle, θ, is the sum of assumed initial eccentricity, e_i, plus these two added quantities. This emphasizes the importance of bracing the ends of bridge beams against rollover as soon as they are erected.

8.10.6 Temporary King-Post Bracing

Long prestressed concrete I-beams are often braced during transportation using a king-post truss system. This system uses external prestressing strands which are partially tensioned against bearing plates at each end of the beam. One or two steel king posts are mounted against each side of the beam at opposing points and harp (push) the strands out to a large eccentricity at or near the mid-point of the beam. This provides a truss-like frame around the beam.

Such bracing is of very little benefit, however. The steel area of the prestressing strands is too small to make a significant contribution to the lateral stiffness of the beam. Temporary prestressing of the top flange, as recommended by Imper and Laszlo (1987), is a more effective way to improve the factor of safety against cracking. Horizontal stiffening trusses fabricated with mild steel chords are also effective.

8.10.7 Lateral Stability Examples

These calculations are based on the example given in Imper and Laszlo (1987). Refer to Imper and Laszlo (1987) and Mast (1993) for additional details.

The following information is provided:

AASHTO-PCI BT-72 bridge beam

Depth, $h = 72$ in.

Top flange width, $b = 42$ in.

Bottom flange width, $b_b = 26$ in.

Unit weight of concrete, $w_c = 155$ pcf

Initial concrete strength, $f'_{ci} = 4{,}500$ psi

Beam cross-sectional area, $A_c = 767$ in.^{2}

Strong axis, $I = 545{,}894$ in.^{4}

$y_b = 36.6$ in.

Weak axis, $I_g = 37{,}634$ in.^{4}

Overall length, $\ell = 136$ ft

Pickup (lift) points, $a = 9$ ft from each end

Harp points, 0.4ℓ

Initial prestress force, $P_i = 1{,}232$ kips (after initial losses)

Location of P_i, $y_{bs} = 5$ in. above soffit at harp points

8.10.7.1 Hanging Beam Example

Find the factors of safety against cracking and against failure when the beam hangs from lifting loops.

1. Check stresses at harp points and required concrete strength:

$$w = \frac{A_c w_c}{144 \times 1{,}000} = 0.8256 \text{ klf}$$

$W = wL = 0.8256(136) = 112.28$ kips

M_g at harp point at $0.4\ell = 15{,}926$ in.-kip

The corresponding concrete stresses:

$f_t = 0.114$ ksi and $f_b = 3.149$ ksi

f'_{ci} required $= f_b/0.6 = 5{,}248$ psi $> 4{,}500$ psi. Use $f'_{ci} = 5.25$ ksi

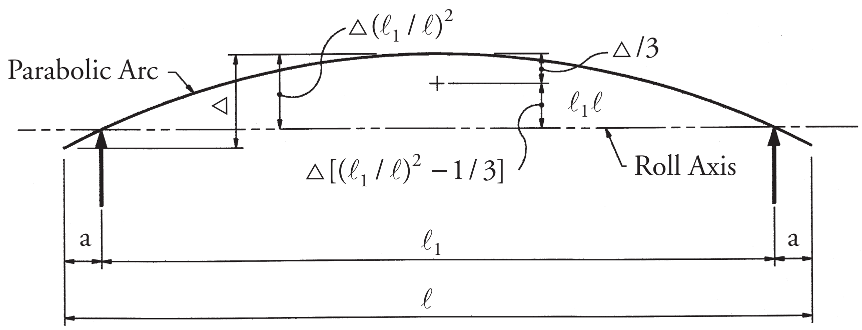
2. Find modulus of elasticity:

   $E_c = 33\ w_c^{1.5}\ \sqrt{f'_{ci}}\ /1000 = 4{,}613$ ksi

3. Compute initial eccentricity, e_i:

   For beams hanging from lifting loops, use a sweep dimension of one-half the PCI sweep tolerance and a lifting loop placement tolerance of 1/4 in. Or, calculate e_i based on observations of the tilt angles of actual beams. To evaluate e_i due to sweep, the distance between the roll axis and the center of gravity of the arc of the curved beam must be found (see **Figure 8.10.7.1-1**). The curved shape is assumed to be a parabola, and the formulas are derived from the properties of a parabola.

Figure 8.10.7.1-1
Offset of Centroid of a Curved Arc



One-half of the PCI sweep tolerance is 1/16 in. per 10 ft, or 0.85 in. for the 136-ft length. The offset factor (see **Figure 8.10.7.1-1**) is $(\ell_1/\ell)^2 - 0.333 = (118/136)^2 - 0.333 = 0.419$. Thus, $e_i = 0.85(0.419) + 0.25 = 0.607$ in.

4. Estimate camber and correct the value of y_r for camber.

   Camber may be estimated from the midspan radius of curvature, R:

$$R = \frac{EI}{M} \qquad \text{(Eq. 8.10.7.1-1)}$$

where

$M = P_i e - M_{gmsp}$

where

M_{gmsp} = self-weight bending moment at midspan

$$= \frac{W}{2}\left[\frac{\ell}{4} - a\right] = \frac{112.28}{2}\left[\frac{136}{4} - 9\right](12) = 16{,}842 \text{ in.-kip}$$

$M = (1{,}232)(31.6) - 16{,}842 = 22{,}089$ in.-kip

$$R = \frac{4{,}613(545{,}894)}{22{,}089} = 114{,}000 \text{ in.}$$

The camber, Δ, measured with respect to the beam ends, is computed as follows:

$$\Delta = \frac{\ell^2}{8R} = \frac{[136(12)]^2}{8(114{,}000)} = 2.92 \text{ in.} \qquad \text{(Eq. 8.10.7.1-2)}$$

The height of the center of gravity of the cambered arc above the roll axis is computed:

$y_r = y_t - \Delta(0.419) = (72 - 36.6) - 2.92(0.419) = 34.18$ in.

Camber has only a small effect on y_r. One may simply subtract an estimate (say between one and two in.) from y_r.

5. Compute $\bar{z}_o$ from Eq. (8.10.1.1-1):

$$\bar{z}_o = \frac{w}{12EI_g \ell}\left(0.1\,\ell_1^5 - a^2\ell_1^3 + 3a^4\ell_1 + 1.2a^5\right) = 10.86 \text{ in.}$$

6. Compute $\theta_i = \dfrac{e_i}{y_r} = \dfrac{0.607}{34.18} = 0.01775$

7. Compute the tilt angle, θ_{max}, at cracking:

$f_r = 7.5\sqrt{f'_{ci}} = 0.543$ ksi (tension), $f_t = 0.114$ ksi (compression) from Step 1.

$$M_{lat} = \frac{(f_r + f_t)(I_g)}{\frac{b}{2}} = 1{,}179 \text{ in.-kip}$$

$$\theta_{max} = \frac{M_{lat}}{M_g} = \frac{1{,}179}{15{,}926} = 0.0740$$

8. Compute factor of safety against cracking, FS_c:

$$FS_c = \frac{1}{\frac{\bar{z}_o}{y_r} + \frac{\theta_i}{\theta_{max}}} = \frac{1}{\frac{10.86}{34.18} + \frac{0.01775}{0.0740}} = 1.79$$

9. Find factor of safety against failure, FS_f:

Assume $FS_f = FS_c = 1.79$. Minimum $FS_c = 1.5$ O.K.

8.10.7.2 Supported Beam Example

Find the factors of safety against cracking and rollover during transportation for the same beam described in Section 8.10.7.

The following information is provided:

f'_c = concrete strength at 28-days = 5,500 psi

Add two strands in top flange, per Imper and Laszlo (1987)

P = prestress force = 1,251.5 kips

y_s = the distance between the center of gravity of the strand to soffit = 7.91 in.

α = superelevation angle = 0.06 radians (different from the 0.08 used in Imper and Laszlo, 1987)

Tractor and steer trailer each with four dual axles and one single axle, stiff suspension

h_r = height of roll center above road = 24 in.

h_{cg} = height of center of gravity of beam above road = 108 in.

1. Estimate K_θ and find r:

   Assume:

   K_θ = 4,500 in.-kip per radian per dual axle

   K_θ = 4.5(4,500) = 20,250 each for tractor and for trailer

   Use two times this constant for total, K_θ, for hauling rig. K_θ = 40,500 in.-kip per radian

   $$r = \frac{K_\theta}{W} = \frac{40{,}500}{112.28} = 360.7 \text{ in.}$$

2. Find tilt angle, θ, from Eq. (8.10.1.2-1):

   $$\theta = \frac{\alpha r + e_i}{r - y - \bar{z}_o}$$

   $y = h_{cg} - h_r = 108 - 24 = 84$ in.

   Increase y by 2%, to allow for camber. Then, y = 85.68 in.

   For shipping, assume PCI sweep tolerances plus 1 in. off-center of truck/trailer. Use offset factor of 0.419 as computed in Step 3. e_i = 1.70(0.419) + 1 = 1.71 in.

   Adjust $\bar{z}_o$ from Step 5, by the square root of ratio of concrete strengths to account for the change in modulus of elasticity.

   $$\bar{z}_o = 10.86\sqrt{\frac{5{,}248}{5{,}500}} = 10.61 \text{ in.}$$

   $$\theta = \frac{0.06(360.7) + 1.71}{360.7 - 85.68 - 10.61} = 0.0883$$

3. Check stresses at harp points:

   $$f_t = \frac{P}{A} - \frac{Pe}{S_t} + \frac{M_g}{S_t} = 0.336 \text{ ksi}$$

   f_b = 2.971 ksi

4. Add lateral bending stress to f_b, and find required concrete strength:

   $M_{lat} = \theta(M_g)$ = 0.0883 (15,926) = 1,406 in.-kip

   f_b = 2.971 + 1,406(13)/37,634 = 3.457 ksi

   $$f'_c = \frac{f_b}{0.6} = 5{,}759 \text{ psi and } E_c = 4{,}833 \text{ ksi}$$

   Adjust $\bar{z}_o$ from Step 5 by the ratio of E_cs:

   $$\bar{z}_o = 10.86\sqrt{\frac{4{,}613}{4{,}833}} = 10.37 \text{ in}$$

5. Estimate camber and correct y for camber:

   The effect of camber on stability during hauling is small. Estimate that camber increases y by 2 in. to 86 in.

6. Find the tilt angle, θ_{max} ,at cracking:

$$f_r = 7.5\sqrt{5{,}759} = 0.569 \text{ ksi (tension)}$$

$$f_t = 0.336 \text{ ksi (compression) from Step 3}$$

$$M_{lat} = \frac{(f_r + f_t)(I_g)}{\frac{b}{2}} = 1{,}622 \text{ in.-kip}$$

$$\theta_{max} = \frac{M_{lat}}{M_g} = \frac{1{,}622}{15{,}926} = 0.1018$$

7. Compute factor of safety against cracking, FS_c, from Eq. (8.10.1.2-2):

$$FS_c = \frac{c_r}{c_a} = \frac{r(\theta_{max} - \alpha)}{\bar{z}_o\theta_{max} + e_i + y\theta_{max}} = \frac{360.7(0.1018 - 0.06)}{10.37(0.1018) + 1.71 + 86(0.1018)}$$

$$= 1.31 > 1.0 \quad \text{OK}$$

8. Find tilt angle, θ'_{max}, at maximum resisting moment arm from Eq. (8.10.1.2-4):

$$\theta'_{max} = \frac{z_{max} - h_r\alpha}{r} + \alpha = \frac{36 - 24(0.06)}{360.7} + 0.06 = 0.1558$$

9. Compute $\bar{z}'_o$ at θ'_{max} from Eq. (8.10.1.2-6):

$\bar{z}'_o = \bar{z}_o(1 + 2.5\theta'_{max}) = 10.37\,[1 + 2.5(0.1558)] = 14.41$ in.

10. Compute factor of safety against rollover, FS_f, from Eq. (8.10.1.2-5):

$$FS_f = \frac{c_r}{c_a} = \frac{r(\theta'_{max} - \alpha)}{\bar{z}'_o\,\theta'_{max} + e_i + y\theta'_{max}} = \frac{360.7(0.1558 - 0.06)}{14.41(0.1558) + 1.71 + 86(0.1558)}$$

$$= 1.99 > 1.5 \quad \text{OK}$$

TABLE OF CONTENTS
DESIGN THEORY AND PROCEDURE

A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = k_{la} k_h k_s	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = k_{cp} k_h k_s	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.11 BENDING MOMENTS AND SHEAR FORCES DUE TO VEHICULAR LIVE LOADS

In designing longitudinal members of bridges, the maximum bending moment and shear force at each section along the span, are computed for live loads. The load position must be determined to give the maximum values of shears and moments. The *Standard Specifications* use the HS20 design truck while the *LRFD Specifications* use the HL-93 loading which is a combination of the HS20 design truck and a lane loading of 0.640 kip/ft. Design for the fatigue limit state in the *LRFD Specifications*, requires that a special design truck be used. This section gives formulas which may be combined to get the maximum bending moments and shear forces due to the above loading cases.

Readers are referred to the *Standard Specifications* for details about the effects of the equivalent lane loading which must also be considered in design. It can be shown that this equivalent lane loading may govern the design of spans longer than 144.5 ft for bending moment and 120 ft for shear force.

8.11.1 HS20 Truck Loading

The following formulas may be used to calculate the maximum bending moment and maximum shear force per lane at any point on a span for the HS20 design truck. Certain limitations apply, as noted in the tables. The computed values should be multiplied by a factor of 1/2 to obtain forces per line of wheels. The formulas are valid only for simple spans and impact is not included (see AASHTO *Manual for Condition Evaluation of Bridges,* AASHTO, 1994).

Table 8.11.1-1 Maximum Bending Moment per Lane for HS20 Truck Load

Load type	x/L	Formula for maximum bending moment, ft-kips	Minimum	
			x,* ft	L, ft
HS20 Truck	0 - 0.333	$\frac{72(x)[(L-x)-9.33]}{L}$	0	28
	0.333 - 0.500	$\frac{72(x)[(L-x)-4.67]}{L} - 112$	14	28

* x is the distance from left support to the section being considered, ft

Table 8.11.1-2 Maximum Shear Force per Lane for HS20 Truck Load

Load type	x/L	Formula for maximum shear force, kips	Minimum		Maximum
			x,* ft	L, ft	L, ft
HS20 Truck	0 - 0.500	$\frac{72[(L-x)-4.67]}{L} - 8$	14	28	42
	0 - 0.500	$\frac{72[(L-x)-9.33]}{L}$	0	42	-

* x is the distance from left support to the section being considered, ft

8.11.2 Lane Loading, 0.640 kip/ft

The following formulas may be used to calculate the maximum bending moment and the maximum shear force per lane at any point on a span for a lane load of 0.640 kip/ft. The formulas are valid only for simple spans and impact is not included.

$$\text{Maximum bending moment} = \frac{0.64(x)(L-x)}{2}, \text{ ft-kips}$$

$$\text{Maximum shear force} = \frac{0.64}{2L}(L - x)^2, \text{ kips}$$

where

x = the distance from left support to the section being considered, ft

L = span, ft

8.11.3 Fatigue Truck Loading

When designing using the *LRFD Specifications*, consideration of the fatigue limit state may be required (see LRFD Article 5.5.3.1). A special fatigue truck load is defined in LRFD Article 3.6.1.4.1. This loading consists of a single design truck which has the same axle weights used in all other limit states, but with a constant spacing of 30.0 ft between the 32.0-kip axles. The following equations may be used to calculate the maximum bending moment per lane at any point on the span for the fatigue truck loading. These values should be multiplied by a factor of 1/2 to obtain values per line of wheels. These formulas are valid only for simple spans and impact is not included.

Table 8.11.3-1 Maximum Bending Moment per Lane for HL-93 Fatigue Truck Loading

Load type	x/L	Formula for maximum bending moment, ft-kips	Minimum	
			x,* ft	L, ft
Fatigue Truck Loading (LRFD)	0 - 0.241	$\frac{72(x)[(L-x)-18.22]}{L}$	0	44
	0.241 - 0.500	$\frac{72(x)[(L-x)-11.78]}{L} - 112$	14	28

* x is the distance from left support to the section being considered, ft

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = k_{la} k_h k_s	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = k_{cp} k_h k_s	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.12 STRUT-AND-TIE MODELING OF DISTURBED REGIONS

Traditionally, models used in the analysis and design of concrete structures have been based on elastic theory and the basic assumption that plane sections remain plane, regardless of the loading. However, it is well known that disturbances do occur in regions near discontinuities, for example, at concentrated loads and abrupt changes in member dimensions. Such regions are referred to as “disturbed regions.”

Methods used to analyze and design disturbed regions must include procedures that reflect the actual flow of stresses in such regions. In considering stress distribution before cracking, it is customary to apply elastic methods of analysis, especially when predicting where significant cracking will occur. Since significant stress redistribution takes place after concrete cracks, elastic methods cannot adequately predict stresses subsequent to cracking.

8.12.1 Introduction

A rational method for dealing with disturbed regions subsequent to cracking is the use of strut-and-tie models. These models can give an excellent representation of the flow of forces in disturbed regions of cracked concrete systems.

The theoretical basis of the method prescribed in the LRFD Specifications [Article 5.8.3.4.2] for the design of a section subjected to combined shear, axial load and flexure, is the modified compression field theory. This method considers the equilibrium of forces acting on the idealized, variable-angle truss, the compatibility of strains, and the effects of cracking of concrete. Simplifications in the method include the use of average values of stresses and strains over a length greater than the crack spacing.

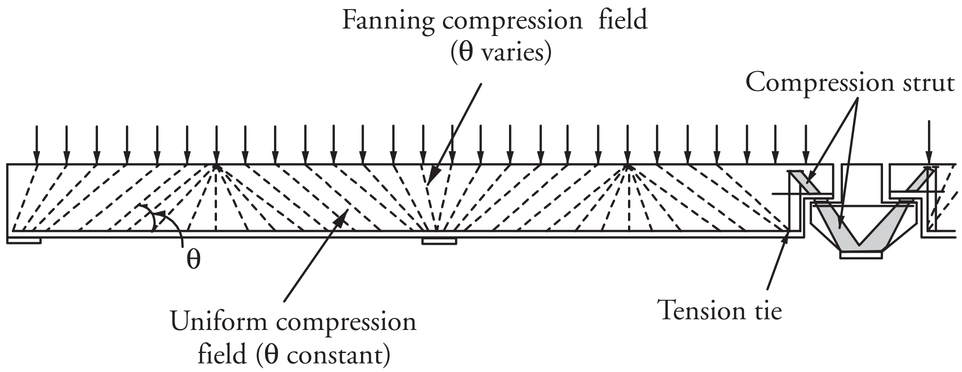
In a typical calculation for shear reinforcement, the sectional dimensions, prestressing and material strengths have been chosen and the shear design involves selection of adequate shear reinforcement and, if necessary, additional longitudinal reinforcement.

Figure 8.12.1-1 shows that there are three types of regions that need to be considered in general shear design of a beam as follows:

1) Regions that can be appropriately treated as a system of struts and ties. This approach is discussed in this section.
2) Disturbed regions of fanning compressive stresses characterized by radiating compressive stresses near supports and regions where the shear changes sign. In such regions the value of θ varies.
3) Regions of uniform compressive stress fields where the value of θ is constant.

The second and third types of regions are discussed in Section 8.4.3 using the general modified compression field theory and the corresponding *LRFD Specifications* procedure.

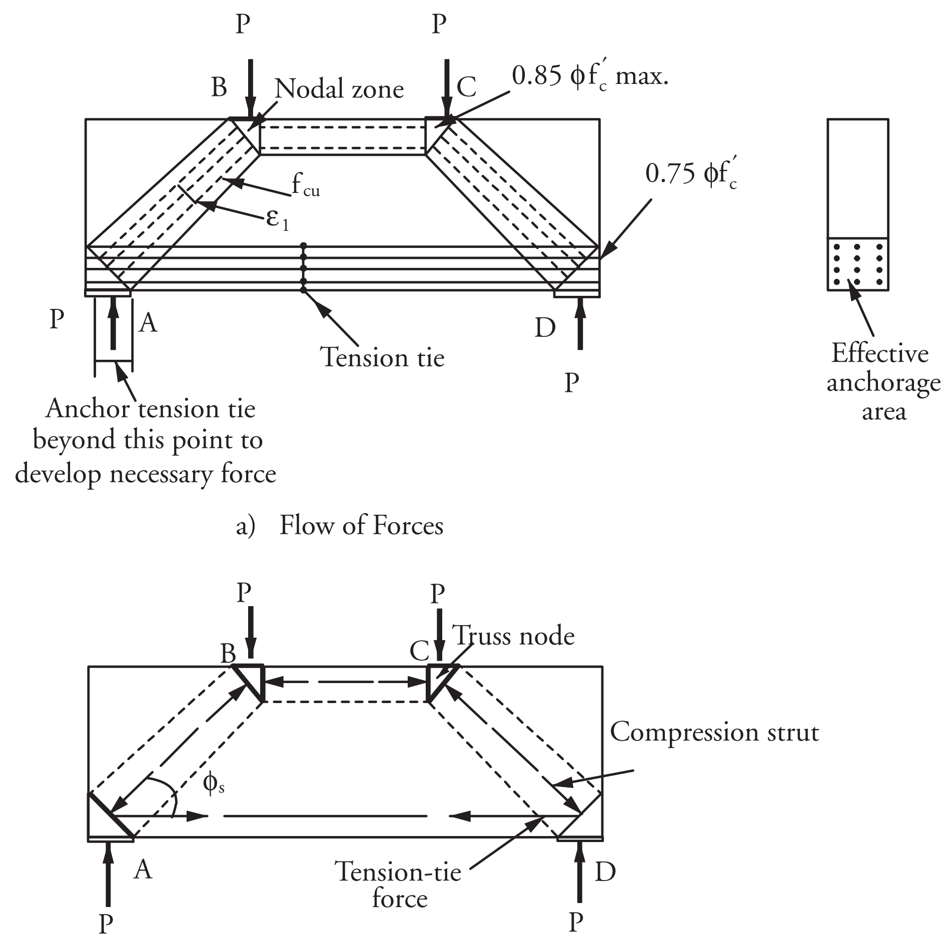
Figure 8.12.1-1
Disturbed Regions and Regions of Uniform Compression Fields



8.12.2 Strut-and-Tie Models

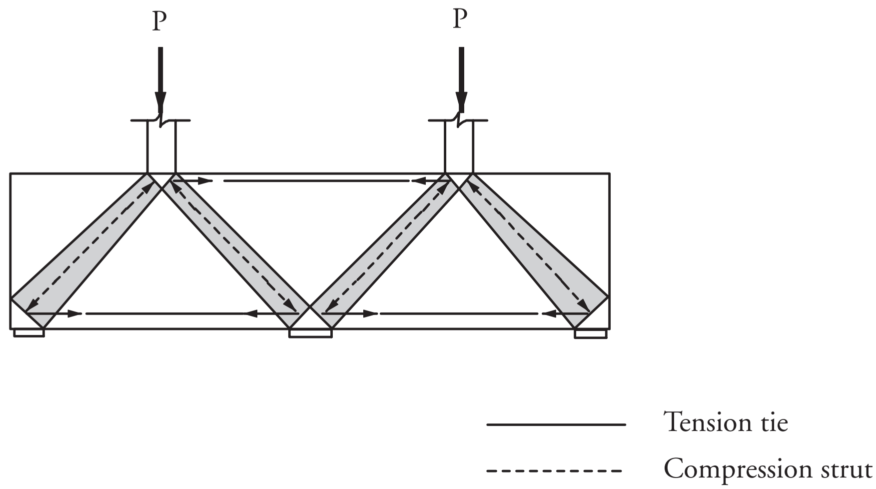
The *LRFD Specifications* encourage the use of strut-and-tie models in design where appropriate. It has been determined through sophisticated analysis and laboratory testing, that cracked reinforced concrete carries load mainly by development of a truss system represented by compressive stresses in the concrete and tensile stresses in the reinforcement. Furthermore, upon the occurrence of significant cracking, the originally curved principal stress trajectories in concrete tend toward straight lines, and it is appropriate to regard the resulting compressive forces as being carried by straight compressive struts. Examples of strut-and-tie modeling of a simply supported and a continuous deep beam are shown in **Figures. 8.12.2-1a-1b** and **8.12.2-2**.

Figure 8.12.2-1a-1b
Strut-and-Tie Model for a Simple Deep Beam



b) Truss Model

Figure 8.12.2-2
Strut-and-Tie Model for a Continuous Deep Beam



Important considerations in strut-and-tie modeling include the geometry of the truss system, the nodal zone and member dimensions, and the strengths of the compression and tension members.

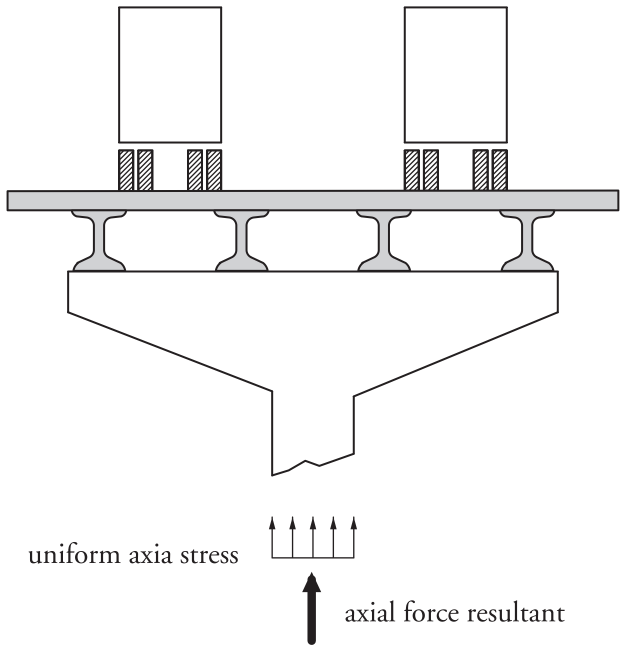
8.12.2.1 Truss Geometry Layout

The significance of using correct geometry in defining a truss should be obvious in the necessity to have a triangularized system of struts and ties. At first glance, the use of a strut-and-tie truss system to resist loads seems like an easy solution that any engineer should be readily able to accomplish. Since the real structure is a continuum, however, there are an infinite variety of trusses that could be designed inside a concrete member. The best or most efficient truss layout will be one that most closely fits the applied load and reaction conditions while resisting forces through the shortest load paths.

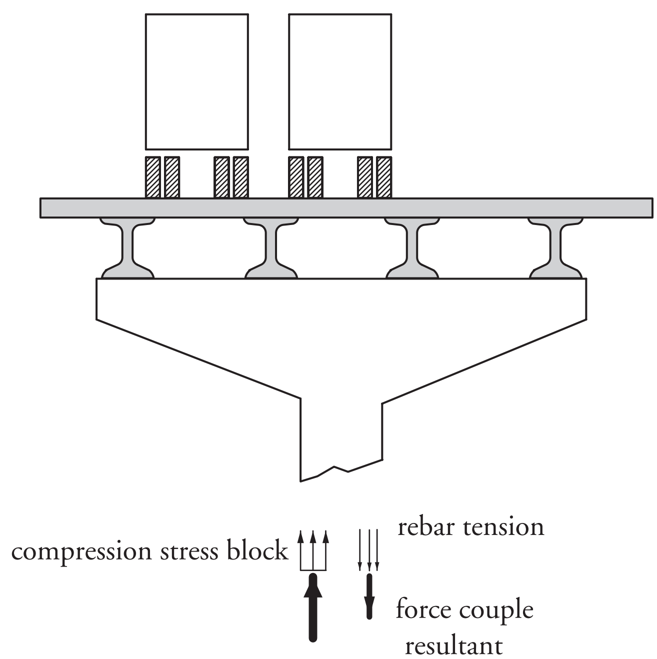
Identification of the existing boundary conditions is the first step in selecting a truss layout for the strut-and-tie system. In the hammerhead pier cap of **Figure 8.12.2.1-1a-1b**, two different sets of boundary conditions are shown depending on the locations of the design lanes and loading on the roadway above. In **Figure 8.12.2.1-1a**, the two 12 ft. design lanes are placed symmetric about the pier centerline and the girder reactions on the pier cap, representing the top boundary condition, are all identical. In **Figure 8.12.2.1-1b**, the two design lanes are shifted to the left side of the roadway and the reactions vary across the top of the pier cap, giving a second top boundary condition.

Regardless of the truss layout that might be selected within the pier cap, the forces in the pier column can be directly calculated: with pure axial compression in the first case and compression plus bending in the second case as shown in **Figure 8.12.2.1-1a-1b**. In the first case, the bottom boundary condition is simply an axial force acting at the middle of the pier. The boundary condition in the second case, however, must be calculated and includes a column compression block and tension component as shown in **Figure 8.12.2.1-1b**. The forces shown in the pier of **Figure 8.12.2.1-1b** are assumed to exist at a distance “d” from the bottom of the pier cap – away from the disturbed region and in the portion of the column assumed to have sectional model behavior.

Figure 8.12.2.1-1a-1b
Pier Cap under Symmetric and Unsymmetric Lane Loading



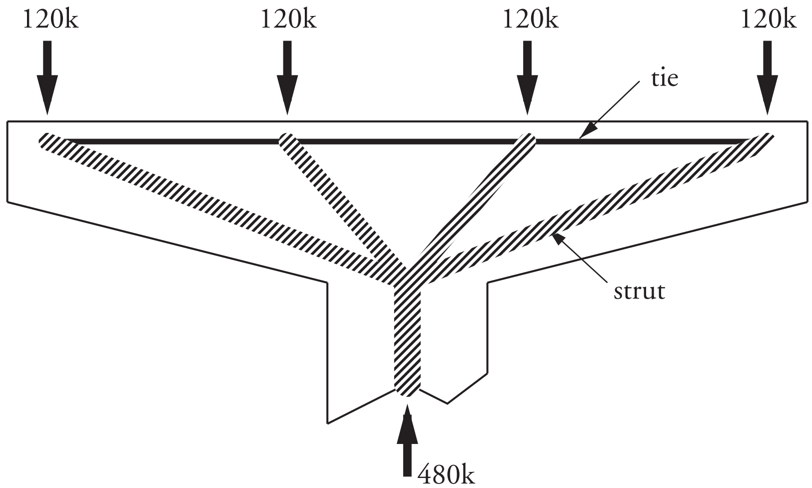
a) Symmetric Loads



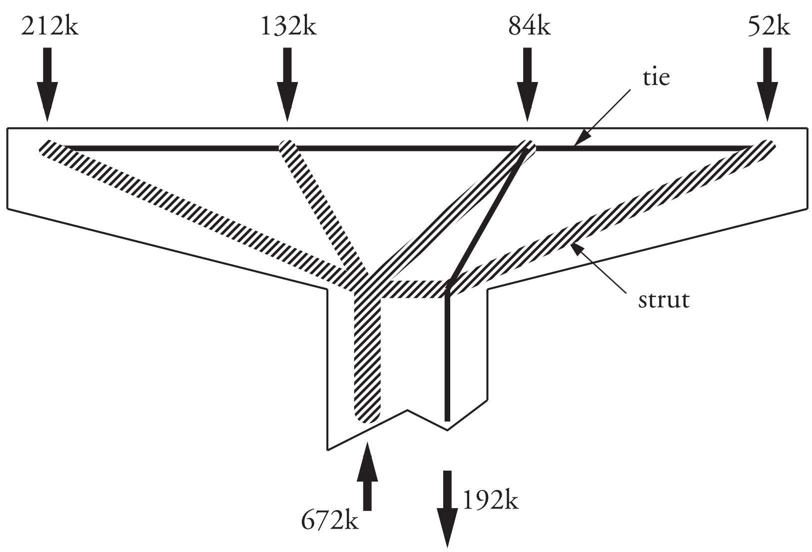
b) Unsymmetric Loads

In the first case of **Figure 8.12.2.1-1a**, the truss layout in the pier cap need only meet the condition of developing a compression thrust at the bottom of the cap. In the second case of **Figure 8.12.2.1-1b**, the truss must develop both the compression and the tension force in the pier column. Clearly two different truss layouts could be designed depending on which set of loads/boundary conditions was being considered as shown in **Figure 8.12.2.1-2a-2b**. The truss in the **Figure 8.12.2.1-2b** would be inverted if the trucks were at the other side of the roadway.

Figure 8.12.2.1-2a-2b Truss Layouts for the Different Load Cases



a) Symmetric Truss



b) Unsymmetric Truss

It is essential in the development of a truss layout for strut-and-tie design that:

(1) all of the possible load combinations be identified.

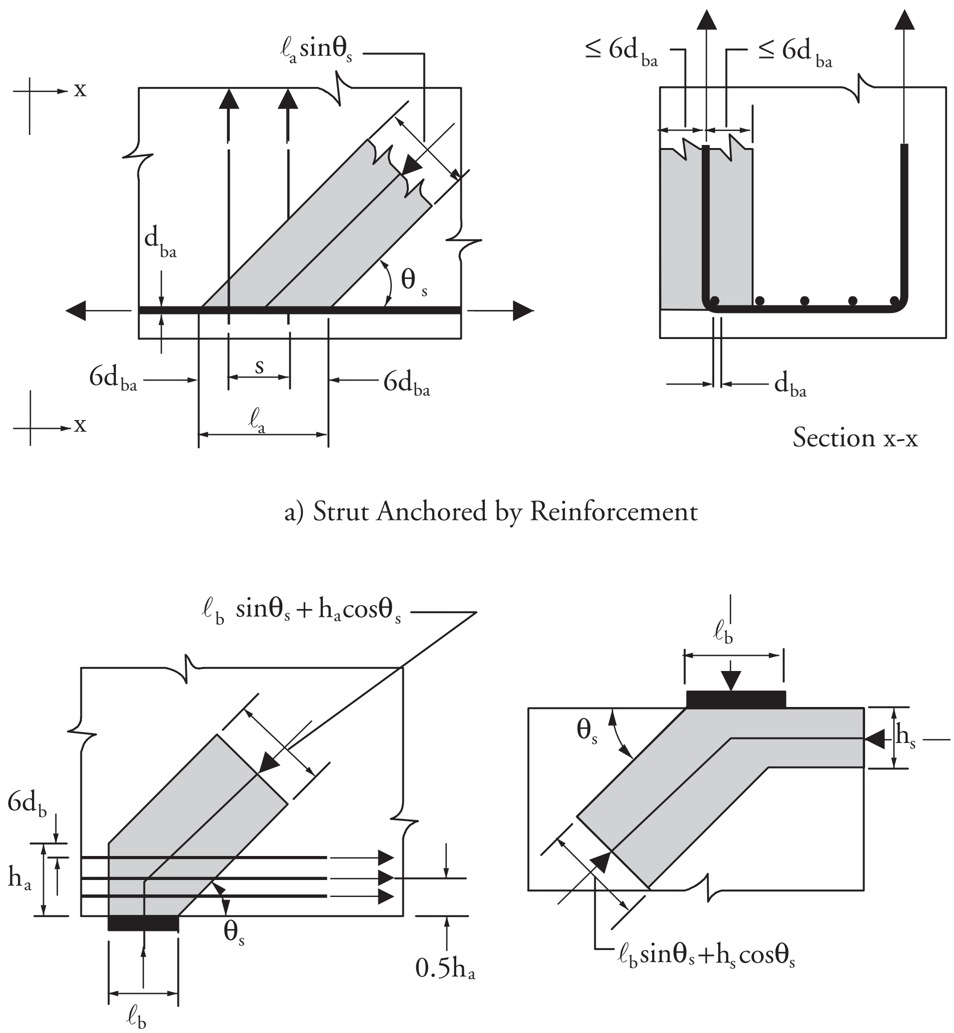
(2) boundary forces, including internal forces from portions of the structure having sectional type behavior, be calculated for each controlling load condition.

(3) appropriate strut-and-tie models be laid out and designed for each set of boundary conditions.

8.12.2.2 Nodal Zone and Member Dimensions

The nodal zones are regions where the struts and ties of the truss join. While the truss diagrams of **Figure 8.12.2.1-2a-2b** idealize the truss members as connecting at points, the actual structure has struts and ties with finite dimensions. The nodal zone sizes are related to both the effective tie member sizes and the mechanism by which exterior loads are transferred into the structure. As shown in **Figure 8.12.2.2-1a-1c**

Figure 8.12.2.2-1a-1c
Effects of Anchorage Conditions on Cross-Sectional Area of Strut



a) Strut Anchored by Reinforcement

b) Strut Anchored by Bearing and Reinforcement

c) Strut Anchored by Bearing and Strut

[*LRFD Specifications* Commentary Figure 5.6.3.3.2-1] the dimensions of the nodal zone and adjoining struts are controlled by the anchorage conditions of reinforcing tie bars or bearing areas of applied loads.

8.12.2.3 Strength of Members

The strength of tension ties depends directly on the type and strength of reinforcing used in the ties. Strengths of the individual truss strut members are normally controlled by the limits on stresses within the nodal zones. The nodal zone compressive stresses are defined by the relation between compressive stress capacity and perpendicular tension strains invoked by compression stress field theory. **Figure 8.12.2-1a** shows the principal tension strain, ε_1, which may exist perpendicular to the compression strut, BP. The strain, ε_1, is dependent on the truss geometry and the tensile strain in adjoining truss members. The adverse effect of this tensile strain in the cracked concrete must be considered in calculating the capacity of a strut. In such struts the limiting compressive stress, f_{cu}, is a function of f'_c and ε_1. The value of ε_1 is, in turn, a function of the tension strain, ε_s, in the cracked concrete in the direction of the tension tie, and the angle between strut and tie.

8.12.3 LRFD Specifications Provisions for Strut-and-Tie Models

LRFD Article 5.6.3.1 states that "strut-and-tie models may be used to determine internal force effects near supports and the points of application of concentrated loads at strength and extreme event limit states." The statement appearing in the second paragraph of this article is stronger, and more specific, namely, "the strut-and-tie model should be considered for the design of deep footings and pile caps or other situations in which the distance between the centers of applied load and the supporting reactions is less than about twice the member thickness."

LRFD Article 5.6.3 provides the following specifications for strut-and-tie modeling.

8.12.3.1 Compression Struts

The factored resistance of strut, P_r, may be calculated as:

$$P_r = \phi P_n \qquad \text{[LRFD Eq. 5.6.3.2-1]}$$

where

ϕ = 0.7 for bearing in concrete and for strut-and-tie models [LRFD Art. 5.5.4.2-1]

P_n = nominal resistance of a compressive strut

8.12.3.1.1 Unreinforced Concrete Struts

The nominal axial resistance of unreinforced struts is calculated as:

$$P_n = f_{cu} A_{cs} \qquad \text{[LRFD Eq. 5.6.3.3.1-1]}$$

where

f_{cu} = limiting compressive stress in strut and is calculated from:

$$f_{cu} = f'_c/(0.8 + 170\varepsilon_1) \le 0.85f'_c \qquad \text{[LRFD Eq. 5.6.3.3.3-1]}$$

where

ε_1 is the principal tensile strain in cracked concrete, and is taken as:

$$\varepsilon_1 = (\varepsilon_s + 0.002)\cot^2\alpha_s \qquad \text{[LRFD Eq. 5.6.3.3.3-2]}$$

where

α_s = smallest angle between the compressive strut and adjoining tension ties, deg

ε_s = tensile strain in the concrete in the direction of the tension tie, in./in.

A_{cs} = effective cross-sectional area of the strut determined from a consideration of the available concrete area and the anchoring or bearing conditions at the ends of the strut [LRFD Art. 5.6.3.3.2.]

For an individual strut, the more basic expression for ε_1 includes an additional term, ε_s, outside the bracket of LRFD Eq. 5.6.3.3.3-2.

For a value of principal tensile strain, $\varepsilon_1 = 0.002$, the concrete in the compression strut can resist a compressive stress of $0.85f'_c$, i.e., the limit for regions of the strut not crossed by or joined to tension ties. It is thus conservatively assumed that the principal compressive strain, ε_2, in the direction of the strut is equal to 0.002.

In the presence of a tension tie at a node, if the reinforcing bars are to yield in tension, there must exist significant tensile strains in the concrete. In LRFD Eq. 5.6.3.3.3-2, as ε_s increases, ε_1 increases, and f_{cu} in LRFD Eq. 5.6.3.3.3-1 decreases. From LRFD Eq. 5.6.3.3.3-2, it is seen that as α_s decreases, $\cot^2\alpha_s$ and ε_1 increase, and therefore f_{cu} decreases. In the limit when $\alpha_s = 0$, the compressive strut direction coincides with that of the tension tie (i.e., incompatibility occurs, and $f_{cu} = 0$ which is an impractical case).

The value of A_{cs} depends on conditions of anchoring of the strut at the node (as shown in **Figure 8.12.2.2-1a-1c**); e.g.

- Strut anchored by reinforcement
- Strut anchored by bearing and reinforcement
- Strut anchored by bearing and strut

The following rules are prescribed for calculating the value of ε_s for substitution in LRFD Eq. 5.6.3.3.3-2:

- For a tension tie consisting of reinforcing bars:
  ε_s = tensile strain in reinforcing bars due to factored loads
- For tension tie consisting of prestressed reinforcement:
  $\varepsilon_s = 0.0$, up to decompression of concrete (i.e., f_{pe})
  $\varepsilon_s = (f_{ps} - f_{pe})/E_p$, beyond decompression

If ε_s varies over the width of the strut, ε_s is taken as the strain at centerline of the strut.

8.12.3.1.2 Reinforced Concrete Struts

For a strut containing longitudinal reinforcement which is detailed to develop its yield stress, the nominal resistanc, e, is calculated as:

$$P_n = f_{cu}A_{cs} + f_yA_{ss} \qquad \text{[LRFD Eq. 5.6.3.3.4-1]}$$

where

A_{ss} = area of reinforcement in strut

A_{cs} = area of concrete strut, calculated as shown earlier

8.12.3.2 Tension Ties

LRFD Article 5.6.3.4.1 states that the nominal strength of a tension tie should be calculated as:

$$P_n = f_yA_{st} + A_{ps}(f_{pe} + f_y) \qquad \text{[LRFD Eq. 5.6.3.4.1-1]}$$

where

f_y = yield strength of longitudinal mild steel reinforcement

f_{pe} = stress in prestressing steel due to prestress (after losses)

A_{st} = area of longitudinal mild steel reinforcement in tie

A_{ps} = area of prestressing steel in tie

In the absence of mild steel, a value of $f_y \approx 60$ ksi may be assumed in the equation, in order to reflect the fact that the stress in the prestressing elements will be increased due to the strain which will cause concrete to crack. [LRFD Art. C5.6.3.4.1]

8.12.3.2.1 Tie Anchorage

The tension tie reinforcement must be anchored in accordance with LRFD Article 5.11 which deals with development of reinforcement. This ensures the satisfactory transfer of the tension force to the node regions.

8.12.3.3 Proportioning Node Regions

In the absence of effective confining reinforcement, it is specified in LRFD Article 5.6.3.5, that the concrete compressive stress in the node regions should not exceed the following:

- $0.85\phi f'_c$ for node regions bounded by compressive struts and bearing areas

- $0.75\phi\, f'_c$ for node regions anchoring only one tension tie
- $0.65\phi\, f'_c$ for node regions anchoring tension ties in more than one direction

where ϕ = resistance factor for bearing on concrete = 0.7 [LRFD Art. 5.5.4.2.1]

Stress limits at a nodal zone are controlled by the type of truss members meeting at the node. At nodes B and C (**Figure 8.12.2-1a**) where compression members meet, and at bearing areas at these locations, a higher compressive stress ($0.85\phi\, f'_c$) is allowed than at A and D where it is necessary to anchor the tension tie, AD. In the latter case, the allowable maximum compressive stress is reduced to $0.75\phi\, f'_c$. This limit is reduced even further to $0.65\phi\, f'_c$ when tension ties converge from more than one direction at a node.

The above reductions in the presence of tension ties reflect the detrimental effect of tensile strain in nodes in which tensile reinforcement is anchored. It can be seen that stresses in nodal zones can be reduced by increasing the size of bearing plates, or by increasing the dimensions of struts and tension ties.

LRFD Commentary Article C5.6.3.5 states that if ties consist of post-tensioned tendons, and if the stress in the concrete does not exceed f_{pc} (at centroid of the tie's cross-section), there is no tensile strain in the nodal zone and the limit for concrete compressive stress may be taken as $0.85\phi\, f'_c$.

8.12.3.4 Crack Control Reinforcement

In order to control crack widths in members designed with the strut-and-tie model (except slabs and footings), and to ensure minimum ductility so that significant redistribution of internal stresses is possible, LRFD Article 5.6.3.6 states that an orthogonal reinforcing grid must be provided near each face. The spacing of bars in such a grid should not exceed 12.0 in., and the ratio of reinforcement area to gross concrete area should exceed 0.003 in each direction.

In general, these crack control requirements lead to a substantial amount of well-distributed reinforcement throughout the member. Accordingly, the *LRFD Specifications* allow for crack control reinforcement located within the region of a tension tie to be included in calculating the resistance of the tie.

8.12.4 Steps for Developing Strut-and-Tie Models

The use of strut-and-tie models typically involves a trial-and-error procedure. The following steps, if followed, should help reduce the effort required:

1) Use strut-and-tie modeling for disturbed regions of the structural member. Solve for internal forces, and their resultants outside of the disturbed regions using sectional analysis with all of the controlling load combinations. These forces from sectional analysis may be considered as boundary forces for the disturbed region model. Apply the resultant forces to the disturbed region along with any external loads that fall on that part of the member.

2) Assume initial models for each of the appropriate controlling load cases and boundary condition force sets. Estimate likely member widths. Elastic stress distribution may be used as a guide. Static equilibrium is then used to determine forces in members due to factored loads. These forces are used in checking member dimensions. It may be necessary to modify the assumed model if the members are determined to be inadequate. A number of appropriate models for different applications are available in the literature [Guyon, (1960); Gergely and Sozen, (1967); Schlaich, et. al., (1987); Collins and Mitchell, (1991); Breen, et. al. (1994)].

3) Draw the strut-and-tie model to a reasonably large scale. This will help avoid errors and give a better appreciation of the proportions of the structure.

4) There is no single strut-and-tie model for a particular system. Generally, the forces will flow in accordance with the pattern of reinforcement. Well-distributed reinforcement should be provided to ensure the redistribution of internal forces in the cracked concrete.

5) Good detailing of the structure is essential to ensure that the assumed flow of forces can be achieved in the cracked structure. Accordingly, reinforcement in tension ties must be effectively anchored to develop the strength of the member. Nodal zones must be checked to ensure satisfactory load transfer between struts and ties.

6) Complicated stress fields such as fans, arches and bands can usually be replaced by simple line struts. Unnecessary complication of the model is not warranted.

8.12.4.1 Design Criteria

Regardless of the strut-and-tie model adopted, the following design criteria must be met:

1. Limits on bearing stresses and on compressive stresses in struts
2. Satisfactory anchorage and careful detailing of tension tie reinforcement
3. Critical examination of nodal zones to determine their maximum capacities
4. Provision of adequate crack control reinforcement throughout, to ensure the redistribution of internal stresses after cracking of concrete

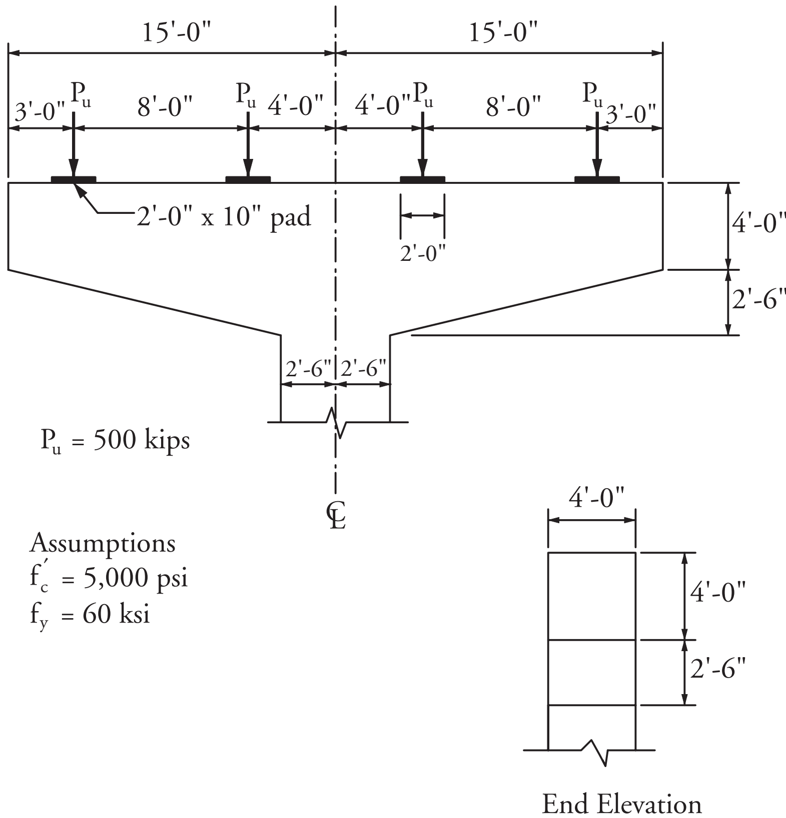
8.12.4.2 Summary of Steps

Step 1 Determine bearing areas	[LRFD Arts. 5.6.3.5 and 5.7.5]
Step 2 Assume appropriate truss geometry (draw a large-scale diagram)	
Step 3 Select tension-tie reinforcement	[LRFD Art. 5.6.3.4]
Select reinforcement distribution	[LRFD Art. 5.6.3.5]
Step 4 Check development of tension-tie reinforcement	[LRFD Arts. 5.6.3.4.2 and 5.11]
Step 5 Check strength of compression struts	[LRFD Art. 5.6.3.3.1]
Step 6 Select crack control reinforcement	[LRFD Art. 5.6.3.6]
Step 7 Detail structure carefully	

8.12.5
Pier Cap Example

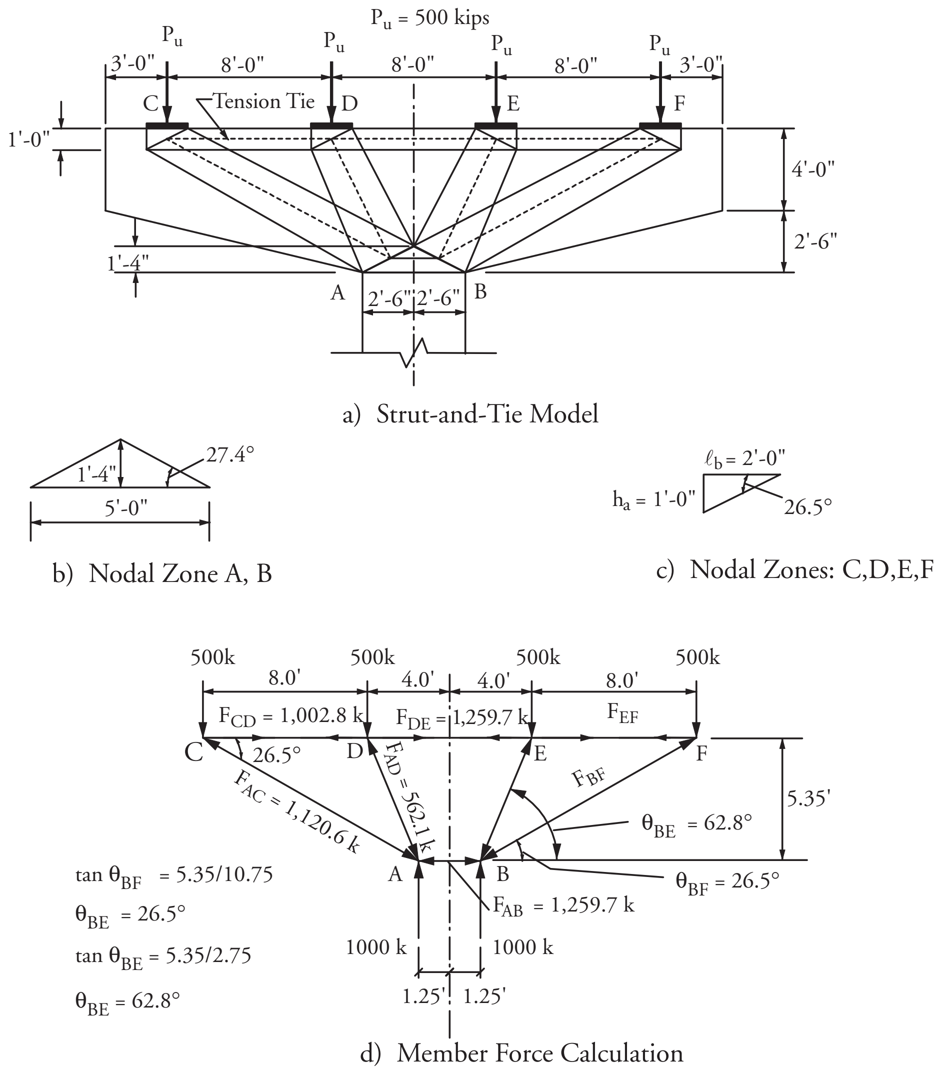
Design the pier cap shown in the figure below.

Figure 8.12.5-1
Pier Cap Dimensions



8.12.5.1 Flow of Forces and Truss Geometry

Figure 8.12.5.1-1a-1d Assumed Truss Geometry



8.12.5.2 Forces in Assumed Truss

Node C:

$$F_{CD} = \frac{500}{\tan(26.5°)} = 1{,}002.8 \text{ kips}$$

$$F_{AC} = \frac{500}{\sin(26.5°)} = -1{,}120.6 \text{ kips}$$

Node D:

$$F_{AD} = \frac{500}{\sin(62.8°)} = -562.1 \text{ kips}$$

$$F_{DE} = F_{CD} + F_{AD}\cos 62.8° = 1{,}002.8 + 562.1\cos 62.8° = 1{,}259.7 \text{ kips}$$

Node A:

$F_{AB} = F_{AD}\cos 62.8° + F_{AC}\cos 26.5° = 562.1\cos 62.8° + 1{,}120.6\cos 26.5° = -1{,}259.7$ kips
$= -F_{DE}$

8.12.5.3
Bearing Stresses

Bearing Stresses at C and D:

$F < P_r = \phi P_n = \phi(0.85f'_c A_1 m)$, assume m = 1 [LRFD Eq.5.7.5-2]

Allowable $F/A_1 = 0.7(0.85 \text{ x } 5) = 2.98$ ksi

Actual $F/A_1 = \dfrac{500 \text{ k}}{24\,(10)} = 2.08$ ksi < maximum allowable O.K.

Bearing Stresses at A and B:

$F/A_1 = \dfrac{2{,}000\text{k}}{60\,(48)} = 0.69$ ksi < maximum allowable O.K.

Therefore, bearing stresses are acceptable.

8.12.5.4
Reinforcement for Tension Tie DE

$F_{DE} = 1{,}259.7$ kips

$\phi f_y A_{st} \geq 1{,}259.7$

$A_{st} \geq 1{,}259.7/0.9(60) = 23.3 \text{ in.}^2$

Because 3'-9" is available for development at C (at inner edge), choose a bar that can be developed in this distance, i.e., choose #10, $\ell_{db} = 43.1$ in. < 45 in. available [LRFD Art. 5.11.2.1].

No. of bars required = 23.3/1.27 = 18.34 bars

Use (20) #10 bars (25.4 in.^{2}) in 2 layers

$F_{CD} = 1{,}002.8$ kips

$A_{st} > 1{,}002.8/0.9(60) = 18.6 \text{ in.}^2$

If (20) #10 bars are used as in DE, $\dfrac{A_s \text{ required}}{A_s \text{ provided}} = \dfrac{18.6}{25.4} = 0.73$

Top bars:

Required development length = 1.3(0.73)(43.1) = 40.9 in. < 45 in. available O.K.

8.12.5.5
Strut Capacities

Note that each strut is traversed by a tie at one end.

Strut AC:

Strut AC is critical due to the small angle it makes with the tension tie, CD.

$F_{AC} = 1{,}120.6$ kips (compression)

End C: (Anchored by bearing and reinforcement)

Tensile strain in tie:

$F_{CD}/A_sE_s = 1{,}002.8/[(20)(1.27)(29{,}000)] = 1.36 \text{ x } 10^{-3}$ in./in.

The tension-tie reinforcing bars are developed in the nodal zone. Therefore, the strain in these bars will increase from zero at the ends to $1.36 \text{ x } 10^{-3}$.

Strain at center of strut, $\varepsilon_s = 1/2(1.36\text{x}10^{-3}) = 0.68 \text{ x } 10^{-3}$

$\varepsilon_1 = (\varepsilon_s + 0.002)\cot^2\alpha_s$

where

$\alpha_s = 26.5°$ [LRFD Art. 5.6.3.3.3]

$\varepsilon_1 = (0.68 \times 10^{-3} + 2.0 \times 10^{-3})\cot^2 26.5° = 10.8 \times 10^{-3}$

$f_{cu} = f'_c/(0.8 + 170\varepsilon_1) \leq 0.85f'_c$ [LRFD Art. 5.6.3.3.3]

$= 5.0/[0.8 + 170(10.8 \times 10^{-3})] = 1.90$ ksi < $(0.85f'_c = 4.25$ ksi)

Capacity of strut, AC at C = $\phi f_{cu}A_{cs}$

where

$A_{cs} = (\ell_b \sin\theta_s + h_a \cos\theta_s)(48)$ **(Figure 8.12.2.2-1c)**

$= (24\sin 26.5° + 12\cos 26.5°) \times 48 = (21.4 \times 48)$ in.^{2}

Capacity of AC at C = (0.70)(1.90)(48 x 21.4)

= 1,366.2 kips > F_{AC} (1,120.6 kips) O.K.

End A of AC is obviously not critical (not crossed by tension tie and wider dimensions than at C).

Strut AD:

Strut AD is anchored by bearing and reinforcement at end A, and crossed by tie at end D.

F_{AD} = 562.1 kips

End D: Tensile strain in tie, DE = $1,259.7/[(20)(1.27)(29,000)] = 1.71 \times 10^{-3}$

Strain at center of strut, $\varepsilon_s = 1/2(1.71 \times 10^{-3}) = 0.86 \times 10^{-3}$

$\varepsilon_1 = (\varepsilon_s + 0.002)\cot^2 \alpha_s$

where

$\alpha_s = 62.8°$

$\varepsilon_1 = (0.86 + 2.0)10^{-3}\cot^2 62.8° = 0.755 \times 10^{-3}$

$f_{cu} = f'_c/(0.8 + 170\varepsilon_1) \leq 0.85f'_c$

$f_{cu} = 5.0/[0.8 + 170(0.755 \times 10^{-3})] = 5.39$ ksi > $0.85f'_c = 4.25$ ksi

Capacity of strut AD at D: $\phi f_{cu}A_{cs}$

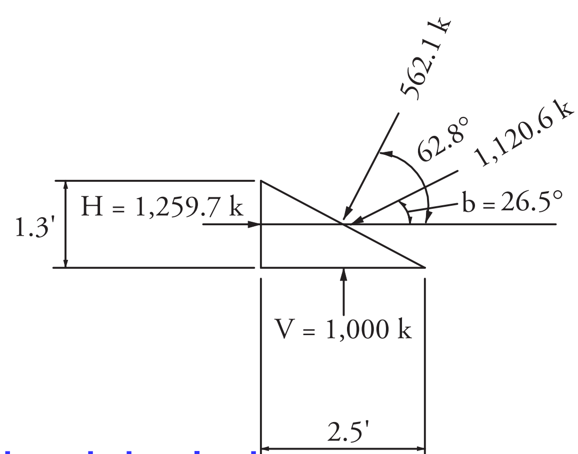
$A_{cs} = (\ell_b\sin\theta_s + h_a\cos\theta_s)(48) = (24\sin 62.8° + 12\cos 62.8°) \times 48 = 1,288$ in.^{2}

Capacity of Strut = (0.7)(4.25)(1,288) = 3,831 kips > F_{AD} = 562.1 k O.K.

End A is obviously not critical (not crossed by tie).

8.12.5.6 Nodal Zone at Pier

Figure 8.12.5.6-1 Nodal Zone at Pier



Horizontal component, H = 562.1cos62.8° + 1,120.6cos26.5° = 1,259.7 kips = F_{AB} O.K.

This nodal zone is bounded only by compressive struts and bearing areas:

Allowable compressive stress, f = $0.85f'_c$ = 2.98 ksi [LRFD Art. 5.6.3.5]

Due to H: F/A = 1,259.7/[(1.3)(12)(48)] = 1.68 ksi < 2.98 O.K.

Due to V: F/A = 1,000/[(2.5)(12)(48)] = 0.69 ksi O.K.

8.12.5.7 Minimum Reinforcement for Crack Control

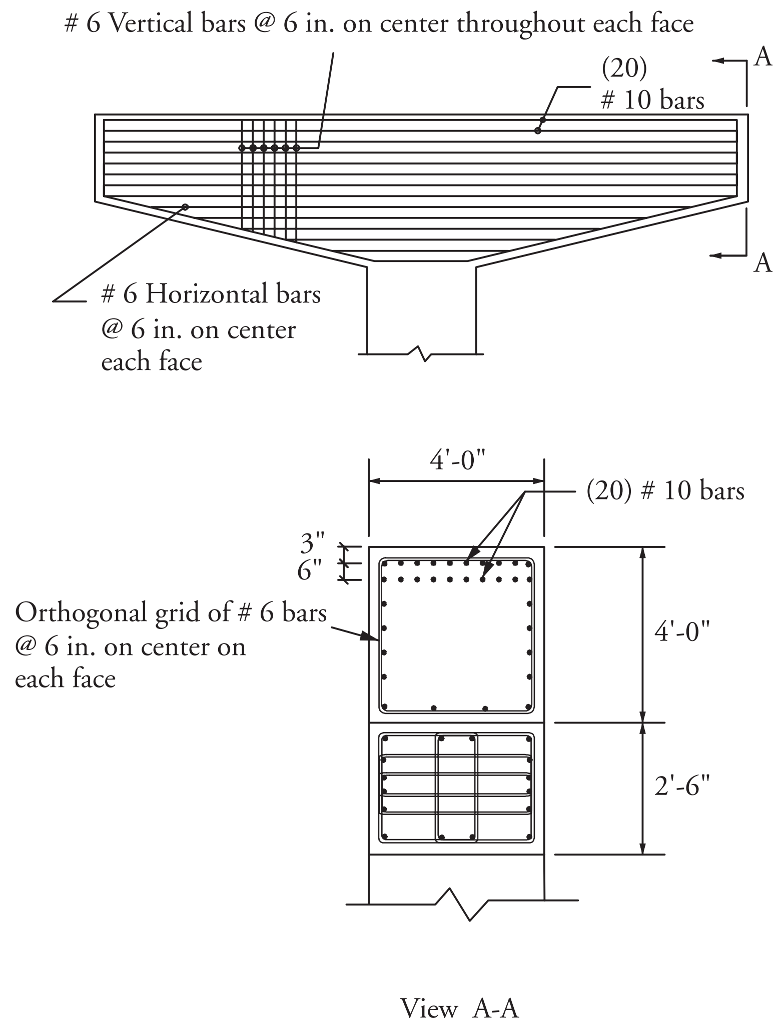
Check at throat and provide this reinforcement throughout (assuming 12 in. spacing).

Minimum A_s required = 0.003(12)(48) = 1.73 in.^{2}/ft [LRFD Art. 5.6.3.6]

Use (2) #9 = 2.00 in.^{2}, #9 each face at 12 in. on center = 2.00 in.^{2}/ft, or #6 bars each face at 6 in. spacing, 4 x 0.44 = 1.76 in.^{2}/ft

Use #6 bars @ 6 in. on center vertically and horizontally.

Figure 8.12.5.7-1 Reinforcement Details



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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.13 DETAILED METHODS OF TIME-DEPENDENT ANALYSIS

Section 8.6 in this chapter presents a variety of practical and relatively simple methods to estimate time-dependent effects in prestressed concrete members. Those methods are suitable for a very wide range of bridge projects but may not be applicable to certain special situations. More detailed methods are available when the designer feels that a more rigorous estimate of time-dependent effects is warranted.

8.13.1 Introduction

The following sections describe a method that can be used to perform time-dependent analysis of a composite prestressed concrete bridge member of any cross-section. This method is based on traditional composite section analysis, using transformed elastic properties of steel elements and any cast-in-place concrete elements. Adjustments are made to the elastic modulus of the concrete elements to reflect creep characteristics. So-called *initial strains* are introduced in the analysis to account for concrete shrinkage, steel relaxation and residual concrete creep. By analyzing discrete cross-sections, and then performing the numerical integration described in Section 8.7.4, whole members may also be analyzed using the methods that follow.

8.13.1.1 Properties of Concrete

The mechanical properties of concrete vary with time. As hydration progresses, compressive strength and modulus of elasticity continually increase, but at a decreasing rate. In addition, it has long been recognized that concrete exhibits creep, defined as the time-dependent increase in strain that occurs while the material is subjected to constant stress. Finally, concrete undergoes shrinkage caused by drying. Chapter 2 and Section 8.6.2.3 provide more detailed discussion of these time-dependent behaviors.

There exists a wide range of methods used to produce precast concrete bridge components. Concrete mixes, aggregates, admixtures, and curing methods all have significant effects on the time-dependent properties of concrete as a structural material. Because of these variations, the recommendations in Chapter 2 and Section 8.6 should be used only as a starting point. For applications where it is critical to accurately predict time-dependent behavior, the properties of the actual materials used should be determined by testing.

8.13.1.1.1 Stress-Strain-Time Relationship

In order to perform time-dependent analysis, it is necessary to establish the stress-strain-time relationship for the concrete material. This relationship will predict the total strain, ε, at a future time, t, that results from a stress increment applied at time, t_o. The total concrete strain at any time, t, can be separated into three components:

- ε_f = the immediate strain due to the applied stress, f
- ε_{cr} = the time-dependent creep strain
- ε_{sh} = free shrinkage strain

It is important to recognize that both the modulus of elasticity, E, and the creep coefficient, C, are functions of time. In addition, because concrete is an aging material, C depends on the loading age, t_o, as well.

a) Constant Stress

Total concrete strain is $(\varepsilon_f + \varepsilon_{cr} + \varepsilon_{sh})$ which is usually expressed as follows:

$$\varepsilon = \frac{f(t_0)}{E_c(t_0)}\left[1 + C(t, t_0)\right] + \varepsilon_{sh} \qquad \text{(Eq. 8.13.1.1.1-1)}$$

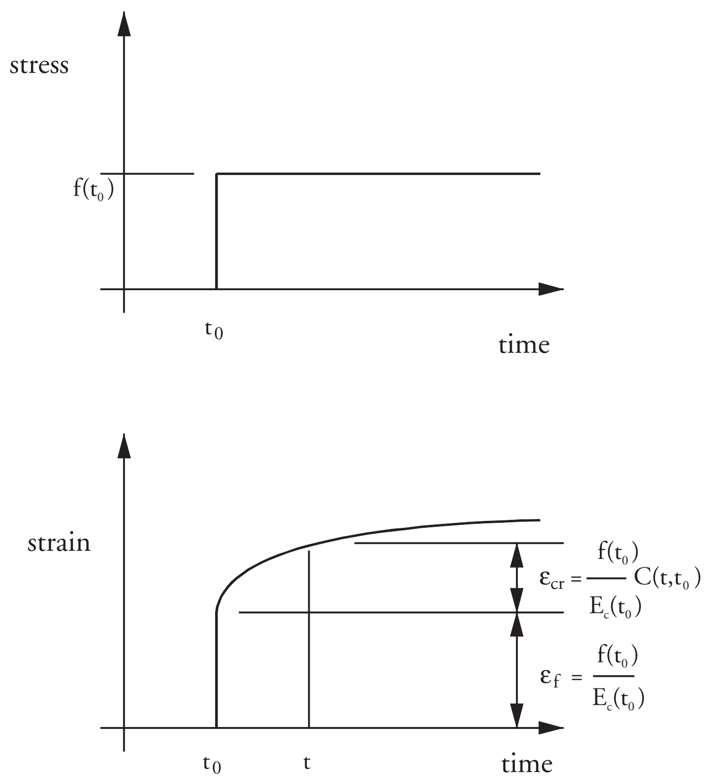
where

$E(t_0)$ = modulus of elasticity at time, t_o, the beginning of the interval

$C(t,t_0)$ = creep coefficient over a time interval from t_0 to t

Eq. (8.13.1.1.1-1) applies as long as stress, f, is a constant, sustained stress. **Figure 8.13.1.1.1-1** shows the gradual development of creep strains with time under the effects of a constant stress.

Figure 8.13.1.1.1-1
Concrete Strain vs. Time
Under Constant Stress

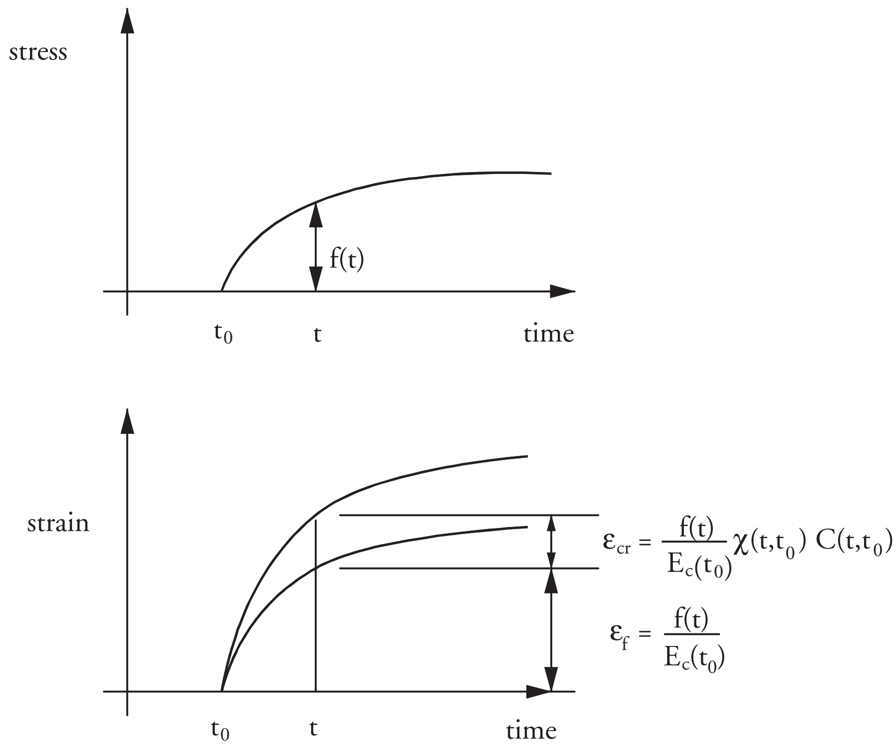


b) Variable Stress

Where the applied stress, f, is variable, Eq. (8.13.1.1.1-1) cannot be used directly. **Figure 8.13.1.1.1-2** depicts the development of creep strains under the effects of an increasing applied stress.

At most stress levels experienced due to service loads, the principle of superposition applies. Using superposition, the effects of a series of applied stress increments can be determined individually, using the above equation, and then combined to give the total time-dependent concrete strain. For a series of stress increments, f_j, applied at times, t_j, the total concrete strain can be expressed as:

Figure 8.13.1.1.1-2
Concrete Strain vs. Time Under Variable Stress



$$\varepsilon = \sum \frac{f(t_j)}{E_c(t_j)}\left[1 + C(t,t_j)\right] + \varepsilon_{sh} \qquad \text{(Eq. 8.13.1.1.1-2)}$$

Therefore, a method for predicting concrete strain, ε, under conditions where stress is not constant, is to break the time interval over which 'f' is applied into many discrete steps and perform a summation using Eq. (8.13.1.1.1-2). While this approach is general and can be easily implemented on a computer, it is not effective for hand calculations. However, an accurate, but simplified method exists and will be discussed further in Section 8.13.1.3.

8.13.1.2 Effective Modulus

The effective-modulus concept is used frequently to simplify creep analysis. The effective modulus is defined as follows:

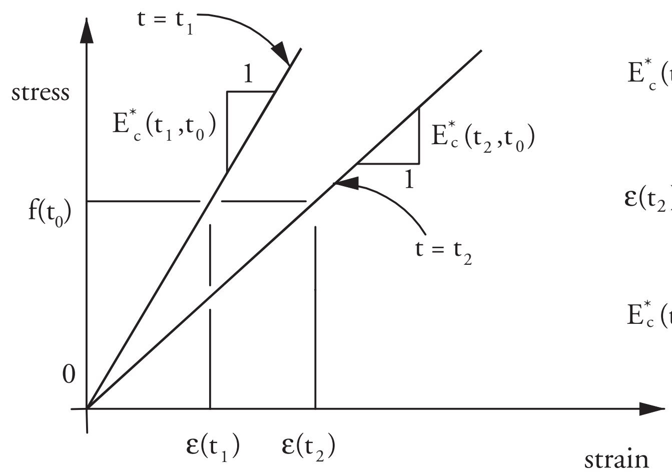
$$E_c^*(t,t_0) = \frac{E_c(t_0)}{1 + C(t,t_0)} \qquad \text{(Eq 8.13.1.2-1)}$$

Comparison with Eq. (8.13.1.1.1-1) shows that E_c^* relates both the immediate strain, ε_f, and the time-dependent creep strain, ε_{cr}, to the applied stress, f. **Figure 8.13.1.2-1** illustrates the effective-modulus concept. Notice that the effective modulus, or the slope of the stress vs. strain curve, depends on both the time of application of the load, t_0, and the time at which strains are to be determined, t_1 or t_2. The use of an effective modulus allows a pseudo-elastic analysis to be performed within a given time interval.

Eq. (8.13.1.1.1-1) can be rewritten to take advantage of the effective-modulus concept:

$$\varepsilon = \frac{f(t)}{E_c^*(t,t_0)} + \varepsilon_{sh} \qquad \text{(Eq. 8.13.1.2-2)}$$

Figure 8.13.1.2-1
Stress vs. Strain for Constant Stress, F_0, Applied at time, t_0



$$\varepsilon(t_1) = \frac{f(t_0)}{E_c(t_0)}[1 + C(t_1,t_0)]$$

$$E_c^*(t_1,t_0) = \frac{E_c(t_0)}{[1 + C(t_1,t_0)]}$$

$$\varepsilon(t_2) = \frac{f(t_0)}{E_c(t_0)}[1 + C(t_2,t_0)]$$

$$E_c^*(t_2,t_0) = \frac{E_c(t_0)}{[1 + C(t_2,t_0)]}$$

8.13.1.3 Age-Adjusted Effective Modulus

Eqs. (8.13.1.1.1-1) and (8.13.1.2-2) are valid only when the stress, f, is constant. In many situations, however, ‘f’ will vary with time. **Figure 8.13.1.1.1-2** graphically depicts this condition.

For instance, consider a hollow precast concrete cylinder that is filled with fresh concrete shortly after the cylinder has been subjected to a constant axial compressive force. When hardened, the cast-in-place concrete fill will be subjected to a load that increases with time as creep strains develop in the surrounding precast cylinder. A similar condition exists in a reinforced concrete member under sustained loads as the reinforcing steel resists creep strains. Solutions of time-dependent problems such as these require the ability to predict creep strains under varying load.

As discussed in Section 8.13.1.1.1, one approach would be to divide the problem into many small time intervals. The stress increment during each interval could be treated as a new load and, since superposition is valid, Eq. (8.13.1.1.1-2) could be used to calculate the total response of the member.

An alternative approach, Bazant (1972), uses the aging coefficient, χ, to adjust the creep coefficient. The aging coefficient accounts for three separate effects:

- When the applied stress, f(t), is increasing, the concrete experiences the maximum force for only an instant at the end of the time interval (t_0,t). At all other times, the concrete experiences a load that is less than the maximum.
- The concrete is gaining strength, and therefore its modulus is increasing with time. Portions of the time-varying load that occur earlier are acting on concrete which is less stiff. Later in the interval, when the loads are larger, the concrete is also stiffer.
- As shown in Chapter 2, for a given concrete in a given environment, the total creep potential for loads applied to young concrete is larger than for the same loads applied to old concrete.

Eq. (8.13.1.3-1) should be used when the stress varies over the interval (t_0, t):

$$\varepsilon = \frac{f(t)}{E_c(t_0)}\left[1+\chi(t,t_0)\,C(t,t_0)\right] + \varepsilon_{sh} \qquad \text{(Eq. 8.13.1.3-1)}$$

The corresponding age-adjusted effective modulus is given by:

$$E_c^*(t,t_0) = \frac{E_c(t_0)}{1+\chi(t,t_0)\,C(t,t_0)} \qquad \text{(Eq. 8.13.1.3-2)}$$

From here on the effective-modulus will be referred to as defined by Eq. (8.13.1.3-2), with the understanding that Eq. (8.13.1.2-1) represents the special case of an instantaneously applied load for which $\chi = 1$.

There are methods available (Bazant, 1972) by which the aging coefficient can be computed precisely for different ages at loading and for different concrete properties. In most practical problems, however, it is sufficiently accurate to use a value of 0.7 or 0.8 for χ, depending on the age of concrete at the beginning of the time interval. For loads applied at a relatively young concrete age, 0.7 should be used. For all other situations, 0.8 is generally sufficiently accurate given all of the other uncertainties present in this type of analysis.

8.13.1.4 Properties of Prestressing Steel

Most prestressing materials, including steel bars and strand, exhibit relaxation. Relaxation is similar to creep but is defined as the loss of stress in a stressed material held at constant length. The following equation may be used to estimate the relaxation, f_r, occurring in steel prestressing materials during the interval (t,t_0):

$$f_r(t,t_0) = \frac{f(t_0)}{K_r}\left[\frac{f(t_0)}{f_y} - 0.55\right]\log_{10}\left(\frac{24t+1}{24t_0+1}\right) \qquad \text{(Eq. 8.13.1.4-1)}$$

$$\text{for } \frac{f(t_0)}{f_y} \geq 0.55$$

where

$f(t_0)$ = tensile stress at the beginning of the interval

f_y = yield strength of the strand

K_r = constant for the material. Values of K_r and f_y for some prestressing strand are provided in **Table 8.13.1.4-1**.

Table 8.13.1.4-1
Values of Material Constant, K_r and Yield Strength, f_y

Grade 270 Strand	K_r	f_y, ksi
Low-Relaxation	45*	243.0
Stress-Relieved (Normal-Relaxation)	10	229.5

** Also, see Note accompanying Eq. (8.6.5.3-1)*

8.13.1.5 Reduced Relaxation under Variable Strain

The relaxation predicted by Eq. (8.13.1.4-1) is the intrinsic relaxation, i.e., the relaxation that occurs under the theoretical condition of constant strain. In an actual prestressed concrete member, strain in the prestressing materials is not constant, and is usually decreasing due to creep and shrinkage of the concrete. Under these circumstances, Eq. (8.13.1.4-1) will somewhat over-predict relaxation. Various researchers, Ghali and Travino (1985), Glodowski and Lorenzetti (1972), Hernandez and Gamble (1975), and others have studied this problem and have proposed various methods of calculating the reduced relaxation that occurs during intervals of decreasing strain.

With modern low-relaxation prestressing materials, however, relaxation effects are very small compared to concrete creep and shrinkage. Therefore, it is sufficiently accurate to adopt a single, standard reduction factor to adjust the intrinsic relaxation during intervals in which the strain is decreasing. In most practical situations, a factor equal to 0.8 may be applied to that portion of the relaxation that occurs under conditions of gradually reducing strain.

8.13.2 Analysis of Composite Cross-Sections

The method of analysis illustrated here is essentially no different than a conventional elastic analysis of a prestressed concrete cross-section using transformed section properties. Instead of a conventional modulus of elasticity, however, the age-adjusted, effective modulus is used for all concrete elements in the section. In addition, initial strains must be considered. The following sections will illustrate the procedure.

8.13.2.1 Initial Strains

An initial strain is defined as one that is not directly due to an applied stress. Other than time-dependent analysis, temperature strain may be the most familiar example of an initial strain.

In time-dependent analysis of concrete members, the initial strains normally considered are:

- free shrinkage of the concrete occurring during the interval in question
- creep strains of the concrete, occurring during the interval in question, that are due to previously applied loads
- the apparent steel strain due to relaxation of prestressing steel during the time interval in question

To incorporate initial strains into cross-section analysis, it is convenient to calculate a fictitious restraining load which will restrain the initial strains described above. The restraining load is then subtracted from any real loads applied to the section. Using the net load, an analysis is performed in a manner similar to conventional transformed section analysis. Finally, the internal forces are calculated using the two components. The internal forces associated with the net load applied to the entire composite section are calculated. These are then added to the individual element restraint forces to give the total actual forces on an individual element of the cross-section. The following section provides a detailed description of the procedure used.

8.13.2.2 Method for Time-Dependent Cross-Section Analysis

Several researchers have published methods to perform the time-dependent analysis of cross-sections of prestressed concrete members. Two approximate methods, suited for manual calculations, and rigorous time-step methods suited only for computerized solutions have been presented. References such as, Branson and Kripanarayanan (1971), Tadros, et al (1975), Tadros, et al (1977A & B), Dilger (1982-both), Tadros, et al (1985), and Collins and Mitchell (1991) can be consulted for additional information.

All of the methods in the cited references, as well as the method presented here, are based on a pseudo-elastic analysis with the following assumptions and conditions:

- the superposition of creep strains from different stress increments is valid
- concrete members remain uncracked
- stress levels are low compared to the compressive strength of the concrete

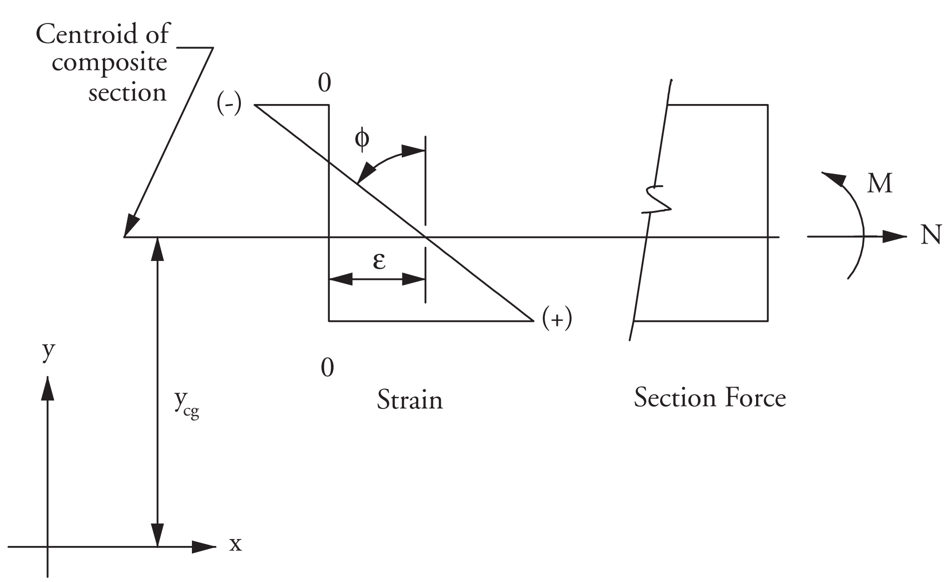
It is necessary to consider the entire history of a cross-section in determining its time-dependent behavior. This history is usually composed of time intervals of varying lengths. Discrete events (such as the release of prestressing or the application of the weight of a cast-in-place topping) mark the beginning and end of each time interval. During the time between these discrete events there is continual creep, shrinkage and relaxation, as well as internal redistribution of stresses. It is convenient to consider each discrete event (such as release of prestressing) as though it occurred during a time interval whose length is zero.

Within a given time interval an elastic analysis with initial strains is performed for the cross-section being analyzed. Transformed composite section properties are recalculated for the analysis in each time interval since the properties of the concrete are time-dependent. A unique set of initial strains, dependent upon all of the stress increments applied during the history of the member, is calculated for each time interval to be analyzed.

The most rigorous methods of time-dependent analysis reduce the time history into many small steps. As the size of the time-step decreases, the accuracy of the analysis increases. One such method is described by Tadros et al (1977B). A slightly less accurate, but greatly simplified method, is presented by Dilger (1982-both) and will be used as the basis for the procedure described here. That method uses creep-transformed section properties based on the age-adjusted, effective modulus for a given time interval.

The sign convention for strain, curvature, and section forces in the following procedure are shown in **Figure 8.13.2.2-1**.

Figure 8.13.2.2-1
Sign Conventions for Composite Section Analysis



8.13.2.2.1 Steps for Analysis

The following steps are repeated for each time interval that is to be analyzed over the entire time history of a single cross-section:

1. Calculate the age-adjusted, effective modulus, E^*_{ck}, for the interval under consideration for each element, k, comprising the composite section. (Take the effective modulus, E^*_c, of the composite section to be that of the concrete beam).
2. Calculate the modular ratio, n_k, for each element in the section.

$$n_k = \frac{E^*_{ck}}{E^*_c} \qquad \text{(Eq. 8.13.2.2.1-1)}$$

3. Calculate the transformed composite section area, A, center of gravity, y, and moment of inertia, I.

$$A = \sum A_k n_k \qquad \text{(Eq. 8.13.2.2.1-2)}$$

$$y = \frac{1}{A}\sum y_k A_k n_k \qquad \text{(Eq. 8.13.2.2.1-3)}$$

$$I = \sum\left[I_k + \left(y - y_k\right)^2 A_k\right] n_k \qquad \text{(Eq. 8.13.2.2.1-4)}$$

4. Calculate the total initial strains, ε_{0k} and ϕ_{0k}, for each element in the composite section. For concrete elements, the total initial strains will be those due to free shrinkage plus those due to creep resulting from previously applied stresses. For prestressed steel elements, the initial strain will be the apparent strain due to relaxation. Typically, non-prestressed steel will have no initial strain. Calculations of initial strains will be presented in the examples that follow.

5. For each element, k, calculate N_{0k} and M_{0k}, the theoretical restraint forces. Sum all of the N_{0k} and M_{0k} over the section to give N_0 and M_0.

$$N_{0k} = -E^*\varepsilon_{0k}A_k \qquad \text{(Eq. 8.13.2.2.1-5)}$$

$$N_0 = \Sigma N_{0k} \qquad \text{(Eq. 8.13.2.2.1-6)}$$

$$M_{0k} = -E^*_{ck}I_k\phi_{0k} \qquad \text{(Eq. 8.13.2.2.1-7)}$$

$$M_0 = \Sigma[M_{0k} - N_{0k}(y_k - y)] \qquad \text{(Eq. 8.13.2.2.1-8)}$$

6. Subtract the restraint forces, N_0 and M_0, from the real applied forces, N and M, and calculate the total strains, ε and ϕ, in the section.

$$\varepsilon = \frac{N - N_0}{E^*_c A} \qquad \text{(Eq. 8.13.2.2.1-9)}$$

$$\phi = \frac{M - M_0}{E^*_c I} \qquad \text{(Eq. 8.13.2.2.1-10)}$$

7. Calculate the element strains for each element in the composite section.

$$\varepsilon_k = \varepsilon - \phi(y_k - y) \qquad \text{(Eq. 8.13.2.2.1-11)}$$

$$\phi_k = \phi \qquad \text{(Eq. 8.13.2.2.1-12)}$$

8. Calculate the element forces, N_k and M_k, and elastic strains, ε_{fk} and ϕ_{fk}, based on the element strains and the effective modulus, E^*_{ck}, for each section element.

$$N_k = E^*_{ck} A_k \varepsilon_k + N_{0k} \quad \text{(Eq. 8.13.2.2.1-13)}$$

$$\varepsilon_{fk} = \frac{N_k}{E^*_{ck} A_k} \quad \text{(Eq. 8.13.2.2.1-14)}$$

$$M_k = E^*_{ck} I_k \phi + M_{0k} \quad \text{(Eq. 8.13.2.2.1-15)}$$

$$\phi_{fk} = \frac{M_k}{E^*_{ck} I_k} \quad \text{(Eq. 8.13.2.2.1-16)}$$

Steps 1 through 8 are repeated for each time interval to be analyzed over the time history of the cross-section.

8.13.2.2.2 Example Calculations

A 12 in.x12 in. concrete prism, reinforced with (4) # 9 reinforcing bars, was loaded with a 216-kip axial compressive force immediately after being wet-cured for seven days. Find the concrete and steel stresses 90 days after loading.

The creep coefficient, C(97,7), is 1.65 and the total free shrinkage strain, ε_{shu}, occurring during this period is -400×10^{-6}. The initial modulus of elasticity of the concrete, E_{ci}, is 3,500 ksi and the modulus of elasticity of the steel bars is 29,000 ksi. The strain in the section immediately after initial loading was calculated to be –0.0003564. The concrete and steel compressive stresses were 1.248 ksi and 10.34 ksi, respectively.

Step 1, Calculate the age-adjusted, effective modulus for the concrete:

$$E^*_c = \frac{3{,}500}{1+(0.7)(1.65)} = 1{,}624 \text{ ksi}$$

Step 2, Calculate the modular ratio for the steel elements:

$$n_s = \frac{29{,}000}{1{,}624} = 17.86$$

Step 3, Calculate the transformed area of the composite section:

$$A = (140)(1.0) + (4.00)(17.86) = 211.4 \text{ in.}^2$$

Step 4, Calculate the initial strain in the concrete (due to both creep and shrinkage):

$$\varepsilon_{0c} = -0.000400 + (1.65)(-0.0003564) = -0.0009881$$

Step 5, Calculate the restraint forces for the concrete element and the composite section:

$$N_0 = N_{0c} = -(1{,}624)(-0.0009881)(140) = 224.7 \text{ kips}$$

Step 6, Calculate the composite section strain:

$$\varepsilon = \frac{0-224.7}{(1{,}624)(211.4)} = -0.0006544$$

Step 7, Calculating the element strains is straightforward for this example. Both the concrete and steel strains are equal to the composite section strain, −0.0006544.

Step 8, Calculate the internal element forces and elastic strains on the concrete and steel:

$$N_c = (-0.0006544)(1{,}624)(140) + 224.7 = 75.9 \text{ kips (tension)}$$

$$N_s = (-0.0006544)(29{,}000)(4.00) = -75.9 \text{ kips (compression)}$$

$$\varepsilon_{fc} = \frac{-75.9}{(140.0)(1{,}624)} = -0.000334$$

$$f_c = \frac{75.9}{140} = 0.542 \text{ ksi (tension)}$$

$$\varepsilon_{fs} = \frac{-75.9}{(29{,}000)(4.00)} = -0.000654$$

$$f_s = \frac{-75.9}{4.00} = -19.0 \text{ ksi (compression)}$$

Therefore, the total concrete and steel stresses at 90 days are 0.706 ksi and 29.3 ksi, respectively. In this example, the initial strains at 90 days were the result of gradual changes that had occurred during the preceding time interval. The aging coefficient in this example was taken to be 0.7 since loading occurred when the concrete was still relatively young.

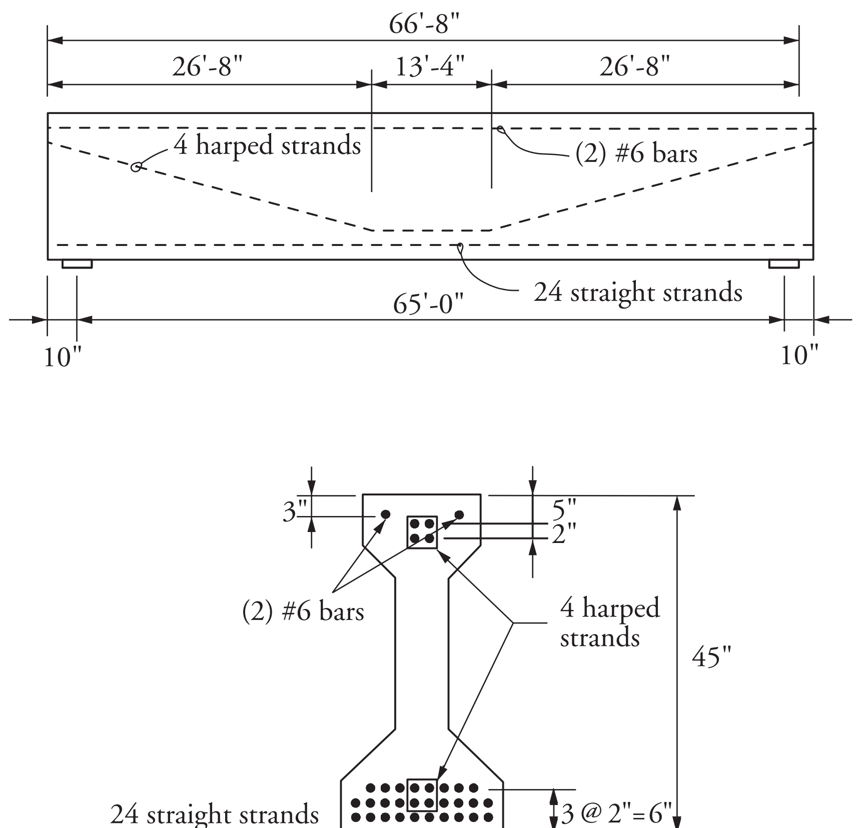


Figure 8.13.3-1 Beam Used for Example Calculations

8.13.3 Analysis of Composite Simple-Span Members

A typical simple-span prestressed concrete bridge beam is shown in **Figure 8.13.3-1**. Detailed information about this bridge is given in Section 8.13.3.2.1. During its history, this beam will experience several different discrete events. Creep, shrinkage and relaxation will continue between these discrete events, accompanied by an internal redistribution of stresses. **Table 8.13.3-1** summarizes the significant time intervals during the life of this typical beam.

Table 8.13.3-1 Beam Lifetime Intervals

Interval	Event	Typical Duration
1	Strand relaxation before transfer	12 to 24 hours
2	Transfer of prestress	0
3	Creep, shrinkage and relaxation of beam after transfer	30 days to 1 year
4	Placement of cast-in-place deck	0
5	Creep, shrinkage and relaxation of composite deck and beam	7 days to 6 months
6	Application of superimposed dead load on the composite deck and beam	0
7	Creep, shrinkage and relaxation of composite deck and beam	25 years or more

The following sections will describe the analyses performed for each time interval during the life of the beam. This is an incremental analysis and the state of stress or strain in the system at any point in time is equal to the sum of the previous intervals.

8.13.3.1 Relaxation of Strands Prior to Transfer

Eq. (8.13.1.4-1) may be used without adjustment to calculate the intrinsic relaxation of the strands prior to release or transfer. While the strands are anchored at the ends of the casting bed, the strain is constant, so the intrinsic relaxation is the correct quantity in this situation.

8.13.3.2 Transfer of Prestress Force

The method described in Section 8.13.2.2 is used to calculate the effects of transferring the prestressing force. Transformed section properties, including the strands, the concrete, and any additional mild steel, are calculated as described. Because this is considered to be an interval of zero duration, the creep coefficient is zero for the concrete. The effective modulus of elasticity of the concrete, therefore, will be equal to the modulus of elasticity at the time of transfer.

The total prestress force in the strands is treated as an external compressive load applied to the transformed section at the centroid of the strands.

8.13.3.2.1 Example Calculation (at Transfer)

Analyze the midspan section of the beam shown in **Figure 8.13.3-1** immediately after the transfer of prestress (note that this is the same beam used in several examples in Section 8.6). To simplify, assume that the strands were tensioned and the concrete cast 18 hours prior to transfer. In practice, it is more likely that strands might be tensioned 18 hours, and concrete cast 12 hours before release. The beam self-weight moment at midspan is 3,694 in.-kips.

Beam data:

Strands:	1/2-in.-diameter strand, Grade 270, low-relaxation
	Eccentricity at midspan = 16.413 in.
	Eccentricity at end of beam = 11.556 in.
Reinforcing Bars:	#6, Grade 60
Beam concrete:	f'_{ci} = 4,950 psi at 18 hours, E_{ci} = 4,054 ksi
	f'_c = 6,750 psi at 28 days, E_c = 4,734 ksi
	$\varepsilon_{shu} = -0.0004$ in./in.
	$C_u = 1.4$
Beam section properties:	AASHTO-PCI Type II
	A = 560 in.^{2}
	I = 125,390 in.^{4}
	y_b = 20.27 in.
Composite section properties:	I_c = 382,372 in.^{4}
	y_{bc} = 36.02 in.
Deck properties:	Width = 104 in.
	Thickness = 8 in.
	f'_c = 4,500 psi at 28 days
	$\varepsilon_{shu} = -0.0004$ in./in.
	$C_u = 1.4$
Dead loads:	Self-weight = 583 plf
	Deck weight = 867 plf
	Haunch weight = 40 plf
	Diaphragm weight = 100 plf
	Superimposed dead load = 360 plf

The total force in the strands, prior to release, is equal to the jacking force less relaxation losses occurring prior to release:

$$f_{pj} = (0.75)(270.0) = 202.5 \text{ ksi}$$

$$f_r = \frac{202.5}{45}\left(\frac{202.5}{243.0} - 0.55\right)\log_{10}\left[\frac{(24)(0.75)+1}{(24)(0.0)+1}\right] = 1.63 \text{ ksi}$$

The force applied to the transformed section at release is:

$$P_i = (202.5 - 1.63)(28)(0.153) = 860.5 \text{ kips} = -N$$

Calculation of transformed composite section properties, i.e. steps 1 through 3 from Section 8.13.2.2.1, is shown in **Table 8.13.3.2.1-1**. The modulus of elasticity of the beam is based on the concrete strength at release. (The section properties of the bare beam have been adjusted in this example to remove the concrete area occupied by strands and mild steel bars. In practice, this refinement may be omitted with no significant loss of accuracy).

Table 8.13.3.2.1-1
Calculation of Transformed Composite Section Properties at Transfer

Item	Area in.^{2} (1)	y_{cg} in. (2)	Moment of Inertia in.^{4} (3)	Modulus ksi (4)	Modular Ratio, n (5)	(1)x(5) in.^{2} (6)	(1)x(5)x(2) in.^{3} (7)	(3)x(5) in.^{4} (8)	[y-(2)]^{2}x(1) in.^{4} (9)	(8)+(9) in.^{4} (10)
Beam	554.836	20.362	123,805	4,054	1.000	554.8	11,298	123,805	207	124,011
Strands	4.284	3.857	10.93	28,500	7.030	30.1	116	77	7,609	7,686
Mild steel	0.880	42.000	0.0	29,000	7.154	6.3	264		3,116	3,116
Composite Section		19.752				591.2	11,678			134,813

Because this is a zero-length interval, there are no initial strains. Steps 4 and 5 may be omitted for zero-length intervals.

Use Eqs. (8.13.2.2.1-9) and (8.13.2.2.1-10) to calculate the strain and curvature of the composite section immediately after release:

$$\varepsilon = \frac{-860.5}{(4{,}054)(591.2)} = -0.000359$$

$$\phi = \frac{3{,}694-(860.5)(19.752-3.857)}{(4{,}054)(134{,}813)} = -1.83 \text{ x } 10^{-5} \text{ in.}^{-1}$$

Next, calculate the individual element strains. The strain due to transfer of prestress at the centroid of the strands is:

$\varepsilon_p = -0.000359 - (-1.83 \text{ x } 10^{-5})(3.857 - 19.752) = -0.000649$

The strain at the centroid of the mild steel bars is:

$\varepsilon_s = -0.000359 - (-1.83 \text{ x } 10^{-5})(42.00 - 19.752) = 0.0000474$

The strain at the centroid of the concrete beam section is:

$\varepsilon_c = -0.000359 - (-1.83 \text{ x } 10^{-5})(20.362 - 19.752) = -0.000348$

Finally, calculate the element forces and elastic strains. For this example, since there are no initial strains, the elastic strains are equal to the total strains that were calculated above. The force on the strands:

$N_p = (28{,}500)(4.284)(-0.000649) = -79.3$ kips

With this information, the remaining stress in the prestressing strands can be calculated:

$$f_p = \frac{860.5-79.3}{4.284} = 182.4 \text{ ksi}$$

The force in the mild steel bars:

$N_s = (29{,}000)(0.88)(0.0000474) = 1.2$ kip

The axial force and moment on the concrete beam section:

$N_c = (4{,}054)(554.8)(-0.000348) = -782.7$ kips

$M_c = (4{,}054)(123{,}805)(-1.83 \text{ x } 10^{-5}) = -9{,}185$ in.-kips

8.13.3.3 Creep, Shrinkage and Relaxation after Transfer

Following the transfer of prestress, but before casting the deck, the beam will undergo gradual changes due to creep and shrinkage of the concrete and relaxation of the prestressing steel. The procedure of Section 8.13.2.2 can be used to analyze these gradual changes. Initial strains due to concrete creep and shrinkage, as well as the apparent strain due to strand relaxation, are included in the analysis. Since the changes occur gradually over this interval, the age-adjusted modulus is used.

8.13.3.3.1 Example Calculation (after Transfer)

Analyze the midspan section of the beam in **Figure 8.13.3-1**, using the results of Example 8.13.3.2.1. Perform the analysis for a time 90 days after casting of the beam.

First, calculate the age-adjusted, effective modulus for the concrete beam. The creep coefficient is:

$$C(90,0.75) = \frac{(90-0.75)^{0.6}}{10+(90-0.75)^{0.6}}(1.4) = 0.836$$

Using an aging coefficient of 0.7, the age-adjusted, effective modulus for the concrete is:

$$E_c^* = \frac{4{,}054}{1+(0.7)(0.836)} = 2{,}558 \text{ ksi}$$

Calculation of the transformed composite section properties is similar to the procedure in Example 8.13.3.2.1 (see **Table 8.13.3.3.1-1**). The modulus of elasticity of the beam concrete is based on the concrete strength at the beginning of the interval, i.e., 4,950 psi.

Table 8.13.3.3.1-1
Calculation of Transformed Composite Section Properties After Transfer

Item	Area in.^{2} (1)	y_{cg} in. (2)	Moment of Inertia in.^{4} (3)	Modulus ksi (4)	Modular Ratio, n (5)	(1)x(5) in.^{2} (6)	(1)x(5)x(2) in.^{3} (7)	(3)x(5) in.^{4} (8)	[y-(2)]^{2}x(1) in.^{4} (9)	(8)+(9) in.^{4} (10)
Beam	554.836	20.362	123,805	2,558	1.000	554.8	11,298	123,805	484	124,289
Strands	4.284	3.857	10.93	28,500	11.142	47.7	184	122	11,574	11,696
Mild Steel	0.880	42.000	0.0	29,000	11.338	10.0	419		5,083	5,083
Composite Section		19.429				612.5	11,901			141,067

Unlike the previous example, there are initial strains to consider in association with the current time interval. First, calculate the initial strain due to shrinkage of the beam concrete:

$$\varepsilon_{sh} = S(90,0.75) = \frac{90-0.75}{55+(90-0.75)}(-0.000400) = -0.000247$$

Next, calculate the creep strain in the beam for this interval. The creep coefficient has already been computed. The elastic strain and curvature from the previous example will be used to compute the creep strains occurring during the current interval:

$$\varepsilon_{cr} = (0.836)(-0.000348) = -0.000291$$

$$\phi_{cr} = (0.836)(-1.83 \text{ x } 10^{-5}) = -1.53 \text{ x } 10^{-5} \text{ in.}^{-1}$$

The theoretical restraint forces for the concrete are calculated next:

$N_c = -(2{,}558)(-0.000247 - 0.000291)(554.8) = 763.7$ kips

$M_{0c} = -(2{,}558)(123{,}805)(-1.53 \text{ x } 10^{-5}) = 4{,}845$ in.-kips

The theoretical restraint force for the strands is due to the apparent strain due to relaxation. Eq. (8.13.1.4-1) is used, along with a reduction factor of 0.8, to compute the reduced relaxation occurring in the strands during the interval:

$$f_r(90{,}0.75) = (0.8)\frac{182.4}{45}\left[\frac{182.4}{243.0} - 0.55\right]\log_{10}\left(\frac{(24)(90)+1}{(24)(0.75)+1}\right) = 1.34 \text{ ksi}$$

The relaxation of stress in the strand is treated as an apparent positive initial strain in the strand, i.e., an apparent increase in strain without a change in stress. Using Eq. (8.13.2.2.1-5), with a positive value for ε_{0p} gives the following value for N_{0p}:

$$N_{0p} = -(28{,}500)\left(\frac{1.34}{28{,}500}\right)(4.284) = -5.7 \text{ kips}$$

Summing the individual restraint forces gives the theoretical restraint forces on the composite transformed section [Eqs. (8.13.2.2.1-6) and (8.13.2.2.1-8)]:

$N_0 = 763.7 + (-5.7) = 758.0$ kips

$M_0 = 4{,}845 - (763.7)(20.362 - 19.429) - (-5.7)(3.857 - 19.429) = 4{,}044$ in.-kips

Eqs (8.13.2.2.1-9) and (8.13.2.2.1-10) are used to compute section strain and curvature:

$$\varepsilon = \frac{(0)-(758.0)}{(2{,}558)(612.5)} = -0.000484$$

$$\phi = \frac{(0.0)-(4{,}044)}{(2{,}558)(141{,}067)} = -1.12 \text{ x} 10^{-5} \text{ in.}^{-1}$$

The element strains in the concrete beam, strands, and mild steel (Eqs. 8.13.2.2.1-11 and 8.13.2.2.1-12) are:

$\varepsilon_c = -0.000484 - (-1.12 \text{ x } 10^{-5})(20.362 - 19.429) = -0.000474$

$\varepsilon_p = -0.000484 - (-1.12 \text{ x } 10^{-5})(3.857 - 19.429) = -0.000658$

$\varepsilon_s = -0.000484 - (-1.12 \text{ x } 10^{-5})(42.0 - 19.429) = -0.000231$

The element forces (Eqs 8.13.2.2.1-13 and 8.12.2.2.1-14) are:

$N_c = (2{,}558)(554.8)(-0.000474) + 763.7 = 91.0$ kips

$N_p = (28{,}500)(4.284)(-0.000658) + (-5.7) = -86.0$ kips

$N_s = (29{,}000)(0.88)(-0.000231) + 0.0 = -5.9$ kips

$M_c = (2{,}558)(123{,}805)(-1.12 \text{ x } 10^{-5}) + 4{,}845 = 1{,}298$ in.-kips

Finally, calculate the elastic strains, i.e. the strains due to stress, in the concrete that occurred during this time interval. These strains will be used to compute creep strains during future time intervals:

$$\varepsilon_{fc} = \frac{91.0}{(2{,}558)(554.8)} = 6.41 \times 10^{-5}$$

$$\phi_{fc} = \frac{1{,}298}{(2{,}558)(123{,}805)} = 4.10 \times 10^{-6} \text{ in.}^{-1}$$

8.13.3.4 Placement of Cast-in-Place Deck

In a typical prestressed concrete beam bridge, the dead weight of a cast-in-place deck, plus intermediate diaphragms (where required) will be carried by the bare precast beam. The placement of these loads on the beam is assumed to occur during an interval of zero length. Analysis of this interval is essentially an elastic analysis using transformed composite section properties calculated on the basis of the modulus of elasticity of the concrete at the time of deck placement.

8.13.3.5 Creep, Shrinkage and Relaxation

Following casting and curing, shrinkage of the deck concrete becomes a significant item affecting the state of stress and strain in the composite system. Since the beam will typically have undergone 40% to 60% of its ultimate shrinkage by the time the slab is cast, the ongoing shrinkage of the slab will usually be larger than the combination of the ongoing shrinkage and creep of the beam. This produces positive curvatures and moments, i.e. tending to cause tension in the bottom of the beam, that gradually diminish with time.

8.13.3.6 Application of Superimposed Dead Load

Application of the superimposed dead load on the composite deck/beam system may occur within 14 days of placement of deck concrete or may be delayed for several months. Concrete barriers and wearing surfaces are the most common instances of superimposed dead loads. Usually, it is assumed that these loads are applied during an interval of zero length, resulting in the performance of an elastic analysis for this interval. The appropriate values of concrete modulus of elasticity for both the deck and the beam, based on their respective ages, are used in calculating the transformed composite section properties.

8.13.3.7 Long-Term Behavior

Following the application of superimposed dead loads, the bridge will typically remain in a constant configuration for several years. During this period, shrinkage of both the deck and beam will continue, but at a steadily decreasing rate. Similarly, creep strains in both the deck and beam will continue to develop. Total creep strains during this interval will be the sum of the creep strains caused by each stress increment applied during the preceding intervals.

By the time the superimposed dead load is applied, 60% or more of the creep due to transfer of prestress probably will have occurred. However, only a small percentage of the creep due to the dead weight of the deck will have occurred at the start of this final interval.

Usually the largest stress increments on the beam are associated with transfer of prestress, application of deck dead weight, and application of superimposed dead loads. It is these large, sudden stress increments which produce the majority of the creep strains. In addition to these stresses, however, we must consider the gradually developing stresses that occur between the major events in the life of the member. These gradually developing stresses are due to restrained or differential shrinkage, relaxation of the strands, and restrained or differential creep. For the purpose of calculating future creep strains, it is customary to assume that these gradually developing stresses can be represented by a sudden stress increment applied at the midpoint of the interval during which they occur. As long as these gradually developing stresses are small compared to the stresses associated with discrete events, the error is small.

8.13.4 Continuous Bridges

In simple-span bridges there will be little or no change in the distribution of forces and moments within the structure as a result of time-dependent deformations. However, multiple-span bridges which are made continuous for live loads and superimposed dead loads, become statically indeterminate after the deck has cured. As a result, any time-dependent deformations that occur after the time that the deck is cured will generally induce forces and moments in the beams (Freyermuth, 1969).

Creep of the beams under the net effects of prestressing, self-weight, deck weight, and superimposed dead loads will tend to produce additional upward camber with time. Shrinkage of the deck concrete will tend to produce downward camber of the composite system with time. In addition, loss of prestress due to creep, shrinkage and relaxation will result in downward camber. Depending on the properties of the concrete materials and the age at which the beams are erected and subsequently made continuous, either positive or negative moments may occur over continuous supports (Oesterle, et al, 1989).

In the situation where beams are made continuous at a relatively young age it is more likely that positive moments will develop with time at the supports. These positive restraint moments are the result of the tendency of the beams to continue to camber upwards as a result of ongoing creep strains associated with the transfer of prestress. Shrinkage of the deck concrete, loss of prestress, and creep strains due to self-weight, deck weight, and superimposed dead loads all have a tendency to reduce this positive moment.

The alternate situation, i.e. where mature beams are erected and made continuous, could result in negative moments at the supports. In this situation, the time-dependent creep strains associated with the transfer of prestress have diminished to the point where the effects that produce downward deflection are more significant. This will induce negative moments as the end rotations that are associated with this sagging at midspan are restrained over the supports.

For the more typical condition of positive moments developing at the piers it is recommended that reinforcing steel be provided to minimize the detrimental effects of cracking at the bottom of the concrete diaphragm. This reinforcement may be accomplished by extending and bending strands from the bottom rows of the beam into the pier diaphragms. Alternatively, mild steel reinforcing bars protruding from the ends of the beams can be extended and bent into the diaphragms. If mild steel bars are utilized, it is essential that these bars extend far enough into the beam to adequately develop the bars. In addition, different length bars should be used to avoid the situation where all the bars terminate at one location.

8.13.4.1 Effectiveness of Continuity

The effects of positive moments, and associated diaphragm cracking, on bridge performance continues to be a hotly debated subject. An argument can be made (Oesterle, et al, 1989) that continuity for live loads becomes unreliable after a small crack has opened near the bottom of the diaphragm. It is pointed out that a finite end rotation is required to close this crack, forcing the beam to carry live loads as a simple-span member. Theoretically, this simple-span action results in live load moments that are significantly higher than those predicted by the design calculations that assumes full continuity.

Countering this argument, however, is the successful experience of the many agencies that routinely design precast, prestressed concrete bridges under the assumption of full continuity for live loads. Distress in the midspan regions of these bridges, predicted by the preceding discussion, has not been reported. In addition, only service load behavior is significantly affected. Under ultimate loads, end rotations of the beams will be large enough to close any crack that may have opened, restoring full continuity. Ultimate capacity, therefore, is relatively unaffected by this phenomenon.

It is unlikely that this issue will be settled completely in the near future. In the meantime, on the basis of the excellent performance of structures of this type, it is recommended that designers continue to rely on continuous action for the design of routine bridges and use details at the piers that have proven to be successful.

8.13.4.2 Applying Time-Dependent Effects

For unusual or special bridges, a time-dependent analysis to predict restraint moments at the piers may be performed according to the procedure in the following section. Construction sequence restrictions, special pier details, and beam design modifications are alternatives the designer may consider should such an analysis predict excessive positive moments.

Specifically, the designer may wish to consider such an analysis when one or more of the following conditions are present:

- Spans 140 feet and longer in humid climates (shorter span lengths should be considered for analysis in arid climates due to increased creep and shrinkage)
- Concrete materials whose creep properties are either unknown, i.e. the mix has not been used previously, or whose creep behavior is known to be poor
- Situations where thermal movements due to daily heating and cooling of the deck are expected to be unusually high

For more information, the reader is advised to consult the references by Mattock (1961), Freyermuth (1969), Oesterle et al, (1989), and Dilger (1982-both) regarding analysis of the effects of creep movements in continuous bridges.

8.13.4.3 Methods of Analysis

The following sections describe two methods to evaluate restraint moments in continuous bridges. The first is a general method and the second is a simplification of the first.

8.13.4.3.1 General Method

The analysis of restraint moments in continuous bridges is a relatively straightforward extension of the methods described in previous sections. Specifically, the following procedure is used:

1. Calculate the time-dependent beam end rotations that would occur under the effects of prestressing, self-weight, and deck weight acting on the simple-span beam using the methods described in Section 8.13.3. Consider only the portions of time-dependent end rotations that occur after the system is made continuous.

2. Using the age-adjusted, effective-modulus method, calculate the rotational stiffness of the beams by conventional stiffness analysis methods. If the beam is prismatic, use of the gross section properties including the deck, is sufficiently accurate. The stiffness factors are given by Eqs. (8.13.4.3.1-1) and (8.13.4.3.1-2) for interior spans and end spans, respectively.

$$M_L = \frac{4E_c^*I}{L}\theta_L + \frac{2E_c^*I}{L}\theta_R \qquad \text{(Eq. 8.13.4.3.1-1)}$$

$$M_R = \frac{2E_c^*I}{L}\theta_L + \frac{4E_c^*I}{L}\theta_R \qquad \text{(Eq. 8.13.4.3.1-2)}$$

where

θ_L = left end rotation of beam due to simple span loads

θ_R = right end rotation of beam due to simple span loads

E_c^* = age-adjusted modulus of concrete

I = moment of inertia of the gross concrete section

L = span length measured center-to-center of the supports for the continuous structure

3. Calculate restraint moments equal to the product of the time-dependent end rotations calculated in step 1 and the rotational stiffness calculated in step 2. Any sign convention may be used, as long as it is consistent.

4. Perform moment distribution analysis for the continuous structure, using the restraint moments as the fixed end moments and the stiffness properties calculated in step 2.

The age-adjusted, effective modulus used in step 2 must be based on gradually varying loads, i.e., use a value of χ that is appropriate for the age and creep coefficient of the concrete. A value of 0.8 may be used with relatively little error.

The effects of permanent loads applied to the structure after continuity is achieved may be computed using a similar analysis. Theoretically, the age-adjusted, effective modulus for this second analysis, however, should be based on a value of χ equal to unity since the application of the load is considered to be instantaneous. As a practical matter, however, this distinction will only affect the distribution of moments in the structure when different creep coefficients are used in different spans. For almost all situations, it would be sufficiently accurate to incorporate the effects of the superimposed dead loads directly into the continuity analysis described above.

8.13.4.3.2 Approximate Method

The above general steps can be further simplified into the following approximate procedures of calculating the restraint moment due to the time-dependent effects.

8.13.4.3.2.1 Restraint Moment Due to Creep

Only loads introduced before continuity can cause time-dependent restraint moment due to creep. Typically, there are pretensioning forces, member self-weight and possibly deck weight. Each loading case is considered separately. The total effect is obtained by simple superposition.

The following assumptions are made. The load is introduced at time, t_0, and the modulus of elasticity of concrete at this time is $E(t_0)$. The continuity is made at time, t_1, and the modulus of elasticity of the concrete at this time is $E(t_1)$. Specifically, the following procedure is used for each load:

1. Calculate time-dependent material properties:

   $C(t,t_0)$ is creep at time, t, for concrete loaded at time, t_0

   $C(t,t_1)$ is creep at time, t, for concrete loaded at time, t_1

   $C(t_1,t_0)$ is creep at time, t_1, for concrete loaded at time, t_0

   Age-adjusted effective modulus for concrete subjected to gradual loading:

$$E_c^*(t,t_1) = \frac{E_c(t_1)}{1+0.7C(t,t_1)} \qquad \text{(Eq. 8.13.4.3.2.1-1)}$$

$$E_c^*(t,t_0) = \frac{E_c(t_0)}{C(t,t_0)-C(t_1,t_0)} \qquad \text{(Eq. 8.13.4.3.2.1-2)}$$

2. Perform elastic analysis, assuming that the load was introduced to a continuous member. Determine the fictitious elastic restraint moments at the supports, M_{el}

3. Determine the time-dependent multiplier, δ_c, corresponding to the load:

$$\delta_c = \frac{E_c^*(t,t_1)}{E_c^*(t,t_0)} \qquad \text{(Eq. 8.13.4.3.2.1-3)}$$

4. Determine the restraining moment, $M_{cr}(t)$:

$$M_{cr}(t) = \delta_c M_{el} \qquad \text{(Eq. 8.13.4.3.2.1-4)}$$

8.13.4.3.2.2
Restraint Moment Due to Differential Shrinkage

The following assumptions are made: (1) The curing of the beam concludes at time, t_2. (2) The curing of the deck ends at time, t_3. Specifically, the following procedure is used for calculating the restraint moment due to differential shrinkage:

1. Calculate time-dependent material properties

   Deck:

   $C_d(t,t_3)$ is the creep at time, t, for deck concrete loaded at time, t_3

   $\varepsilon_{shd}(t,t_3)$ is the shrinkage strain of the deck from time t_3 to time, t

   $E_{cd}(t_3)$ is the modulus of elasticity for deck concrete at time, t_3

   Beam:

   $C_b(t,t_3)$ is the creep at time, t, for beam concrete loaded at time, t_3

   $\varepsilon_{shb}(t,t_2)$ is the shrinkage strain of the beam from time t_2 to time, t

   $\varepsilon_{shb}(t_3,t_2)$ is the shrinkage strain of the beam from time t_2 to time, t_3

   $E_{cb}(t_3)$ is the modulus of elasticity for beam concrete at time, t_3

   The age-adjusted, effective modulus for concrete subjected to gradual loading:

$$E_{cd}^* = \frac{E_{cd}(t_3)}{1+0.7C_d(t,t_3)} \qquad \text{(Eq. 8.13.4.3.2.2-1)}$$

$$E_{cb}^* = \frac{E_{cb}(t_3)}{1+0.7C_b(t,t_3)} \qquad \text{(Eq. 8.13.4.3.2.2-2)}$$

2. Calculate the shrinkage moment, M_{sh}:

$$M_{sh} = Sh_d E^*_{cd} \varepsilon_{shd}(t,t_3)\left(y_{tc} - \frac{h_d}{2}\right) - AE^*_{cb}[\varepsilon_{shb}(t,t_2) - \varepsilon_{shb}(t_3,t_2)](y_{bc} - y_b)$$

(Eq. 8.13.4.3.2.2-3)

where

S = beam spacing

h_d = deck thickness

y_{tc} = distance from centroidal axis of the composite section to the top of the deck

A = gross area of the non-composite beam

y_{bc} = distance from centroidal axis of composite section to the bottom of the beam

y_b = distance from centroidal axis of non-composite section to the bottom of the beam

3. Perform moment distribution analysis for the continuous structure, using the shrinkage moments as the fixed end moments and the stiffness properties calculated from the composite section. The moment at the supports after moment distribution is the restraint moment, $M_{sr}(t)$, due to the differential shrinkage.

Should this analysis predict net positive moments at the piers, the results should probably be treated as an upper bound to the actual moments in the structure. It is likely that the non-prestressed section of the diaphragm between the ends of the beams would experience some cracking at relatively low moments. This would have the effect of introducing a slightly “softer” joint than the fully continuous joint that is assumed by this analysis.

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@Seismicisolation

NOTATION

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

NOTATION

DESIGN THEORY AND PROCEDURE

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.2 FLEXURE

The design of prestressed concrete members in flexure normally starts with determination of the required prestressing level to satisfy service conditions, using "allowable stress design" (ASD). All load stages that may be critical during the life of the structure from the time prestressing is first applied are considered. This is followed by a strength check of the entire member under the influence of factored loads, using "load factor design" (LFD). ASD controls the design of most prestressed concrete bridges. Except for rare situations where strand development length is inadequate, and for some adjacent box applications, LFD seldom requires the addition of reinforcement or other design changes. As a result, the flexural strength of prestressed concrete bridges may be significantly larger than that required. This gives prestressed concrete bridges greater reserve strength than either structural steel or reinforced concrete and may be part of the reason that prestressed concrete beams in bridge structures last longer. Another significant fact is that prestressed concrete members are essentially "proof tested" during fabrication. When prestress strands are released in the plant, the prestress level is the highest a member will ever experience while the concrete strength is at its lowest.

8.2.1 Allowable Stress Design (ASD)

Various load combinations are considered in design. A load factor of 1.0 is used to reflect the actual unfactored loading on the structure. There are exceptions to this unity factor as explained in Chapter 7 and later in Section 8.2.1.2. The basic assumptions for flexural design are:

a.) Plane sections remain plane and strains vary linearly over the entire member depth regardless of load level. Therefore, composite members consisting of precast concrete beams and cast-in-place decks must be adequately connected so that this assumption is valid and all elements respond to superimposed loads as one unit.

b.) Before cracking, stress is linearly proportional to strain; i.e. $f = \varepsilon E$ where f is stress, E is modulus of elasticity and ε is strain.

c.) After cracking, tension in the concrete is neglected.

d.) Spans made continuous for live load through placement of reinforcing bars in the deck slab, or by other means not involving prestressing, are assumed to be treated as prestressed members in the positive moment zone between supports and as conventionally reinforced members in the negative moment zones over the supports. Therefore, no allowable tension limit is imposed on the top fiber stresses in the negative moment zone. However, crack width, fatigue and ultimate strength should be checked.

8.2.1.1 Theory

The various stages of loading for a prestressed concrete beam bridge are shown in **Figure 8.2.1.1-1** and **Figure 8.2.1.1-2**.

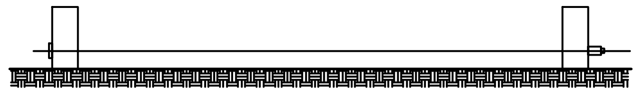
8.2.1.1.1 Stage 1 Loading

Stage 1 involves tensioning the strand in the prestressing bed. The tensile stress in the strand is higher at this stage than at any other stage during the service life of the member. Seating losses in the bed, relaxation losses, and any temperature rise reduce the stress in the strand. However, if the temperature drops, or harped strand is deflected after tensioning, the stress in the strand will rise. Producers take these factors into account as part of manufacturing and quality control processes, so the designers do not need to be concerned with controlling strand stresses before release.

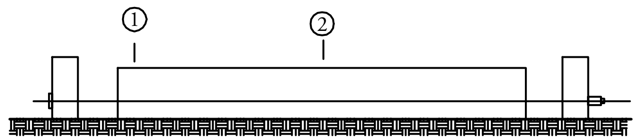
8.2.1.1.2 Stage 2 Loading

Concrete is placed in the forms at Stage 2, and cured until it reaches the initial strength required by design.

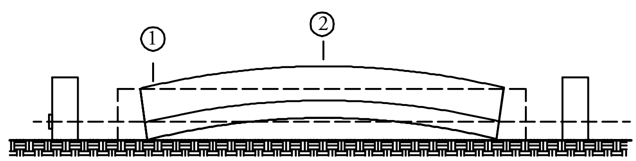
Figure 8.2.1.1-1
Loading Stages of a Precast Prestressed Concrete Bridge Beam



Stage 1: Tensioning of prestressing strands in stressing bed before casting concrete



Stage 2: Placement of concrete in forms and around tensioned strands



Stage 3: Release of strands causing shortening of member

Stage 4: Member placed on piers and/or abutments and deck slab, if any, cast

Stage 5: Full service load after all prestress losses

Figure 8.2.1.1-2
Loading Stages, Stress Diagrams and Corresponding Allowable Stress Limits from Standard Specifications

Stage	1	2	3	4	5
	Tensioning of prestressing strands	Concrete placement	Release of strands	Member installation	Full load
Location 1 (at transfer length)	——	——	$< 7.5\sqrt{f'_{ci}}$ $< 0.6\, f'_{ci}$		
Location 2 (at midspan)	——	——	$< 0.6\, f'_{ci}$		$< 0.6\, f'_c$ $< 6.0\sqrt{f'_c}$

8.2.1.1.3
Stage 3 Loading

The force is transfered to the concrete in Stage 3. This is accomplished by flame cutting or by a gradual release of the jacking force at the stressing abutment (see Chapter 3, Sect. 3.3.2.8 for details). As the prestress is gradually released, the concrete member begins to shorten and camber. When the prestress is fully released, the member resists its own weight and the prestress force. The stress distribution is shown in **Figure 8.2.1.1-2.** Regions near the ends of the member do not receive the benefit of bending stresses due to member weight. Therefore, they may be more critical at release than the midspan section. It should be noted that the very end of the member has zero stresses. A finite distance from the end, called the transfer length, is required for the prestress to be fully transferred to the concrete through bond between the concrete and steel. Thus, for straight strands that are bonded throughout member length, the critical section for release stresses is Section 1-1 shown in **Figure 8.2.1.1-1**. In most applications, this is not the most effective utilization of available prestress.

There are several methods to relieve excessive stresses at Section 1-1. They include the following: 1) harping, where some of the strands are deflected upward from one or two points in the member in order to decrease the end eccentricity, 2) debonding, where the strands are kept straight but are wrapped in plastic over a predetermined distance to eliminate concrete bond, or 3) a combination of harping and debonding. The amount of harping or debonding is a design parameter intended to address the maximum allowed concrete compressive or tensile stress near the ends of a member.

Recent studies, Pang (1997), Huo and Tadros (1997-A), reveal that concrete is capable of resisting more compression than the $0.60f'_{ci}$ allowed by the *Standard Specifications*. If the results of this research are adopted, designers should expect some relief of the requirements of harping and/or debonding. Concrete tensile stresses in the top fibers at Section 1-1 in Stage 3 are also a critical design parameter. Often these stresses are controlled by providing straight top strands. It is advisable to use strand whenever possible (as opposed to bars) due to availability and economy. Top strand may be stressed to a nominal 10 to 15 ksi tension, unless higher prestress is needed by design to control bottom concrete compression. The nominal amount of tension in the strands provides taut straight lines which may be useful in providing firm and accurate anchors for attachment of mild reinforcement.

8.2.1.1.4 Stage 4 Loading

Stage 4 represents conditions several weeks to several months after prestress release. Camber growth and prestress losses are design factors at this stage. If a cast-in-place composite deck is placed, its top surface must follow the required roadway profile. Field adjustments to the haunch (fillet) thickness above the beam top flanges are usually needed to provide the required grade. Reliable estimates of deflection and camber are needed to prevent fillet thickness from being excessive, to avoid intrusion of the beam flange into the deck, or to avoid adjustments of beam seats or roadway approaches. Stresses at this stage are generally not critical.

8.2.1.1.5 Stage 5 Loading

Stage 5 is assumed to occur after an extended period of time during which all prestress losses have occurred and loads are at their maximum. In contrast to Stage 3, this is the condition described as "service load after losses," or "maximum service load, minimum prestress." The tensile stress in the bottom fibers of the midspan section generally controls the design.

8.2.1.1.5.1 Tensile Stresses - Normal Strength Concrete

The tensile stress limit varies from zero to $6\sqrt{f'_c}$ (psi) depending on the severity of exposure and local practices. Generally, it is not advisable to exceed $6\sqrt{f'_c}$ (psi) as cracking might occur under service loads. Some engineers have proposed that prestressed concrete members be allowed to crack, similar to conventional reinforced concrete design. However, until crack control, fatigue and deflection control issues are well researched and design criteria established, the stress should be maintained below cracking in the positive moment zone at service load levels.

8.2.1.1.5.2 Tensile Stresses - High Strength Concrete

It should be noted that for high strength concrete in the 8,000 to 14,000 psi range, the modulus of rupture of concrete has been observed to vary from $8\sqrt{f'_c}$ to $12\sqrt{f'_c}$ as reported by Ralls (1996), Tadros (1996) and others. The major benefit of using high strength concrete in bridge beams is the increase in compressive stress limits which are linearly related to the concrete strength. Increasing the tensile stress limits according to modified formulas would have little impact, (Huo and Tadros (1997)), and is not permitted in the current *Standard Specifications*. Nevertheless, it is reassuring to know that the margin of safety against flexural tension cracking increases with increased compressive strength.

8.2.1.1.5.3 Tensile Stresses-LRFD Specifications

The *LRFD Specifications* require only 80% of the live load moments to be applied to the bridge when checking the tensile stress at service conditions. This reduced live load was determined by comparing a number of bridges designed by both the *Standard Specifications* and the *LRFD Specifications*. It is an acknowledgment of the satisfactory service performance of the very large number of bridges designed by the *Standard Specifications*. Designs using the two Specifications give approximately the same number of strands, except for long spans where the *LRFD Specifications* may still be too conservative in requiring more.

8.2.1.2 Allowable Concrete Stresses

8.2.1.2.1 Standard Specifications

The allowable stresses in a prestressed beam immediately after release due to the prestressing force and the self-weight of the beam and after initial losses due to elastic shortening and strand relaxation have occurred are given below: [STD Art. 9.15.2.1]

1. Compression in pretensioned members, $0.60f'_{ci}$
2. Tension:
   a) In tension areas with no bonded reinforcement, 200 psi or $3\sqrt{f'_{ci}}$, psi
   b) Where the calculated tensile stress exceeds the above value, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile tensile stress should not exceed, $7.5\sqrt{f'_{ci}}$, psi

The allowable stresses in a prestressed beam due to the combined effects of prestressing, self-weight and externally applied loads after all losses have occurred, are given below: [STD Art. 9.15.2.2]

1. Compression for pretensioned members:
   a) The compressive stresses under all load combinations, except as stated in (b) and (c), $0.60f'_c$
   b) The compressive stresses due to effective prestress plus permanent (dead) loads, $0.40f'_c$
   c) The compressive stresses due to live loads plus one-half of the sum of compressive stresses due to prestress and permanent (dead) loads, $0.40f'_c$
2. Tension in the precompressed tensile zone:
   a) For members with bonded reinforcement (includes bonded prestressed strands), $6\sqrt{f'_c}$, psi
   b) For severe corrosive exposure conditions, such as coastal areas, $3\sqrt{f'_c}$, psi

8.2.1.2.2
LRFD Specifications

Stress limits for concrete at release: [LRFD Art. 5.9.4]

1. Compression for pretensioned or post-tensioned members, $0.60f'_{ci}$
2. Tension:
   a) in areas without bonded reinforcement, $0.0948\sqrt{f'_{ci}} \leq 0.2$ ksi
   b) in areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi, $0.24\sqrt{f'_{ci}}$, ksi

Stress limits for concrete at service limit state for fully prestressed components are given in LRFD Article 5.9.4.2 (for more information about Load Combinations, see Section 7.3.2).

1. Compression using the service limit state Load Combination I:
   a) due to effective prestress and permanent (dead) load, (i.e. beam self-weight, deck slab weight, diaphragm weight, wearing surface and barrier weights), $0.45f'_c$
   b) due to effective prestress and permanent and transient loads (i.e., all dead loads and live loads) and during shipping and handling, $0.60\phi_w f'_c$
   c) due to live load and one-half the sum of effective prestress and permanent loads, $0.40f'_c$
2. Tension using the service limit state Load Combination III, where only 80% of the live load effects are considered:
   a) for components with bonded prestressing tendons or reinforcement subjected to not worse than moderate corrosion conditions, $0.19\sqrt{f'_c}$, ksi
   b) for components with bonded prestressing tendons or reinforcement subjected to severe corrosive conditions, $0.0948\sqrt{f'_c}$, ksi
   c) for components with unbonded prestressing, no tension is allowed

The reduction factor, ϕ_w, should be taken equal to 1.0 when the web and flange slenderness ratio, calculated according to LRFD Article 5.7.4.7.1 for hollow rectangular cross-sections, is not greater than 15. For most beams, $\phi_w = 1$.

8.2.1.3
Design Procedure

Generally, the tensile stresses at midspan due to full dead and live loads plus effective prestress (after losses) controls the design.

1. Compute the tensile stress due to beam self-weight plus any other non-composite loads such as the deck, deck forms, haunches, diaphragms, etc., if any, applied to the beam section only.
2. Compute the tensile stress due to superimposed dead loads plus live load (*Standard Specifications*) or 0.8 live load (*LRFD Specifications*) applied to the composite section.
3. The net stress, f_b, due to loads in Steps 1 plus 2, minus the allowable tensile stress is the stress that needs to be offset by prestressing:

$$\frac{P_{se}}{A} + \frac{P_{se}\,e_c}{S_b}$$

where P_{se} is the effective prestress, e_c is strand eccentricity at midspan, and A and S_b are beam area and bottom fiber modulus.

Solve for P_{se}. The estimated number of strands = P_{se}/(area of one strand)(f_{pe}), where f_{pe} is the effective prestress after all losses which may be approximated as 160 ksi for Grade 270 strand.

4. Perform a detailed calculation of prestress losses and repeat Step 3 if necessary.
5. Check stresses at the ends (transfer length) and midspan at transfer and at service. Check stresses at the harp point at transfer. Under typical load conditions, stresses at harp points do not govern at service loads and are therefore not checked. Determine the amount of harping and/or debonding required to control stresses at the end of the beam. This may be done by computing a required "e" for the selected P_{se} when harping is used, or by computing the required P_{se} for a given "e" when debonding is used.
6. Check strength.
7. If necessary, revise number of strands and repeat Steps 4 and 5.

8.2.1.4 Composite Section Properties

8.2.1.4.1 Theory

Certain bridge superstructures, such as I-beams and spread box beams, require cast-in-place (CIP) concrete deck slabs to provide a continuous riding surface. Sometimes, a CIP topping is provided for adjacent precast concrete members, such as solid slabs, voided slab beams and box beams. When the CIP topping is adequately bonded or connected to the precast concrete member, it provides a "composite section" which is capable of resisting superimposed loads introduced after the deck concrete has cured.

Satisfactory composite action is achieved through verification that the interface shear is adequately resisted through bond between the precast and the CIP concrete, and the addition of shear connectors where needed. Composite (horizontal) shear design is considered in Section 8.5.

Once the composite deck has hardened, the member with deck is considered to act as a unit. The assumption that plane sections remain plane after bending is assumed valid for the entire depth of the composite member, at all loading stages through ultimate capacity.

All loads placed on the bridge after the deck has hardened are applied to the composite member. Since the deck concrete usually has a lower strength than the precast concrete, its modulus of elasticity is also lower. The analysis for service loads is simplified by transforming the deck concrete into equivalent beam concrete to obtain a section with uniform material properties. This is done by reducing the width of the CIP concrete using the modular ratio, n, of the CIP to precast concrete. It is generally acceptable to use the modular ratio for the 28-day strength. In reality, the two concretes begin to interact with one another upon initial set of the deck concrete.

Designers are advised to specify sufficient field curing procedures for the deck concrete. The concrete surface should be covered with wet blankets as soon as it is able to accept them, and continued for a period of at least 7 days. This is important to avoid premature shrinkage cracks in the CIP deck, especially over the piers in multi-span bridges with continuously cast decks. Time dependent analysis that accounts for differential creep and shrinkage of the two concretes may alter the stresses obtained from the elastic analysis given below. However, analysis which includes these time dependent effects is complex and requires specialized computer programs, such as CREEP3 described in work by Tadros (1977-B) and Abdel-Karim and Tadros (1993).

8.2.1.4.2 Procedure

1. Compute modular ratio (n) between slab and beam concrete:

$$n = \frac{E_c(\text{slab})}{E_c(\text{beam})}$$

2. Compute effective flange width: [STD Art. 9.8]

For composite prestressed concrete where slab or flanges are assumed to act integrally with the precast beam, the effective flange width may be calculated as follows:

Standard Specifications

- The effective web width needed for computation of the effective flange width should be the lesser of 6(maximum thickness of the flange) + web width + fillets or total width of the top flange. [STD Art. 9.8.3.1]
- The effective flange width of the composite section should be the lesser of 1/4 span length, 1/2 the clear distance on each side of the effective web width plus the effective web width, or 12(slab thickness) plus the effective web width. [STD Art. 9.8.3.2]

LRFD Specifications [LRFD Art. 4.6.2.6.1]

- For interior beams, effective flange width should be the least of 1/4 span length, 12(effective slab thickness) plus greater of web thickness or 0.5(beam top flange width), or average spacing between beams
- For exterior beams, effective flange width may be taken as one-half the effective width of the adjacent interior beam, plus the least of one-eighth of the effective span length or 6.0 times the average thickness of the slab, plus the greater of half the web thickness or one-quarter of the width of the beam top flange, or width of the overhang
- For closed precast concrete boxes, the distance between the outside of webs at their tops will be used in lieu of the web thickness, and the spacing will be taken as the spacing between the centerline of boxes.

3. Compute transformed section properties:

Transformed flange width = (n)(effective flange width)

If the haunch is considered in the composite section properties, its width should be transformed before it is used in calculations. Note that the haunch thickness should not be included unless the design drawings show a minimum thickness specified after adjustment for camber and deflection.

8.2.1.5 Harped Strand Considerations

When concrete stresses exceed allowable limits, strand harping becomes an attractive option to reduce prestress eccentricity. The designer should be familiar with the practice and limitations of local producers when considering whether or not the calculated force and harp angle can be tolerated. The following are some options to consider if the hold-down force exceeds that which the fabricators can accommodate:

1. Split the strands into two groups with separate hold-downs.
2. Change slope of harp by moving harp points closer to centerline of the beam, or by lowering harp elevation at beam ends, or both. Also, refer to Chapter 3 for additional discussion on uplift force and harp angle.
3. Decrease the number of harped strands.
4. Use debonding instead of harping or combine debonding with harping to reduce harping requirements. Refer to Chapter 3, Fabrication, Section 3.3.2.2 for additional details.

8.2.1.6 Debonded Strand Considerations

An alternative to strand harping is to reduce the total prestress force by debonding some strands at the ends of members. After prestress is released to the concrete member, the debonded length of the strand has zero stress. Strand debonding may be more economical for some precast producers than harping. However, designers should take into account the effects of the reduction of precompression, (P/A), as well as the loss of the vertical component of prestress which contributes to shear resistance near the member ends. In addition, the calculated strand development length at the end of a debonded strand is required to be doubled by the *Standard Specifications*. Debonded strands have been shown by recent studies, Russell and Burns (1993, 1994-A and 1994-B), to perform well and their use is encouraged whenever possible. The *Standard Specifications* do not contain specific requirements regarding the maximum number and distribution of debonded strands. However, LRFD Article 5.11.4.3 provides the following rules if debonded prestressing strands are used:

1. The number of partially debonded strands should not exceed 25% of the total number of strands.
2. The number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row.
3. Debonded strands should be symmetrically distributed about the centerline of the member.
4. Exterior strands in each horizontal row should be fully bonded.

The 25% limitation of rule 1 appears to be too conservative according to current practice in several states and the studies by Russell and Burns (1993, 1994-A and 1994-B).

It is good practice to limit the number of debonded strands that are terminated at any section to 40% of the shielded strands, or 4, whichever is greater.

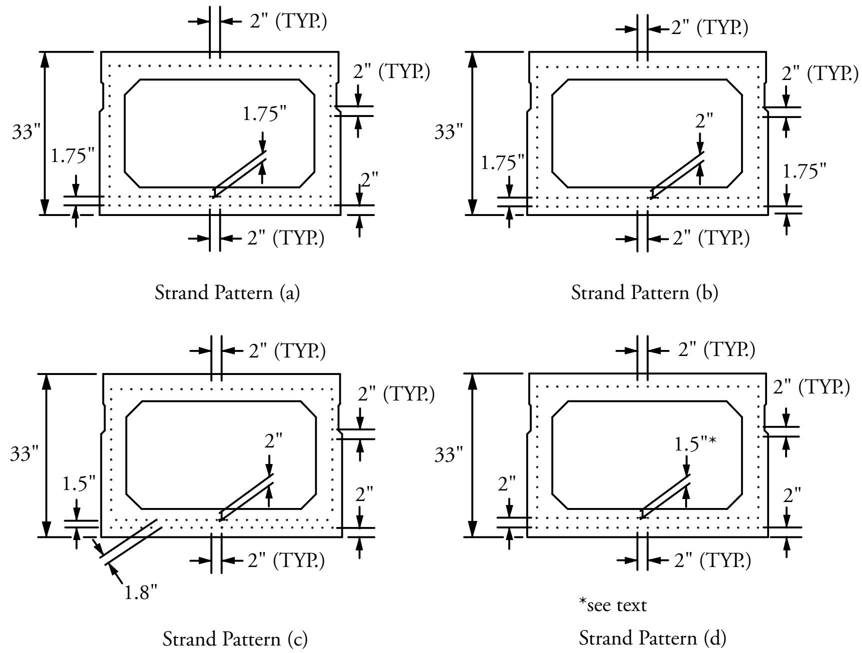
8.2.1.7 Minimum Strand Cover and Spacing

The *Standard Specifications* require a minimum concrete cover over strands of 1.50 in. The *LRFD Specifications* are unclear regarding concrete cover over prestressing strand in precast concrete beams. For precast soffit form panels (stay-in-place deck panels), the minimum cover is 0.80 in. and for members subject to exterior exposure, the minimum is 2.0 in. regardless of whether the member is precast or cast-in-place. It is recommended here to use the 1.50 in. minimum cover specified in the *Standard Specifications* for bridge beams.

Figure 8.2.1.7-1 shows four possible strand patterns to accomplish various strand spacing and cover requirements. Dimensions are to centerlines of strands. Pattern (d) would require a thicker bottom flange with adjusted void depth if 1.5 in. clear cover is required over the second row to the void.

The Federal Highway Administration has approved use of 1/2-in.-diameter strand at a spacing of 1.75 in., and 0.6-in.-diameter strand at 2.00 in. on center. As a result, box beams, for example, may have two layers of 1/2-in.-diameter strands in the bottom flange using one of the alternative patterns shown in **Figure 8.2.1.7-1**. If the vertical strand spacing is desired to be 2 in., the bottom flange thickness may have to be increased to satisfy the minimum cover requirements.

Figure 8.2.1.7-1
Alternative 1/2-in.-Diameter Strand Patterns for Typical AASHTO Box Beam

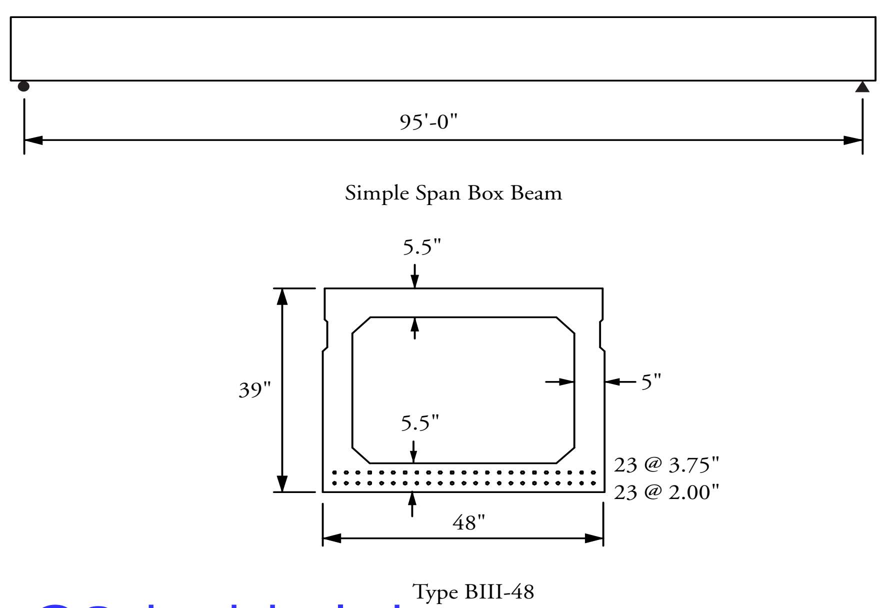


8.2.1.8
Design Example

The following information is given for the box beam design example shown in **Figure 8.2.1.8-1**.

- design span, L = 95.0 ft
- self-weight of the beam = 0.847 kip/ft
- superimposed live load = 1.840 kip/ft
- concrete strength at release, f'_{ci} = 4,000 psi
- 28-day concrete strength, f'_c = 5,000 psi
- prestressing strands: 1/2" diameter, low-relaxation, 270 ksi steel
- stress in the strand just after release, f_{si} = 192 ksi
- strand stress after all losses, f_{se} = 172 ksi

Figure 8.2.1.8-1
Elevation and Cross-Section of the Box Beam



- area of box beam cross-section, A = 813 in.^{2}
- section modulus for the extreme bottom fiber of the precast section, S_b = 8,728 in.^{3}
- section modulus for the extreme top fiber of the precast section, S_t = 8,542 in.^{3}
- distance from the centroid of the cross section to extreme bottom fiber, y_b = 19.29 in.

Note that the numbers of strand used in this example and in the examples of Sections 8.2.2.4 and 8.2.2.6 is unusually large. They have been selected to illustrate the capacity of standard precast concrete beam shapes and to demonstrate how to resolve challenging design issues.

8.2.1.8.1 Design Requirement 1

Determine the amount of prestressing force required to produce a tensile stress in the bottom fiber at the midspan section under all loads equal to $6\sqrt{f'_c} = 424$ psi.

First calculate the moments due to self-weight and superimposed live load using the following equation at different points along the span, L.

$$M_x = 0.5wx(L - x) \qquad \text{(Eq. 8.2.1.8.1-1)}$$

where

x = distance between the support and the point on the span under consideration

w = uniformly distributed load

Eq. (8.2.1.8.1-1) above, reduces to $\frac{wL^2}{8}$ when x = L/2. The general formulation is given here because it is needed to calculate subsequent requirements.

The equation for bottom fiber stress due to applied loads for non-composite sections is:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}e}{S_b} - \frac{M_g + M_{LL}}{S_b} \qquad \text{(Eq. 8.2.1.8.1-2)}$$

where

P_{se} = effective prestress force after all losses

e = strand eccentricity at the section being considered (assumed at 16.42 in.)

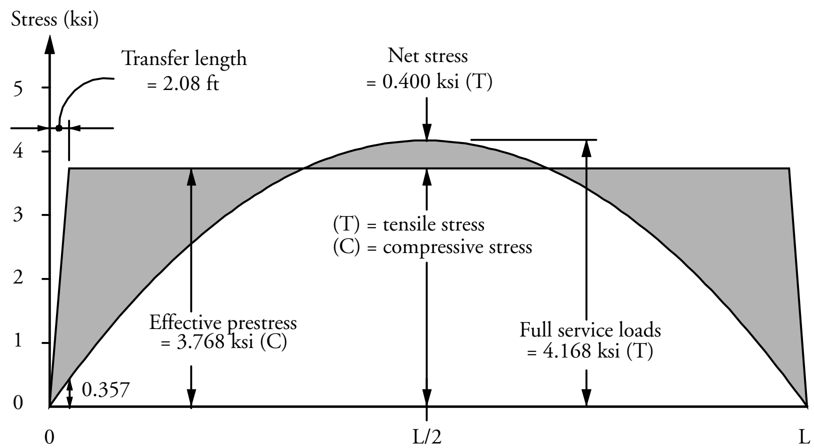
M_g = bending moment due to self-weight

M_{LL} = bending moment due to live load

The calculations for the required number of strands are computed here at the mid-span section. Other design problems may have to be checked at other locations where tensile stresses are higher, e.g. members with harped strands or exterior span members made continuous with interior spans.

Next, the value of P_{se} can be obtained using Eq. (8.2.1.8-2). Set the bottom stress equal to 424 psi (T) and solve for P_{se}. This will yield P_{se} equal to 1,203 kips. Since the prestressing force per strand is equal to 0.153 (172) = 26.32 kips, the required number of strands is 45.7 (use 46 strands) as shown in **Figure 8.2.1.8-1**. **Figure 8.2.1.8.1-1** shows the bottom fiber stress distribution due to full service loads plus the effective prestress force, assuming that the strands are fully-bonded straight strands for the entire length of the member. Note that this figure and the following figures show the stress diagrams due to prestress and due to gravity loads superimposed on each other to get an appreciation for the relative impact of the various components on the net stresses.

Figure (8.2.1.8.1-1) Bottom Fiber Stress Distribution Due to Full Service Load Plus Effective Prestress



8.2.1.8.2 Design Requirement 2

Find the compressive stress in the top fibers at midspan due to the full service loads and the effective prestress force.

Using the formulas in Section 8.2.1.8.1 with the proper section modulus and stress signs and $P_{se} = (46)(0.153)(172) = 1{,}211$ kips, the top fiber stress distribution is given in **Figure 8.2.1.8.2-1**. The net compressive stress is equal to 3.420 ksi. This exceeds the allowable compressive stress which is equal to $0.60f'_c = 0.6(5.000) = 3.000$ ksi

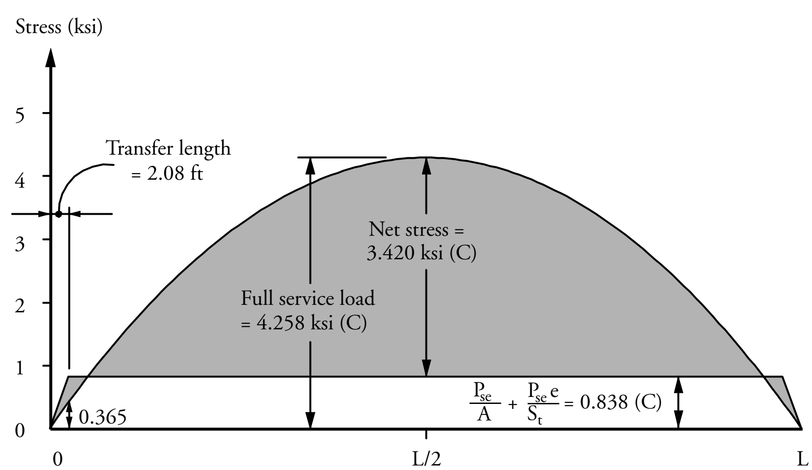
This problem may be solved by increasing the 28-day concrete strength to:

$$\frac{3{,}420}{0.6} = 5{,}700 \text{ psi.}$$

Another option is to add bottom strands if space is available. Note that in this section, all practical available strand locations are utilized for purposes of illustration. Follow-up examples show that other design criteria are not met and indicate how to address deficiencies.

Note that the *Standard Specifications* and the *LRFD Specifications* require additional compressive stress checks due to other loading combinations.

Figure 8.2.1.8.2-1 Top Fiber Stress Distribution Due to Full Service Load Plus Effective Prestress

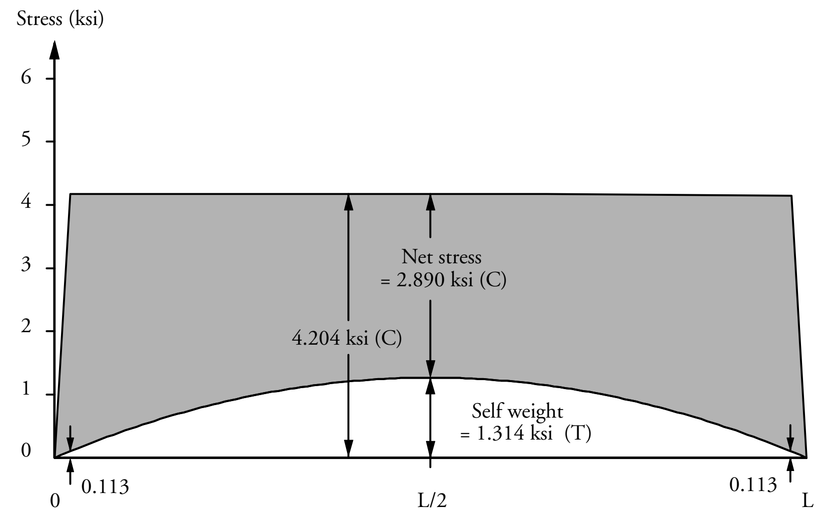


8.2.1.8.3 Design Requirement 3

Determine the compressive stress in the bottom fibers due to the member weight and the initial prestress force and find solutions for excessive stresses.

Using Eq. (8.2.1.8.1-2), with $(M_g + M_{LL})$ replaced by (M_g) only and with P_{se} replaced by $P_{si} = 46(0.153)(192) = 1{,}351$ kips, the net bottom fiber stress can be calculated to be 2.890 ksi as shown in **Figure 8.2.1.8.3-1**. This exceeds the allowable compressive stress which is equal to $0.60f'_{ci} = 0.6(4.000) = 2.400$ ksi.

Figure 8.2.1.8.3-1
Bottom Fiber Stress Distribution Due to Self-Weight Plus Initial Prestress, Using Fully Bonded Straight Strand



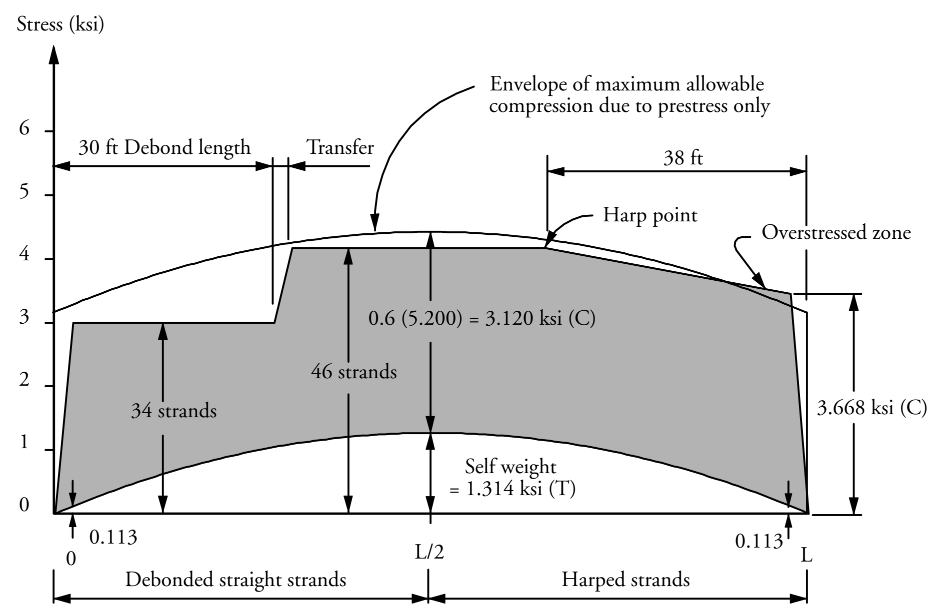
This stress would be acceptable if the initial concrete strength at release (f'_{ci}), is increased to:

$$\frac{2{,}890}{0.6} = 4{,}817 \text{ psi.}$$

Note that f'_{ci} = 4,817 psi would satisfy the stress limits only at the midspan section. It must be increased somewhat to allow the 46 strands to be used for a reasonable distance within the middle zone of the member as illustrated in **Figure 8.2.1.8.3-1**.

Assume f'_{ci} = 5,200 psi and proceed with strand debonding accordingly. Of course, this would imply that f'_c at service would have to be specified to be at least equal to 5,200 psi. Please note that the stress diagram has been modified to reflect gradual transfer of the prestress over a transfer length equal to 25 in. as shown in **Figure 8.2.1.8.3-2**. A strand debonding pattern is attempted for the left half of the beam and a strand harping pattern for the right half. Obviously, only one solution would be used for both halves of a beam in actual design.

Figure 8.2.1.8.3-2
Bottom Fiber Stress Distribution Due to Self-Weight Plus Initial Prestress, Using Debonded or Harped Strands



8.2.1.8.3.1
Strand Debonding

The diagram in **Figure 8.2.1.8.3-2** should be utilized to determine the number and length of strands to be debonded at the member end. The stress diagram due to initial prestress should always be within the maximum compressive stress envelope shown. The number of debonded strands and the arrangement within a section should be carefully determined to avoid possible stress concentrations. The *LRFD Specifications* give guidelines that should be followed. **Figure 8.2.1.8.3-2** shows that debonding 12 strands for a length equal to 30 ft from the end is sufficient. In actual design, it would be advisable to debond the strands in three groups of 4 strands each over lengths of about 10, 20 and 30 ft.

8.2.1.8.3.2
Harped Strands

A similar analysis can be done for the right half of the beam, with harped strand. Strand harping offers two advantages over strand debonding:

- the average prestress, P/A is higher
- the vertical prestress component due to harping produces a shear force that "balances" part of the shear due to gravity load

However, there are two disadvantages:

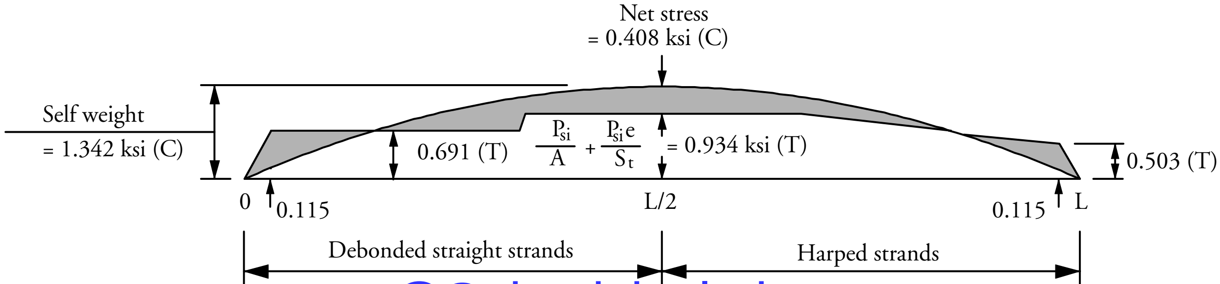
- hold down devices and the labor involved in harping may make it a more expensive solution
- only a limited number of strands can be harped

The maximum number of strands to be harped is dictated by their location. Only strands that can be raised into the webs may be harped. Therefore, using the strand pattern shown in **Figure 8.2.1.8-1**, the maximum number of strands that can be harped is four in each web. It is common practice to harp the strands at 0.30 to 0.45L from the member end. Use 0.40L for this example. Also, harp using the maximum possible slope, which corresponds to minimum cover required of the top layer of harped strands, assumed here to be 3 in. Also note that prestressing bed capacity may control the maximum hold-down force of harped strands. Based on this configuration and using P_{si} = 1,351 kips, e = 13.69 in., and M_g = 81.85 ft-kips, the concrete compressive stress at release = 3.668 ksi which exceeds the allowable limit. This solution is therefore unsatisfactory. To allow a harped-strand solution to work, a number of options may be exercised. They include rearranging the pattern of the 46 strands to allow more strands to be harped in the webs.

8.2.1.8.3.3
Other Methods to Control Stresses

It is possible to combine harping strands with a minor amount of debonding, which seems attractive for this example as there is only a small region where the stress limits are exceeded. Finally, it is possible to control the release stresses by means of temporary pretensioned straight top strands. This option involves shielding the strands for most of the member length, except perhaps 5 to 10 ft at each end of the member. The

Figure 8.2.1.8.4-1
Top Fiber Stress Distribution Due to Self-Weight Plus Initial Prestress, Using Debonded or Harped Strands



shielding will allow this temporary prestress to be eliminated in most of the member length after it is no longer needed. When enough gravity load is introduced, when the concrete strength is increased, or when time dependent losses take effect, the tension in these strands can be released by cutting them through a pre-formed pocket. Sometimes, it may be acceptable to avoid cutting top strands if the compression due to effective prestress plus full loads is not critical.

8.2.1.8.4
Design Requirement 4

Calculate the net stress in the top fibers due to the self-weight of the beam and the initial prestress force. Consider both patterns of strand debonding and harping.

Using Eq. (8.2.1.8.1-2), modified with $P_{se} = P_{si} = 1{,}351$ kips and S_b replaced with $S_t = 8{,}542$ in.^{3}, the top fiber stress can be obtained, as shown in **Figure 8.2.1.8.4-1**. The two solutions have been shown here for comparison purposes. At midspan, there is a net compressive stress of 408 psi, while a net tension of 200 psi or $3\sqrt{f'_{ci}} = 3\sqrt{5,200}$ = 216 psi is allowed. There is tension, however, at other locations as shown in the diagram. If the 200 psi limit is exceeded, the AASHTO Specifications allow the stress to be as high as $7.5\sqrt{f'_{ci}} = 7.5\sqrt{5,200} = 541$ psi if bonded reinforcement is provided to resist the entire tension force at the section being considered.

8.2.1.9
Fatigue

The *Standard Specifications* do not require a fatigue check for prestressed concrete members. The *LRFD Specifications*, however, require a fatigue check according to Article 5.5.3.1. Current practice with precast, pretensioned concrete members does not permit cracking of concrete due to service loads. Accordingly, the tensile stress range in the strand due to live loads is only n times the corresponding concrete stress, where n is modular ratio between steel and concrete. This is always lower than the 10 ksi fatigue limit stress range. Therefore, fatigue in strands should not be considered in members designed to be uncracked under service load conditions. Fatigue of concrete in compression is very unlikely to occur in actual practice. However, this issue is addressed in both the *Standard Specifications* and *LRFD Specifications* by setting a maximum concrete compressive stress limit due to full live load combined with 1/2 of permanent loads plus effective prestress.

8.2.2 Flexural Strength Design

8.2.2.1
Theory

Approximate formulas for pretensioning steel stress at ultimate flexure are given in the *Standard Specifications* (in this manual, Section 8.2.2.2, STD Eq. 9-17) and *LRFD Specifications* (Section 8.2.2.3, LRFD Eq. 5.7.3.1.1-1). Use of these formulas simplifies the process of calculating the flexural strength, M_n, by eliminating consideration of nonlinear material properties of both concrete and prestressing steel at ultimate conditions. However, due to their simplified nature, these formulas should be used with caution especially beyond the limits for which they were developed. As will be shown in Section 8.2.2.5, the general strain compatibility approach can be used to avoid difficulties in applying the approximate formulas or inaccuracies associated with their use.

In addition to the standard assumptions used in flexural, strength analysis, e.g., equivalent rectangular stress block with ultimate concrete strain of 0.003, the approximate formulas (STD Eq. 9-17 and LRFD Eq. 5.7.3.1.1-1) for calculation of strand stress at ultimate flexure are based on the following simplified assumptions:

1.) The compression zone is either rectangular (*Standard Specifications*) or T-shaped (*LRFD Specifications*).

2.) The compression zone is within only one type of concrete; for composite members, it is assumed to be within the deck concrete.

3.) Only fully-tensioned strands near the tension face of the member may be used. No strands near the compression face of the member, or uniformly distributed in the cross-section, can be accurately accounted for.

4.) Effective pretension is not less than 50% of the ultimate strength of the strands.

5.) Steel content must be below the amount that causes the steel stress being predicted to be lower than the yield strength. In other words, the formulas are only intended as an interpolation function between the yield and ultimate strengths of the steel.

Examples are given in Section 8.2.2.4 to illustrate how to apply the approximate procedures, and in Section 8.2.2.6 to discuss their accuracy compared to the more general strain compatibility procedure.

8.2.2.2 Standard Specifications

8.2.2.2.1 Ultimate Moment Capacity

8.2.2.2.1.1 Required Parameters

For bonded members with prestressing only, the average stress in prestressing reinforcement at ultimate load, f^*_{su}, is: [STD Art. 9.17.4.1]

$$f^*_{su} = f'_s\left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right) \qquad \text{[STD Eq. 9-17]}$$

The depth of equivalent rectangular stress block, assuming a rectangular section, is computed by:

$$a = \frac{A^*_s f^*_{su}}{0.85 f'_c b} \qquad \text{[STD Art. 9.17.2]}$$

where

a = depth of stress block

γ^* = 0.28 for low-relaxation strand

β_1 = ratio of depth of equivalent compression zone to depth: [STD Art. 8.16.2.7]
for $f'_c \leq 4$ ksi, $\beta_1 = 0.85$;
for 4 ksi $< f'_c <$ 8 ksi, $\beta = 0.85 - 0.05(f'_c[\text{ksi}] - 4)$;
for $f'_c \geq 8$ ksi, $\beta_1 = 0.65$

f'_c = 28-day concrete strength of compression fiber (slab concrete for composite members)

ρ^* = ratio of prestressing strands = $A^*_s/(bd)$

A^*_s = area of prestressed reinforcement

b = effective flange width

d = distance from top of slab to centroid of prestressing force = beam depth (h) + slab thickness − y_{bs}

y_{bs} = distance from center of gravity of the strand to the bottom fiber of the beam

Note that actual flange width, rather than the transformed width used in allowable strength design, must be used here. Transformation using modular ratio is only related to linear elastic behavior and should never be used in strength analysis.

8.2.2.2.1.2 Rectangular Section

For sections with prestressing strand only and the depth of the equivalent rectangular stress block less than the flange thickness, the design flexural strength should be taken as:

$$\phi M_n = \phi A_s^* f_{su}^* d \left(1 - 0.6 \frac{\rho^* f_{su}^*}{f_c'}\right) \quad \text{[STD Eq. 9-13]}$$

where

ϕ = strength reduction factor = 1.0

M_n = nominal moment strength of a section

8.2.2.2.1.3 Flanged Section

For sections with prestressing strand only and the depth of the equivalent rectangular stress block greater than the flange thickness, the design flexural strength should be taken as:

$$\phi M_n = \phi \left\{ A_{sr} f_{su}^* d \left[1 - 0.6 \left(\frac{A_{sr} f_{su}^*}{b' d f_c'} \right) \right] + 0.85 f_c' (b - b')(t)(d - 0.5t) \right\}$$

[STD Eq. 9-14]

where

A_{sr} = steel area required to develop the compressive strength of the web of a flanged section

$= A_s^* - A_{sf}$ [STD Eq. 9-15]

A_{sf} = the steel area required to develop the ultimate compressive strength of the overhanging portions of the flange

$= 0.85 f_c'(b - b')t/f_{su}^*$ [STD Eq. 9-16]

b' = width of web for a flanged section

t = thickness of flange of a flanged section

[STD Art. 9.18.1]

8.2.2.2.2 Maximum Reinforcement Limit

Prestressed concrete members should be designed so that the steel is yielding as ultimate capacity is approached. For rectangular sections, the maximum reinforcement index to ensure steel yielding is:

$$\rho^* \frac{f_{su}^*}{f_c'} \leq 0.36\beta_1 \quad \text{[STD Eq. 9-20]}$$

For flanged sections, the maximum reinforcement is:

$$A_{sr} \frac{f_{su}^*}{b' d f_c'} \leq 0.36\beta_1 \quad \text{[STD Eq. 9-21]}$$

If the reinforcement index is greater than $0.36\beta_1$, the *Standard Specifications* still permit this condition provided that the flexural strength is assumed to be controlled by the capacity of the concrete compression part of the moment couple. For rectangular sections, the design flexural strength may be estimated as:

$$\phi M_n = \phi[(0.36\beta_1 - 0.08\beta_1^2) f_c' b d^2] \quad \text{[STD Eq. 9-22]}$$

For flanged sections, the design flexural strength should be taken as

$$\phi M_n = \phi[(0.36\beta_1 - 0.08\beta_1^2) f_c' b d^2 + 0.85 f_c'(b - b')t(d - 0.5t)] \quad \text{[STD Eq. 9-23]}$$

8.2.2.2.3
Minimum Reinforcement Limit

[STD Art. 9.18.2]

The total amount of prestressed and nonprestressed reinforcement should be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment, M^*_{cr}:

$\phi M_n \geq 1.2M^*_{cr}$

The cracking moment, M^*_{cr}, may be computed by:

$$M^*_{cr} = (f_r + f_{pe})S_{bc} - M_{d/nc}\left(\frac{S_{bc}}{S_b} - 1\right) \qquad \text{[STD Art. 9.18.2]}$$

where

M^*_{cr} = cracking moment

f_r = modulus of rupture [STD Art. 9.15.2.3]

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied load

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

S_{bc} = section modulus of composite section

S_b = section modulus of non-composite section

$M_{d/nc}$ = non-composite dead load moment at the section

Alternatively, the requirements of STD Article 9.18.2.1 may be waived if the area of prestressed and non-prestressed reinforcement provided is at least one-third greater than that required by analysis.

8.2.2.3
LRFD Specifications

8.2.2.3.1
Nominal Flexural Resistance

8.2.2.3.1.1
Required Parameters

The average stress in bonded prestressing steel is:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{[LRFD Eq. 5.7.3.1.1-1]}$$

Assuming rectangular section behavior, the neutral axis depth, c, is computed as:

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{0.85f'_c\beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{[LRFD Eq. 5.7.3.1.1-4]}$$

where

c = distance between the neutral axis and the compressive face

A_{ps} = area of prestressing steel

f_{pu} = specified tensile strength of prestressing steel

A_s = area of mild steel tension reinforcement

f_y = yield strength of tension reinforcement

A'_s = area of compression reinforcement

f'_y = yield strength of compression reinforcement

b = width of compression flange

k = factor related to type of strand:

$$= 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$ [LRFD Eq. 5.7.3.1.1-2]

= 0.28 for low-relaxation strand

f_{py} = yield strength of prestressing steel

d_p = distance from extreme compression fiber to the centroid of the prestressing strand

The depth of the compression block may be computed by, $a = \beta_1 c$. If the depth of the compression flange is less than c, as computed by LRFD Eq. 5.7.3.1.1-4, flanged section behavior must be used with c calculated by:

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y' - 0.85\beta_1 f_c'(b - b_w)h_f}{0.85 f_c' \beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1-3]

where

b_w = width of web

β_1 = ratio of depth of equivalent compression zone to depth (see Sect. 8.2.2.2.1.1)

8.2.2.3.1.2 Rectangular Sections

The nominal flexural capacity of a rectangular section is computed using the following equation according to LRFD Article 5.7.3.2.3:

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s' f_y'\left(d' - \frac{a}{2}\right)$$ (Eq. 8.2.2.3.1.2-1)

8.2.2.3.1.3 Flanged Sections

The nominal flexural capacity of a flanged section is computed using the following equation:

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s' f_y'\left(d' - \frac{a}{2}\right) + 0.85 f_c'(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

[LRFD Eq. 5.7.3.2.2-1]

where

f_{ps} = average stress in prestressing steel

a = depth of the equivalent stress block = $(\beta_1 c)$

A_s = area of nonprestressed tension reinforcement

d_s = distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement

A_s' = area of compression reinforcement

d' = distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement

Factored flexural resistance:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where ϕ = resistance factor = 1.00 [LRFD Art. 5.5.4.2.1]

[LRFD Art. 5.7.3.3.1]

8.2.2.3.2 Maximum Reinforcement Limit

The maximum amount of prestressed and nonprestressed reinforcement should be such that:

$$\frac{c}{d_e} \leq 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

where $d_e = \dfrac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$ [LRFD Eq. 5.7.3.3.1-2]

The commentary to the *LRFD Specifications* allows use of the same flexural strength equations as in the *Standard Specifications*, STD Eqs. 9-22 and 9-23, in cases where the maximum reinforcement limit is exceeded.

8.2.2.3.3 Minimum Reinforcement Limit

[LRFD Art. 5.7.3.3.2]

At any section, the amount of prestressed and nonprestressed reinforcement should be adequate to developed a factored flexural resistance, M_r, at least equal to the lesser of 1.2 times the cracking moment determined on the basis of elastic stress distribution, or 1.33 times the factored moment required by the applicable strength load combinations.

The *LRFD Specifications* formula for computing the cracking moment, M_{cr}, is the same as the equation given in the *Standard Specifications*.

$$M_{cr} = (f_r + f_{pe})S_{bc} - M_{d/nc}\left(\frac{S_{bc}}{S_b} - 1\right)$$

where, f_r = modulus of rupture = $0.24\sqrt{f'_c}$ [LRFD Art. 5.4.2.6]

Contrary to the *Standard Specifications*, the *LRFD Specifications* require that this criterion be met at all sections.

8.2.2.4 Flexural Strength Design Example

Consider the information given for the design example in Section 8.2.1.8. Use f'_c = 5,800 psi, and 46 strands as shown in **Figure 8.2.1.8-1**.

8.2.2.4.1 Design Requirement 1

Does the midspan section have adequate flexural strength to resist a factored moment, M_u = 4,900 kip-ft?

8.2.2.4.1.1 Standard Specifications

Assume the depth of the compression block, a, falls within the top flange.

Compute the average stress in the prestressing steel at ultimate load, f^*_{su}, using STD Eq. 9-17 with f'_s = 270 ksi, γ^* = 0.28, β_1 = 0.76, and:

$$\rho^* = \frac{A^*_s}{bd} = \frac{46(0.153)}{48.0(36.13)} = 0.00406$$

Therefore, $f^*_{su} = 270\left(1 - \dfrac{0.28(0.00406)(270)}{0.76(5.8)}\right) = 251$ ksi.

The compression block depth, $a = \dfrac{A^*_s f^*_{su}}{0.85 f'_c b} = \dfrac{46(0.153)(251)}{0.85(5.8)(48)} = 7.47$ in.

This is larger than the top flange thickness (5.50 in.). Therefore, the section behaves as a flanged section, with

b = 48.00 in.

b' = 2(5) = 10.00 in.

$t = 5.50$ in.

$A_{sf} = 0.85f'_c(b - b')/f^*_{su} = 4.110$ in.^{2}

Thus $A_{sr} = A^*_s - A_{sf} = 2.928$ in.^{2}

The corresponding reinforcement index, $A_{sr}f^*_{su}/(b'df'_c) = 0.35$. This exceeds the maximum steel index of $0.36\beta_1 = 0.36(0.76) = 0.27$. Thus, the section must be designed as an over-reinforced section. Using STD Eq. 9-23, $\phi M_n = 4{,}301$ ft-kips. Note that when reinforcement amounts greater than the maximum limit are used, their effectiveness is significantly diminished. Such design is rare as it is generally uneconomical.

The design capacity, $\phi M_n = 4{,}301$ ft-kips, is less than the required capacity of 4,900 ft-kips. The capacity may be improved by increasing the compressive strength of the deck concrete, f'_c. Increasing f'_c would reduce the reinforcement index and improve the lever arm distance between the center of the strand group and the center of the compression block. Use value of $f'_c = 8{,}500$ psi. This significantly larger value than 5,800 psi was chosen for the purpose of comparison of the results with *LRFD Specifications* and strain compatibility solutions given later. The values of β_1 and f^*_{su} become 0.65 and 255 ksi, respectively. The corresponding $a = 5.18$ in. is less than the top flange thickness of 5.5 in. Therefore, $\phi M_n = 5{,}009$ ft-kips is > 4,900 ft-kips, which is acceptable. The reinforcement index, $\rho^* f^*_{su}/f'_c = 0.12$, is much lower than the limit, $0.36\beta_1 = 0.23$.

It should be noted that it is not unusual to have flexural strength rather than service limit state stresses control the design of adjacent box beam bridges. This is an indication of the efficiency of this system as further illustrated by Examples 9.1 and 9.2.

8.2.2.4.1.2 LRFD Specifications

Using LRFD Eq. 5.7.3.1.1-3, the neutral axis depth, $c = 21.40$ in.

where

$A_{ps} = 7.038$ in.^{2}

$f_{pu} = 270$ ksi

$\beta_1 = 0.76$

$f'_c = 5.8$ ksi

$(b - b_w) = 38$ in.

$h_f = 5.50$ in.

$b_w = 10.00$ in.

$k = 2(1.04 - 0.9) = 0.28$

$d_p = 36.13$ in.

The maximum reinforcement limit, $c/d_p = 0.59$, which is greater than the maximum value of 0.42. The section is over-reinforced and LRFD Eq. C5.7.3.3.1-2, which is identical to STD Eq. 9-23, must be used. The resulting ϕM_n would therefore be identical to that obtained earlier.

If f'_c is increased to 8.5 ksi, the neutral axis depth = 14.89 in., and $c/d_p = 0.41$ which is slightly less than the maximum value. Thus, the section is under-reinforced and LRFD Eq. 5.7.3.2.2-1 may be used.

Using LRFD Eq. 5.7.3.2.2-1 with $a = \beta_1 c = 9.68$ in., $f_{ps} = 270[1 - 0.28(14.89)/36.13] = 239$ ksi with $\phi M_n = 4{,}557$ ft-kips. This value is less than the capacity needed. Note that the values of a, f_{ps}, and ϕM_n are considerably different from the corresponding *Standard Specifications* results. These differences will be discussed further in the example in Section 8.2.2.6.

8.2.2.4.2 Design Requirement 2

Does the beam have adequate flexural capacity, ϕM_u, in the end regions?

Assume that the strand development length = 7 ft for bonded strands and 14 ft for debonded strands. Determine the envelope of the flexural capacity along the span length. Assume 12 of the 46 strands are debonded, six in each row, see **Figures 8.2.1.8.3-2 and 8.2.1.8-1**.

Calculate the capacity for the 34 bonded strands when fully developed at 7 ft from the end of the beam. Assume the depth of the compression block, a, falls within the top flange.

Compute the average stress in the prestressing steel at ultimate load, f^*_{su} using STD Eq. 9-17, with $f'_s = 270$ ksi, $f'_c = 8{,}500$ psi, $\gamma^* = 0.28$, $\beta_1 = 0.65$, and

$$\rho^* = \frac{A^*_s}{bd} = \frac{34(0.153)}{48.0(36.13)} = 0.00300.$$

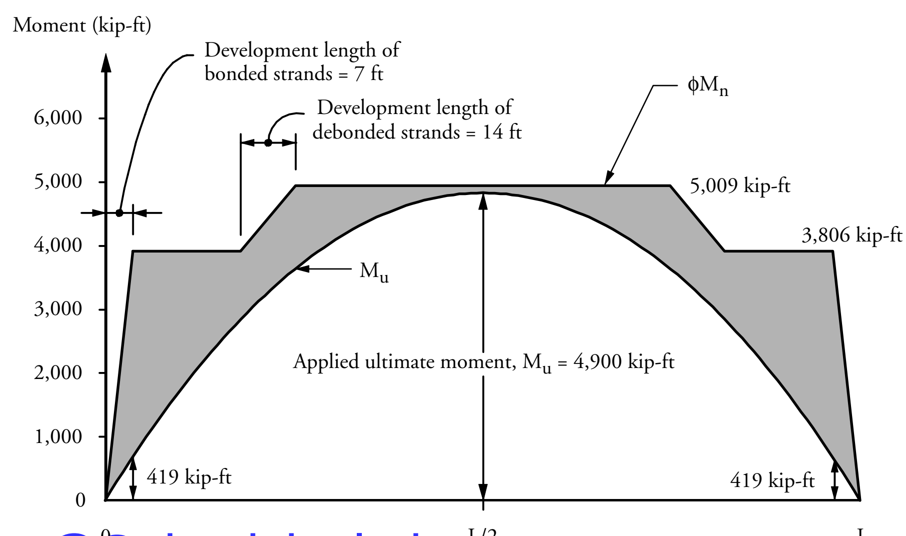
Therefore, $f^*_{su} = 258$ ksi.

The compression block depth, $a = \dfrac{A^*_s f^*_{su}}{0.85 f'_c b} = \dfrac{34(0.153)(258)}{0.85(8.5)(48)} = 3.87$ in.

This is less than the flange thickness (5.50 in.). Therefore, the section is considered a rectangular section, and the corresponding ϕM_n (STD Eq. 9-13) is 3,806 ft-kips. The ultimate moment capacity diagram is shown in **Figure 8.2.2.4.2-1**. Note that even though 14 ft is a very conservative estimate of development length for debonded strands, it has little impact on the flexural strength of this member.

Calculations using the *LRFD Specifications* would give the same results.

Figure 8.2.2.4.2-1
Ultimate Moment Capacity for the Beam



8.2.2.5 Strain Compatibility Approach

The strain compatibility approach is based on three well accepted fundamental assumptions:

- plane sections remain plane after bending
- compatibility of strains, i.e., full bond between steel and concrete at the section being considered
- equilibrium of forces within a section

In addition, the standard assumptions of concrete stresses at ultimate flexure being represented by a rectangular stress block is adopted, with the intensity = $0.85f'_c$ and depth $a = \beta_1 c$ where c is neutral axis depth and β_1 is a coefficient defined in Section 8.2.2.2.1.1. The steel stress-strain relationship may be defined using any representative formula or graph. For 270 ksi, low-relaxation strands:

$$f_{si} = \varepsilon_{si}[887 + 27{,}613/\{1 + (112.4\varepsilon_{si})^{7.36}\}^{1/7.36}] \leq 270 \text{ ksi} \qquad \text{(Eq. 8.2.2.5-1)}$$

where f_{si} is the stress in a given layer of reinforcement whose strain is ε_{si} and ε_{si} is the strain in a given layer of reinforcement.

The above "power formula" is based on a lower bound curve fitting of actual stress-strain relationships and on satisfaction of the minimum ASTM limits (Devalapura and Tadros, 1992). Alternatively, the graph given in Fig. 2.11-1, which is taken from the *PCI Design Handbook* may be used.

For mild reinforcement, an elastic-plastic stress-strain relationship is assumed:

$$f_{si} = E_s\varepsilon_{si} \leq f_y \qquad \text{(Eq. 8.2.2.5-2)}$$

In order to maintain equilibrium, the sum of the tension and compression forces must equal zero. The sum of the moments of these forces about any horizontal axis is equal to the moment acting on the section for the assumed conditions. The process is iterative due to the non-linearity of the stress-strain relationship of the prestressing steel. The following 6 steps, adapted from Skogman, et al (1988), demonstrate the application of this approach:

Step 1: Assume a neutral axis depth c and substitute in Eq (8.2.2.5-3) to obtain the corresponding strain in each steel layer "i". A layer "i" is defined here as a group of bars or tendons with the same stress-strain properties (prestressing strand or mild reinforcement), the same effective prestress, and which can be assumed to have a combined area with a single centroid. The strain in each layer of steel can be estimated using the equation:

$$\varepsilon_{si} = 0.003\left(\frac{d_i}{c} - 1\right) + \left(\frac{f_{se}}{E_s}\right)_i \qquad \text{(Eq. 8.2.2.5-3)}$$

where

d_i = depth of steel layer from extreme compression fiber

f_{se} = effective prestress. For partially tensioned tendons or for non-tensioned reinforcing bars, f_{se} may be assumed = f_{pi} – 25,000 psi where f_{pi} is initial tension (assumed zero for non-tensioned reinforcing bars).

Step 2: Use Eq. (8.2.2.5-1) or Eq. (8.2.2.5-2) to estimate the stress in each steel layer.

Step 3: Use equilibrium of forces to check assumed neutral axis depth:

$$\Sigma A_{si} f_{si} + \Sigma F_{cj} = 0 \qquad \text{(Eq. 8.2.2.5-4)}$$

where i refers to steel "layer" and j refers to concrete components within the compression block.

Each concrete component would have a force:

$$F_{cj} = 0.85 f'_{cj} A_{cj} \qquad \text{(Eq. 8.2.2.5-5)}$$

For example, the cross-section shown in **Figure 8.2.2.5-1** has three steel "layers:"

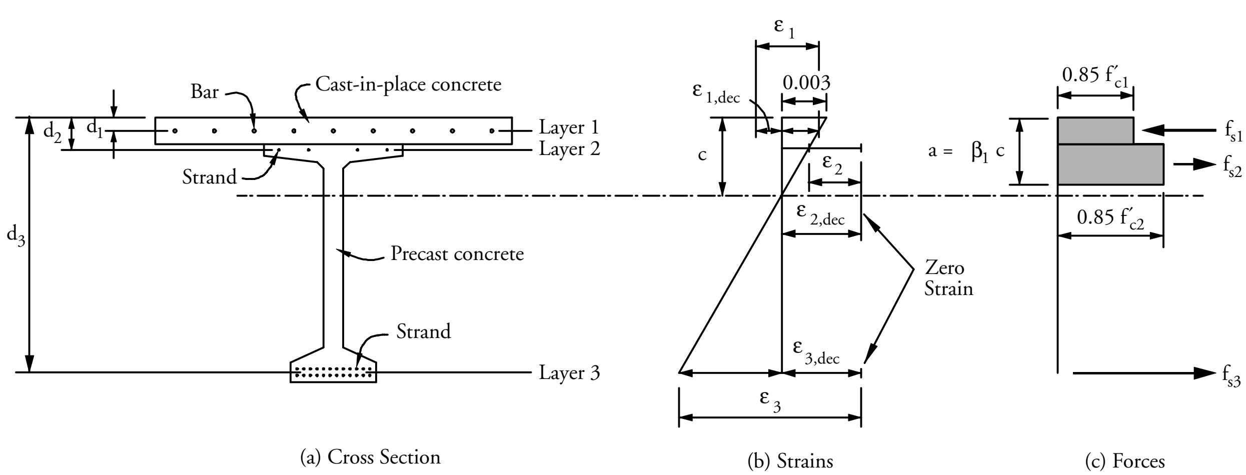
- bottom flange group of strands
- top flange group of strands
- group of deck reinforcing bars

It has three concrete components:

- the cast-in-place deck
- the overhanging portions of the top beam flange
- the portion of the beam web within the compression block depth

The flange overhangs may also be subdivided into rectangular and triangular components, although the additional calculations will not significantly affect the accuracy in this case.

Figure 8.2.2.5-1
Flexural Strength Relationships for Strain Compatibility Analysis



In composite construction, the stress block factor, β_1, may be different for different components of the compression block. In this case an average β_1 may be assumed as follows:

$$\beta_{1ave} = \sum_j (f'_c A_c \beta_1)_j / \sum_j (f'_c A_c)_j \qquad \text{(Eq. 8.2.2.5-6)}$$

Step 4: Revise "c" and repeat Steps 1-3, until Eq 8.2.2.5-4 is satisfied.

Step 5: Calculate the nominal flexural capacity by summing moments of all forces about any horizontal axis. If the top fiber is used,

$$M_n = \sum_i A_{si} f_{si} d_i + \sum_j F_{cj} d_j \qquad \text{(Eq. 8.2.2.5-7)}$$

Step 6: Calculate the design moment capacity ϕM_n. Both the *Standard Specifications* and the *LRFD Specifications* state that ϕ may be taken equal to 1.0 for precast concrete flexural members. Both Specifications are silent relative to ϕ values when the tension reinforcement exceeds the maximum reinforcement limit. However, the ACI Code, 318-95 (1995) addresses this situation in the adoption of the Unified Design Provisions which were developed by Mast (1992). Drawing from the ACI approach, it is recommended that ϕ be reduced linearly from $\phi = 1.0$ for net tensile strain of 0.005 to $\phi = 0.7$ for net tensile strain of 0.002 in the "extreme" tension steel. The "extreme" tension steel is the "layer" of strands or bars nearest the tension face of the member at the cross-section being considered. The net tensile strain is the tensile strain at nominal strength exclusive of strains due to effective prestress which may be obtained using the first term in Eq. (8.2.2.5-3). Therefore, use the following expression to determine the value of ϕ:

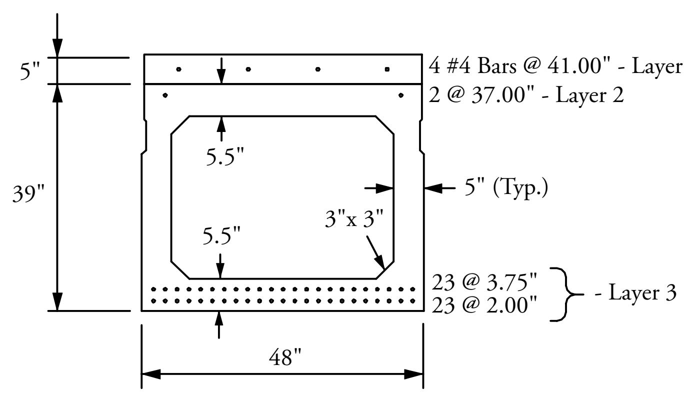
$$0.7 \le \phi = 0.50 + 0.30 \left(\frac{d_{ext}}{c} - 1\right) \le 1.0 \qquad \text{(Eq. 8.2.2.5-8)}$$

where d_{ext} is d_i of the extreme "layer" of reinforcement.

8.2.2.6 Design Example – Strain Compatibility

Consider the precast concrete cross-section shown in **Figure 8.2.1.8-1**. In addition to the 46 strands in the bottom of the section, two strands initially tensioned at 15,000 psi are provided near the top as shown in **Figure 8.2.2.6-1**. The section is made composite with a 5.00-in.-thick cast-in-place slab whose f'_c is 3,000 psi and reinforced with 4 #4 Grade 60 bars. The modulus of elasticity of the pretensioning strands is 28,500 ksi and of the reinforcing bars is 29,000 ksi.

Figure 8.2.2.6-1
Cross-Section of the Box Beam



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8.2.2.6.1 Part 1 – Flexural Capacity

Determine the flexural capacity and the corresponding steel stresses.

The steps given in Section 8.2.2.5 may be followed iteratively until a solution is achieved. For brevity, only the last iteration will be shown here.

Step 1: Neutral axis depth = c = 12.31 in.

For layer 1: d_1 = 3 in., f_{se1} = –25 ksi, E_s = 29,000 ksi, and ε_{s1} from Eq. (8.2.2.5-3), ε_{s1} = –0.00313 (positive sign indicates tension). Similarly, ε_{s2} = –0.00164 and ε_{s3} = +0.0131.

Step 2: Steel stresses, Eq. (8.2.2.5-2) yields f_{s1} = –60 ksi, and Eq. (8.2.2.5-1) yields

f_{s2} = –46.9 ksi and f_{s3} = +255.4 ksi.

Step 3: Check equilibrium of forces using Eq. (8.2.2.5-4).

$$\sum_i A_{si} f_{si} = 4(0.2)(-60) + 2(0.153)(-46.9) + 46(0.153)(255.4) = -48 - 14 + 1{,}798$$
$$= 1{,}736 \text{ kips}$$

This must be equal and opposite to $\sum_j F_{cj}$.

The coefficient β_1 must first be averaged over the two concrete materials since the depth of the compression block is greater than the depth of the cast-in-place topping. β_1 of the 5,800 psi precast concrete is 0.76 and of the 3,000 psi cast-in-place concrete is 0.85. Using an initial β_{1ave} of 0.80, a = β_1c = 9.85 in. Substituting into Eq. (8.2.2.5-6):

$$\beta_{1ave} = [(3)(5)(48)(0.85) + (5.8)(4.85)(48)(0.76)]/[(3)(5)(48) + (5.8)(4.85)(48)] = 0.79$$

Thus, revised a = 9.72 in., c = 9.72/0.79 = 12.30 in. and

$$\sum_j F_{cj} = -[0.85(3)(5)(48) + 0.85(5.8)(9.72 - 5)(48)] = -[612 + 1{,}117] = -1{,}729 \text{ kips}$$

$$\sum_i A_{si} f_{si} + \sum_j F_{cj} = 1{,}736 - 1{,}729 = 7 \text{ kips} \cong 0.0 \qquad \text{O.K.}$$

Step 4: No revision of c or further iteration is needed.

Step 5: Taking moments about the top fiber:

$$M_n = \sum_i A_{si} f_{si} d_i + \sum_j F_{cj} d_j$$
$$= [-48(3) - 14(7) + 1{,}798(41.13) - 612(2.5) - 1{,}117(7.36)]/12 = 5{,}333 \text{ ft-kips}$$

The approximate formulas of the AASHTO Specifications are not suitable for a flanged cross-section in which the compression zone extends below the flange. They would only be valid if the cast-in-place topping and if the top two strands did not exist, as explained in Section 8.2.2.1.

8.2.2.6.2 Part 2 – Comparative Results

Table 8.2.2.6.2-1 compares results of the strain compatibility approach for f'_c = 5.8 ksi and 8.5 ksi with the results of flexural design of the example of Section 8.2.2.4 using both the *Standard Specifications* and the *LRFD Specifications.*

Note that the comparisons are for an untopped box beam and not the beam with deck slab in the preceeding section.

Table 8.2.2.6.2-1 Flexural Capacity Prediction by Various Methods

	f'_c = 5.8 ksi			f'_c = 8.5 ksi		
	STD Spec.	LRFD Spec.	Strain Comp.	STD Spec.	LRFD Spec.	Strain Comp.
Neutral axis depth, c, in.	9.83	21.40	16.39	7.97	14.89	8.12
Compression block depth, a, in.	7.47	16.26	12.46	5.18	9.68	5.28
Steel stress at ultimate flexure, ksi	251	225	240	255	239	260
ϕM_n, ft-kips	4,301	4,301	4,505	5,009	4,557	5,106
	95%	95%	100%	98%	89%	100%

The table clearly shows the advantage of using the accurate strain compatibility approach. For $f'_c = 5.8$ ksi, the approximate approach utilizes an equation that is not even a function of the steel provided. For $f'_c = 8.5$ ksi, the *Standard Specifications* give results that are much closer to the strain compatibility approach than the results of the *LRFD Specifications*. Part of the reason is the estimation of the neutral axis depth which is excessive, resulting in a low steel stress and a correspondingly low ϕM_n.

Some designers compound the errors resulting from the approximate procedures by lumping all pretensioning steel in a section into a single location for the purpose of establishing the effective depth. This is incorrect. Only the reinforcement near the tension face of the member should be considered in determining the steel stress using STD Eq. 9-17 and LRFD Eq. 5.7.3.1.1-1.

8.2.3 Design of Negative Moment Regions for Members Made Continuous for Live Loads

8.2.3.1 Strength Design

Where continuity at interior supports under live load and composite dead loads is desired at interior support, negative moment reinforcement may be provided within the cast-in-place deck slab. The negative moment section is designed as a reinforced concrete section using the compressive strength of the beam concrete regardless of the strength of the cast-in-place concrete. [STD Art. 9.7.2.3.2]

Use the width of the bottom flange as the width of the concrete compressive stress block, b. Determine the required steel in the deck to resist the total factored negative moment, assuming that the compression block is uniform:

$$R_n = \frac{M_u}{\phi b d^2} \qquad \text{(Eq. 8.2.3.1-1)}$$

where

R_n = strength design factor

M_u = total factored negative moment

d = distance from extreme compression fiber to centroid of the negative moment reinforcing for precast beam bridges made continuous

ϕ = strength reduction factor = 0.9 [LRFD Art. 5.5.4.2.1]

This value is consistent with cast-in-place concrete construction, rather than $\phi = 1.0$ for precast members. This is reasonable as the main reinforcement is placed in the field.

Estimate the required area of steel using the following equation:

$$\rho = \frac{1}{m}\left[1 - \sqrt{\left(1 - \frac{2mR_n}{f_y}\right)}\right] \qquad \text{(Eq. 8.2.3.1-2)}$$

where

$$m = \frac{f_y}{0.85\, f'_c} \qquad \text{(Eq. 8.2.3.1-3)}$$

f_y = yield stress of nonprestressed conventional reinforcement

f'_c = concrete compressive strength at 28 days for the beam

The steel area, $A_s = \rho bd$. Alternatively, A_s may be determined using one of several approximate methods. For example,

$$A_s \cong \frac{M_u}{0.9\phi f_y d} \qquad \text{(Eq. 8.2.3.1-4)}$$

The above equation implies that the lever arm between the tension and compression stress resultants is approximately 0.9d.

The design moment strength, ϕM_n, may be computed by:

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad \text{[STD Eq. 8-16]}$$

where

a = depth of compression block = $\dfrac{A_s f_y}{0.85\, f'_c b}$ [STD Eq. 8-17]

A_s = area of nonprestressed tension reinforcement

If the depth of the compression block is larger than the thickness of the bottom flange, flanged section analysis similar to that used for the positive moment section would be required.

8.2.3.2
Reinforcement Limits - Standard Specifications

• Maximum reinforcement [STD Art. 8.16.3.1]

$$\rho_b = \frac{0.85\,\beta_1 f'_c}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) \qquad \text{[STD Eq. 8-18]}$$

$$\rho_{max} = 0.75\rho_b \qquad \text{[STD Art. 8.16.3.1.1]}$$

• Minimum reinforcement [STD Art. 8.17.1]

The total amount of nonprestressed reinforcement must be adequate to develop an ultimate moment (ϕM_n) at the critical section at least 1.2 times the cracking moment. The cracking moment may be calculated as for a prestressed concrete section except $f_{pe} = 0$.

$$\phi M_n \geq 1.2 M_{cr}$$

The specifications allow this requirement to be waived if the amount of reinforcement is at least one-third greater than that required by analysis.

8.2.3.3
Reinforcement Limits - LRFD Specifications

• Maximum reinforcement [LRFD Art. 5.7.3.3.1]

The maximum amount of prestressed (if any) and nonprestressed reinforcement resisting negative moment should be such that:

$$\frac{c}{d_e} \leq 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

This is the same requirement as that given in Section 8.2.2.3.2, as the *LRFD Specifications* intend to provide for unified treatment of prestressed and conventionally reinforced concrete.

• Minimum reinforcement [LRFD Art. 5.7.3.3.2].

Requirements for minimum reinforcement in the negative moment region are the same as in the positive moment region given in Section 8.2.2.3.3.

8.2.3.4 Serviceability

The deck slab is not prestressed and therefore is not subject to the tensile stress limits specified under service load conditions for prestressed concrete members. Distribution of the flexural reinforcement in the deck slab should be checked in order to control cracking. The best crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension [STD 8.17.2.1]. Several bars at moderate spacing are more effective in controlling cracking than one or two larger bars of equivalent area. Crack width is controlled by:

- steel stress
- thickness of concrete cover
- area of concrete surrounding each individual reinforcing bar
- surface condition of the reinforcing bars

The *Standard Specifications* and the *LRFD Specifications* use the same approach for crack control. The tensile stress in the mild reinforcement at service loads, f_s or f_{sa}, should not exceed:

$$f_s = \frac{z}{\left(d_c A\right)^{1/3}} \le 0.6\, f_y \qquad \text{[STD Eq. 8.61]}$$

$$f_{sa} = \frac{Z}{\left(d_c A\right)^{1/3}} \le 0.6\, f_y \qquad \text{[LRFD Eq. 5.7.3.4-1]}$$

where

d_c = depth of concrete from extreme tension fiber to center of bar

A = area of concrete having the same centroid as the tensile reinforcement and bounded by the surfaces of the cross-section and a straight line parallel to the neutral axis, divided by the number of bars

Z = z = crack width parameter

In situations where the concrete surface is subject to severe exposure conditions, a maximum value of Z = 130 kip/in. is used in design. For moderate exposure conditions, a maximum value of Z = 170 kip/in. is used.

8.2.3.5 Fatigue in Deck Reinforcement

The longitudinal deck reinforcement in the negative moment zone over the piers must be checked for fatigue. This portion of the deck is likely to crack due to service loads and the steel stress range may be significant. The stress range in reinforcement is limited by:

$$f_f = 21 - 0.33 f_{min} + 8\left(\frac{r}{h}\right) \qquad \text{[STD Eq. 8-60, LRFD Eq. 5.5.3.2-1]}$$

where

f_f = stress range

f_{min} = algebraic minimum stress level, positive if tension, negative if compression

r/h = ratio of base radius to height of rolled-on transverse deformations; if the actual value is not known, 0.3 may be used.

For stress calculation according to the *LRFD Specifications*, the special fatigue truck loading must be introduced to the continuous structure.

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

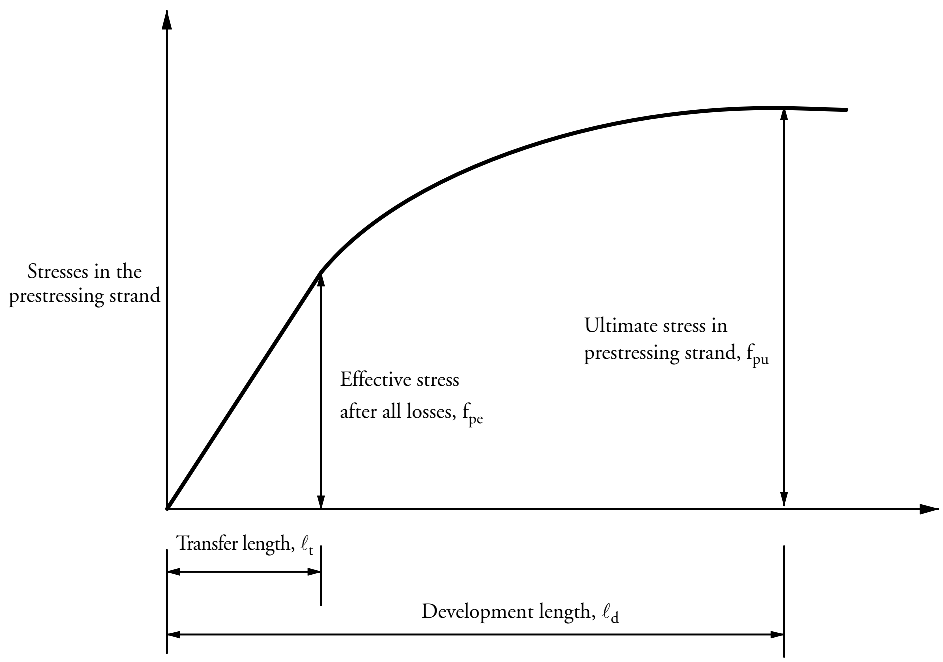
Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.3 STRAND TRANSFER AND DEVELOPMENT LENGTHS

The transfer length, ℓ_t, is the length of strand over which the prestress force in pretensioned members is transferred to the concrete by bond and friction. The development length, ℓ_d, is the length of strand required to develop the stress in the strand corresponding to the full flexural strength of the member. The transfer length is included as part of the development length. These two parameters are used differently in design as discussed below. **Figure 8.3-1** illustrates the relationship between the transfer and development lengths, and the strand stress.

Figure 8.3-1
Strand Transfer and Development Lengths



Much research has been conducted in recent years on methods of predicting ℓ_t and ℓ_d. Prediction formulas have been developed with no clear consensus among researchers. It should be emphasized, however, that the impact of variability of the transfer length on design of bridge beams is very small, and is limited to the 2 to 3 ft at the end of a member. The impact of variability of development length on bridge beams is also small. An over-estimation of ℓ_d will not significantly increase the cost of beams. However, ℓ_d may become a significant design parameter for some prestressed concrete members, such as deck panels, which have very short spans, and piles, which may have their largest bending moment at the pile/cap interface.

8.3.1 Strand Transfer Length

8.3.1.1 Impact on Design

Transfer length is the bonded length of strand required to transfer the prestress force in the strand to the surrounding concrete in a pretensioned member. At any section which falls within the transfer length, the prestress force should be reduced in proportion to its distance from the end of the member. Specifically, within the transfer length, the stress in the strand is assumed to vary linearly from zero at the end of member, or the point where the strand is bonded if debonding is used, to the full effective prestress force at the end of the transfer length.

Overestimation of transfer length is generally conservative for shear design but may be unconservative in flexural design of the end regions. Shear strength is reduced within the transfer length due to the reduced precompression in the concrete. On the other hand, the reduced prestress force in the transfer length zone protects the end of the beam from excessive tensile stresses. Such excessive stresses may require that the end of the beam be reinforced with additional bonded steel reinforcement near the top fibers.

8.3.1.2
Specifications

STD Article 9.20.2.4 states that the transfer length be equal to 50 times the diameter of the strand while LRFD Article 5.8.2.3 requires a transfer length of 60 times the diameter of the strand.

8.3.1.3
Factors Affecting Transfer Length

The transfer length for prestressing strand is affected by many parameters. Some of the most important are:

- type of prestressing strand
- strand diameter
- strand stress level
- surface condition of strand (i.e., clean, oiled, rusted, epoxy-coated, etc.)
- concrete strength
- type of loading (i.e., static, repeated or impact)
- method of strand detensioning (i.e., gradual or sudden)
- confining reinforcement around strand
- consolidation and consistency of concrete around strand
- concrete cover around the strand
- strand spacing
- time-dependent effects
- vertical location in concrete (top versus bottom locations)

8.3.1.4
Research Results

The Federal Highway Administration (FHWA), in 1996, approved the use of 1/2-in.-diameter strands at a center-to-center spacing of 1.75 in. and 0.6-in.-diameter strands at a spacing of 2 in. These spacings are less than the 4 strand diameters previously required in the *Standard Specifications*. This decision was based on studies, which demonstrate that the transfer length for the more closely spaced strands remains conservatively estimated using the relationship found in the *Standard Specifications*. The use of 0.6-in. strand in pretensioned bridge members, (which had earlier been prohibited by FHWA), and especially its use at the reduced spacing of 2 in., is expected to substantially increase the use of 0.6-in.-diameter strands. With only a 20% increase in diameter from 0.5 to 0.6-in., the prestress force per strand is increased by 40%. Using 0.6-in. strands at a 2 in. spacing, it is possible now to increase the amount of pretensioning force by up to 40% and still preserve the same prestress eccentricity. This will dramatically improve the load carrying capacity of a given cross-section. For more information on this research, see Section 8.3.2.4.

8.3.1.5
Recommendations

The current recommendations of the *Standard* and the *LRFD Specifications* to use 50 or 60 strand diameters are adequate for design of typical structures. For unusually short span products or for strands with marginal surface conditions, this transfer length may not be adequate. For high strength concrete, the provisions in both specifications may overestimate the transfer length.

8.3.1.6 End Zone Reinforcement

The *Standard Specifications* and the *LRFD Specifications* have essentially the same requirements for providing vertical reinforcement near the ends of pretensioned members. Both require that an area of mild reinforcing steel be provided to resist 4% of the prestressing force. The stress in the mild reinforcement resisting this force, is limited to 20,000 psi. This reinforcement is usually provided as stirrups and must be placed within a distance equal to h/4 from the beam end, where h is the overall depth of the pretensioned element.

The origin of this requirement is not well known. It appears to be reasonable for modest levels of prestressing. However, in recent years, larger prestressing forces are being used with high performance concrete. This is especially true for sections such as the NU Bulb-Tee beams*, where up to 58 strands can be placed in the bottom flange. When this large number of strands is used with relatively shallow beams, such as the 43.3 in. deep NU-1100, the specifications require that as much as 3.6 in.^{2} of reinforcement be placed within a distance of 9.0 in. from the end of the beam. It is very difficult to satisfy this requirement and provide adequate clearance to place and consolidate the concrete.

Until this issue is investigated further, designers should be aware that the most critical time is the time of prestress release. Areas of end zone reinforcement that are less than the required areas and that have been consistently used in actual production without objectionable cracking at member end should be acceptable.

* a family of metric-dimensioned beams developed at the University of Nebraska

8.3.2 Strand Development Length

8.3.2.1 Impact on Design

Strand development length is the length required for bond to develop the strand tension at ultimate flexure. As shown in Section 8.2.2, this tension is generally lower than the specified ultimate strength of the strand and may be as low as its yield point. For bridge beams, the development length is insignificant unless the bridge beams are less than about 24 ft in length, or unless the beams are subjected to large bending moments near their ends. The development length becomes significant in deck panels used as stay-in-place forms.

8.3.2.2 Standard Specifications

Equations for development length were based on tests performed on normal weight concrete. The *Standard Specifications* require that the strand be embedded beyond the critical section for a distance of at least the development length, which can be computed by the equation:

$$\ell_d = \left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \qquad \text{[STD Eq. 9-42]}$$

where

f_{ps} = average stress in prestressing steel at ultimate load

f_{pe} = effective stress in the prestressing steel after losses

d_b = nominal strand diameter

The *Standard Specifications* require that debonded strands be designed for twice the development length of bonded strands. This requirement appears to be unnecessarily conservative. However, it should be followed until research justifies its modification.

When the critical section occurs at a location where the full development length is not available because of proximity to the end of the member, the available stress in the strand must be reduced. The *Standard Specifications* provide the following equation for computing the available stress for such a condition:

$$f_{su}^{*} = \frac{\ell_x}{d_b} + \frac{2}{3} f_{pe} \qquad \text{[STD Eq. 9-19]}$$

where ℓ_x is the distance from the critical section to the end of the prestressing strand and f_{pe} is in ksi.

8.3.2.3
LRFD Specifications

The equation for development length in the *LRFD Specifications* is very similar to the *Standard Specifications*. However, based on work by Cousins et al. (1986) indicating that the existing equation was unconservative, the Federal Highway Administration (FHWA) imposed a 1.6 multiplier on the AASHTO equation. As a result, the *LRFD Specifications* include a K factor in the equation as follows:

$$\ell_d \geq K\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b \qquad \text{[LRFD Eq. 5.11.4.2-1]}$$

where

K = 1.6 for bonded strands in precast, prestressed beams

In lieu of the first equation, an alternative equation may be used for pretensioned beams as follows:

$$\ell_d \geq \frac{4 f_{pbt} d}{f_c'} + \frac{6.4\left(f_{ps} - f_{pe}\right)\left(d_b\right)}{f_c'} + 10 \qquad \text{[LRFD Eq. 5.11.4.2-2]}$$

where

f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in LRFD Table 5.9.3-1

This equation was developed by the Federal Highway Administration (Lane, 1998) and is intended to reflect the effect of higher strength concrete up to 10,000 psi. For strands with 12 in. or more of concrete cast below the strands, a 1.3 multiplier must be applied to LRFD Eq. 5.11.4.2-2 to compensate for the lower bond strengths that can exist in “top” reinforcement.

When a portion of the strand is debonded or “shielded” and where tension exists in the precompressed tensile zone under service loads, the development length must be determined using LRFD Eq. 5.11.4.2-1 with a value of K = 2.0.

8.3.2.4
Factors Affecting Development Length

The development length of the strand depends on a number of factors in addition to the factors already stated for the transfer length. These factors include:

- The difference between the stress in the prestressing steel at the ultimate member strength and the effective prestress after all losses. As indicated earlier, the stress at ultimate strength may range from the yield point to ultimate strength of the strand.
- Use of bonded or debonded prestressing steel.
- Flexure-shear interaction.

8.3.2.5
Bond Studies

Numerous studies have been conducted on both 1/2 in. and 0.6-in.-diameter strand, often with conflicting conclusions. A summary of research on 1/2-in. strand was reported by Buckner (1994 and 1995). He proposed modification of the ℓ_d formula which takes into account the effect of strand stress at ultimate flexure. He suggests that a strand with a stress at ultimate flexure close to the ultimate strength of the steel should have a development length almost twice as long as a strand with a stress at ultimate flexure equal to the yield point of the steel.

Shahawy (2001) suggests that for members with depths greater than 24 in., the STD Eq. 9-42 results in an unacceptably low development length while LRFD Eq. 5.11.4.2-1 (with K = 1.6) and LRFD Eq. 5.11.4.2-2 yield conservative results. For members with depths less than 24 in., the K multiplier in LRFD Eq. 5.11.4.2-1 is not warranted and LRFD Eq. 5.11.4.2-2 is extremely conservative. Shahawy also concludes that flexure-shear interaction has a significant effect on the development length of prestressing strands and should be incorporated into the design equations. Research findings remain under study by AASHTO, FHWA, PCI and others.

8.3.2.6
Recommendations

It is recommended that the formula given in the *LRFD Specifications* be used unless an improved formula emerges. If the formula in the *Standard Specifications* is used, a multiplier of 1.6 should be considered for member depths greater than 24 in. Even though the factor of 2 applied to debonded strands may be too conservative, it is not expected to have significant impact on bridge beam design.

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.4 SHEAR

The design and analysis of precast, prestressed concrete bridge members for vertical shear is presented in this section. Design and analysis for combined torsion and shear are not included. The applicable sections in the *Standard Specifications* and the *LRFD Specifications* are covered in detail. In addition, a simplified procedure given in the AASHTO *Standard Specifications for Highway Bridges*, 1979 Interim Revisions, is presented because it is permitted as an alternative to the procedure given in the *Standard Specifications*.

Generally, the design of vertical web reinforcement is one of the last steps performed in the design of a prestressed concrete bridge beam. The precast member cross-section, beam spacing, span geometry and flexural reinforcement have already been established. Unlike flexural design, for which conditions at both service and factored load are evaluated, shear design is only evaluated for factored loads (strength limit state).

Shear design is essentially based on the truss analogy which has been used for concrete design since the early 1900s. In the truss analogy, a concrete member resists loads by a truss composed of concrete "compression struts" and steel "tension ties." However, while this model is an effective tool for estimating the ultimate shear capacity of concrete members, it may be overly conservative in calculating the cracking shear capacity when compared to test results.

Therefore, both the *Standard* and *LRFD Specifications* provide three shear design methods which attempt to provide more realistic estimates of shear capacity of a concrete member by adding a concrete contribution to the basic truss analogy. And thus, the nominal shear strength, V_n, is considered to be a combination of the concrete contribution, V_c, and web reinforcement contribution, V_s. In members with harped strand, the vertical component of the prestressing force, V_p, is also considered to resist the factored shear force. The nominal shear resistance can, therefore, be expressed as:

$$V_n = V_c + V_s + V_p$$

The factored shear force at the section under investigation must be less than or equal to the nominal shear resistance reduced by a resistance factor, ϕ:

$$V_u \leq \phi V_n = \phi(V_c + V_s + V_p)$$

While both the *Standard Specifications* and the *LRFD Specifications* are based on the truss analogy as discussed above, there is a significant difference in the way in which it is used. The *Standard Specifications* shear design provisions may be interpreted to use a truss model with the inclination of the diagonal compression struts fixed at 45° in all cases. This limitation is appropriate for reinforced concrete members but may be too conservative in prestressed concrete applications. The *LRFD Specifications* use variable inclination truss analogy.

To ensure ductile behavior, the designer must properly detail the web reinforcement to provide adequate development and to satisfy maximum and minimum limits on the quantity and spacing of the reinforcement. Each of the shear design procedures is discussed in detail in the following sections.

8.4.1 Standard Specifications

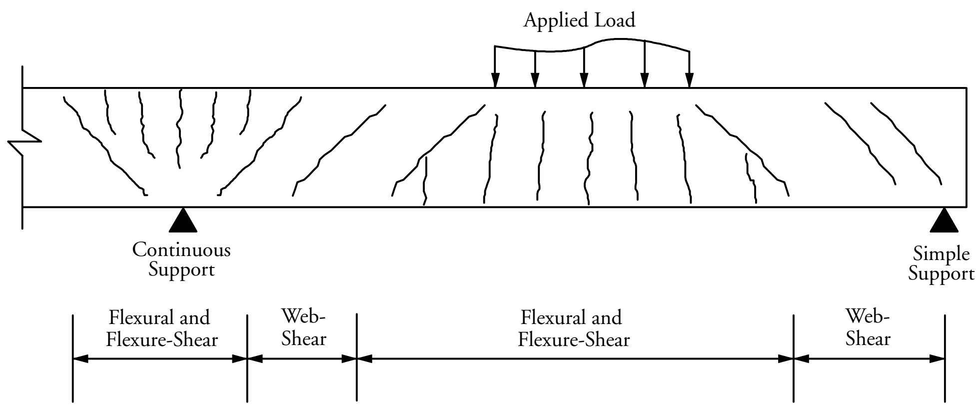
Procedures for shear design of precast, prestressed concrete members are given in STD Article 9.20. Members must be designed so that:

$$V_u \le \phi(V_c + V_s) \qquad \text{[STD Eq. 9-26]}$$

The concrete contribution, V_c, is taken as the shear required to produce shear cracking. Two types of shear cracks have been identified: flexure-shear and web-shear, as illustrated in **Figure 8.4.1-1**.

Flexure-shear cracks dominate the behavior of the portion of the beam where high bending stresses coincide with significant shear stresses. Web-shear cracks form in regions of high shear and small bending stresses such as near the support of a simply-supported beam. The shear that produces these two types of cracking are V_{ci} and V_{cw} respectively. Therefore, V_c is taken as the lesser of V_{ci} and V_{cw}. This procedure was developed from research conducted at the University of Illinois from the late 1950s through the mid-1960s.

Figure 8.4.1-1
Types of Cracking in Concrete Beams



Procedures for computing the shear capacity associated with the two types of shear cracking and the contribution of web reinforcement are presented below.

8.4.1.1 Flexure-Shear Strength, V_{ci}

A flexure-shear crack is initiated by a flexure crack forming at a distance d/2 from the section being considered. As the shear increases, the flexure crack inclines and becomes a shear crack with a horizontal projection equal to the distance d.

Let M and V be the moment and the shear force at the section of investigation. Set the moment at d/2 from this section equal to the cracking moment, M_{cr}. The value of M_{cr} can be calculated using the expression:

$$M_{cr} = M - V\left(\frac{d}{2}\right) \qquad \text{[STD Eq. 9-27a]}$$

Dividing both sides of the equation by V and solving for V, the shear required to produce the flexure crack at d/2 from the section of investigation can be expressed as:

$$V = \frac{M_{cr}}{\left(\frac{M}{V}\right) - \left(\frac{d}{2}\right)} \qquad \text{[STD Eq. 9-27b]}$$

It has been found that an additional increment of shear equal to $1.0\sqrt{f'_c}\,b'd$ causes shear failures of beams without web reinforcement. Therefore, it was recommended that the nominal shear strength carried by the concrete where flexure-shear cracking occurred can be computed using the equation:

$$V_n = 1.0\sqrt{f'_c}\,b'd + V_d + \frac{M_{cr}}{\frac{M}{V} - \frac{d}{2}} \qquad \text{[STD Eq. 9-27c]}$$

The dead load effects were kept separate for simplicity.

This equation has been modified to its current form, shown below, by conservatively dropping the d/2 term in the denominator of the third term and using a constant of 0.6 in the first term to provide a lower bound of the test data, resulting in the equation:

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad \text{[STD Eq. 9-27]}$$

where

V_d = shear force at the section of investigation due to the unfactored dead load

M_{max} = maximum factored moment at the section due to externally applied loads

V_i = factored shear force at the section that occurs simultaneously with M_{max}
For convenience, some designers use the maximum shear force envelope to determine V_i in the $(V_i M_{cr}/M_{max})$ term. This approximation should be acceptable in most designs.

M_{cr} = moment due to external load required to crack the concrete at the critical section. Cracking is assumed to occur at a tensile stress of $6\sqrt{f'_c}$ in the extreme fiber.

Setting the stress in the extreme tension fiber equal to the cracking stress, $6\sqrt{f'_c}$, the following expression can be derived:

$$6\sqrt{f'_c} = -f_{pe} + f_d + \frac{M_{cr} y_t}{I} \qquad \text{(Eq. 8.4.1.1-1)}$$

Solving for M_{cr},

$$M_{cr} = \frac{I}{y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right) \qquad \text{[STD Eq. 9-28]}$$

The term f_{pe} and f_d are the stresses at the extreme tension fiber due to the effective prestress forces only, after all loses, and due to the total unfactored dead load, respectively. The ratio I/y_t is the section modulus at the extreme tension fiber of the section resisting externally applied loads.

The AASHTO specifications state that V_{ci} need not be taken less than $1.7\sqrt{f'_c}\,b'd$ and that d need not be taken less than 0.8h, where h is the height of the section. It can be seen that at locations where the maximum moment approaches zero, such as at an inflection point, STD Eq. 9-27 approaches infinity. The code does not place an upper limit on V_{ci}. However, as V_{ci} becomes large, V_c will be controlled by V_{cw} as determined in Section 8.4.1.2.

8.4.1.2 Web-Shear Strength, V_{cw}

Assuming the web is uncracked, elastic analysis principles can be applied to predict web-shear cracking. Using Mohr's circle, the shear force required to cause a principal tensile stress equal to the splitting, or tensile strength of the concrete at the centroid of the section can be computed using the following expression:

$$V_{cw} = \frac{I\,b'}{Q} f'_t \left(1 + \frac{f_{pc}}{f'_t}\right)^{0.5} \qquad \text{(Eq. 8.4.1.2-1)}$$

where

I = moment of inertia of the section

b' = web width

Q = area moment of the part of the cross-section above or below the centroid

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at the centroid of the cross-section resisting externally applied loads

f'_t = strength of the concrete in diagonal tension

Typical values for f'_t range between 4 and 5 times the square root of the compressive strength for field-cured concrete.

Based on this analysis, the orientation of the web-shear crack can be predicted using the following expression:

$$\theta = \frac{1}{2}\tan^{-1}\left(\frac{2Q}{Ib'}\frac{V_{cw}}{f_{pc}}\right) \qquad \text{(Eq. 8.4.1.2-2)}$$

If the value of f_{pc} is zero, as is the case for reinforced concrete beams, the equation predicts a 45° crack. The presence of a longitudinal prestress force flattens this angle. Values of θ for typical prestressed beams range from 20° to 40°.

Eq. (8.4.1.2-1) was derived for non-composite beams. The equation for composite sections becomes slightly more complicated. Therefore, STD Eq. 9-29 was developed to approximate the elastic solution for both non-composite and composite sections.

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b'd + V_p \qquad \text{[STD Eq. 9-29]}$$

In the cases where harped strand is used, the term V_p is the vertical component of the prestress force. Where strands are straight, V_p is zero. The effective depth, d, is the distance from the extreme compression fiber to the centroid of the prestressing strand or 0.8 times the total depth, whichever is greater. The concrete strength, f'_c, is taken as the concrete strength of the web of the precast beam. For composite sections, the value of f_{pc} is evaluated at the centroid of the composite section unless this point is above the junction of web and flange. For this situation, the value of f_{pc} is computed at the junction. The term f_{pc} includes the effect of the prestress force after losses and the stresses due to any loads applied to the member as a non-composite section. If the critical section is within the transfer length of the prestressing strand which is defined to be 50 strand diameters, a reduced effective prestress force must be used to compute f_{pc} (see Sect. 8.3.1).

8.4.1.3 Web Reinforcement Contribution, V_s

Web reinforcement in the form of vertical stirrups is typically required to increase the strength of the beam to resist the factored shear force. The web reinforcement contribution, V_s, is computed using:

$$V_s = \frac{A_v f_{sy} d}{s} \qquad \text{[STD Eq. 9-30]}$$

where

A_v = area of web reinforcement

d = distance from extreme compression fiber to centroid of the pretensioning force

f_{sy} = yield strength of nonprestressed conventional web reinforcement in tension

s = longitudinal spacing of web reinforcement

This equation assumes a 45° crack with a horizontal projection equal to the effective depth, d. Assuming a constant stirrup spacing, the number of stirrups crossing the crack is d/s. Therefore, the vertical shear is resisted by d/s stirrups, each contributing a force equal to $A_v f_{sy}$. The effective depth d need not be taken less than 0.8 times the height of the section. Over-reinforcement of the web, which can lead to brittle web-crushing shear failure, is prevented by requiring V_s to be less than or equal to:

$$8\sqrt{f'_c}\,b'd \qquad \text{[STD Art. 9.20.3.1]}$$

The design yield strength of web reinforcement is limited to 60,000 psi.
[STD Art. 9.20.3.4]

8.4.1.3.1 Minimum Spacing Requirements

The spacing of web reinforcement shall not exceed 0.75h or 24 inches. If V_s is greater than:

$$4\sqrt{f'_c}\,b'd$$

then the maximum spacing is reduced by one-half to 0.375h or 12 in.

[STD Art. 9.20.3.2]

8.4.1.3.2 Minimum Shear Reinforcement

STD Article 9.20.3.3 requires that a minimum area of web reinforcement be provided to prevent sudden failure of the beam due to the formation of shear cracks. This minimum area is computed by the expression:

$$A_v \geq \frac{50b's}{f_{sy}} \qquad \text{[STD Eq. 9-31]}$$

This equation can also be expressed by combining STD Eqs. 9-30 and 9-31 and limiting V_s to:

$$V_s = 50b'd \qquad \text{(Eq. 8.4.1.3.2-1)}$$

The constant 50 carries the units of psi.

For voided slab beams where $V_u \leq \phi V_c/2$, shear reinforcement may be omitted completely. [STD Art. 9.20.1.1]

8.4.1.4 Application of Standard Specifications to Continuous Spans

A common application of precast concrete beam construction is to make the beams continuous for loads applied to the composite structure. This is accomplished by connecting the ends of beams with a diaphragm and making the composite deck continuous over the interior supports. Live loads and composite dead loads such as barrier rails, sidewalks, and future wearing surface are applied to the continuous structure.

This method of construction eliminates unnecessary joints over the bents and takes advantage of continuity. The continuity of the superstructure is especially important for resisting and distributing forces due to extreme events, such as earthquakes or vessel impact.

The application of V_{ci} and V_{cw} is not recommended for this type of construction in areas near interior supports. This is because those procedures were based on extrapolation of empirically deduced equations using simple-span testing. The designer is directed to use one of two approaches for such regions:

a) Provisions of the 1979 Interim Revisions (see Sect. 8.4.2).

b) Design the quarter-spans closest to interior supports as conventionally reinforced members.

However, it is recognized that the V_{ci} and V_{cw} equations are commonly used in practice for continuous spans with no known problems.

8.4.2 1979 Interim Specifications

In a footnote to STD Article 9.20, web reinforcement may be designed using the method presented in the 1979 Interim Revisions to the *Standard Specifications.*

In the 1979 Interim Revisions, the following equation is given for computing the concrete contribution to shear strength:

$$V_c = 0.06f'_c b'jd \le 180b'jd \quad \text{(Eq. 8.4.2-1)}$$

where j = ratio of the distance between the centroids of the compression force and tension steel, and effective depth, d

Note that concrete strengths above 3,000 psi do not increase the value of V_c due to the maximum limit.

The required shear contribution of web reinforcement is computed using the expression:

$$\phi V_s = V_u - \phi(V_c + V_p) \quad \text{(Eq. 8.4.2-2)}$$

where V_p is the vertical component of the prestress force.

The area of web reinforcement is computed using the expression given in Article 1.16.13 of the 1979 Interim Revisions:

$$A_v = \frac{[V_u - \phi(V_c + V_p)]s}{2\phi f_{sy} jd} \quad \text{(Eq. 8.4.2-3)}$$

where

V_u = factored dead and live load shear

ϕ = resistance factor for shear = 0.9

Solving for V_u, the following equation is obtained:

$$V_u = \phi\left(V_c + V_p + 2\frac{A_v f_{sy} jd}{s}\right) \quad \text{(Eq. 8.4.2-4)}$$

The *"Tentative Recommendations for Prestressed Concrete"* published in 1958 by the ACI-ASCE Joint Committee 323, serves as the basis for the shear design provision of the 1979 Interim Revisions. The factor of 2 in the shear reinforcement term represents the assumed benefit of prestressed concrete. For typical sections, a value of 0.9 can be conservatively used to estimate j. The spacing of the web reinforcement is limited to 3/4 the section height. For simply supported spans, it is recommended that shear be designed in the middle half of the beam. The required stirrup spacing computed at the quarter-point is used from the quarter-point to the support. For continuous spans, web reinforcement must be evaluated for the full length of interior spans and 3/4 of the exterior span. This approach assumes that regions of high shear and high flexural stresses are most critical for web reinforcement design.

A minimum amount of web reinforcement is required by these provision equal to:

$$A_v = \frac{100b's}{f_{sy}} \qquad \text{(Eq. 8.4.2-5)}$$

The coefficient 100 in the equation above was further reduced to the traditional 50 psi in the 1980 Interim Revisions. PCI supported this change (see Design Supplement to Short Span Bridges, 1985).

8.4.3 LRFD Specifications

There are two methods of shear design presented in the *LRFD Specifications*. The most general method is the strut-and-tie model. This model can be applied to any design situation, including members with irregular cross-sections or discontinuities. It is also used to design a member for all load effects, not just shear. This method is discussed in Section 8.12.

The method used for typical shear design is the sectional design model, or modified compression field theory developed by Collins, Mitchell and others. This method is based on the variable angle truss model in which the inclination of the diagonal compression field is allowed to vary. This differs from the approach used in the *Standard Specifications* in which this angle is always assumed to be 45°. This is especially significant for prestressed concrete members where the inclination is typically 20° to 40° degrees due to the effect of the prestressing force.

This model also differs from the shear design method found in the *Standard Specifications* because the concrete contribution, V_c, is attributed to tension being carried across the compression diagonals. This contribution has been determined experimentally and has been related to the strain in the tension side of the member. In general, the higher the strain in the tension side at ultimate, the wider the shear cracks, and in turn the smaller the concrete contribution.

It is significant to note that the concrete contribution, V_c, is what sets the sectional design model apart from the strut-and-tie model which is discussed in Section 8.12. Both models are based on the variable-angle truss analogy in which a concrete member resists loads by a truss composed of concrete "compression struts" and steel "tension ties." While this model is an effective tool in estimating the shear capacity of concrete members, it has been found to underestimate V_c when compared to test results. Therefore, the sectional design method can be expected to give higher capacities than the strut-and-tie model.

Chapter 5 of the *LRFD Specifications* is applicable to conventionally reinforced concrete, and fully-prestressed concrete, as well as partially-prestressed concrete design. "Flexural Regions" as defined in LRFD Article 5.8.1.1 are discussed below. Design of regions near discontinuities are covered in Section 8.12.

8.4.3.1 Shear Design Provisions

The *LRFD Specifications*, Article 5.8.3, introduces the sectional design model. Subsections 1 and 2 describe the applicable geometry required to use this technique to design web reinforcement.

8.4.3.1.1 Nominal Shear Resistance

The nominal resistance is taken the lesser of:

$V_n = V_c + V_s + V_p$, or, [LRFD Eq. 5.8.3.3-1]

$V_n = 0.25f'_c b_v d_v + V_p$ [LRFD Eq. 5.8.3.3-2]

where

b_v = effective web width

d_v = effective shear depth

LRFD Eq. 5.8.3.3-2 represents an upper limit of V_n to assure that the concrete in the web will not crush prior to yield of the transverse reinforcement.

8.4.3.1.2 Concrete Contribution, V_c

The *LRFD Specifications* defines the concrete contribution as the nominal shear resistance provided by the tensile stresses in the concrete. This resistance is computed using the following equation:

$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v$ [LRFD Eq. 5.8.3.3-3]

The units used in the *LRFD Specifications* are kips and inches. The factor 0.0316 is equal to

$$\frac{1}{\sqrt{1{,}000}}$$

which converts the expression from psi to ksi units for the concrete compressive strength, f'_c.

8.4.3.1.3 Web Reinforcement Contribution, V_s

The contribution of the web reinforcement is given by the general equation:

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$ [LRFD Eq. 5.8.3.3-4]

where the angles, θ and α, represent the inclination of the diagonal compressive stresses measured from the horizontal beam axis and the angle of the web reinforcement relative to the horizontal beam axis, respectively.

For cases of vertical web reinforcement, the expression for V_s simplifies to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$ [LRFD Eq. C5.8.3.3-1]

Transverse shear reinforcement should be provided when:

$V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

When the reaction introduces compression into the end of the member, LRFD Article 5.8.3.2 specifies that the critical section for shear is the larger of $0.5d_v\cot\theta$, or d_v, measured from the face of the support, where d_v and θ are measured at the critical section for shear.

8.4.3.1.4 Values of β and θ

To determine the nominal resistance, the design engineer must determine β and θ from LRFD Article 5.8.3.4. For mildly reinforced concrete sections, the values of β and θ are 2 and 45° respectively. These will produce results similar to the *Standard Specifications*. However, for prestressed concrete, the engineer can take advantage of the precompression and use lower angles of θ, which optimizes the web reinforcement.

8.4.3.2 Design Procedure

To design the member for shear, the designer first determines the factored shear due to applied loads at the section under investigation. The critical section is located at the larger of d_v or $0.5d_v\cot\theta$ from the face of support. The effective shear depth, d_v, is taken as the distance between the resultants of the tensile and compressive forces due to flexure. It need not be taken less than the greater of $0.9d_e$ or $0.72h$. When strands are straight and compression stays in the top flange, d_v is easily calculated as d_e–a/2. When determining d_e, only the steel on the tension side should be considered. However, determination of d_v can get complicated with harped strands as d_v depends on the location of the critical section, which in turn is a function of d_v.

For T-beam analysis, the resultant of the compression force is not at a/2. Computer programs may be used to perform these calculations, but some simplifications are warranted for hand calculations. For example, the critical section can initially be assumed to be at 0.72h, and the value of d_v can be determined at that location. If this value of d_v is larger than 0.72h or $0.9d_e$, then the designer may elect to choose a new location using the value d_v just determined. When calculating d_v, it is convenient to use the depth of the compression block, a, at midspan without introducing significant error.

The shear contribution from harped strand V_p, is then computed.

Next, the average factored shear stress is calculated using:

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \qquad \text{[LRFD Eq. 5.8.2.9-1)}$$

The quantity v_u/f'_c is then computed, and a value of θ is assumed. For prestressed members, a good initial estimate for θ is 25°.

For sections containing at least the minimum transverse reinforcement specified in LRFD Article 5.8.2.5, the longitudinal strain is calculated at middepth of the member using:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5\left(V_u - V_p\right)\cot\theta - A_{ps}f_{po}}{2\left(E_sA_s + E_pA_{ps}\right)} \qquad \text{[LRFD Eq. 5.8.3.4.2-1]}$$

The initial value of ε_x should be taken not greater than 0.001.

The specifications indicate that the area of prestressing steel, A_{ps}, must account for lack of development near the ends of prestressed beams. Any mild reinforcement or strand in the compression zone of the member, which is taken as one-half of the overall depth (h/2), should be neglected when computing A_s and A_{ps} for use in this calculation. This is very important when evaluating members with harped strand, since near the end of typical beams, harped strands are near the top of the beam. Because of this, it is recommended that the straight and harped strands be considered separately in the analysis. It is the physical location of each strand that is important and not the centroid of the group.

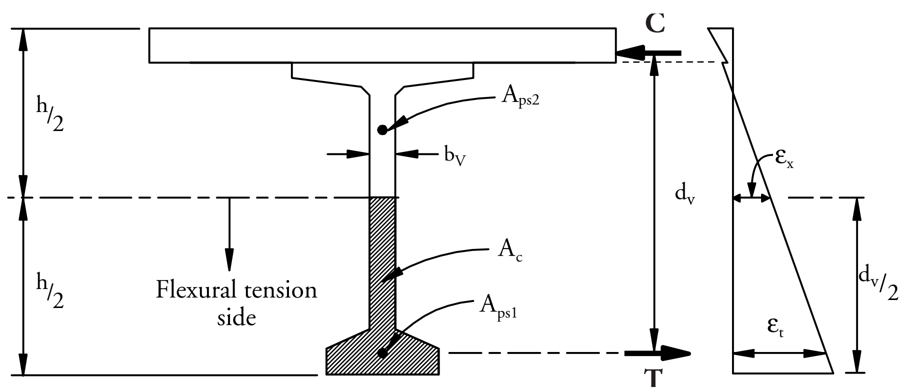
The variable, f_{po}, represents the modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete. For usual levels of prestressing, the *LRFD Specifications* suggests a value of $0.7f_{pu}$ is appropriate for both pretensioned and post-tensioned members. However, for pretensioned members, LRFD Article C5.8.3.4.2 indicates that f_{po} can be taken as the stress in the strands when the concrete is cast around them, which is the jacking stress, f_{pj} which equals $0.75f_{pu}$. Therefore, it is recommended that for usual pretensioned beams with low-relaxation strands, the value of f_{po} should be taken as $0.75f_{pu}$. Within the transfer length, f_{po} should be increased linearly from zero to its full value along the transfer length.

If the longitudinal strain in the tensile reinforcement (ε_x) calculated using LRFD Eq. 5.8.3.4.2-1 is negative, ε_x should be recomputed using the following equation:

$$\varepsilon_x = \frac{\left(\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}\right)}{2\left(E_cA_c + E_sA_s + E_pA_{ps}\right)} \qquad \text{[LRFD Eq. 5.8.3.4.2-3]}$$

where A_c represents the area of concrete on the flexural tension side of the member as shown in **Figure 8.4.3.2-1**. LRFD Table 5.8.3.4.2-1 is then entered with the values of v_u/f'_c and ε_x. The table is set up so that interpolation is not necessary and is not recommended for hand calculations. However, linear interpolation between values given in the table is acceptable and is often performed by computer programs.

Figure 8.4.3.2-1 Illustration of Shear Parameters



The value of θ determined using v_u/f'_c and ε_x is compared to the assumed value of θ. If the values match, V_c is calculated using Eq. (5.8.3.3-3) with the value of β from the table. If they do not match, the value of θ taken from the table is used for another iteration. Of the quantities computed thus far, only ε_x will change with a new value for θ, so the effort required for additional iterations is minor.

The 2003 Interim Revisions introduced a simplification in the commentary to avoid iteration in determining ε_x and θ. In the numerator of the LRFD equations for ε_x, the term $0.5\cot\theta$ can be assumed equal to 1.0, which is the same as assuming $\theta = 26.6°$. When ε_x is determined using this simplification to the equations, the values of β and θ read from LRFD tables can be used directly without returning to the strain equations to calculate a revised value of ε_x. The flow chart in LRFD Figure C5.8.3.4.2-5 that was revised with the 2003 Interim Revisions is based on this simplified assumption, although the flow chart previously contained in Figure C5.8.3.4.2-5 still accurately reflects the design procedure without the simplification.

As noted above, LRFD Eq. 5.8.3.4.2-1 is used to calculate ε_x for sections containing adequate transverse reinforcement, which is usually the case for prestressed members. For these sections, the longitudinal strain is calculated near mid-depth of the member where the strain is less than maximum to account for the redistribution of shear stresses. Sections with less than the minimum transverse reinforcement have less capacity for redistribution of shear stresses, and the longitudinal strain is calculated at the location in the web that is subject to the highest longitudinal tensile strain. For these sections LRFD Eq. 5.8.3.4.2-2 and LRFD Table 5.8.3.4.2-2 are used.

After V_c has been computed, V_s is calculated using LRFD Eq. 5.8.3.3-4. The quantity of shear reinforcement is then calculated using LRFD Eq. C5.8.3.3.-1 with the value of θ from the table.

After determining the amount of shear reinforcement needed, the designer should check the maximum spacing allowed by the specifications as given in LRFD Article 5.8.2.7. Also, the amount of shear reinforcement should be checked to ensure that it is equal to or larger than the minimum value required by the specifications, which is:

$$A_v = 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \qquad \text{[LRFD Eq. 5.8.2.5-1]}$$

8.4.3.3 Longitudinal Reinforcement Requirement

The longitudinal (flexural) reinforcement must also be able to resist additional force due to shear, i.e., the horizontal component of the diagonal compression field. The tensile capacity of the reinforcement on the flexural tension side of the member, taking into account any lack of full development of that reinforcement, must be greater than or equal to the force T, calculated as:

$$T = \left[\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta\right] \qquad \text{[LRFD Eq. 5.8.3.5-1]}$$

The tensile capacity of the reinforcement can be determined by using the appropriate values for $A_s f_y + A_{ps} f_{ps}$. V_s is given by LRFD Eq. 5.8.3.3-4 except that V_s may not be greater than V_u/θ.

Satisfying this equation is very important for prestressed concrete beams, especially near non-continuous supports where a substantial portion of the prestressing strands are harped. Harped strands are not effective in contributing to this longitudinal reinforcement requirement since they are above midheight of the member.

The *LRFD Specifications* require that this criterion also be checked at the face of the bearing. At this section, which usually lies within the transfer length of the strands, the effective prestressing force in the strands is not fully developed. Thus, the term f_{ps} should be calculated as a portion of the effective prestress force based on linear variation starting from zero at the end of the beam to full effective prestress at the transfer length. The designer should not be confused by the term f_{ps}, which generally refers to the prestress force at Strength Limit State, because the strands at this section do not have enough development length to provide such level of prestress. If the strands are well anchored at the end of the member, by embedment in a diaphragm or by use of a mechanical device, the stress in the strands, f_{ps}, can be considered to equal the stress in the strands at Strength Limit State.

8.4.4 Comparison of Shear Design Methods

A recent study by Ma, et al (1997) compared the design of two example bridges using various shear design methods. The purpose of the study was to determine the significance of varying the method of shear design on the overall cost of precast concrete beams. Two cross-sections were considered in the comparison: the box beam of Examples 9.1 and 9.2 in Chapter 9 and the I-beam shown in **Figure 8.4.4-1**. It consists of two spans made continuous for superimposed loads using a rigid diaphragm and conventional deck reinforcement over the pier. Note that the box beam used in Examples 9.1 and 9.2 is a simple span with conventional concrete strength, while the I-beam is a continuous span with higher concrete strength. The shear design methods of the *Standard Specifications, LRFD Specifications*, and 1979 AASHTO Interim Revisions were used. As shown in the preceding sections, the methods give identical values for V_p. The basic formulation for computing V_s is the same, but there are differences in the assumption of the angle θ between the diagonal compression strut and the horizontal axis of the member. Methods for computing V_c, however, are very different and the resulting values are also significantly different.

Figures 8.4.4-2 and **8.4.4-3** show the different values of V_c and A_v/s along the first span of the two-span I-beam bridge. Although, V_c varies significantly from one method to the other, the overall cost of the beam caused by the variation of the total amount of shear reinforcement is affected by a maximum of 1.1 percent for the I-beam and 0.4 percent for the box beam, as shown in **Figure 8.4.4-4**. These cost estimates were based on an average total fabricated beam cost of \$600/yd^3 and reinforcing bar cost of \$0.50/lb. Based on this limited study, it would seem reasonable to conclude that complex shear design may not be warranted for box or I-beam members.

Figure 8.4.4-1
Elevation and Cross-Section of an Example Bridge Beam

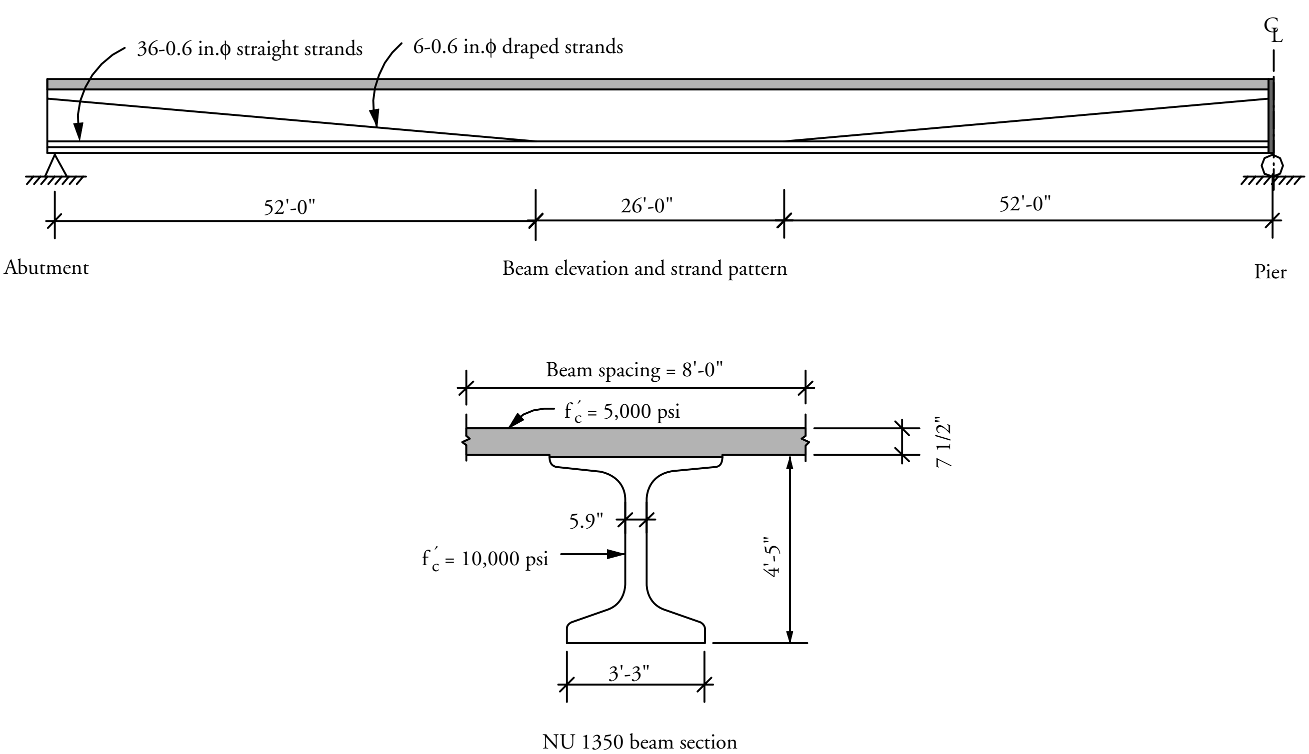


Figure 8.4.4.-2
Calculated V_c Values for Different Design Methods

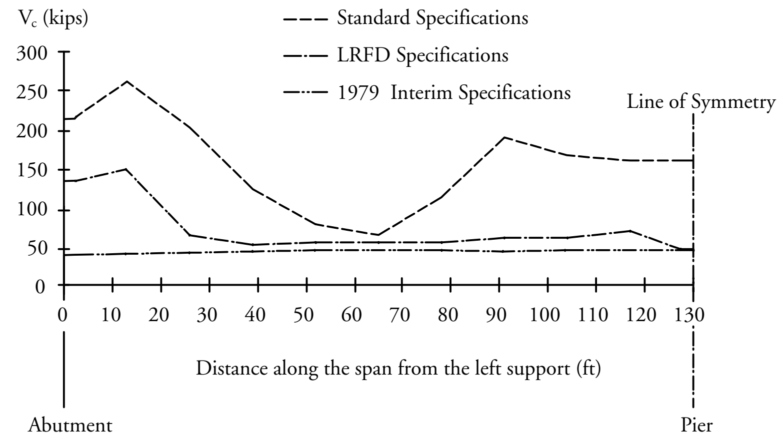


Figure 8.4.4.-3
Shear Reinforcement and Costs for Different Design Methods

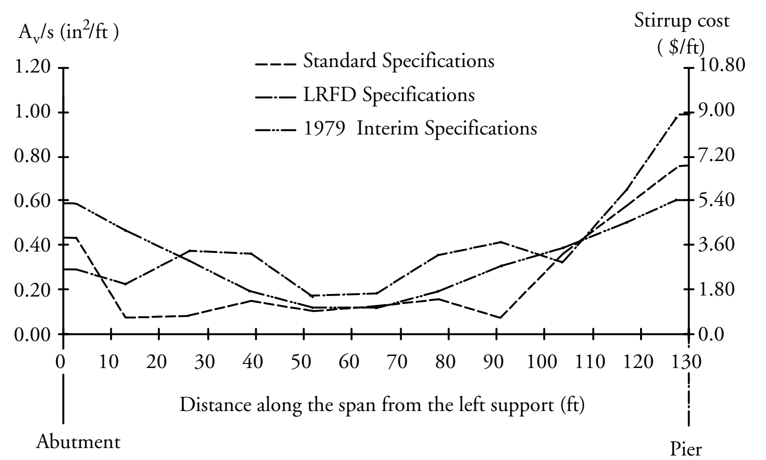
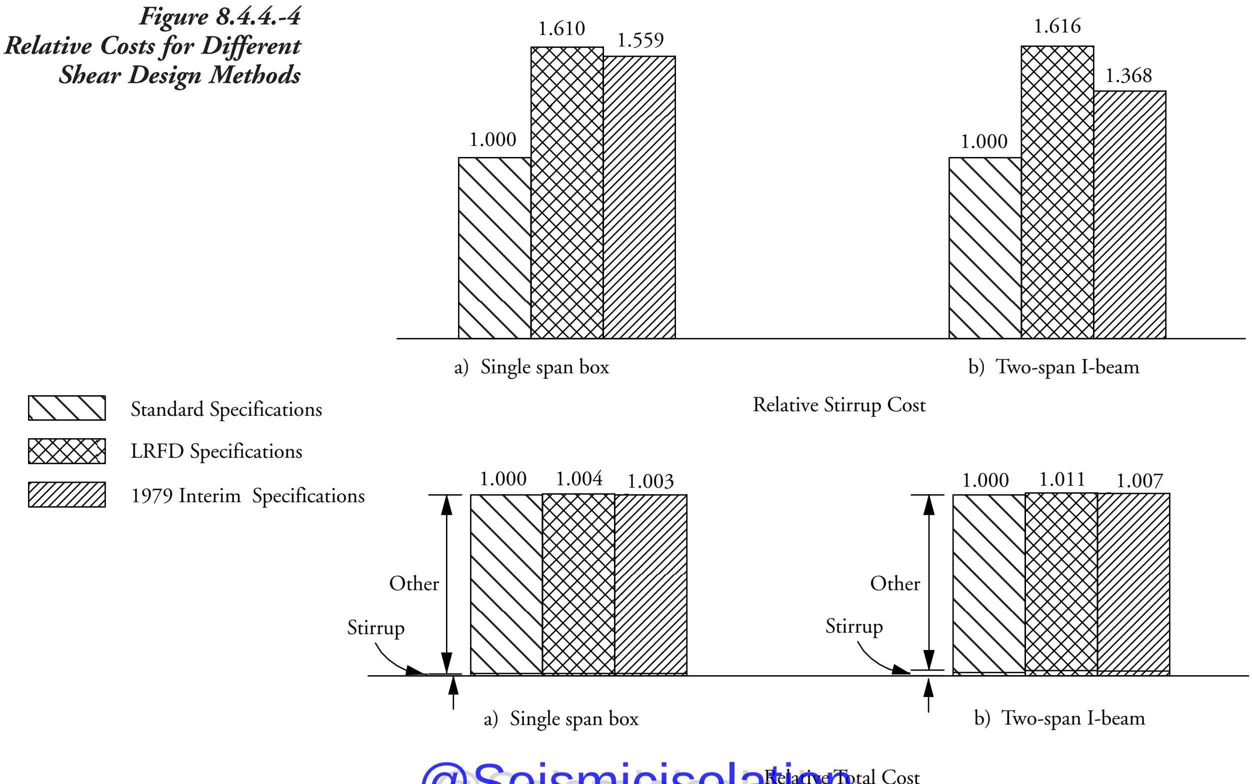


Figure 8.4.4.-4
Relative Costs for Different Shear Design Methods



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NOTATION

DESIGN THEORY AND PROCEDURE

A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.5 HORIZONTAL INTERFACE SHEAR

8.5.1 Theory

Cast-in-place concrete decks designed to act compositely with precast concrete beams must be able to resist the horizontal shearing forces at the interface between the two elements. The basic strength equation for the design of the interface between the deck and beam is:

$V_u \leq \phi V_{nh}$

where [STD Eq. 9-31a]

V_u = factored shear force acting on the interface

ϕ = strength reduction factor

V_{nh} = nominal shear capacity of the interface

Design is carried out at various locations along the span, similar to vertical shear design.

Theoretical calculation of the shearing force acting on the interface at a given section is not simple because the section does not behave as a linear elastic material near ultimate capacity. If it did, the shear stress, horizontal or vertical, at any fiber in a cross-section would be calculated from the familiar equation:

$$v_h = \frac{VQ}{Ib} \qquad \text{(Eq. 8.5.1-1)}$$

where

V = vertical shear force at the section

I = moment of inertia

b = section width at the fiber being considered

Q = first moment of the area above (or below) the fiber being considered

However, at ultimate conditions, the material is no longer elastic and the concrete may be cracked at the section being considered. Further, the composite cross-section consists of two different types of concrete with different properties. Therefore, application of the above equation to design at ultimate, without modification, would yield questionable results.

Loov and Patnaik (1994) determined that the above equation may yield adequate results if both the cracked section moment of inertia and area moment of a transformed composite section are used. The section would be transformed using the slab-to-beam modular ratio used in flexural design by the allowable stress method. However, this approach is still too complicated. It confuses the calculations at two limit states: service and ultimate.

Kamel (1996) used equilibrium of forces to show that:

$$v_h = V/(jd)b_v \qquad \text{(Eq. 8.5.1-2)}$$

where

V = factored vertical shear at the section in question
d = effective depth of the member
jd = distance between the tension and compression resultant stresses in the section. This is the same distance as d_v used in the *LRFD Specifications*.
b_v = section width at the interface between the precast and the cast-in-place concrete. It is important to understand that b_v is not the web width.

Another important issue is which loads should be used to calculate V_u at a section. Neither the *Standard Specifications* nor the *LRFD Specifications* give guidance in this regard. While most designers would use all loads to compute V_u, a strong case can be made for excluding the self-weight of the precast concrete member, and the weight of the deck since they are present prior to composite action taking effect. Some designers and agencies, such as the Illinois Department of Transportation, use only the composite loads, which include the superimposed dead loads (barriers, wearing surface, etc.) and the live loads. Fortunately, the amount of reinforcement required, even with consideration of all loads, is reasonable in practical applications.

To determine the shear capacity of the interface, the *LRFD Specifications* uses a form of the well-established shear friction theory, while the *Standard Specifications* uses an empirical approach based on several investigations, for example, Birkeland and Birkeland (1966), Mast (1968), Kriz and Raths (1965) and Hofbeck, et al (1969).

The requirements of the *Standard Specifications* are stated in terms of vertical shear while those in the *LRFD Specifications* are stated in terms of horizontal (interface) shear. This difference in presentation, which has caused confusion among designers, will be numerically illustrated in Section 8.5.4.

8.5.2 Standard Specifications

The *Standard Specifications* does not identify the location of the critical section. For convenience, it may be assumed to be the same location as the critical section for vertical shear. Other sections, generally at tenth-point intervals along the span, are also designed for composite-action shear. This may be necessary to ensure that adequate reinforcement is provided for horizontal shear because reinforcement for vertical shear, which is extended into the deck and used for horizontal shear reinforcement, may vary along the length of the member.

Composite sections are designed for horizontal shear at the interface between the precast beam and deck using the equation:

$$V_u \le \phi V_{nh} \qquad \text{[STD Eq. 9-31a]}$$

where

V_u = factored vertical shear force acting at the section

V_{nh} = nominal horizontal shear strength

ϕ = strength reduction factor = 0.90

Note that the horizontal shear strength is compared to the factored vertical shear rather than a factored horizontal shear.

The nominal shear capacity is obtained from one of the following conditions given in STD Article 9.20.4.3:

a) when the contact surface is intentionally roughened but minimum vertical ties are not provided:

$$V_{nh} = 80 b_v d \qquad \text{(Eq. 8.5.2-1)}$$

b) when minimum ties are provide but the contact surface is not intentionally roughened:

$$V_{nh} = 80 b_v d \qquad \text{(Eq. 8.5.2-2)}$$

c) when the contact surface is intentionally roughened to a minimum amplitude of ¼ in. and minimum vertical ties are provided:

$$V_{nh} = 350 b_v d \qquad \text{(Eq. 8.5.2-3)}$$

d) when required area of ties, A_{vh}, exceeds the minimum area:

$$V_{nh} = 330 b_v d + 0.40 A_{vh} f_y d/s \qquad \text{(Eq. 8.5.2-4)}$$

The minimum area of ties is:

$$A_{vh} = 50\frac{b_v s}{f_y} \qquad \text{[STD Art. 9.20.4.5]}$$

For the above equations,

b_v = width of cross-section at the contact surface being investigated for horizontal shear

d = distance from extreme compression fiber to centroid of the prestressing force. As for vertical shear design, d need not be taken less than 0.80h.

s = maximum spacing not to exceed 4 times the least web width of support element, nor 24 in. [STD Art. 9.20.4.5]

Note that Eq. (8.5.2-4) reduces to Eq. (8.5.2-3) when A_{vh} is set equal to $50b_v s/f_y$. Eq. (8.5.2-4) is the mathematical representation of the provisions of STD Article 9.20.4.3 (d).

Editions of the *Standard Specifications* prior to 1996, required that all vertical shear reinforcement in a beam extend into the cast-in-place slab. However, that requirement was removed beginning with the 16th Edition because large amounts of tie reinforcement may hinder efficient deck removal when the cast-in-place deck requires replacement. Therefore, only the minimum vertical shear reinforcement necessary to satisfy the horizontal shear requirements must extend into the deck. In lieu of extending vertical shear reinforcement, independent reinforcement may be provided.

8.5.3 LRFD Specifications

LRFD Article C5.8.4.1 provides guidance for computing horizontal shear due to factored loads. In the commentary, the horizontal shear is calculated per unit of girder length and is also compared with the applied factored shear force on a per unit length basis. The factored vertical shear force is V_u. The factored shear force per unit length, V_h, is given by the following:

$$V_h = \frac{V_u}{d_v} \qquad \text{[LRFD Eq. C5.8.4.1-1]}$$

where

V_h = factored horizontal shear force per unit length of beam

V_u = factored vertical shear force at specified section due to superimposed loads

d_v = the distance between resultants of tensile and compressive forces

(Note: Through the 2003 Interim Revisions, the *LRFD Specifications* uses the notation d_e, which is felt to be in error. It is expected that the specifications will be revised.)

Required strength ≤ nominal strength, or:

$$V_h \leq \phi V_n \qquad \text{(Eq. 8.5.3-1)}$$

where V_n = nominal shear resistance of the interface surface

$= cA_{cv} + \mu[A_{vf} f_y + P_c]$ [LRFD Eq. 5.8.4.1-1]

$= cb_v + \mu[A_{vf} f_y + P_c]$ per unit length [LRFD Eq. C5.8.4.1-9]

where

c = cohesion factor

= 0.100 ksi for this case [LRFD Art. 5.8.4.2]

b_v = interface width

μ = friction factor

= 1.0 for this case [LRFD Art. 5.8.4.2]

A_{cv} = interface area of concrete engaged in shear transfer

A_{vf} = area of shear reinforcement crossing the shear plane where the shear plane under consideration must be consistent with the units used

P_c = permanent net compressive force normal to the shear plane (may be conservatively neglected)

f_y = yield strength of shear reinforcement

While the same notation is used for A_{vf} and P_c in the preceding equations, the values differ because the second equation is expressed on a per unit length basis. Units must be consistent for each term within the equations.

The values for c and μ apply when the top surface is intentionally roughened to an amplitude of ¼ in. Typically, any compressive force across the interface is neglected (i.e., $P_c = 0$).

Therefore, for normal weight concrete cast against hardened, roughened, normal weight concrete, the above relationships may be reduced to the following formula:

$V_h \leq \phi(0.1b_v + A_{vf}f_y)$ per unit length (Eq. 8.5.3-2)

where the minimum $A_{vf} = (0.05b_v)/f_y$ per unit length [LRFD Eq. 5.8.4.1-4]

Nominal shear resistance stated on a per unit length basis must also satisfy:

$V_n \leq 0.2f'_c b_v$, and, [LRFD Eq. 5.8.4.1-2]

$V_n \leq 0.8b_v$ [LRFD Eq. 5.8.4.1-3]

If the width of the contact surface exceeds 48 in., a minimum of four bars should be used for each row and consideration should be given to locating one bar in each outstanding portion of the flange.

The minimum reinforcement required of A_{vf} may be waived where $V_h/\phi b_v$ is less than 0.100 ksi. This is consistent with the *Standard Specifications*, the ACI Code and other references. It would seem to be impractical and an unnecessary expense to provide connectors in a number of common applications, such as precast stay-in-place panels if the interface stress is lower than 0.10 ksi.

8.5.4 Comparison of Design Specifications

As discussed in Section 8.5.1, the results of design for horizontal shear according to the *Standard* and *LRFD Specifications* can be quite different. Part of the difference is caused by the strength prediction formulas, as shown in **Figure 8.5.4-1**. Designers must be careful to recognize that the factored vertical shear force used in the *Standard Specifications* is not the same as the interface shear force used in the *LRFD Specifications*. These differences are demonstrated in the following example.

Consider an I-beam with a 20-in.-wide top flange and an 8"-thick composite cast-in-place concrete deck. The effective depth of the overall section is 48 in. These two elements are connected using 2 #4, Grade 60 bars spaced at 24 in.

The maximum nominal shear capacity according to the *Standard Specifications* is:

$\phi V_{nh} = \phi(0.35)(b_v)(d) = 0.85(0.35)(20)(48) = 286$ kips (Eq. 8.5.2-1)

The maximum nominal shear capacity according to the *LRFD Specifications* is:

$\phi V_n = \phi(cb_v + \mu A_{vf}f_y)$ [LRFD Eq. C.5.8.4.1-9]

$= 0.9[0.1(20) + 1.0(0.4/24)(60)]$

$= 2.7$ kips/in.

For comparison with the *Standard Specifications*, the results of ϕV_n from the *LRFD Specifications* must be multiplied by d_v. For simplicity, d_v can be taken as the distance between the resultant of the tensile force and the mid-thickness of the deck.

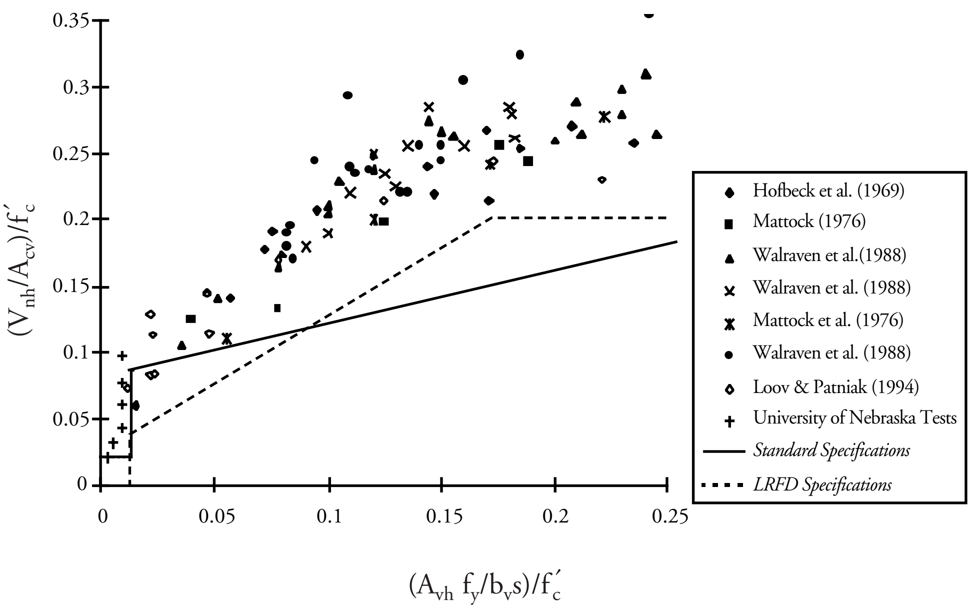
Therefore:

$d_v + 48 - 8/2 = 44$ in.

$\phi V_n d_v = 2.7(44) = 119$ kips

Recent studies, summarized by Kamel (1996) indicate that the 350 psi stress allowed by the *Standard Specifications* is higher than some of the experimental results, see **Figure 8.5.4-1**.

Figure 8.5.4-1
Specifications Equations for Horizontal Shear Strength Versus Test Results (Kamel, 1996)



It is the opinion of the Manual authors that the *LRFD Specifications* provisions are more theoretically correct than the provisions in the *Standard Specifications*, especially if the designer designs for the composite loads only, i.e. the loads introduced after the deck slab has hardened.

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.6 LOSS OF PRESTRESS

8.6.1 Introduction

Concrete is a material that exhibits time-dependent behavior. Under the effects of sustained stress, "creep" causes concrete to experience ongoing strains. Even when no loads are present, concrete specimens will undergo "shrinkage" strains. Prestressing steel, when strained at levels normal in prestressed concrete bridge members, exhibits a gradual loss of stress under constant strain that is referred to as "relaxation." **Chapter 2** and Section 8.13 provide equations, representative material constants and more information related to predicting creep, shrinkage and relaxation.

Under the combined effects of creep and shrinkage of concrete and the relaxation of prestressing steel, prestressed concrete members gradually deform with time. These time-dependent changes manifest themselves in the shortening of the member, in some loss of prestress and, therefore, in a change in camber or deflection.

Several techniques are available to the designer to account for these effects. Approximate methods suitable for conventional designs are given in this section. More detailed methods suitable for unusual or complex designs are presented in Section 8.13.

8.6.2 Definition

Loss of prestress is defined as the difference between the initial stress in the strands (just after seating of strands in the anchorage), and the effective prestress in the member (at a time when concrete stresses are to be calculated). This definition of loss of prestress includes both instantaneous (elastic) losses and losses that are time dependent.

In post-tensioning applications, friction between the tendon and the duct as well as anchorage seating losses during the post-tensioning operation must be considered in design. Post-tensioning applications are included in **Chapter 11**.

8.6.3 Significance of Losses on Design

For design, there are two important stages in the life of a prestressed concrete bridge beam when loss of prestress plays a significant role. First, a reasonable estimate of the prestress level is needed immediately following transfer of prestress. This is to avoid overstressing the concrete beam when the prestress force is the highest and concrete strength is lowest. The second stage that requires an estimate of effective prestress is under long-term service conditions. This is required to ensure that calculated stresses in the concrete are below the limits prescribed by the specifications. Other design criteria, including such critical matters as ultimate flexural strength and ultimate shear strength, are relatively insensitive to the designer's estimate of loss of prestress. It is primarily the unfactored service load stresses in the bottom flange of a concrete beam that the designer is attempting to control through estimates of loss of prestress.

8.6.4 Effects of Estimation of Losses

It is important to recognize the variables which affect the loss of prestress in a beam. Some of the important variables affecting time-dependent behavior, and therefore loss of prestress, are the concrete's modulus of elasticity, and creep and shrinkage properties. These variables can be somewhat unpredictable for a given concrete mixture and cannot be fully controlled by the designer. Therefore, the estimation of loss of prestress should not be overemphasized at the expense of other more important issues during the design process.

8.6.4.1 Effects at Transfer

An exception to the previous statement should be made with regard to estimation of loss of prestress at the time of transfer. It is important that losses at release (transfer) not be grossly overestimated. If this were to happen, sudden failure of the member at prestress transfer, while not likely, could possibly occur. At least, significant cracking could be expected. Fortunately, estimates of losses at this time are subject to fewer unknown variables, allowing a more accurate estimate to be made in almost all cases.

8.6.4.2 Effect on Production Costs

In the case of overestimation of loss of prestress, the beam is likely to be constructed with more strands than necessary. The increase in strand cost is usually not significant, but if additional strands are provided, the concrete release strength may have to be increased to accommodate the larger prestress force. This could require a longer curing time or a more expensive concrete mix. An increase in beam costs may result, particularly if the increase in release strength requires an additional day to complete the casting cycle.

8.6.4.3 Effect on Camber

Another issue associated with overestimation of loss of prestress, and the addition of strands, is unexpected camber. Excessive camber of a bridge member can cause problems for constructors and can result in the need to adjust bridge grades. Finally, if a designer significantly overestimates loss of prestress for a member near the limits of its span range, it may be necessary to specify a larger beam. Foundation and substructure costs may also increase.

From the above discussion, it should be apparent that it is not beneficial to overestimate loss of prestress. The designer, therefore, is cautioned against overestimation.

8.6.4.4 Effect of Underestimating Losses

Conversely, underestimation of loss of prestress can theoretically result in excessive tensile stresses in the concrete member under service conditions. No known instances of problems resulting from this condition have been reported. The ultimate flexural strength requirements of bridge design specifications serve to impose a lower limit on the number of strands in a member, thereby indirectly preventing a low estimate for loss of prestress from having a significant adverse impact on a member. Underestimating losses may also result in lower than anticipated camber or even an unsightly sag in the final structure, but again, this is unlikely to occur in practice.

8.6.5 Prediction of Creep, Shrinkage and Relaxation Material Properties

Time-dependent analysis for prestress loss, deflection, and for restraint moments in jointless bridges require knowledge of the material properties of concrete and steel. These properties include modulus of elasticity, creep and shrinkage of concrete and relaxation of prestressing steel. More detailed discussion of these properties is given in Chapter 2. The following sections present prediction formulas for use in lieu of actual test data. These formulas are used in the analysis of prestress losses and other time-dependent effects in this section and in Section 8.13.

Several prediction formulas exist for creep and shrinkage. They are primarily based on either ACI Committee 209 or CEB-FIP model code. The formulas recommended by ACI 209 are given below. It is the designer's option to use either the ACI Committee 209 or CEB-FIP formulas. It is important, however, to recognize that one should not combine information from both sources in one design, as the ACI procedure tends to give higher shrinkage and lower creep values than the CEB-FIP method. The *LRFD Specifications* includes formulas for creep and shrinkage prediction that are primarily based on the ACI method. For the reasons indicated below, it is recommended that the formulas given here be used. They are based on recommendations by ACI Committee 209, modified by Huo (1997) to account for high strength concrete. As **Figures 8.6.5-1** and **8.6.5-2** show, the formulas given here represent the best correlation with test results. Otherwise, the *LRFD Specifications* shrinkage formulas should be modified to account for the higher concrete strengths used in precast concrete products. The *LRFD Specifications* creep formulas already include a strength modifier.

Figure 8.6.5-1
Comparison Between Experimental Results and Different Formulae for Prediction of Creep Coefficient (Huo, et al, 2001)

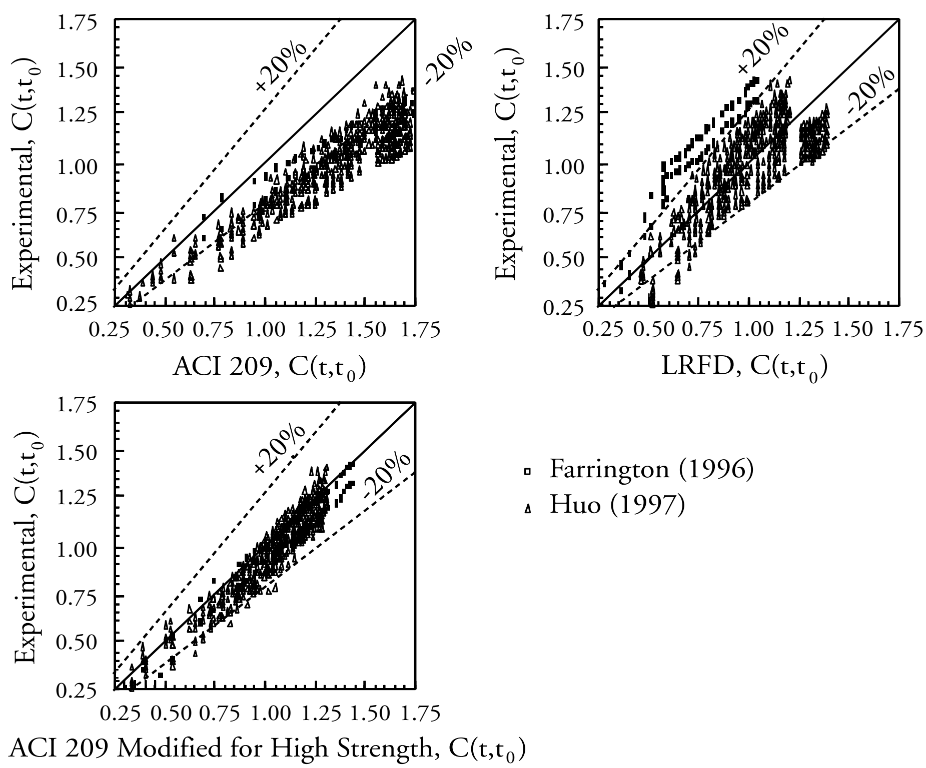
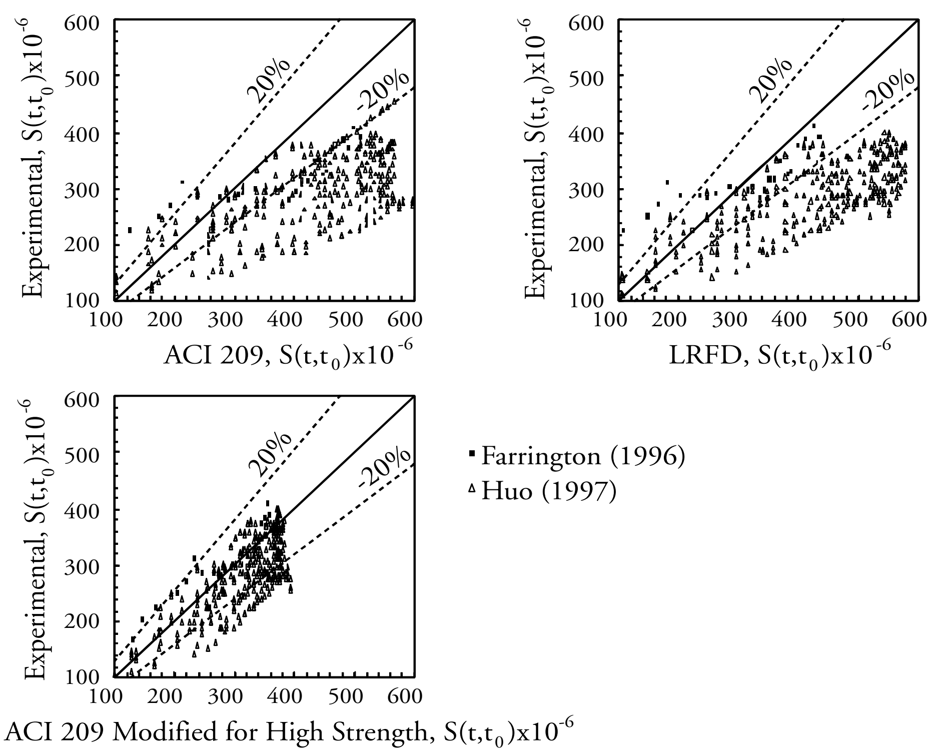


Figure 8.6.5-2
Comparison Between Experimental Results and Different Formulae for Prediction of Shrinkage Coefficient (Huo, et al, 2001)



8.6.5.1
Prediction of Creep Coefficient of Concrete

Prediction of creep coefficient of concrete members is presented in detail in Chapter 2 for members whose specified compressive strength is in the range of 3 to 5 ksi. In this strength range, the creep coefficient $C(t,t_0)$ for concrete loaded at an age of t_0, in days, for a loading duration of $(t\text{-}t_0)$ days, is:

$$C(t,t_0) = \frac{(t-t_0)^{0.6}}{10+(t-t_0)^{0.6}} C_u \qquad \text{(Eq. 2.5.8.1-1)}$$

where $C_u = 1.88k_c$ (Eq. 2.5.8.1-2)

where k_c = product of applicable correction factors = $k_{la}\, k_h\, k_s$ (Eq. 2.5.8.1-2a)

where

k_{la} = correction factor for loading age

k_h = correction factor for relative humidity

k_s = correction factor for size of member

Eq. (2.5.8.1-1) is applicable to members moist-cured for 7 days or heat-cured for 1 to 3 days.

8.6.5.1.1
Creep Modification Factors

The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2** or by using Eq. (2.5.7.1-3d). The size correction factor, k_s, depends on the volume-to-surface area of the member and may be taken from **Table 2.5.7.1-3** or by using Eq. (2.5.7.1-3f). The correction factor for loading age, k_{la}, can be calculated using Eq. (2.5.8.1-2b) for steam-cured concrete or Eq. (2.5.8.1-2c) for moist-cured concrete or may be taken from **Table 2.5.8.1-1**.

8.6.5.1.2
Modification Factors for Strength

Experimental studies by Huo (1997) produced two additional modifiers to allow for application to members with concrete strengths up to 12,000 psi. First, the term $(12 – 0.5f'_c)$ is proposed to be used in place of the number "10" in Eq. 2.5.8.1-1 to reflect the fact that the creep coefficient initially develops at a faster rate in higher strength concrete. Second, an additional modifier, k_{st}, is introduced to reflect the experimental observation that higher strength concrete exhibits lower ultimate creep.

Correction factor for concrete strength: $k_{st} = (1.18 – 0.045f'_c)$ (Eq. 8.6.5.1.2-1)

where f'_c = concrete strength at 28 days, ksi

Therefore, Eq. 2.5.8.1-1 is modified to the following equation for use for members with f'_c in the range of from 4 to 12 ksi:

$$C(t,t_0) = 1.88k_{la}\, k_h\, k_s\, k_{st} \frac{(t-t_0)^{0.6}}{(12-0.5f'_c)+(t-t_0)^{0.6}} \qquad \text{(Eq. 8.6.5.1.2-2)}$$

8.6.5.1.3
Creep Example

Determine creep at 2 years for 7.0 ksi concrete in an environment of 70% relative humidity. The concrete was steam-cured and initially loaded at 2 days. The member volume-to-surface area ratio is 1.5 in.

Thus,

$t = 2(365) = 730$ days

$t_0 = 2$ days

$H = 70\%$

$f'_c = 7.0$ ksi

Therefore,

$k_{la} = 1.13(2)^{-0.094} = 1.06$

$k_h = 1.00$

$$k_s = \frac{2}{3}(1+1.13e^{-0.54(1.5)}) = 1.00$$

$k_{st} = 1.18 - 0.045(7.0) = 0.87$

$k_c = (1.06)(1.00)(1.00)(0.87) = 0.92$

$C_u = 1.88(0.92) = 1.73$

$$C(730, 2) = \frac{(728)^{0.6}}{\left[12-0.5(7)\right]+(728)^{0.6}}(1.73) = 1.49$$

The *LRFD Specifications* provides a correction factor for concrete strength that has a similar effect to that given by the correction factor, k_{st}, of Huo (1997), i.e. reduction of the overall creep for increased concrete strength. It is based on Nilson (1987). However, the *LRFD Specifications* creep prediction formula tends to somewhat underestimate creep growth at early concrete ages.

8.6.5.2 Prediction of Shrinkage Coefficient of Concrete

Prediction of the shrinkage coefficient for concrete members is presented in detail in Chapter 2 for members whose specified compressive strength is in the 3 to 5 ksi range. For this strength range, the shrinkage coefficient, $S(t,t_0)$, can be calculated using the following equations.

For concrete members subjected to steam curing for 1 to 3 days:

Shrinkage, S, in the period $(t-t_0)$:

$$S(t,t_0) = \frac{(t-t_0)}{55+(t-t_0)} S_u \qquad \text{(Eq. 2.5.7.1-1)}$$

For concrete members moist-cured for 7 days:

Shrinkage, S, in the period $(t-t_0)$:

$$S(t,t_0) = \frac{(t-7)}{35+(t-7)} S_u \qquad \text{(Eq. 2.5.7.1-2)}$$

where S_u = ultimate shrinkage strain = $545k_{sh} \times 10^{-6}$ (Eq. 2.5.7.1-3)

where k_{sh} = product of applicable corrections factors = $k_{cp}\ k_h\ k_s$ (Eq. 2.5.7.1-3a)

where

k_{cp} = correction factor for curing period

k_h = correction factor for relative humidity

k_s = correction factor for size of member

8.6.5.2.1 Shrinkage Modification Factors

Correction factor, k_{cp}, for a different curing period than 7 days of moist curing may be taken from **Table 2.5.7.1-1**. This correction factor, k_{cp}, may be taken equal to 1.0 for steam-cured members. The relative humidity correction factor, k_h, may be taken from **Table 2.5.7.1-2** or calculated by using Eq. (2.5.7.1-3b) and Eq. (2.5.7.1-3c). The size correction factor, k_s, depends on the volume-to-surface area of the member and may be taken from **Table 2.5.7.1-3** or calculated by using Eq. (2.5.7.1-3e).

8.6.5.2.2 Modification Factors for Strength

Experimental studies by Huo (1997) have produced factors that permit using the above shrinkage prediction procedure for concrete strengths up to 12.0 ksi. Two additional modifiers were introduced. The first was to add $(10–2.5f'_c)$ to the denominator of the time function to reflect more rapid shrinkage at an early age of high strength concrete. The second modifier is a multiplier, k_{st}, to reflect a reduced ultimate shrinkage with higher strength concrete.

Ultimate shrinkage correction factor for concrete strength:

$$k_{st} = (1.2 - 0.05f'_c) \qquad \text{(Eq. 8.6.5.2.2-1)}$$

where f'_c = concrete strength at 28 days, ksi

Therefore, the Eqs. 2.5.7.1-1 and 2.5.7.1-2 are modified to the following equations for use with members with f'_c in the 4 to 12 ksi range:

For moist-cured concrete:

$$S(t,t_0) = 545 \times 10^{-6} k_{cp} k_h k_s k_{st} \frac{(t-7)}{(45-2.5f'_c)+(t-7)} \qquad \text{(Eq. 8.6.5.2.2-2)}$$

For steam-cured concrete:

$$S(t,t_0) = 545 \times 10^{-6} k_{cp} k_h k_s k_{st} \frac{(t-t_0)}{(65-2.5f'_c)+(t-t_0)} \qquad \text{(Eq. 8.6.5.2.2-3)}$$

8.6.5.2.3 Shrinkage Example

Compute the shrinkage for 7,000 psi concrete, moist-cured for 1 day and subjected to an environment of 70% relative humidity for 2 years. The volume-to-surface area ratio is 1.5 in.

Thus,

t = 2(365) = 730 days

t_0 = 1 day

H = 70%

Therefore,

k_{cp} = 1.0

$k_h = 1.00$

$k_s = 1.20e^{-0.12(1.5)} = 1.00$

$k_{st} = 1.2 - 0.05(7.0) = 0.85$

$k_{sh} = (1.0)(1.00)(1.00)(0.85) = 0.85$

$S_u = 545(0.85) \text{ x } 10^{-6} = 463 \text{ x } 10^{-6}$

$$S(730,1) = \frac{(730-1)}{\left[65-2.5(7)\right]+(730-1)}(463 \text{ x } 10^{-6}) = 435 \text{ x } 10^{-6}$$

The *LRFD Specifications* significantly overestimates long-term shrinkage in precast, prestressed concrete members as no correction factors are given for concrete strength.

8.6.5.3
Prediction of Relaxation of Prestressing Steel

The intrinsic relaxation of low-relaxation prestressing steel can be calculated using the following equation, based on the prestressing force in the strand immediately after anchoring:

$$L_r = \frac{\log_{10}(24t)}{K_r}\left(\frac{f_i}{f_{py}} - 0.55\right)(f_i) \qquad \text{(Eq. 8.6.5.3-1)}$$

where

L_r = intrinsic relaxation of the strand without accounting for member shortening due to creep and shrinkage

f_i = initial stress at the beginning of the relaxation loss period

f_{py} = yield strength of the strand

t = time in days for which relaxation loss is calculated

K_r = 45 for low-relaxation strand

= 10 for normal-relaxation (stress-relieved) strand

Note: The PCI Committee Report on Prestress Losses (1975) recommends a value for K_r of 45 for low-relaxation strand. However, the *LRFD Specifications* uses 40 (see Section 8.13, **Table 8.13.1.4-1**). A value of 45 is both commonly used and recommended.

8.6.6
Methods for Estimating Losses

A variety of methods are available to predict loss of prestress. They fall into three main categories, listed in order of increasing complexity:

1. lump sum estimate methods
2. rational approximate methods
3. detailed time-dependent analyses

Both the *Standard Specifications* and *LRFD Specifications* provide methods in the first two categories above. The time-dependent analysis procedure described in Section 8.13 fits into the third category. Over the last several decades, a wide variety of additional techniques and procedures have been suggested for estimating loss of prestress.

The ACI-ASCE Committee 423 method (Zia, et al, 1979) is similar to both the *Standard Specifications* and *LRFD Specifications* methods, but uses slightly different equations. The PCI method (PCI Committee on Prestress Losses, 1975) provides a detailed step-by-step analysis. Another method, developed by Branson and

Kripanaranayan (1971), appears in the ACI 209 Committee Report (1992). The interested reader is referred to the cited references for additional details.

8.6.7 Elastic Shortening Loss

Elastic shortening is the immediate shortening of the member under the application of prestressing force. Elastic shortening at the transfer of pretensioning occurs instantaneously and is not a time-dependent effect. Elastic shortening losses differ from time-dependent losses in that they are partially recoverable. A partial recovery of elastic shortening losses may occur due to elastic gain when loads are superimposed on the member.

8.6.7.1 Computation of Elastic Shortening Loss

The elastic shortening loss is computed using the same equation in both the *Standard* and *LRFD Specifications*. That equation, however, provides an approximate value for this loss, or requires iteration to obtain an exact value. Alternatively, Eqs. (8.6.7.1-1) through (8.6.7.1-3) which follow, provide a method for determination of the elastic shortening loss that yield an exact value without iteration.

$$ES = f_{pi} - f_{po} \qquad \text{(Eq. 8.6.7.1-1)}$$

where

f_{pi} = initial stress immediately before transfer

f_{po} = stress in prestressing steel immediately after release

$$f_{po} = \frac{f_{pi} + \dfrac{E_p}{E_{ci}}\dfrac{M_{sw}e}{I}}{1 + \alpha} \qquad \text{(Eq. 8.6.7.1-2)}$$

where

E_{ci} = modulus of elasticity of the beam concrete at transfer

E_p = modulus of elasticity of pretensioning tendons

M_{sw} = moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at the time of transfer

e = eccentricity of prestressing steel with respect to centroid of section

$$\alpha = \frac{E_p}{E_{ci}}\frac{A_p}{A}\left(1 + \frac{e^2}{r^2}\right) \qquad \text{(Eq. 8.6.7.1-3)}$$

and where r = radius of gyration of the gross cross-section = $\sqrt{\dfrac{I}{A}}$

8.6.7.2 Elastic Shortening Example

Calculate elastic shortening loss at midspan for the beam shown in **Figure 8.6.7.2-1**. Assume that the strands were tensioned 18 hours prior to release. For a span length of 65 ft, the self-weight moment at midspan is 3,694 in.-kips.

Some designers use the full length of the member to calculate effects at the time of prestress release, since the member cambers and becomes supported at its ends. This is not a universally accepted approach. For convenience of discussion in the following sections, the span between centerlines of bearings will be used throughout this section.

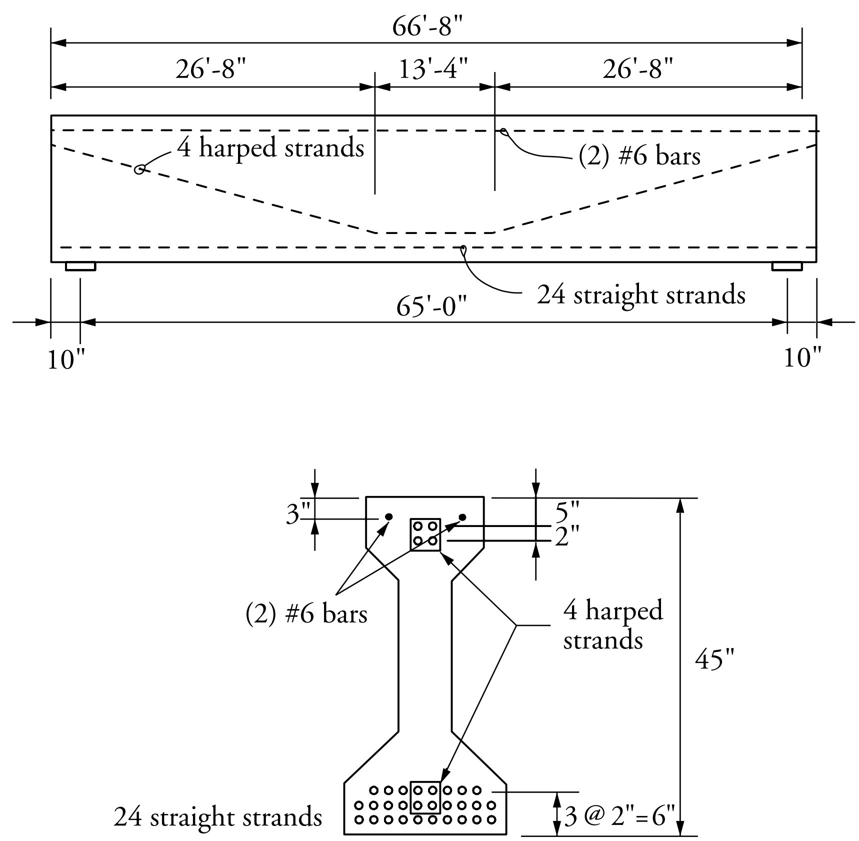


Figure 8.6.7.2-1 Beam Configuration Used in Elastic Shortening Example

Input data

Strands:	1/2-in.-diameter strand, Grade 270, low-relaxation
	Eccentricity at midspan = 16.413 in.
	Eccentricity at end of beam = 11.556 in.
	A_p = 28 strands x 0.153 = 4.284 in.^{2}
Bars:	#6, Grade 60
Beam concrete:	f'_{ci} = 4,950 psi at 18 hours, E_{ci} = 4,054 ksi
	f'_c = 6,750 psi at 28 days, E_c = 4,734 ksi
	$\varepsilon_{shu} = -0.0004$ in./in.
	$C_u = 1.4$
Beam section properties:	Type III AASHTO-PCI
	A = 560 in.^{2}
	I = 125,390 in.^{4}
	y_b = 20.27 in.
Composite section properties:	I_c = 382,372 in.^{4}
	y_{bc} = 36.02 in.
Deck properties:	Width = 104 in.
	Thickness = 8 in.
	f'_c = 4,500 psi at 28 days
	$\varepsilon_{shu} = -0.0004$ in./in.
	$C_u = 1.4$

Dead loads:
- Self-weight = 583 plf
- Deck weight = 867 plf
- Haunch weight = 40 plf
- Diaphragm weight = 100 plf
- Superimposed dead load = 360 plf

The total force in the strands, at the time of transfer, is equal to the force immediately after the strands are anchored less the relaxation loss occurring prior to transfer. Relaxation loss prior to transfer is computed using equation 8.6.5.3-1. Using an anchoring stress of $0.75f_{pu}$ = 202.5 ksi and assuming the strands are anchored 18 hours (0.75 days) prior to transfer, L_r is calculated to be 1.60 ksi. Therefore, f_{pi}, the strand stress immediately before transfer, is taken as 202.5 − 1.6 = 200.9 ksi.

Next, α is calculated using an eccentricity of the prestressing steel, e, of 16.413 in. The modulus of elasticity of the concrete at transfer, E_{ci}, is 4,054 ksi, and the modulus of elasticity of the prestressing strand, E_p, is 28,500 ksi.

$$\alpha = \frac{28{,}500}{4{,}054}\,\frac{4.284}{560}\left(1+\frac{(16.413)^2}{223.91}\right) = 0.1185$$

Calculate the strand stress immediately after transfer:

$$f_{po} = \frac{200.9 + \dfrac{28{,}500}{4{,}054}\,\dfrac{(3{,}694)(16.413)}{125{,}390}}{1 + 0.1185} = 182.7 \text{ ksi}$$

Elastic shortening loss, ES, is calculated to be:

ES = 200.9 − 182.7 = 18.2 ksi

8.6.8 Losses from the Standard Specifications

The Eighth Edition of the Standard Specifications, published in 1961, was the first AASHO (now AASHTO) specifications to contain a reference to loss of prestress. In that edition, "loss of prestress due to all causes except friction" was assumed to be 35,000 psi for normal relaxation strand for pretensioned applications. The 1961 *Standard Specifications* also recommended that the method presented in the ACI-ASCE Joint Committee 423 Report [1958] was recommended if a "more exact" estimate was required.

In the 1971 Interim Revisions, a method was introduced where the total losses were estimated as the sum of individual components. These provisions marked the first use, in the *Standard Specifications*, of a detailed method of estimating loss of prestress.

The following equation, which remains essentially unchanged since its introduction in the 1971 Interim Revisions, is used for estimating total loss of prestress:

$$\Delta f_s = SH + ES + CR_C + CR_S \qquad \text{[STD Eq. 9-3]}$$

where

Δf_s = total loss of prestress

SH = loss of pretension due to concrete shrinkage

ES = loss of pretension due to elastic shortening

CR_C = loss of pretension due to creep of concrete

CR_S = loss of pretension due to relaxation of prestressing steel

All values are in psi.

8.6.8.1 Shrinkage Loss

The equation for estimating loss due to shrinkage of concrete, SH, is roughly based on an ultimate concrete shrinkage strain of approximately –0.00042 and a modulus of elasticity of approximately 28,000 ksi for prestressing strands. Correction factors for different average ambient relative humidities were applied resulting in three values of SH, depending on regional average humidity. A straight line was drawn to best fit the three discrete values, resulting in the following equation for pretensioned members:

$$SH = 17{,}000 - 150RH \qquad \text{[STD Eq. 9-4]}$$

where RH = mean annual ambient relative humidity, percent.

8.6.8.2 Elastic Shortening Loss

The *Standard Specifications* provides the following equation for estimating elastic shortening loss for pretensioned members:

$$ES = \frac{E_s}{E_{ci}} f_{cir} \qquad \text{[STD Eq. 9-6]}$$

where f_{cir} = average concrete stress at the center of gravity of pretensioning steel due to pretensioning force and dead load of beam immediately after transfer.

Since f_{cir} depends on the as-yet-unknown ES, an approximation for f_{cir} must be made in [STD Eq. 9-6]. Representative starting values for f_{cir} are provided in the *Standard Specifications* for both low-relaxation and stress-relieved strand.

It is recommended, however, that the more exact method represented by Eqs. (8.6.7.1-1) through (8.6.7.1-3) be used in lieu of the procedure in the *Standard Specifications* for estimating ES, as shown in Example 8.6.7.2.

8.6.8.3 Creep Loss

Loss due to creep of concrete is given by:

$$CR_C = 12f_{cir} - 7f_{cds} \qquad \text{[STD Eq. 9-9]}$$

where f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time pretensioning force is applied.

This equation was developed by Gamble (1972) and was based on field measurements of a series of bridges in Illinois. The first term in [STD Eq. 9-9] is based on a creep coefficient of approximately 1.7 and a modular ratio of 7. The second term in the equation represents the instantaneous elastic stress increase in the bonded tendons after application of additional dead loads. (It should be noted that creep due to f_{cds} would have a tendency to further increase the second term, resulting in lower predictions for CR_C. [STD Eq. 9-9] represents a conservative approach, i.e. assuming a creep coefficient of zero for these later permanent loads.

8.6.8.4 Strand Relaxation Loss

For stress-relieved strands, total loss due to relaxation of the prestressing steel is given by:

$CR_S = 20{,}000 - 0.4ES - 0.2(SH + CR_C)$ [STD Eq. 9-10]

This equation was also developed by Gamble (1972). The 0.4 multiplier on the ES term was based on analytical studies of total relaxation of stress-relieved strands with varying values of elastic shortening loss. Gamble reasoned that shrinkage and creep losses, occurring gradually and at a time when relaxation would occur at a reduced rate, would produce only approximately one-half as much reduction in relaxation as would elastic shortening loss that occurs very soon after the strand is stressed. Hence the factor of 0.2 on SH and CR_C.

For low-relaxation strand, total loss due to relaxation is given by:

$CR_S = 5{,}000 - 0.1ES - 0.05(SH + CR_C)$ [STD Eq. 9-10A]

This equation is based on the equation for stress relieved strand, but is adjusted for intrinsic relaxation in low-relaxation strand of approximately 25% that of stress-relieved strand.

8.6.8.5 Lump Sum Losses

As an alternative to the procedure described in the preceding paragraphs, the *Standard Specifications* permits the use of a single value of lump sum total losses equal to 45,000 psi for pretensioned beams of “usual design” and a concrete strength of 5,000 psi. No corrections are provided for other concrete strengths.

8.6.9 Standard Specifications Example

Calculate the total loss of prestress at midspan for the beam shown in **Figure 8.6.7.2-1** in accordance with the *Standard Specifications*. The midspan moment due to deck, haunch and diaphragms applied to the plain beam is 6,379 in.-kips. The midspan moment due to dead loads applied to the composite section is 2,280 in.-kips. The average relative humidity is 70%. The strands are initially tensioned to a stress equal to 0.75(270) = 202.5 ksi.

Use the results of Example 8.6.7.2 for elastic shortening loss. This is a more accurate method than the one given in the *Standard Specifications*. Therefore, ES = 18,200 psi.

Again, using Example 8.6.7.2 and STD Eq. 9-6 rearranged, the concrete stress at the centroid of the prestressing steel at the time of prestress release is:

$$f_{cir} = (E_{ci}/E_p)ES = \frac{4{,}054}{28{,}500}(18{,}200) = 2{,}589 \text{ psi}$$

Loss due to shrinkage of concrete:

$SH = 17{,}000 - (150)(70) = 6{,}500$ psi

The determination of loss due to creep of concrete requires the calculation of f_{cds}, the concrete stress due to permanent dead loads other than self-weight of the beam.

Calculation of f_{cds} requires application of the standard equation $\left(\frac{My}{I}\right)$ applied after the release of prestress, where y refers to the distance from the centroid of the section to the centroid of the prestressing steel. The calculation must take into account all loading stages. First, calculate the component of f_{cds} due to loads on the plain beam. These are the deck weight = 867 plf, the haunch weight = 40 plf and the diaphragm weight = 100 plf and the resulting moment = 6,379 in.-kip.

$$f_{cds1} = \frac{1,000(6,379)(16.413)}{125,390} = 835 \text{ psi}$$

Next, calculate concrete stresses due to dead loads on the composite section. The composite section moment of inertia is 382,372 in.^{4} and the composite section centroid is 36.02 in. from the bottom flange. Thus, the centroidal distance to the steel is [36.02 – (20.27 – 16.413)] = 32.163 in. Calculating the stress for the effect of a superimposed load of 360 plf gives:

$$f_{cds2} = \frac{1,000(2,280)(32.163)}{382,372} = 192 \text{ psi}$$

f_{cds} = 835 + 192 = 1,027 psi

So, CR_C = (12)(2,589) – (7)(1,027) = 23,879 psi

Finally, the total relaxation of steel is calculated:

CR_S = 5,000 – (0.1)(18,200) – (0.05)(6,500 + 23,879) = 1,661 psi

So, the total loss of prestress for this beam is:

Δf_s = 6,500 + 18,200 + 23,879 + 1,661 = 50,240 psi

The effective prestress remaining in the beam after all losses is (202.50 – 50.24) = 152.26 ksi, or a total loss equal to 24.8% of the initial prestress. Alternatively, use of the lump sum total loss estimate of 45,000 psi is permitted by the *Standard Specifications*. The detailed estimate results in a predicted prestress loss approximately 2.6% of the initial prestress force higher than the lump sum estimate.

8.6.10 Losses from the LRFD Specifications

The *LRFD Specifications* provides two methods for estimating time-dependent losses. The first is the lump sum method with adjustments for PPR (partial prestressing ratio) and concrete compressive strength, as shown in **Table 8.6.10-1**. The table values do not include the loss due to elastic shortening, which must still be accounted for.

Table 8.6.10-1
Time-Dependent Losses, ksi
[LRFD Table 5.9.5.3-1]

Type of Beam Section	Level	For wire and strands with f_{pu} = 235, 250 or 270 ksi	For bars with f_{pu} = 145 or 160 ksi
Rectangular Beam and Solid Slab	Upper bound Average	29.0 + 4.0 PPR 26.0 + 4.0 PPR	19.0 + 6.0 PPR
Box Beam	Upper bound Average	21.0 + 4.0 PPR 19.0 + 4.0 PPR	15.0
I-Beam	Average	$33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\text{PPR}$	19.0 + 6.0 PPR
Single-Tee, Double-Tee, Hollow-Core and Voided Slab	Upper bound	$39.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\text{PPR}$	$31.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\text{PPR}$
	Average	$33.0\left[1.0-0.15\frac{f'_c-6.0}{6.0}\right]+6.0\text{PPR}$	

The partial prestress ratio, PPR, is determined using the *LRFD Specifications* equation:

$$PPR = \frac{A_{ps} f_{py}}{A_{ps} f_{py} + A_s f_y} \qquad \text{[LRFD Eq. 5.5.4.2.1-2]}$$

where

A_s = area of nonprestressed tension reinforcement

A_{ps} = area of pretensioning steel

f_y = yield strength of reinforcing bars

f_{py} = yield strength of pretensioning steel

The second method is an approximate rational method, nearly identical to the *Standard Specifications* equations. This "refined" method is described in the following paragraphs.

The provisions in the *LRFD Specifications* for the estimation of loss of prestress are patterned closely after the *Standard Specifications*. The only changes are minor differences in the computation of the loss due to relaxation of prestressing steel.

Using the notation of the *LRFD Specifications*, total loss of prestress is given by:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} \qquad \text{[LRFD Eq. 5.9.5.1-1]}$$

where

Δf_{pT} = total loss of pretensioning steel stress

Δf_{pES} = loss of pretensioning steel stress due to elastic shortening

Δf_{pSR} = loss of pretensioning steel stress due to shrinkage of concrete

Δf_{pCR} = loss of pretensioning steel stress due to creep of concrete

Δf_{pR2} = loss of pretensioning steel stress due to relaxation of prestressing steel

All stresses are in ksi.

8.6.10.1 Elastic Shortening Loss

The elastic shortening loss is given by:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \qquad \text{[LRFD Eq. 5.9.5.2.3a-1]}$$

where f_{cgp} = concrete stress at the center of gravity of the pretensioning tendons due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment.

This is the same as the definition of f_{cir} in the *Standard Specifications*. As in the *Standard Specifications*, approximate values of f_{cgp} are provided.

In the 2001 Interim Revisions, an alternative to LRFD Eq. 5.9.5.2.3a-1 was introduced in the Commentary for pretensioned members as follows:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_m{}^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m{}^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}} \qquad \text{[LRFD Eq. C5.9.5.2.3a-1]}$$

where

A_{ps} = area of prestressing steel, in.^{2}

A_g = gross area of section, in.^{2}

E_{ci} = modulus of elasticity of concrete at transfer, ksi

E_p = modulus of elasticity of prestressing tendons, ksi

e_m = average eccentricity at midspan, in.

f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in LRFD Table 5.9.3-1, ksi

I_g = moment of inertia of the gross concrete section, in.^{4}

M_g = midspan moment due to member self-weight, in.-kips

This closed form equation for prestress loss due to elastic shortening, eliminates trial and error calculations.

It is recommended, however, that the "exact" method of Eqs. (8.6.7.1-1) through (8.6.7.1-3) be used for estimating elastic shortening loss.

8.6.10.2 Shrinkage and Creep Losses

Losses due to shrinkage and creep of concrete are given by the following equations, which are the same as in the *Standard Specifications*:

$$\Delta f_{pSR} = 17.0 - 0.15H \qquad \text{[LRFD Eq. 5.9.5.4.2-1]}$$

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} \qquad \text{[LRFD Eq. 5.9.5.4.3-1]}$$

where

H = average annual ambient relative humidity, %

Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load present at the time the pretensioning force is applied. This quantity is identical to the f_{cds} defined in the *Standard Specifications*.

8.6.10.3 Strand Relaxation Loss

Prior to the *LRFD Specifications*, relaxation of the strands before transfer was not specifically addressed in the bridge design codes.

Typical design practice, however, involves making an estimate of the intrinsic strand relaxation that is known to occur during the 12- to 18-hour period between anchoring of the stressed strands and the transfer of prestress to the member. Many designers add this relaxation, Δf_{pR1}, which is typically on the order of 1.5 ksi, to the computed elastic shortening loss, Δf_{pES}, to determine the initial prestress loss used to check beam stresses at transfer.

The equation for relaxation losses in the *Standard Specifications* is understood to include both the intrinsic relaxation (prior to transfer) described above and the long-term relaxation that occurs during the service life of the beam. Therefore, the relaxation prior to transfer is not used to compute the total prestress losses for design of the beam for service loads using the *Standard Specifications.*

The *LRFD Specifications* state that those relaxation losses prior to transfer are accounted for during fabrication of a prestressed member. However, this is not standard practice in the industry. The only adjustments to the prestressing force that are commonly made by producers are those required for temperature compensation, bed or form deformation, and chuck seating (see Sect. 3.3.2.7).

It is expected that a future revision to the *LRFD Specifications* will remove the statement that relaxation losses prior to transfer are accounted for in fabrication. In the meantime, this statement in the *LRFD Specifications* should be disregarded.

Based on a long history of successful designs using the approach for computing prestress losses in the *Standard Specifications*, the following are recommended when using the *LRFD Specifications:*

- Assume strands will be tensioned to $0.75f_{pu}$ upon anchoring (unless it is known that another value is to be used).
- Compute Δf_{pR1} and add to elastic shortening loss, Δf_{pES}, to estimate prestress losses at transfer, Δf_{pi}, and use Δf_{pi} to check beam stresses at transfer.
- Use Δf_{pR2} as the total relaxation loss in computing total prestress losses, Δf_{pT}, for the service load design of the member.

The term Δf_{pR1} is the same as L_r in Eq. (8.6.5.3-1).

The relaxation loss used for service load design of the member, is given by the following equations:

For normal-relaxation (stressed-relieved) strands,

$$\Delta f_{pR2} = 20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR}) \qquad \text{[LRFD Eq. 5.9.5.4.4c-1]}$$

For low-relaxation strands, the relaxation loss is 30% of Δf_{pR2}, or [LRFD Art.5.9.5.4.4c]

$$\Delta f_{pR2} = 6.0 - 0.12\Delta f_{pES} - 0.06(\Delta f_{pSR} + \Delta f_{pCR}) \qquad \text{(Eq. 8.6.10.3-1)}$$

Note that the *LRFD Specifications* total loss due to relaxation, for low-relaxation strand, is 30% of the loss for normal-relaxation strand rather than a factor of 25% in the *Standard Specifications.*

8.6.10.4 Washington State Study

A study performed by Seguirant (1998) for the Washington State Department of Transportation has demonstrated the significant overestimation of pretension loss that can result from using the *LRFD Specifications* refined method, as shown in **Figure 8.6.10.4-1**. A similar trend would be expected when the *Standard Specifications* equations are used, since both sets of equations were developed for concrete strengths and prestress levels much lower than currently possible. This overestimation is more significant with higher levels of pretension. It can result in loss of span capacity as shown in **Figure 8.6.10.4-2**. Therefore, it is recommended that designers avoid using the refined *LRFD Specifications* method, especially for members with high pretensioning forces.

Figure 8.6.10.4-1 Pretension Loss Comparison

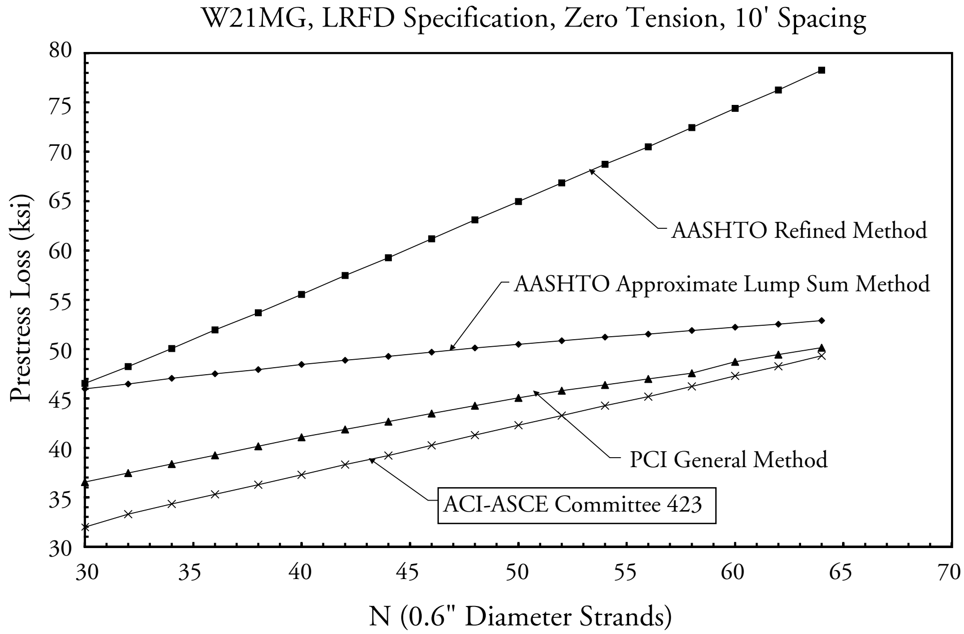
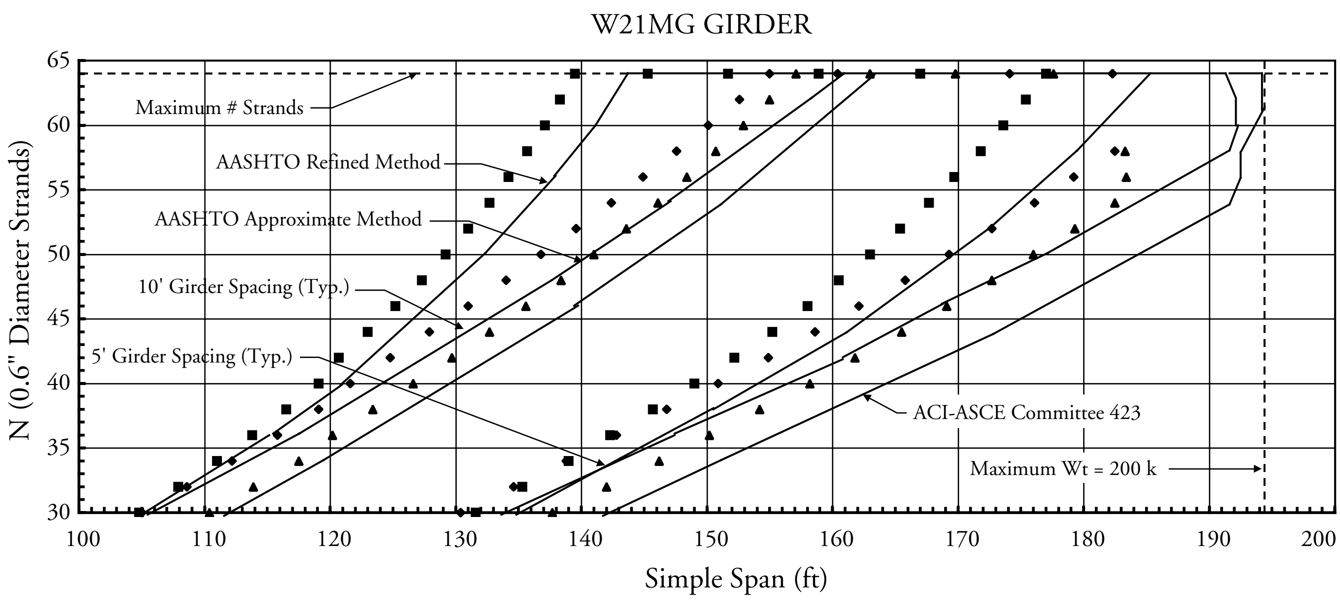


Figure 8.6.10.4-2 Span Capability vs. Pretension Loss Comparison



8.6.11
LRFD Specifications Example

Repeat Example 8.6.9, using the *LRFD Specifications*, with the above refinements:

Use the method described in Section 8.6.7 to estimate the elastic shortening loss with the same assumption in Section 8.6.7.2.

$\alpha = 0.1185$ (see Example 8.6.7.2)
$f_{pi} = 200.9$ ksi
$f_{po} = 182.7$ ksi
$\Delta f_{pES} = 200.9 - 182.7 = 18.20$ ksi

$$f_{cgp} = \frac{4{,}054}{28{,}500}(18.20) = 2.59 \text{ ksi}$$

Loss due to shrinkage of concrete:
$\Delta f_{pSR} = 17.0 - 0.150(70) = 6.50$ ksi

Loss due to creep of concrete:

$$\Delta f_{cdp1} = \frac{(6{,}379)(16.413)}{125{,}390} = 0.835 \text{ ksi}$$

$$\Delta f_{cdp2} = \frac{(2{,}280)(32.163)}{382{,}372} = 0.192 \text{ ksi}$$

$\Delta f_{pCR} = (12.0)(2.59) - (7.0)(0.835 + 0.192) = 23.89$ ksi

Finally, calculate the relaxation loss that occurs after transfer of prestress:
$\Delta f_{pR2} = 6.0 - (0.12)(18.20) - (0.06)(6.50 + 23.89) = 1.99$ ksi

The total loss of prestress, according to the *LRFD Specifications*, is:
$\Delta f_{pT} = 18.20 + 6.50 + 23.89 + 1.99 = 50.58$ ksi

8.6.12 Losses by the Tadros Method

In addition to the methods defined above from the *Standard Specifications* and *LRFD Specifications*, there are other rational methods to calculate losses that can be used for a wide variety of structures. Once such method (Tadros, et al, 1985), is based on the age-adjusted effective modulus that will be described in Section 8.13. While the Tadros Method is capable of considering the presence of both prestressed and non-prestressed steel, the following equations will be limited to the situation where only prestressing steel is considered.

Step 1 in the Tadros Method is to calculate loss due to elastic shortening. It is recommended that Eqs. (8.6.7.1-1) through (8.6.7.1-3) be used for this purpose.

The following steps in the method require the use of a coefficient, K, from Eq. (8.6.12-1). The coefficient K is used to adjust the losses due to shrinkage, creep and relaxation to reflect the small regain in steel stress that is due to the continuous interaction between the concrete and steel components of the member. The value of K to be used for time-dependent losses is:

$$K = \frac{1}{1 + \frac{E_p}{E^*_c}\frac{A_p}{A}\left(1 + \frac{e_p^2}{r^2}\right)} \qquad \text{(Eq. 8.6.12-1)}$$

where

E^*_c = age-adjusted effective modulus of the concrete for a gradually applied load at the time of transfer of prestress

e_p = eccentricity of the prestressing strands with respect to the centroid of the section

Refer to Section 8.13 for more information on the age-adjusted effective modulus.

Loss due to shrinkage of concrete is given by:

$$SH = K\varepsilon_{shu} E_p \qquad \text{(Eq. 8.6.12-2)}$$

where ε_{shu} = ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity

Loss due to creep of concrete is given by:

$$CR_C = E_p\left[KC_u\frac{f_{cir}}{E_{ci}} - \left(1 + KC'_u\right)\frac{f_{cds}}{E_c}\right] \qquad \text{(Eq. 8.6.12-3)}$$

where

f_{cir} and f_{cds} are as defined for the *Standard Specifications* method

C'_u = ultimate creep coefficient for concrete at time of application of the superimposed dead loads

Relaxation loss is given by:

$$CR_S = \psi KL_r \qquad \text{(Eq. 8.6.12-4)}$$

The relaxation reduction factor, ψ, reflects the fact that the actual relaxation will be less than the intrinsic relaxation due to the steady decrease in strand stress due to shrinkage and creep losses:

$$\psi = 1 - 3\left(\frac{SH + CR_C}{f_{po}}\right) \qquad \text{(Eq. 8.6.12-5)}$$

8.6.12.1 Tadros Method Example

Use the Tadros Method to calculate loss of prestress for the beam in Examples 8.6.7.2, 8.6.9 and 8.6.11. The following information is given:

E_{ci} = 4,054 ksi

E_c = 4,734 ksi

C_u = 1.4

H = 70%

C'_u = 1.0

ε_{shu} = 0.0004 in./in.

$$r^2 = \frac{I}{A} = \frac{125{,}390}{560} = 223.91 \text{ in.}^2$$

χ = aging coefficient = 0.7 which may be assumed constant for all precast, pretensioned concrete applications.

First, calculate the age-adjusted effective modulus for the beam concrete for loads that are gradually applied beginning at the time of transfer of prestress:

$$E^*_c = \frac{E_{ci}}{(1+\chi C_u)} = \frac{4{,}054}{1 + (0.7)(1.4)} = 2{,}047 \text{ ksi} \qquad \text{(Eq. 8.6.12.1-1)}$$

Next, calculate K from Eq. (8.6.12-1):

$$K = \frac{1}{1 + \left(\frac{28{,}500}{2{,}047}\right)\left(\frac{4.284}{560.0}\right)\left[1 + \frac{(16.413)^2}{223.91}\right]} = 0.8099$$

Loss due to shrinkage of concrete is calculated by Eq. (8.6.12-2):

SH = (0.8099)(0.0004)(28,500) = 9.23 ksi

Loss due to creep of concrete is calculated by Eq. (8.6.12-3):

f_{cir} = 2.589 ksi (see Example in Sect. 8.6.9)

f_{cds} = 1.027 ksi (see Example in Sect. 8.6.9)

$$CR_C = 28{,}500\left[(0.8099)(1.4)\frac{2.589}{4{,}054} - [1+(0.8099)(1.0)]\frac{1.027}{4{,}734}\right] = 9.45 \text{ ksi}$$

Finally, calculate the relaxation loss occurring after transfer. For the total intrinsic relaxation loss, assume a structure life of 50 years.

The stress immediately after transfer is (see Example 8.6.7.2):

f_{po} = 182.7 ksi

Using Equation (8.6.5.3-1):

$$L_r = \frac{182.7}{45}\left[\frac{182.7}{243.0} - 0.55\right]\log_{10}\left[24(50)(365)\right] = 4.62 \text{ ksi}$$

Since the prestressing steel is not isolated, but rather embedded in a creeping and shrinking concrete, its intrinsic loss must be reduced by the factor calculated from Eq. (8.6.12-5):

$$\psi = 1 - 3\left[\frac{9.23 + 9.45}{182.7}\right] = 0.693$$

Therefore the loss of prestress due to steel relaxation, given by Eq. (8.6.12-4) is:

CR_s = (0.693)(0.8099)(4.62) = 2.60 ksi

Using the Tadros Method, the total loss of prestress that occurs after transfer is:

Δf_{p2} = 9.23 + 9.45 + 2.60 = 21.28 ksi

The net effective prestress, therefore, is:

f_{pe} = 182.7 – 21.28 = 161.42 ksi

For comparison with the previous examples, the total loss of prestress is:

Δf_p = 202.50 – 161.42 = 41.08 ksi

This value is lower than the values produced by both the *Standard* and *LRFD Specifications*. It implies that the lump sum loss of 45 ksi allowed by the *Standard Specifications* is more accurate for this example than the detailed methods of both specifications. The result of this example compares almost exactly with the results of a multiple time-step analysis performed by the program TDA (Gallt, 1996). The computer analysis, using identical parameters, yields an estimated loss at midspan of 41.33 ksi, which includes relaxation loss before transfer.

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A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

NOTATION

DESIGN THEORY AND PROCEDURE

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone. [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer —

f_{pj} = stress in the pretensioning steel at jacking [LRFD]

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = modulus of rupture of concrete [STD], [LRFD]

f_s = allowable stress in steel under service loads —

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress —

f_{si} = effective initial pretension stress —

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

$f(t_j)$ = stress at time t_j —

$f_r(t,t_0)$ = relaxation stress at a certain time —

$f(t_0)$ = tensile stress at the beginning of the interval —

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity [LRFD]

h = length of a single segment —

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_{cg} = height of center of gravity of beam above road —

h_d = deck thickness —

h_f = compression flange depth [LRFD]

h_r = height of roll center above road —

I = moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam [STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.7 CAMBER AND DEFLECTION

Generally, there are three sets of beam deformations of interest to the designer:

- vertical deflections (typically at midspan)
- end rotations
- axial shortening

Of these, midspan deflection, or camber, is usually of greatest interest. Excessive, unexpected camber at the time of erection may require adjustment of bridge grades to prevent intrusion of the beam top flange into the deck. Additionally, estimates of the final midspan deflections under the action of permanent dead load and live load may be required to ensure serviceability of the bridge.

End rotations are of importance when continuity is introduced at the time of casting the deck. When these rotations are restrained or partially restrained by adjacent spans, secondary time-dependent stresses are introduced in the structure. These stresses must be considered in the design of connections and detailing of the end regions of beams.

Finally, axial shortening of precast, prestressed bridge members must be considered when designing bearings and expansion devices. This information is also helpful in assessing the impact of superstructure restraint against shortening in jointless bridge systems.

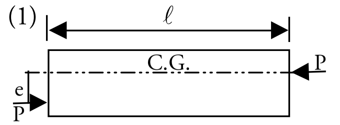
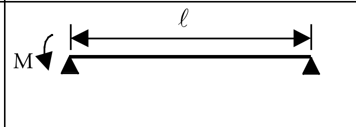
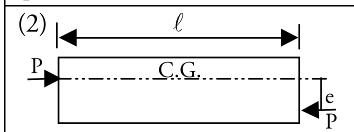
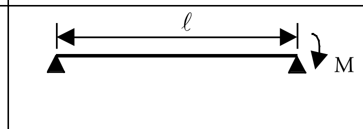
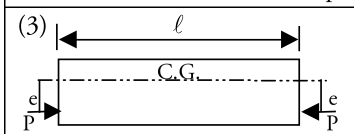
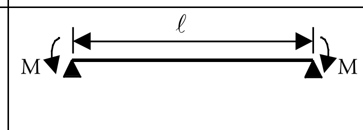
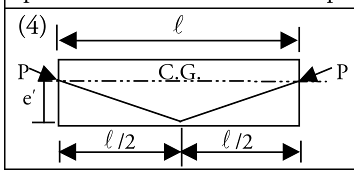
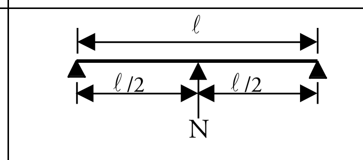
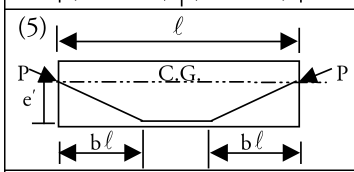
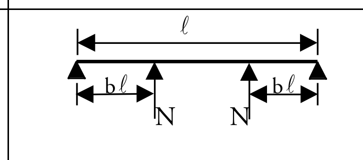
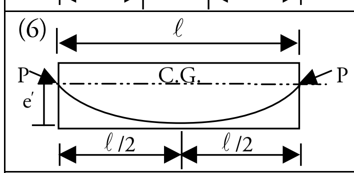
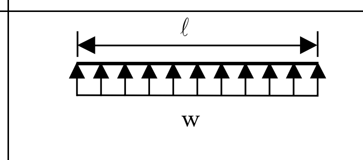
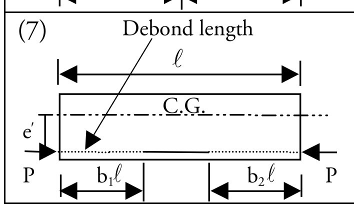
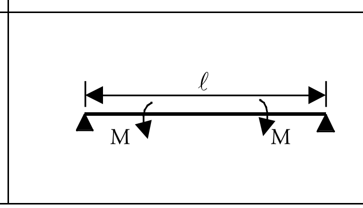
This section discusses the computations of camber and deflection including the changes that occur in these quantities with time. The methods that are available to estimate long-term cambers and other deflections of precast, prestressed members fall into three categories, listed in order of increasing complexity and accuracy:

- multiplier methods
- improved multiplier methods, based on estimates of loss of prestress
- detailed analytical methods

Camber in a prestressed beam occurs immediately upon the transfer of the prestressing force. The magnitude of the initial camber is dependent on the length, weight and moment of inertia of the member; the modulus of elasticity of the concrete; and the arrangement and amount of prestressing. Values for several prestressing arrangements are give in **Table 8.7-1**. The modulus of elasticity of the concrete usually cannot be predicted with precision at the time of the design of the member. The standard prediction formulas are based on values assumed by the designer for concrete unit weight and strength at the time of prestress transfer. These assumed values do not include actual material properties, nor account for such important factors as type of aggregates and ratio of coarse-to-fine aggregate. For these reasons, initial camber predictions using assumed material properties must be regarded as estimates and the designer is cautioned against placing a high degree of confidence in calculated initial cambers.

After release, camber generally increases with time. Creep of the concrete is primarily responsible for this camber growth. Simultaneously, the gradual loss of prestress due to creep, shrinkage and strand relaxation has the effect of reducing the initial rate of growth of camber. The magnitude and rates of both creep and shrinkage, and therefore changes in camber, are affected by environmental conditions such as ambient relative humidity and temperature.

Table 8.7-1
*Camber (deflection) and rotation coefficients for prestress force and loads**

Prestress Pattern	Equivalent Moment or Load	Equivalent Loading	Camber	End rotation	
(1) 	M = Pe		$+\dfrac{M\ell^2}{16EI}$	$+\dfrac{M\ell}{3EI}$	$-\dfrac{M\ell}{6EI}$
(2) 	M = Pe		$+\dfrac{M\ell^2}{16EI}$	$+\dfrac{M\ell}{6EI}$	$-\dfrac{M\ell}{3EI}$
(3) 	M = Pe		$+\dfrac{M\ell^2}{8EI}$	$+\dfrac{M\ell}{2EI}$	$-\dfrac{M\ell}{2EI}$
(4) 	$N = \dfrac{4Pe'}{\ell}$		$+\dfrac{N\ell^3}{48EI}$	$+\dfrac{N\ell^2}{16EI}$	$-\dfrac{N\ell^2}{16EI}$
(5) 	$N = \dfrac{Pe'}{b\ \ell}$		$+\dfrac{b(3-4b^2)N\ell^3}{24EI}$	$+\dfrac{b(1-b)N\ell^2}{2EI}$	$-\dfrac{b(1-b)N\ell^2}{2EI}$
(6) 	$w = \dfrac{8Pe'}{\ell^2}$		$+\dfrac{5w\ell^4}{384EI}$	$+\dfrac{w\ell^3}{24EI}$	$-\dfrac{w\ell^3}{24EI}$
(7) 	$M = Pe'$		$+\dfrac{M\ell^2}{8EI}(1-2b_1^2-2b_2^2)$	$+\dfrac{M\ell}{2EI}\left[(1-2b_1)^2-b_2^2\right]$	$-\dfrac{M\ell}{2EI}\left[(1-2b_1)^2-b_2^2\right]$

* The tabulated values apply to the effects of prestressing. By adjusting the directional rotation, they may also be used for the effects of loads. For patterns 4 to 7, superimpose on 1, 2 or 3 for other C.G. locations

From the preceding discussion, it should be obvious that the task of predicting both initial camber and the growth of camber with time is difficult because of the large number of random variables that affect this behavior which are beyond the designer's control. Estimates of these effects should be recognized as being approximations only.

8.7.1 Multiplier Method

Perhaps the most popular method for predicting time-dependent camber of precast, prestressed members is the set of multipliers given in **Table 8.7.1-1** (Martin, 1977). The use of this method is fairly straightforward. First, elastic deflections caused by the effects of prestressing, beam self-weight, and other dead loads are calculated using conventional elastic analysis techniques. These are multiplied by the appropriate factors selected from **Table 8.7.1-1** to determine the deflections that will occur as a result of time-dependent behavior.

Table 8.7.1-1 Suggested Multipliers to be Used as a Guide in Estimating Long-Term Cambers and Deflections for Typical Members

		Without Composite Topping	With Composite Topping
	At erection:		
(1)	Deflection (↓) component – apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
(2)	Camber (↑) component – apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
	Final:		
(3)	Deflection (↓) component – apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40
(4)	Camber (↑) component – apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
(5)	Deflection (↓) component – apply to elastic deflection due to superimposed dead load only	3.00	3.00
(6)	Deflection (↓) component – apply to elastic deflection caused by the composite topping	---	2.30

This method gives reasonable estimates for cambers at the time of erection. The method does not, however, properly account for the significant effects of a large cast-in-place deck. The presence of a deck, once cured, drastically changes the stiffness of a typical bridge member. This has the effect of restraining the beam creep strains that are the result of prestressing, member self-weight, and the dead load of the deck itself. Also, differential creep and shrinkage between the precast and the cast-in-place concretes can produce significant changes in member deformation. The multipliers for long-term deflection suggested by this method, therefore, should not be used for bridge beams with structurally composite cast-in-place decks.

In addition, it is not recommended that prestressing levels be increased in order to reduce or eliminate long-term downward deflection that might be predicted if the multipliers in **Table 8.7.1-1** are used.

8.7.2 Improved Multiplier Method

An improved multiplier method (Tadros, et al, 1985) is similar, in many respects, to the method described in the preceding section. However, this method provides two improvements:

1. If reliable estimates are available for the creep coefficient of the actual concrete mix, or if high performance concrete with a significantly lower creep coefficient is used, these more accurate coefficients may be substituted;
2. The component of deflection due to the loss of prestress can be based on the actual calculated value for that loss.

The recommended multipliers for this method are given in **Table 8.7.2-1**.

Table 8.7.2-1 Time-Dependent Multipliers for Deflections Using the Improved Multiplier Method

	Erection Time		Final Time	
Load condition	Formula	Average	Formula	Average
Initial prestress	$1+C_a$	1.96	$1+C_u$	2.88
Prestress loss	$\alpha(1+\chi C_a)$	1.00	$(1+\chi C_a)$	2.32
Self-weight	$1+C_a$	1.96	$1+C_u$	2.88
Dead load on plain beam	1.00	1.00	$1+C'_u$	2.50
Dead load on composite beam	1.00	1.00	$1+C'_u$	2.50

C_u = ultimate creep coefficient for loads applied immediately after transfer. Average value is 1.88.

C'_u = ultimate creep coefficient for loads applied at time of erection. Average value is 1.50.

C_a = creep coefficient for loading applied immediately after transfer and strains measured at time of erection. Average value is 0.96.

α_a = time-dependent prestress loss at erection divided by total time-dependent prestress loss. Average value is 0.60.

χ = Bazant's aging coefficient. Average value is 0.70.

Average values are based on 70 percent relative humidity, average member thicknesses, concrete age at release of 1 to 3 days and age at erection of 40 to 60 days.

Table 8.7.2-1 provides creep multipliers to be applied to the calculated deflections due to initial prestress, loss of prestress and dead loads.

Once the time-dependent loss of prestress has been determined (using any one of the available methods for estimating losses), an effective negative prestressing force, ΔP_c, is calculated as the product of the losses and the area of the prestressing material. The deflection due to this apparent negative prestressing force is calculated in the same way that the initial deflection due to transfer of prestress is calculated (see **Table 8.7-1**).

Note that only the time-dependent portion of the loss of prestress is considered in the calculation for ΔP_c. Losses that occur before or at release, such as elastic shortening and the portion of relaxation that occurs prior to release, are not included.

Unfortunately, the multipliers given by Tadros, et al (1985) are limited to totally precast concrete members. Therefore, for bridges with composite cast-in-place concrete decks, use of these multipliers should be limited to estimating deflections before the cast-in-place deck begins to act compositely with the precast concrete beam. A need still exists for a simple multiplier procedure for use in estimating the final time-dependent deflection. If necessary, the detailed computer analysis methods of Section 8.13 may be employed.

Note that the final time-dependent deflection is not required to be checked by the Specifications. The only value of computing final deflection is to ensure that the structure does not develop excessive sag. Of course, instantaneous deflection due to live loads should be limited as a serviceability criterion. However, elastic live load deflection can be determined by conventional structural analysis techniques, and vibration of conventionally designed and constructed prestressed concrete bridges has not been reported to be a problem.

8.7.3 Examples

Calculate initial and erection cambers, as well as the immediate camber after construction of the deck for the beam shown in **Figure 8.6.7.2-1**. Use both the multiplier method and the improved multiplier method.

8.7.3.1 Multiplier Method Example

Use the following information to calculate initial camber:

Span = 65.00 ft

Self-weight = 583 plf

E_{ci} = 4,054 ksi

P_o = (182.7)(4.284) = 782.7 kips (Example 8.6.7.2)

Using **Table 8.7-1**, Case 5:

b = 26.67/66.67 = 0.4

e′ = 16.413 − 11.556 = 4.857 in.

$$N = \frac{(782.7)(4.857)}{(12)(0.4)(65)} = 12.18 \text{ kips}$$

$$\text{camber} = \frac{(0.4)[3-4(0.4)^2](12.18)[(65)(12)]^3}{(24)(4{,}054)(125{,}390)} = 0.447 \text{ in.} \quad \uparrow$$

Using **Table 8.7-1**, Case 3:

e = 11.556 in.

M = (782.7)(11.556) = 9,045 in.-kips

$$\text{camber} = \frac{(9{,}045)[(65)(12)]^2}{(8)(4{,}054)(125{,}390)} = 1.353 \text{ in.} \quad \uparrow$$

Total initial camber due to prestress:

camber = 0.447 + 1.353 = 1.800 in. ↑

Deflection due to self-weight:

$$\Delta = \frac{-(5)(0.583/12)[(65)(12)]^4}{(384)(4{,}054)(125{,}390)} = -0.461 \text{ in.} \quad \downarrow$$

So, net camber at transfer = 1.800 – 0.461 = 1.339 in. ↑

Applying the multipliers at erection from **Table 8.7.1-1** to the initial values computed above, erection camber = (1.80)(1.800) – (1.85)(0.461) = 2.387 in. ↑

Deflection due to deck, haunch, and diaphragm weights:

E_c = 4,734 ksi

$$\Delta = \frac{-(5)[(0.867+0.040+0.100)/12][(65)(12)]^4}{(384)(4{,}734)(125{,}390)} = -0.681 \text{ in.} \downarrow$$

Net camber immediately after application of deck weight:

Camber = 2.387 – 0.681 = 1.706 in. ↑

8.7.3.2
Improved Multiplier Method Example

Use of this method allows refinement based on actual creep characteristics. For this example, C_u = 1.4 and C'_u = 1.0 (see the Example of Sect. 8.6.12.1). C_a, the creep coefficient for deflections at time of erection (90 days) due to loads applied at release, is calculated using prediction equations to be 0.84.

Assumed values of the aging coefficient, χ = 0.7, and ratio of prestress loss at erection to total prestress loss, α_a = 0.6 are good average values for most precast, pretensioned concrete bridge applications.

At time of erection:

Load	Deflection	Multiplier		Result	
initial prestress	1.800	1 + 0.84	= 1.84	3.312	↑
prestress loss	–0.210	(0.6)[1 + 0.7(0.84)]	= 0.95	–0.200	↓
self-weight	–0.461	1 + 0.84	= 1.84	–0.848	↓
dead loads on plain beam	–0.681		= 1.00	–0.681	↓

Net camber immediately after application of all permanent loads = 1.583 in.↑

Note that the deflection due to prestress loss is obtained using simple proportioning of the initial prestress camber, (–21.28/182.7)(1.800) = –0.210 in. See Example 8.6.12.1, Δf_{p2}, for value of prestress loss.

8.7.4
Camber and Deflection Estimates Using Numerical Integration

For structures that are expected to be particularly sensitive to time-dependent deflection such as highly prestressed slender members, and when the designer has access to reliable estimates for creep and shrinkage properties of the concrete, the methods of cross-section analysis described in Section 8.13 can also be used to estimate deflections. In particular, if an estimate of long-term deflections is required, this method should be considered. As discussed above, the multiplier methods, in general, do not give good estimates of long-term deflections for systems where the cast-in-place deck represents a significant increase in the stiffness of the beam.

To estimate beam deformations, several cross-sections along the length of the member are analyzed using the methods of Section 8.13, or another method that can provide reliable time-dependent curvatures. Numerical integration of curvatures and strains along the member, using the principle of virtual work, is used to calculate rotations and deflections.

A variety of techniques are available for performing the required numerical integration. Simpson's rule is recommended because it is reasonably accurate, it is familiar to many designers, and it makes use of analyses of cross-sections that fall at regular intervals along the length of the member.

The procedure for estimating deflection using numerical integration is as follows:

1. Select equally-spaced sections for analysis. Simpson's rule requires an odd number of sections, resulting in an even number of segments. As the minimum, a section at each support and a section at midspan could be analyzed. More sections will usually result in improved accuracy. The designer should take full advantage of symmetry.

2. Perform time-dependent analyses of each of the selected sections using the creep-transformed section properties method described in Section 8.13 or another reliable method.

3. Use the principle of virtual work and Simpson's rule to determine the deflections of interest. **Figure 8.7.4-1** provides the virtual work expressions for calculating midspan deflection, end rotations, and axial shortening deformations. The following equation defines Simpson's rule:

$$S_N = \frac{h}{3}\left(f_0 + 4f_1 + 2f_2 + \ldots + 4f_{N-1} + f_N\right) \qquad \text{(Eq. 8.7.4-1)}$$

where

S_N = the value of the integral

N = number of segments between nodes (must be even number)

h = length of a single segment

f_i = value of integrated function at node i = 0, 1, 2...N-1, or N

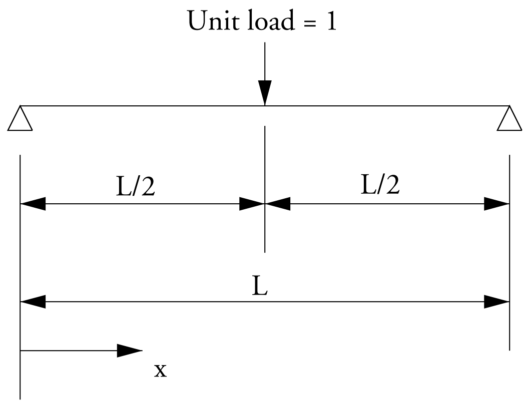
For the case of N = 2, i.e., using sections at the ends and midspan of the member only, Simpson's rule should not be used for the calculation of midspan deflection. For this special case, a better result for midspan deflection is obtained using:

$$\Delta = \left(\phi_0 + 5\phi_c\right)\frac{L^2}{48} \qquad \text{(Eq. 8.7.4-2)}$$

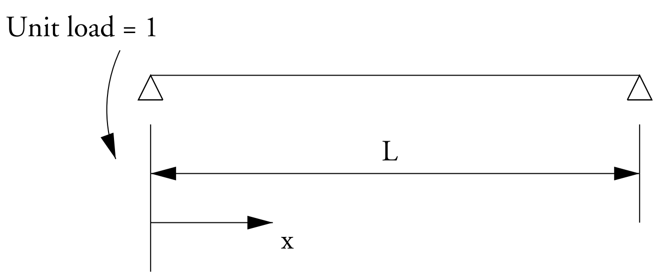
where ϕ_0 and ϕ_c are the curvatures at the support and midspan, respectively.

For calculation of end rotations and axial shortening with N = 2, and for the calculation of midspan deflection for N = 4 or more, however, Simpson's rule should be used.

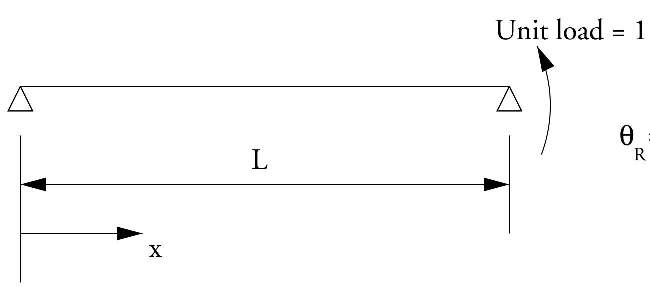
Figure 8.7.4-1
Virtual Work Expressions for Beam Deflections



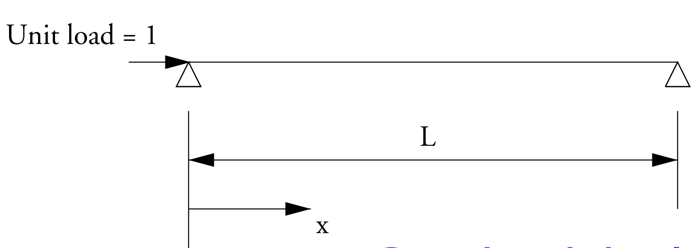
$$\Delta = \int_0^{L/2} (x/2)\phi(x)dx + \int_{L/2}^{L} (L-x)/2\phi(x)dx$$



$$\theta_L = -\int_0^L [1-(\tfrac{x}{L})]\phi(x)dx$$



$$\theta_R = -\int_0^L (\tfrac{x}{L})\phi(x)dx$$



$$\Delta x = \int_0^L \varepsilon(x)dx$$

8.7.4.1 Numerical Integration Example

A computerized time-dependent analysis of the beam in **Figure 8.6.7.2-1** has been performed using TDA (Gallt, 1996). A total of nine sections were analyzed, resulting in N = 8 for Simpson's rule integration. The following curvatures are predicted at a girder age of 90 days, immediately after the slab is placed:

Location, in.	Curvature Immediately after Slab Placement x 10^{-5} in.$^{-1}$	Final Long-Term Curvature x10^{-5} in.$^{-1}$
10.00(bearing)	2.968	2.862
107.50	2.405	2.045
205.00	2.087	1.553
302.50	2.020	1.393
400.00 (midspan)	1.942	1.281
497.50	2.020	1.393
595.00	2.087	1.553
692.50	2.405	2.045
790.00(bearing)	2.968	2.862

First calculate midspan deflections for the case of N = 2, immediatlely after application of permanent loads:

$$\Delta = [+2.968+(5)(1.942)]\text{x}10^{-5}\left[\frac{(780)^2}{48}\right] = 1.607 \text{ in.} \quad \uparrow$$

Similarly, for final deflection

$$\Delta = [+2.862+(5)(1.281)]\text{x}10^{-5}\left[\frac{(780)^2}{48}\right] = 1.175 \text{ in.} \quad \uparrow$$

Now compare these results to a more refined case where N = 8. The length of each section, h, is equal to 97.5 in.

N	x	f_N x 10^{-5}		Wt	Wt x f_N
0	0.0	1/2(0.0)(2.968) =	0.0	1	0.0
1	97.5	1/2(97.5)(2.405) =	0.0011724	4	0.004690
2	195.0	1/2(195.0)(2.087) =	0.0020348	2	0.004070
3	292.5	1/2(292.5)(2.020) =	0.0029543	4	0.011817
4	390.0	1/2(390.0)(1.942) =	0.0037869	2	0.007574
5	487.5	1/2(780.0 – 487.5)(2.020) =	0.0029543	4	0.011817
6	585.0	1/2(780.0 – 585.0)(2.087) =	0.0020348	2	0.004070
7	682.5	1/2(780.0 – 682.5)(2.405) =	0.0011724	4	0.004690
8	780.0	1/2(780.0 – 780.0)(2.968) =	0.0	1	0.0
					0.04873

Therefore, deflection after slab placement, $\Delta = \frac{97.5}{3}(0.04873) = 1.584$ in. $\uparrow$

A similar procedure is followed to estimate long-term camber:

These results show that the N = 2 case provides results that are nearly as good as the N = 8 case. For many designs the simpler N = 2 case is adequate. Camber by this method, immediately after slab placement, compares favorably with the values of 1.706 in. and 1.583 in. obtained by the more approximate multiplier method and the improved multiplier method, respectively.

The numerical integration method is attractive in that it can account for curvature variations along the span, e.g. due to cracking if cracking is allowed to take place under service loads when the member is designed to be "partially prestressed." The principal value of the method in common applications is in its ability to account for differential creep and shrinkage due to composite action and to account for continuity in continuous bridge superstructures. Curvature calculation using creep-transformed section properties is covered in Section 8.13.

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DESIGN THEORY AND PROCEDURE

A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c = age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing —

E^*_{cb} = age-adjusted, effective modulus of elasticity of the beam —

E^*_{cd} = age-adjusted, effective modulus of elasticity of the deck —

$E^*_c(t,t_0)$ = effective modulus of elasticity at certain time —

E^*_{ck} = age-adjusted, effective modulus of element k —

e = eccentricity of prestressing strands —

e_c = eccentricity of the strand at midspan —

e_g = distance between the centers of gravity of the beam and the slab [LRFD]

e_i = initial lateral eccentricity of the center of gravity with respect to the roll axis —

e_m = average accentricity at midspan [LRFD]

e_p = eccentricity of the prestressing strands with respect to the centroid of the section —

FS_c = factor of safety against cracking —

FS_f = factor of safety against failure —

F_b = allowable tensile stress in the precompressed tension zone at service loads —

F_{cj} = force in concrete for the j th component —

F_{pi} = total force in strands before release —

f = stress —

f_b = concrete stress at the bottom fiber of the beam —

f'_c = specified concrete strength at 28 days [STD]

f'_c = specified compressive strength at 28 days [LRFD]

f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied [STD]

f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer [STD]

f'_{ci} = concrete strength at transfer [STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning (transfer) [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_{cu} = the limiting concrete compressive stress for designing by strut-and-tie model [LRFD]

f_f = stress range [STD]

f_{min} = algebraic minimum stress level [STD]

f_{pbt} = stress in prestressing steel immediately prior to transfer [LRFD]

f_{pc} = compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads [STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.8 DECK SLAB DESIGN

8.8.1 Introduction

This section considers concrete slabs that act compositely with precast beams and where the slab span and main reinforcement are transverse to traffic. Cast-in-place (CIP) concrete is sometimes used as a topping on longitudinal, "full-deck" members such as adjacent box beams, double tees and deck bulb-tees. However, this type of deck slab generally does not require flexural design and is not covered in this section. The majority of deck slabs in new bridge construction use CIP concrete with or without precast stay-in-place (SIP) deck panels. The CIP topping provides flexibility to adjust for roadway profile and for differences in beam elevations. The use of precast SIP panels is gaining popularity due to their cost-effectiveness and improvement in jobsite construction safety.

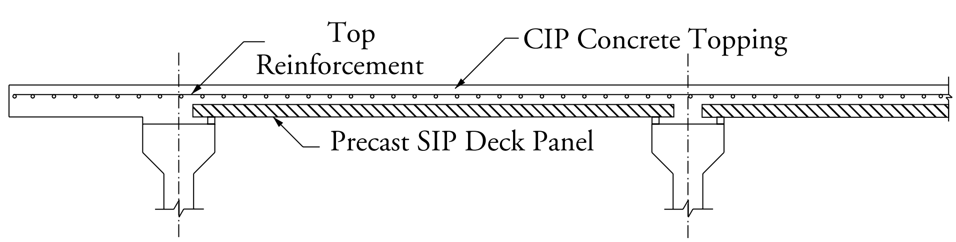
This section focuses on the design of CIP decks using precast SIP panels according to both the *Standard Specifications* and *LRFD Specifications*. In addition, a subsection summarizes the "Empirical Design Method" of full-depth CIP slabs. This method is becoming more popular due to the relatively small amount of reinforcement it requires. However, at this time, the *LRFD Specifications* do not permit this method for design of precast SIP deck panel systems. Also in this section, two new precast concrete deck systems will be introduced. The first system is an improved SIP panel that allows for better construction speed and structural performance than for the conventional SIP panel system. The second is a full-depth precast, prestressed concrete panel that is best suited for rapid replacement of high-traffic bridge decks.

8.8.2 Design of Bridge Decks Using Precast Panels

A precast SIP deck panel system typically consists of thin precast, concentrically prestressed, concrete panels which span between supporting beams, and a CIP concrete topping which acts compositely with the SIP panels to form the fully composite deck. Precast concrete panels as thin as 2.5 in. have successfully been used. Because most panels are thin, strict quality control practices are recommended to avoid panel cracking or camber (PCI Committee Report, 1988). The prestress force should be released as gradually as possible. The strands should be maintained concentric with the concrete cross-section. Research by Kumar (1996), has shown that prestressed SIP deck panels with a 0.05 to 0.075-in. amplitude, broom-finished surface do not require horizontal shear connectors to achieve full composite action with the CIP topping providing the nominal horizontal shear stress is less than 116 psi.

Positive moment sections between the supporting beams are designed as prestressed concrete composite sections with the prestressing strands as the main reinforcement. Negative moment sections over beam lines are designed as conventionally reinforced sections with the reinforcing bars in the topping slab as the main reinforcement as shown in **Figure 8.8.2-1**.

Fig. 8.8.2-1 Cross-Section of CIP Deck with Precast SIP Panel



8.8.2.1 Determining Prestress Force

The first step in design is to estimate the required amount of prestress force. This estimate is governed by the allowable tensile stress in the precast SIP panel due to service loads at the maximum positive moment section. The weight of the precast SIP deck panel and the CIP topping act on the non-composite section, i.e. the precast SIP deck panel alone. The superimposed dead loads (wearing surface, barriers, etc.) and live loads act on the precast SIP panel-CIP topping composite section. After the required prestress force is determined, the unfactored load stresses and the ultimate flexural capacity at various construction stages are checked.

8.8.2.2 Service Load Stresses and Flexural Strength

Service load stresses should be checked in the panel and in the completed deck at a number of stages. The first stage is at the time of prestress transfer or release. The strands are normally concentric within the precast SIP deck panel. Therefore, prestress introduces uniformly distributed compressive stresses. However, accidental misplacement of the strands may be conservatively assumed to produce 0.25 in. prestress eccentricity.

The second loading stage occurs at the time of topping placement. Loads at this stage act only on the SIP panel. Service load stresses and ultimate capacity of the precast SIP panel should be checked due to the weight of the precast panel and the CIP topping in addition to a construction load, estimated as 50 psf unless a more accurate figure is available. The construction load represents people, material and equipment used to place, finish and cure the topping but it does not include concentrated loads representing finishing machine reactions. Special brackets directly supported on beam seats are used to resist finishing machine loads. Other loads at this stage act only on the SIP panel.

The third loading stage occurs after the CIP topping cures and the superimposed loads are introduced. At this stage, the stresses are calculated using a transformed section analysis similar to that done in composite I-beam analysis. The ultimate strength of the composite section at the maximum positive moment section, is checked against factored dead and live loads. One of the most important issues in determining the flexural strength of the positive moment section is the strand development length. Since the strands are terminated at panel ends over beam lines, the maximum positive moment sections may be closer to the end of the panel than the development length of the strands. Thus, only partial strand development can be expected. In this situation, the stress in the pretensioning steel at factored load, f^*_{su}, should be limited to:

$$f^*_{su} = \left(\frac{L_x}{D} + \frac{2}{3} f_{se}\right) \qquad \text{[STD Art. 9.17.4.2]}$$

where

f^*_{su} = average stress in pretensioning steel at ultimate load

L_x = distance from end to center of the panel, in.

D = nominal diameter of the strand, in.

f_{se} = effective final pretension stress (after losses), ksi

Non-prestressing reinforcement provided in the CIP topping, is determined on the basis of flexural strength. The critical sections over interior beams are designed for superimposed dead and live loads. In addition, the region near the exterior beams should be designed for crash loading combined with dead and live loads. Design Examples 9.7 and 9.8 in Chapter 9, give complete details of the design of overhangs for this type of loading. In negative moment zones, proper distribution of the flexural reinforcement is required to control top fiber cracking.

8.8.2.3 Standard Specifications

8.8.2.3.1 Minimum Thickness

Table 8.9.2 in the *Standard Specifications* gives the minimum recommended total deck thickness that can be used without deflection calculations. Thinner slabs may be used if the deflection due to service live load plus impact is limited to 1/800 of the span for spans between supporting beams and 1/300 of the cantilever arm for cantilevers (STD Art. 8.9.3).

Some agencies specify that 1/2 to 3/4 in. of the slab thickness be considered as an integral layer of wearing surface. In this case, the full thickness should be used for calculation of loads and the reduced thickness for determining section properties.

The Standard Specifications do not give any recommendations concerning the thickness of SIP panels. Although thinner panels have been successfully used, common practice indicates that 3.0 in. and 3.5 in., with 3/8 in. and 1/2 in. diameter strands respectively, are acceptable minimum thicknesses.

8.8.2.3.2 Live Load

STD Article 3.7 specifies the standard truck load that should be used as a live load to compute the bending moments at any section. There are four standard classes of highway loading: H15, H20, HS15 and HS20. Most state agencies use the HS20 standard truck, although some states use the non-standard HS25 truck, which is 125% of the HS20 standard truck. The bending moment due to

$$\text{truck loading at various locations} = \left(\frac{S+2}{32}\right)P \qquad \text{[STD Eqs. 3.15 and 3.16]}$$

where

S = effective span length, ft

P = load on rear wheel of truck = 16,000 lb for H20 and HS20

The effective span length for continuous span slabs, as specified in STD Article 3.24.1.2, depends on the slenderness of the top flange of the support. For slender top flanges, where the ratio of top flange width to minimum thickness of the flange is greater than 4.0, S is taken as the distance between edges of top flange plus one-half of the width of beam top flange. If the ratio of top flange width to minimum thickness is less than 4.0, S is taken as the distance between edges of the top flange.

A continuity factor of 0.8 is applied to the live load bending moment for slabs that are made continuous over three or more supports. The live load moments are increased to allow for impact effects, as given in STD Article 3.8.2, which always results in an impact fraction of 0.3. Each section of the deck should be designed so that the factored moment due to applied loads is less than or equal to the capacity of the section.

8.8.2.3.3 Reinforcement Requirements

At negative moment locations, minimum reinforcement should be provided so that the design moment capacity is not less than 1.2 times the cracking moment (STD Art. 8.17.1). Alternatively, according to STD Article 8.17.1.2, the minimum reinforcement requirement may be satisfied by providing at least one-third more reinforcement than required by analysis.

At positive sections, a minimum reinforcement of 0.11 in.^{2}/ft in the SIP panel is specified (STD Art. 9.24.2). Also, STD Article 9.18.2.4 states that a minimum amount of longitudinal reinforcement of 0.25 in.^{2}/ft should be provided in the CIP section for lateral distribution of concentrated loads and for crack control.

STD Article 8.22 specifies a minimum concrete clear cover of 2.0 in. to the top reinforcement for deck slabs in mild climates and 2.5 in. for deck slabs exposed to deicing salts. For bottom reinforcement, the minimum specified clear cover is 1.0 in.

8.8.2.3.4 Shear Design

Article 3.24.4 in the *Standard Specifications* states that slabs designed for bending moment in accordance with STD Article 3.24.3 need not be checked for shear or bond. If elected by the designer, one-way and two-way shear capacities can be checked as given in STD Articles 8.16.6.2.1 and 8.16.6.6.2 respectively. Two-way shear is calculated assuming that the tire contact area is rectangular where the dimensions of the contact area depend on the wheel load of the specified truck (STD Art. 3.30).

8.8.2.3.5 Crack Control

The negative moment areas over the beams, designed as conventionally reinforced concrete, are expected to crack under service load conditions. Thus, crack control criteria given in STD Article 8.16.8.4 must be satisfied. In areas where deicing salts are used, a number of state agencies specify that slab reinforcement must be epoxy-coated. The *Standard Specifications* allow the use of epoxy-coated reinforcement. However, STD Article 8.25.2.3 requires that the basic development length should be increased by a factor of 1.15 or 1.5 depending on the concrete cover and the clear spacing between epoxy-coated bars.

8.8.2.4 LRFD Specifications

An entire section in the *LRFD Specifications*, Section 9, is devoted to deck systems. Three levels of analysis are permitted in the *LRFD Specifications*:

(1) refined analysis

(2) approximate analysis, generally known as the Strip Method

(3) the Empirical Method

As noted earlier, the specifications do not permit the empirical method to be used in the design of SIP panel deck systems.

8.8.2.4.1 LRFD Specifications Refined Analysis

LRFD Articles 4.4 and 4.6.3.2 allow the use of refined methods of analysis. These methods should satisfy the requirements of equilibrium and compatibility and utilize stress-strain relationships for the proposed materials. Refined analysis methods include, but are not limited to:

(1) grillage analogy method

(2) finite strip method

(3) finite element method

However, some conditions should be considered which accurately model the behavior of the deck slabs as observed in actual bridges. These conditions are:

(1) flexural and torsional deformation of the deck in skewed bridges

(2) in-plane shear deformation, which affects the effective width of composite bridge decks

(3) locations of flexural discontinuity through which shear is transmitted, should be modeled as hinges

(4) wheel loads should be modeled as patch loads over the tire contact area, given by the specifications, extended by half of the deck depth on all four sides

A structurally continuous railing, barrier or median, acting compositely with the supporting components, can be considered to be structurally active at service and fatigue limit states.

8.8.2.4.2
LRFD Specifications Strip Method

In this method, the deck slab is divided into strips perpendicular to the supporting beams. To calculate the bending moments and shear forces, the strips are treated as a continuous member and the supporting beams are assumed to be infinitely rigid. The width of the strip is determined so that the effects of flexure in the secondary direction and of torsion are accounted for to obtain flexural force effects approximating those that would be provided by refined methods of analysis. However, the strip method model was developed based on non-skewed bridges, thus, more accurate analysis may be warranted for end zones of skewed bridges.

8.8.2.4.2.1
Minimum Thickness

LRFD Article 9.7.1.1 states that the depth of the concrete deck, excluding any provision for grinding, grooving and sacrificial surface, should not be less than 7.0 in. LRFD Article 2.5.2.4 states that concrete decks without an initial overlay should have an additional thickness of 1/2 in. to allow for correction of the deck profile by grinding and to compensate for thickness loss due to abrasion. For concrete deck overhangs which support a deck-mounted post system or concrete parapets or barriers, a minimum depth of 8.0 in. is required (LRFD Art. 13.7.3.1.2). LRFD Article 9.7.4.3.1 states that the thickness of the precast SIP deck panel should neither exceed 55% of the total slab depth nor be less than 3.5-in. thick. However, as noted earlier, SIP panels 3.0 in. thick or even as thin as 2.5 in. have been used in recent years with satisfactory performance.

8.8.2.4.2.2
Minimum Concrete Cover

LRFD Article 5.12.3 provides minimum concrete cover requirements similar to those given in the *Standard Specifications*. The minimum cover values are stated for concrete mixes with water-cement ratios from 0.40 to 0.50. For concrete mixes with different water-cement ratios, a modification factor is provided. When epoxy-coated bars are used, the *LRFD Specifications* allow the minimum cover requirement for uncoated bars in interior exposure to be used. However, special provisions for development length and lap splices for coated reinforcement must be satisfied as given in LRFD Article 5.11.

8.8.2.4.2.3
Live Load

The standard live load used in the *LRFD Specifications* is the HL-93, which consists of the combination of a design truck or tandem, and a design lane load (LRFD Art. 3.6.1.2). However, LRFD Article 3.6.1.3.3 states that for deck slabs where the strips are in the transverse direction of the bridge and their span does not exceed 15 feet, only the wheels of the 32.0-kip axle of the design truck, spaced at 6.0 ft, need be considered. If the transverse strip span exceeds 15 feet, both the 32-kip axle and the design lane load should be used. One or more design lanes may be assumed to be loaded simultaneously. Within each design lane, the truck axle can be positioned so that the center of a wheel is not closer than 2.0 ft to the edge of the lane nor closer than 1.0 ft to the face of the curb or rail. The location of the design lanes can be shifted laterally relative to the longitudinal axis of the deck, to produce the maximum force effects. LRFD Article 4.6.2.1.3 gives the width of the strip in inches as:

Width = 26.0 + 6.6S (for positive moment calculations)

Width = 48.0 + 3.0S (for negative moment calculations)

where S = span of the slab between beam centerlines, ft

For overhangs, the negative moment strip width is:

Width = 45.0 + 10.0X

where X = distance from the wheel load to point of support, ft

These equations are based on three-dimensional finite element analyses of a large number of one- and two-span bridges covering the practical range of number of beams, beam stiffness, beam spacing, span length, and slab thickness. Because a three-dimensional analysis was used to develop the strip width equations, the effects of flexure in the secondary direction and torsion are already accounted for. The maximum positive and negative bending moments calculated using the strip method are considered to apply to all regions along the bridge length. Dynamic allowance of 33% and a multiple presence factor as specified in LRFD Articles 3.6.2 and 3.6.1.1.2 should be included in design. Table A4.1-1 in Appendix A4 of the *LRFD Specifications* gives the maximum design bending moment values for different beam arrangements, where the multiple presence factors and the dynamic load allowance are included in the tabulated values. Appendix A4 is applicable only to bridge decks supported on at least three parallel beams and having a width measured between the centerlines of the exterior beams not less than 14 ft.

8.8.2.4.2.4 Location of Critical Sections

For precast I-beam bridges, the location of the design section for negative moments and shear forces may be taken as one-third of the flange width, but not more than 15 in. from the support centerline (LRFD Art. 4.6.2.1.6).

8.8.2.4.2.5 Design Criteria

LRFD Article 9.7.4.1 states that prestressed concrete stay-in-place panels should to be elastic under construction loads. Construction loads include the weight of the SIP panel, weight of the CIP topping, and an additional 0.050 ksf. Flexural stresses due to unfactored construction loads should not exceed 75% of the steel yield strength or 65% of the 28-day compressive strength for concrete in compression, or the modulus of rupture for concrete in tension. Also, LRFD Article 9.7.4.1 states that elastic deflection caused by the weights of the panel, the plastic concrete and reinforcement should not exceed:

(a) Span length/180 with an upper limit of 0.25 in. for span length of 10 ft or less

(b) Span length/240 with an upper limit of 0.75 in. for span length greater than 10 ft.

At service loads, the stresses in the composite section have to checked under Service I Limit State for prestressed concrete in compression. For Service III Limit State, which is used to check tensile stresses in the precast SIP panel, the full live load moment should be used, i.e. the 0.8 factor associated with live load should be replaced by 1.0. This is because the 0.8 factor was developed for application only to longitudinal prestressed concrete beams.

Finally, Strength I Limit State is used to check the ultimate flexural capacity of the composite section. Check stress in prestressing steel according to the available development length, ℓ_d, as follows:

$$\ell_d = K\left(f_{ps} - \frac{2}{3}f_{pe}\right)d_b \qquad \text{[LRFD Eq. 5.11.4.2-1]}$$

$$\text{or, } f_{ps} = \frac{\ell_d}{Kd_b} + \frac{2}{3}f_{pe}$$

where

d_b = nominal strand diameter

f_{pe} = effective stress in prestressing steel after losses

ℓ_d = available development length at midspan of the SIP panel

K = 1.6 for precast, prestressed slabs

8.8.2.4.2.6 Reinforcement Requirements

The provisions for minimum reinforcement in the *LRFD Specifications* are the same as those in the *Standard Specifications*. The maximum amount of reinforcement, according to LRFD Article 5.7.3.3, should be such that:

$$\frac{c}{d_e} \le 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

where

c = distance from the extreme compression fiber to the neutral axis, in.

d_e = effective depth from the extreme compression fiber to the centroid of the force in the tension reinforcement, in.

The *LRFD Specifications* do not give guidance for the required amount of distribution reinforcement for the concrete SIP panel system that provides for the lateral distribution of concentrated live loads. However, LRFD Article 9.7.3.2 specifies the minimum amount of reinforcement in the longitudinal direction, for slabs which have four layers of reinforcement, as $220/\sqrt{S} \le 67\%$ of the primary reinforcement, where S = span between the inside faces of the beam webs, ft.

Applying this provision yields a higher amount of longitudinal reinforcement than that required by the Empirical Design method or the Standard Specifications (see Example 9.8 in Chapter 9).

8.8.2.4.2.7 Shear Design

Two-way shear should be checked assuming that the contact area of the tire is rectangular. LRFD Article 3.6.1.2.5 gives the dimensions of the contact area for a tire pressure of 0.125 ksi. The two-way shear capacity is given in LRFD Article 5.13.3.6.3. One-way shear should be checked as specified in LRFD Article 5.13.3.6.2.

8.8.2.4.2.8 Crack Control

For crack control in the negative moment areas, provisions of LRFD Article 5.7.3.4 should be applied. Because positive moment regions for precast SIP deck panel systems are prestressed, they are designed to be uncracked under service load conditions.

In order to control possible cracking due to shrinkage and temperature changes, a minimum amount of reinforcement, in each direction, should be provided:

$$A_s \ge 0.11\, A_g/f_y \qquad \text{(LRFD Eq. 5.10.8.2-1)}$$

where

A_g = gross area of section, in.^{2}

f_y = specified minimum yield strength of reinforcing bars, ksi

This reinforcement should be equally distributed on both faces and should not be spaced farther apart than three times the slab thickness or 18.0 in. It is reasonable to waive this requirement in precast, prestressed concrete panels in the direction of prestress.

8.8.3 Other Precast Bridge Deck Systems

Rapid replacement of bridge decks is becoming increasingly important in high traffic areas due to public intolerance to extended bridge closures. This section covers two new bridge deck systems developed at the University of Nebraska for rapid replacement of bridge decks (Tadros, 1998). The first system is a continuous precast concrete stay-in-place panel system, called NUDECK (Badie, 1998). It is intended for applications similar to the conventional SIP panel previously discussed. The second system is a full-depth precast concrete panel system intended for very rapid construction (Takashi, 1998). The following sections briefly introduce these two new systems.

8.8.3.1 Continuous Precast Concrete SIP Panel System, NUDECK

Although conventional SIP precast panels have proven cost-effective and have been widely used in Florida, Missouri, Tennessee, Texas and several other states, they do have drawbacks. These include:

(1) the need for forming overhangs with wood forming

(2) the possible appearance of reflective cracking over joints between SIP panels

(3) the lack of development of the pretensioning strands in the SIP panel caused by strand discontinuity at beam lines and relatively small beam spacing.

The continuous stay-in-place (CSIP) system – NUDECK – has the following advantages:

(1) the CSIP panel covers the entire width of the bridge eliminating the necessity of forming the overhang

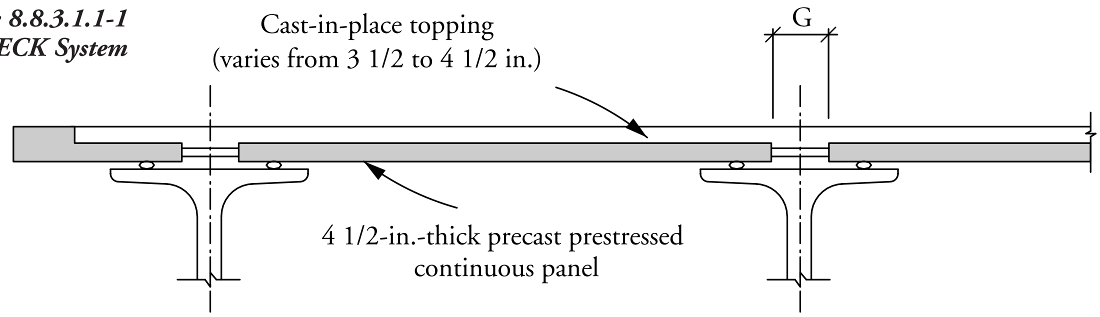
(2) the CSIP panel is continuous longitudinally and transversely which results in minimized reflective cracks, full development of the pretensioning reinforcement, and better live load distribution

Cost studies conducted by contractors and consulting engineers (Tadros, 1998), estimated that the NUDECK system would be cost-competitive with CIP systems. The slight increase in panel cost would be offset by the reduction in field costs due to installation of fewer pieces and elimination of overhang forming. However, the novelty of the system, panel forming challenges and panel weight are potential disadvantages of this system.

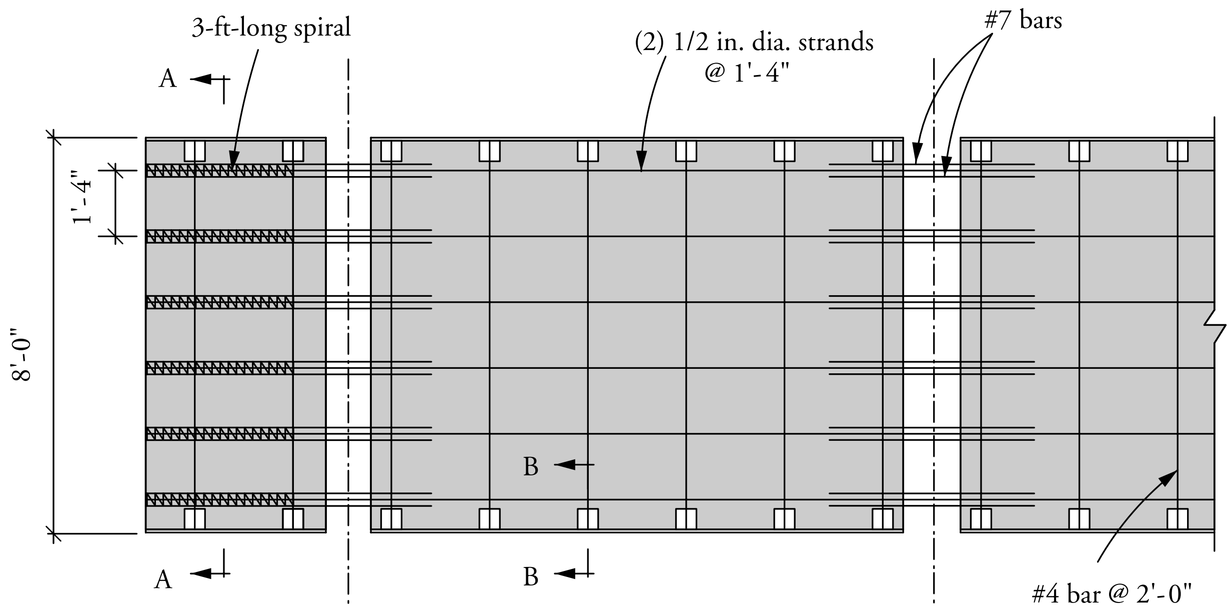
8.8.3.1.1 Description of NUDECK

Figure 8.8.3.1.1-1 shows a cross-section of a bridge and a plan view of the precast panel. The system consists of a 4.5-in.-thick precast panel and a 3.5 to 4.5-in.-thick CIP reinforced topping. The length of the panel in the direction of traffic can vary from 8 to 12 ft depending on the transportation and lifting equipment available in the field. At each beam position there is a full-length gap to accommodate shear connectors. The width of the gap, G, depends on the shear connector detail used in the precast beam. As an example, for a beam spacing of 12 ft and overhang of 4 ft subjected to HS-25 truck loading, an 8-ft-wide panel would require (12) 1/2-in.-diam-

Figure 8.8.3.1.1-1
The NUDECK System



(a) Cross Section of the NUDECK System



(b) Plan View of the NUDECK Panel

eter strands and a 28-day concrete compressive strength of 8,000 psi. The strands are located in two layers and uniformly spaced at 16 in. A minimum clear concrete cover of 1 in. is used for both the top and bottom layers of strands.

In order to maintain the gap over the beam, and to transmit the pretensioning force from one section to another across the gap, 24 short pieces of No. 7 reinforcing bars are used in two layers. These bars transmit the prestress compression force across the gap. To maintain continuity in the longitudinal direction between the adjacent precast panels, shear keys and reinforced pockets are provided as shown in **Figures 8.8.3.1.1-2** and **8.8.3.1.1-3**. The panel is reinforced longitudinally with No. 4 bars spaced at 2 ft at the location of the pockets. To provide for full tension development of the No. 4 bars, they are spliced using an innovative confinement technique as shown in **Figure 8.8.3.1.1-4**. A pocket, only 5-in. deep, is needed to fully develop the No. 4 bar. The panels are erected using shims and leveling bolts. The longitudinal gaps are then filled with fine-aggregate concrete. When the concrete attains a strength of 4,000 psi, the finishing machine can then be installed and the CIP topping cast in one continuous operation. Full-scale laboratory testing (Yehia, 1998) has shown this system has almost two-times the load capacity of an equivalent conventional SIP panel system.

Figure 8.8.3.1.1-2
Cross-Section of the NUDECK Panel

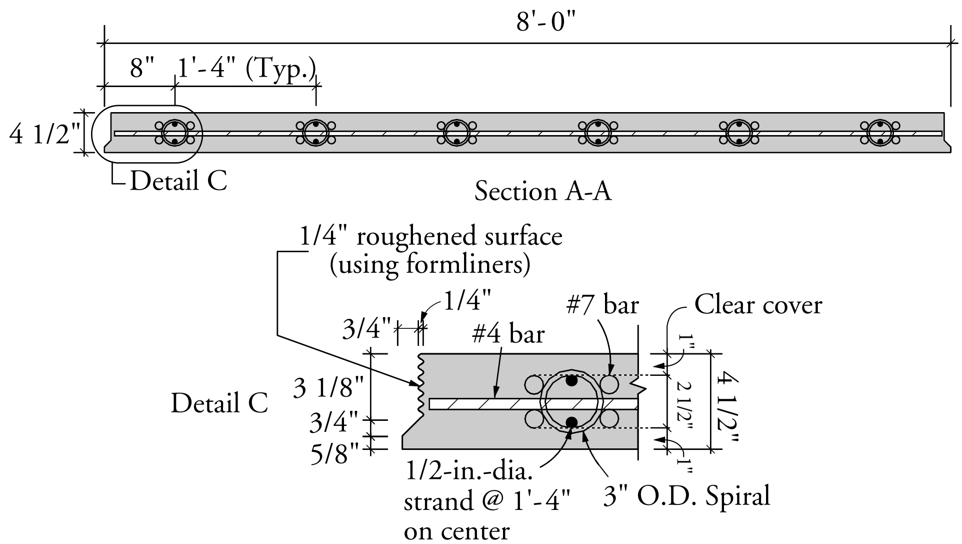


Figure 8.8.3.1.1-3
Details of Reinforced Pockets

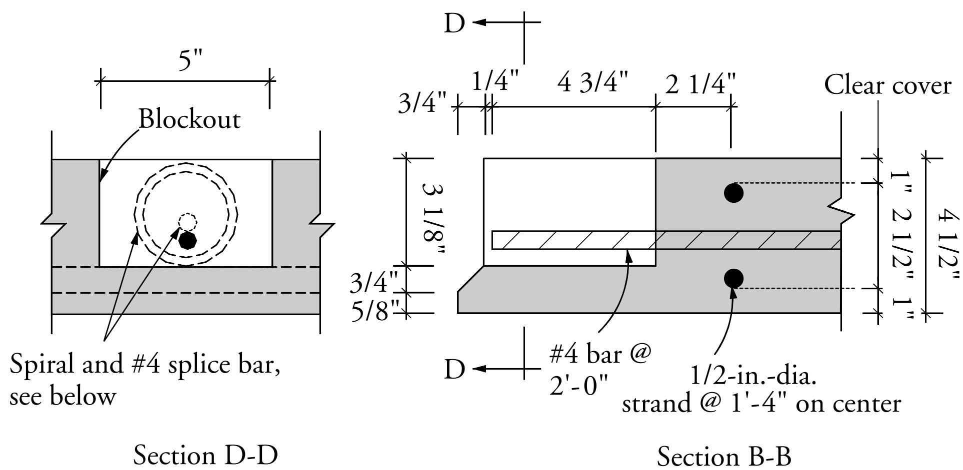


Figure 8.8.3.1.1-4
Panel-to-Panel Connection (At 2'-0" Centers)

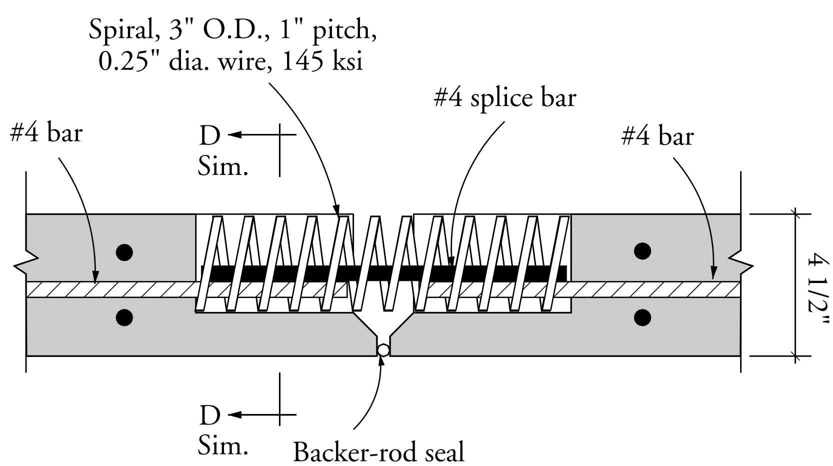
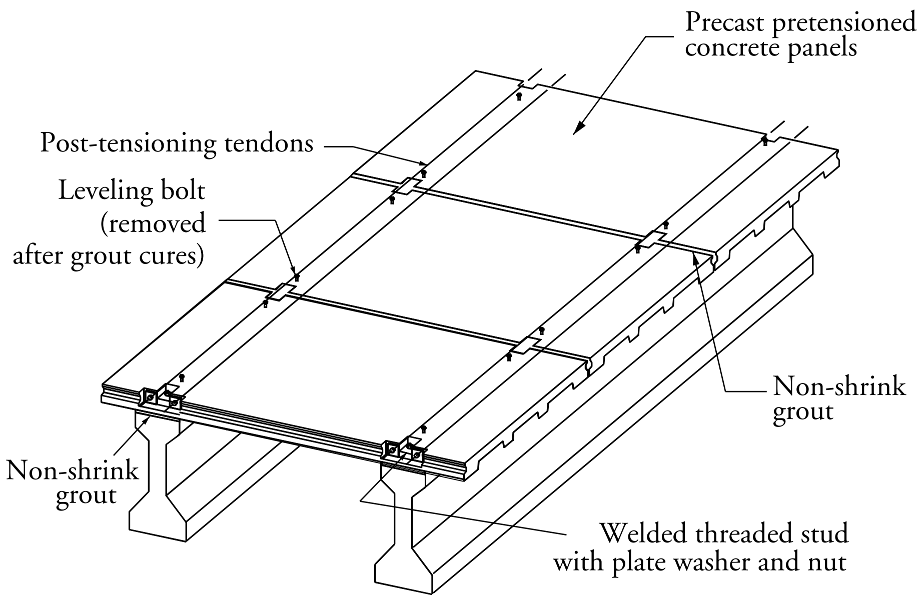


Figure 8.8.3.2-1
Overview of Full-Depth Panel System



8.8.3.2 Full-Depth Precast Concrete Panels

An overview of this system is shown in **Figure 8.8.3.2-1**. It consists mainly of precast, transversely pretensioned concrete panels, welded threaded studs, grout-filled shear keys, leveling bolts, and longitudinal post-tensioning tendons.

The overall geometry is determined by the arrangement of pretensioning strands for positive moments and to provide an adequate compressive zone for negative moments. One layer of welded wire reinforcement is provided in the upper portion of the slab. Pretensioning strands are arranged in two layers and eccentricity is minimized because the panel is subjected to both negative and positive moments. Two important functions of the transverse joints between panels are to transfer live loads and to prevent water leakage. For these two requirements, a shear key with a rapid-set, non-shrink grout is used. Longitudinal post-tensioning is applied after the transverse shear keys are grouted but before the deck is made composite with the underlying beams.

The full-depth precast pretensioned system has the following benefits:

- It has an equivalent slab thickness of 5.9 in., which makes it significantly lighter than other systems
- The system is prestressed both directions, resulting in superior performance compared to conventionally-reinforced decks
- The system does not need a CIP topping, which reduces the time of construction
- The panel includes 1/2 in. extra cover to be used for grinding the deck to a smooth surface
- The panels can be rapidly produced and constructed, or removed
- The grouted, post-tensioned transverse joints between panels prevent cracking and possible leakage throughout the service life of the deck
- Deflection under service load is small in comparison to non-prestressed systems

Full-scale fatigue and ultimate strength testing has demonstrated superior performance of this system. No cracks or joint leakage were observed after two million cycles of loading. The strength of the system was governed by punching shear of the slab at about 5 times the maximum wheel load of an HS25 truck.

The disadvantages of this system include the following:

- The deck surface is required to be ground in order to attain a smooth riding surface
- Longitudinal post-tensioning significantly increases the number of construction steps required
- Panel weight requires availability of cranes

8.8.4 LRFD Specifications Empirical Design Method

The empirical procedure of the *LRFD Specifications* [LRFD Art. 9.7.2] is attractive in that it provides less reinforcement than analytical methods, including finite element and strip analysis. Less steel should result in less deck deterioration due to reinforcement corrosion. The method is based on full-scale testing, conducted primarily in Ontario, Canada. The empirical design method may be used only if certain specified conditions are met. If the specified amount of reinforcement is provided, the deck is considered to satisfy all design requirements without need for design calculations. The conditions are:

- the supporting components are steel and/or concrete beams
- the deck is fully cast-in-place and water-cured
- the deck is of uniform depth, except for haunches at beam flanges and other local thickening
- the ratio of effective length, between inside faces of beam webs, to the design depth does not exceed 18.0 and is not less than 6.0
- core depth of the slab, between the extreme faces of top and bottom reinforcement, is not less than 4.0 in.
- the effective length, between the inside faces of the beam webs, does not exceed 13.5 ft
- the minimum depth of the slab is not less than 7.0 in. excluding a sacrificial wearing surface where applicable
- there is an overhang beyond the centerline of the outside beam of at least 5 times the depth of the slab. This condition is satisfied if the overhang is at least 3 times the depth of the slab, and a structurally continuous concrete barrier is made composite with the overhang
- the specified 28-day strength of the deck concrete is not less than 4.0 ksi
- the deck is made composite with the supporting structural components
- the reinforcement required consists of four layers
- minimum amount of reinforcement is 0.27 in.^{2}/ft for each bottom layer and 0.18 in.^{2}/ft for each top layer
- maximum spacing of bars is 18 in.

The provisions of the empirical design method are not applied to overhangs. The overhang should be designed for all of the following cases:

- wheel loads for decks with discontinuous railings and barriers using the equivalent strip method

- equivalent line loads for decks with continuous barriers (LRFD Art. 3.6.1.3.4)
- collision loads using a failure mechanism

Note that negative overhang moments require reinforcement that must be extended into the adjacent span.

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NOTATION

DESIGN THEORY AND PROCEDURE

A	= area of cross-section of the precast beam	[STD], [LRFD]
A	= distance to pickup points from each end of the beam	—
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_c	= area of beam cross-section	—
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_{cs}	= cross-sectional area of a concrete strut	[LRFD]
A_g	= gross area of section	[LRFD]
A_k	= area of cross-section of element k	—
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_s	= area of non-pretensioning tension reinforcement	[STD], [LRFD]
A_s	= total area of vertical reinforcement located within a distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= area of steel required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= area of steel required to develop the compressive strength of the web of a flanged section	[STD]
A_{ss}	= area of reinforcement in strut	[LRFD]
A_{st}	= area of longitudinal mild steel reinforcement in tie	[LRFD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance s	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	—
$A_{v\text{-}min}$	= minimum area of web reinforcement	—
a	= depth of the compression block	[STD]
a	= depth of the equivalent rectangular stress block	[LRFD]
a	= length of overhang	—
b	= effective flange width	—
b	= width of beam	[STD]
b	= width of top flange of beam	—
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_b	= width of bottom flange of beam	—
b_v	= width of cross-section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
C_a	= creep coefficient for deflection at time of erection due to loads applied at release	—

CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
$C(t,t_0)$	= creep coefficient of the concrete member at a certain age	—
$C(t,t_j)$	= creep coefficient at time t_j ($j = 0,1,2,\ldots$)	—
$C_b(t,t_3)$	= creep at time t for beam concrete loaded at time t_3	—
$C_d(t,t_3)$	= creep at time t for deck concrete loaded at time t_3	—
C_u	= ultimate creep coefficient for concrete at time of release of prestressing	—
C'_u	= ultimate creep coefficient for concrete at time of application of superimposed dead loads	—
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= nominal diameter of the strand	[STD]
DC	= dead load of structural components and non-structural attachments	[LRFD]
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compression fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[STD], [LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tension reinforcement	[LRFD]
d_{ext}	= depth of the extreme steel layer from extreme compression fiber	—
d_i	= depth of steel layer from extreme compression fiber	—
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_s	= distance from extreme compression fiber to the centroid of nonprestressed tensile reinforcement	[LRFD]
d_v	= effective shear depth	[LRFD]
d'	= distance from extreme compression fiber to the centroid of nonprestressed compression reinforcement	[LRFD]
E	= modulus of elasticity	—
E_c	= modulus of elasticity of concrete	[STD], [LRFD]
$E_{cb}(t_3)$	= age-adjusted modulus of elasticity for beam concrete at time t_3	—
$E_{cd}(t_3)$	= age-adjusted modulus of elasticity for deck concrete at time t_3	—
$E_c(t_j)$	= modulus of elasticity at time t_j ($j = 0,1,2,\ldots$)	—
$E_c(t_0)$	= initial modulus of elasticity	—
$E_c(t,t_0)$	= modulus of elasticity at a certain time	—
E_{ci}	= modulus of elasticity of the beam concrete at transfer	—
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]

E^*_c	= age-adjusted, effective modulus of elasticity of concrete for a gradually applied load at the time of transfer of prestressing	—
E^*_{cb}	= age-adjusted, effective modulus of elasticity of the beam	—
E^*_{cd}	= age-adjusted, effective modulus of elasticity of the deck	—
$E^*_c(t,t_0)$	= effective modulus of elasticity at certain time	—
E^*_{ck}	= age-adjusted, effective modulus of element k	—
e	= eccentricity of prestressing strands	—
e_c	= eccentricity of the strand at midspan	—
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
e_i	= initial lateral eccentricity of the center of gravity with respect to the roll axis	—
e_m	= average accentricity at midspan	[LRFD]
e_p	= eccentricity of the prestressing strands with respect to the centroid of the section	—
FS_c	= factor of safety against cracking	—
FS_f	= factor of safety against failure	—
F_b	= allowable tensile stress in the precompressed tension zone at service loads	—
F_{cj}	= force in concrete for the j th component	—
F_{pi}	= total force in strands before release	—
f	= stress	—
f_b	= concrete stress at the bottom fiber of the beam	—
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at transfer	[STD]
f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning (transfer)	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_{cu}	= the limiting concrete compressive stress for designing by strut-and-tie model	[LRFD]
f_f	= stress range	[STD]
f_{min}	= algebraic minimum stress level	[STD]
f_{pbt}	= stress in prestressing steel immediately prior to transfer	[LRFD]
f_{pc}	= compressive stress in concrete (after allowance for all pretensioning losses) at centroid of cross-section resisting externally applied loads	[STD]

f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	—
f_{pj}	= stress in the pretensioning steel at jacking	[LRFD]
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel under service loads	—
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	—
f_{si}	= effective initial pretension stress	—
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
$f(t_j)$	= stress at time t_j	—
$f_r(t,t_0)$	= relaxation stress at a certain time	—
$f(t_0)$	= tensile stress at the beginning of the interval	—
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity	[LRFD]
h	= length of a single segment	—
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]
h_{cg}	= height of center of gravity of beam above road	—
h_d	= deck thickness	—
h_f	= compression flange depth	[LRFD]
h_r	= height of roll center above road	—
I	= moment of inertia about the centroid of the non-composite precast beam, major axis moment of inertia of beam	[STD], [LRFD]

I	= impact fraction	[STD]
I_k	= moment of inertia of element k	—
IM	= dynamic load allowance	[LRFD]
I_{eff}	= effective cracked section lateral (minor axis) moment of inertia	—
I_g	= gross lateral (minor axis) moment of inertia	—
K	= factor used for calculating time-dependent losses	—
K_r	= factor used for calculating relaxation loss in strand that occurs prior to transfer	—
K_θ	= sum of rotational spring constants of supports	—
k	= factor used in calculation of average stress in pretensioning steel for strength limit state; factor related to type of strand [LRFD]	
k_c	= product of applicable correction factors for creep = $k_{la}\ k_h\ k_s$	—
k_{cp}	= correction factor for curing period	—
k_{la}	= correction factor for loading age	—
k_h	= correction factor for relative humidity	—
k_s	= correction factor for size of member	—
k_{sh}	= product of applicable correction factors for shrinkage = $k_{cp}\ k_h\ k_s$	—
k_{st}	= correction factor for concrete strength	—
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	—
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
LL	= vehicular live load	[LRFD]
L_r	= intrinsic relaxation of the strand	—
L_x	= distance from end of prestressing strand to center of the panel	[STD]
ℓ	= overall length of beam	—
ℓ_d	= development length	—
ℓ_t	= transfer length	—
M_c	= moment in concrete beam section	—
M_{cr}	= cracking moment	[LRFD]
$M_{cr}(t)$	= restraint moment due to creep at time t	—
M^*_{cr}	= cracking moment	[STD]
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_{el}	= fictious elastic restraint moment at the supports	—
M_g	= unfactored bending moment due to beam self-weight	—
M_g	= self-weight bending moment of beam at harp point	—
M_{gmsp}	= self-weight bending moment at midspan	—
M_k	= element moment	—

M_{lat}	= lateral bending moment at cracking	—
M_{LL}	= unfactored bending moment due to lane load per beam	—
M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	—
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_{sh}	= shrinkage moment	—
$M_{sr}(t)$	= restraint moment due to differential shrinkage at time t	—
M_{sw}	= moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—
M_u	= factored bending moment at section	[STD], [LRFD]
M_x	= bending moment at a distance x from the support	—
M_0	= theoretical total moment in sections	—
M_{0k}	= theoretical moment in section of element k	—
m	= stress ratio	—
N	= number of segments between nodes (must be even number)	—
N_k	= element normal force	—
N_c	= internal element force in concrete	—
N_s	= internal element force in steel	—
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
N_{0k}	= theoretical normal force in section of element k, positive when tensile	—
N_0	= theoretical total normal force in sections	—
n	= modular ratio between slab and beam materials	[STD], [LRFD]
n_k	= modular ratio of element k	—
n_s	= modular ratio of steel element	—
PPR	= partial prestress ratio	[LRFD]
P_c	= permanent net compression force	[LRFD]
P_n	= nominal axial resistance of strut or tie	[LRFD]
P_r	= factored axial resistance of strut or tie	[LRFD]
P_{se}	= effective pretension force after allowing for all losses	—
P_{si}	= effective pretension force after allowing for the initial losses	—
Q	= first moment of inertia of the area above the fiber being considered	—
R	= radius of curvature	—
RH	= relative humidity	[STD]
R_n	= strength design factor	—
R_u	= flexural resistance factor	—
r	= radius of gyration of the gross cross-section	—
r	= radius of stability	
S	= width of precast beam	[STD]
S	= spacing of beams	[STD], [LRFD]

S	= slab span	[LRFD]
S	= span between the inside faces of the beam webs	[LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	—
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam	—
SH	= loss of pretension due to concrete shrinkage	[STD]
S_N	= the value of the integral	—
$S(t,t_0)$	= shrinkage coefficient at a certain age	—
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	—
S_u	= ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
t	= time, days; age of concrete at the time of determination of creep effects, days; age of concrete at time of determination of shrinkage effects, days; time after loading, days	—
t	= thickness of web	—
t	= thickness of an element of the beam	—
t_f	= thickness of flange	—
t_0	= age of concrete when curing ends; age of concrete when load is initially applied, days	—
t_s	= cast-in-place concrete slab thickness	—
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component of the effective pretensioning force, in the direction of the applied shear, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD], [LRFD]

V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
v_u	= average factored shear stress	[LRFD]
W	= total weight of beam	—
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w	= weight per unit length of beam	—
w_c	= unit weight of concrete	[STD], [LRFD]
x	= distance from the support to the section under question	—
y	= height of center of gravity of beam above roll axis (beam supported from below)	—
y_b	= distance from centroid to the extreme bottom fiber of the non-composite beam	—
y_{bc}	= distance from centroid to the bottom of beam of the composite section	—
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	—
y_k	= distance of the centroid of element k from edge	—
y_r	= height of roll axis above center of gravity of beam (hanging beam)	—
y_s	= height above soffit of centroid of prestressing force	—
y_t	= distance from centroid to the extreme top fiber of the non-composite beam	—
y_{tc}	= distance from centroid to the top of deck of the composite section	—
$\bar{z}$	= lateral deflection of center of gravity of beam	—
z_{max}	= distance from centerline of vehicle to center of dual tires	—
$\bar{z}_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally	—
$\bar{z}'_o$	= theoretical lateral deflection of center of gravity of beam with the full dead weight applied laterally, computed using I_{eff} for tilt angle θ under consideration	—
α	= super-elevation angle or tilt angle of support in radians	—
α	= factor used in calculating elastic shortening loss	—
α	= coefficient defined by (Eq. 8.6.2.5.1-3) to account for interaction between steel and concrete in pretensioning loss calculations	—
α_s	= angle between compressive strut and adjoining tension tie	[LRFD]
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
δ_c	= time-dependent multiplier	—
Δ	= deflection	—
Δ	= camber measured with respect to the beam-ends	—

NOTATION

DESIGN THEORY AND PROCEDURE

Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]
Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δf_s	= total loss of prestress	—
ε	= strain	—
ε_c	= strain in concrete beam	—
ε_{cr}	= the time dependent creep strain	—
ε_f	= the immediate strain due to the applied stress f	—
ε_{fc}	= elastic strain in concrete	—
ε_{fk}	= element strain	—
ε_{fs}	= elastic strain in steel	—
ε_k	= strain in element k	—
ε_p	= strain in prestressing steel	—
ε_s	= strain in mild steel	—
ε_s	= tensile strain in cracked concrete in direction of tensile tie	[LRFD]
ε_{sh}	= free shrinkage strain	—
$\varepsilon_{shb}(t,t_2)$	= shrinkage strain of the beam from time t_2 to time t	—
$\varepsilon_{shb}(t_3,t_2)$	= shrinkage strain of the beam from time t_2 to time t_3	—
$\varepsilon_{shd}(t,t_3)$	= shrinkage strain of the deck from time t_3 to time t	—
ε_{shu}	= ultimate free shrinkage strain in the concrete, adjusted for member size and relative humidity	—
ε_{si}	= strain in tendons corresponding to initial effective pretension stress	—
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
ε_{0c}	= initial strain in concrete	—
ε_1	= principal tensile strain in cracked concrete due to factored loads	[LRFD]
γ^*	= factor for type of pretensioning reinforcement	[STD]
ϕ	= strength reduction factor	[STD]
ϕ	= resistance factor	[LRFD]
ϕ	= curvature	—
ϕ_c	= curvature at midspan	—
ϕ_{cr}	= curvature due to creep	—
ϕ_{fk}	= element curvature	—

ϕ_k	= curvature of element k	—
ϕ_0	= curvature at support	—
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
θ	= roll angle of major axis of beam with respect to vertical	—
θ_L	= left end rotation of beam due to simple span loads	—
θ_R	= right end rotation of beam due to simple span loads	—
θ_i	= initial roll angle of a rigid beam	—
θ_{max}	= tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture	—
θ'_{max}	= tilt angle at maximum factor of safety against failure	—
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= ratio of pretensioning reinforcement	[STD]
ψ	= a factor that reflects the fact that the actual relaxation is less than the intrinsic relaxation	—
χ	= aging coefficient	—
$\chi(t,t_0)$	= aging coefficient at certain time	—

8.9 TRANSVERSE DESIGN OF ADJACENT BOX BEAM BRIDGES

8.9.1 Background

Adjacent box beam bridges are constructed by placing precast, prestressed concrete box beams next to each other so that a deck slab is not required to complete the structure. The small longitudinal joint between beams – the "shear key" or "keyway" – is normally filled with grout. Often, a composite concrete topping or a non-structural asphalt concrete overlay is used to provide the riding surface. Typically, longitudinal keyways are dimensioned for standard products shown in Appendix B. Transverse connections are made between beams to prevent differential deflection and to improve the distribution of live loads.

The design of the transverse connections between adjacent box beams has been identified as an important issue by bridge owners and designers. Without an adequate transverse connection, beams will not deflect equally under live loads. Differential movement between beams may lead to longitudinal cracking of the grouted keyways and reflective cracking in the overlay, if one is present. Recent surveys of adjacent box beam bridges have revealed that cracks of these types are a recurring problem in some areas. In rural locations where deicing chemicals are not used, such cracks may be tolerated. However, in most locations, these cracks should be prevented because water and deicing chemicals may penetrate the cracks and cause concrete staining and eventually structural deterioration of the box beams from corrosion of reinforcement and subsequent spalling of the concrete cover.

In addition, a transverse connection between box beams is necessary to provide effective load transfer between beams. Without adequate transverse connection, live load cannot be distributed across the bridge. Each beam in that case would have to be designed to resist the full effect of a set of wheel loads.

8.9.1.1 Current Practice

When a transverse connection is provided between adjacent box beams, it is typically made using threaded rods, post-tensioning bars or strands, or welded connections. A 5- or 6-in.-thick reinforced concrete composite slab may also be used to provide a transverse connection between adjacent members. When a structural concrete topping is not used, a non-structural overlay, such as a 2.0-in.-thick asphalt concrete wearing surface, is often applied as a final riding surface. However, for some secondary roads, a topping is not used because the surface of the precast beam is more than adequate to use for a riding surface.

The number and location of transverse ties, the erection details, and procedures for installing transverse connections vary from state to state. Several different types of connections have been found to provide good performance, although in other cases, similar details and procedures may perform very differently. The selection of a system for connecting adjacent box beam bridges depends on initial cost, long-term maintenance costs, experience of the owner, capabilities of local contractors, and availability of materials.

8.9.1.2 Ontario Bridge Design Code Procedure

The Ontario, Canada, Bridge Design Code provides a procedure for the design of adjacent box beam bridges. The Ontario Code assumes that the load is transferred from one beam to another primarily through transverse shear; transverse flexural rigidity is neglected, (Bakht, 1983). Charts are provided to determine the transverse shear force to be resisted. A reinforced concrete structural slab with a minimum thickness of 5.9 in. is required to be placed on the bridge to provide the shear transfer between beams. Therefore, the grouted keyway is not relied upon to transfer shear between boxes.

8.9.2 Empirical Design

Several users of adjacent beams have developed empirical design guidelines for transverse connection of adjacent slabs and box beams, with varying degree of success. One procedure, developed by the State of Oregon and refined over many years of practice, has demonstrated satisfactory field performance in controlling longitudinal cracking and moisture leakage between beams. This method is described in detail in the following sections.

8.9.2.1 Tie System

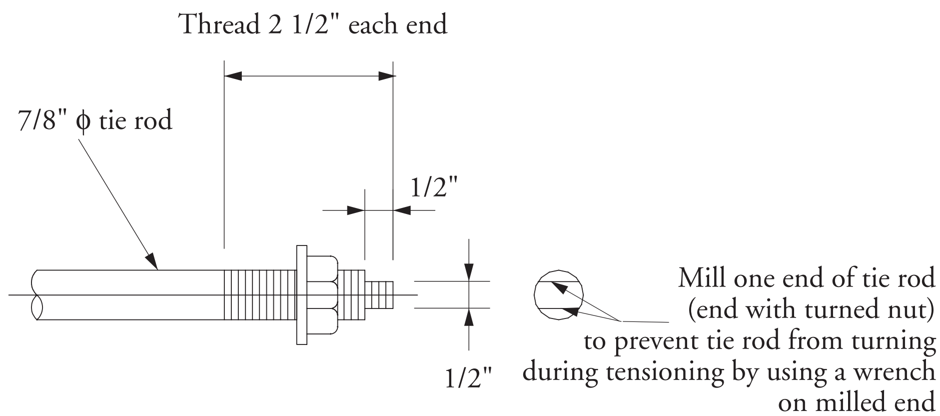
Transverse ties should be capable of providing a total transverse force at least equal to the weight of each beam. The ties are provided in the form of 1 or 2 rods at mid-depth of the member at locations along the span according to **Table 8.9.2.1-1**. This spacing and number of tie rods has been found to produce satisfactory field performance.

Table 8.9.2.1-1 Number and Spacing of Tie Rods

Span, ft	Number of rods and spacing
≤ 20	One at midspan
>20, but ≤ 40	One at third points
>40, but ≤ 70	Two at third points
>70, but ≤ 100	Two rods at ≤ 24 ft spacing, with first set at 8 ft from end

A 7/8 in. diameter, 8'-2" long smooth rod with 2 1/2" threaded length is used for each location. The rod material is ASTM A449 high strength steel. Each rod is tensioned to 39.25 kips, using a torque wrench and a direct tension indicating (DTI) washer, conforming to ASTM F959. A heavy hexagonal nut, conforming to ASTM A194 and a 5"x5"x1" ASTM A36 bearing plate complete the tensioning-and-anchorage assembly, as shown in **Figure 8.9.2.1-1**. On the non-tensioning end of the rod, the DTI washer is replaced with a hardened steel ASTM F436 flat circular washer. All hardware is hot-dip galvanized after fabrication.

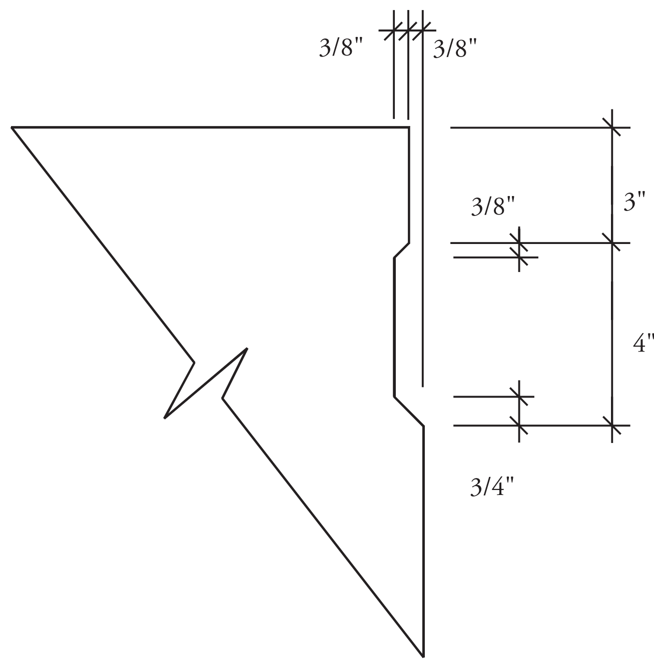
Figure 8.9.2.1-1 Hardware Used for Tie Rods



8.9.2.2 Production

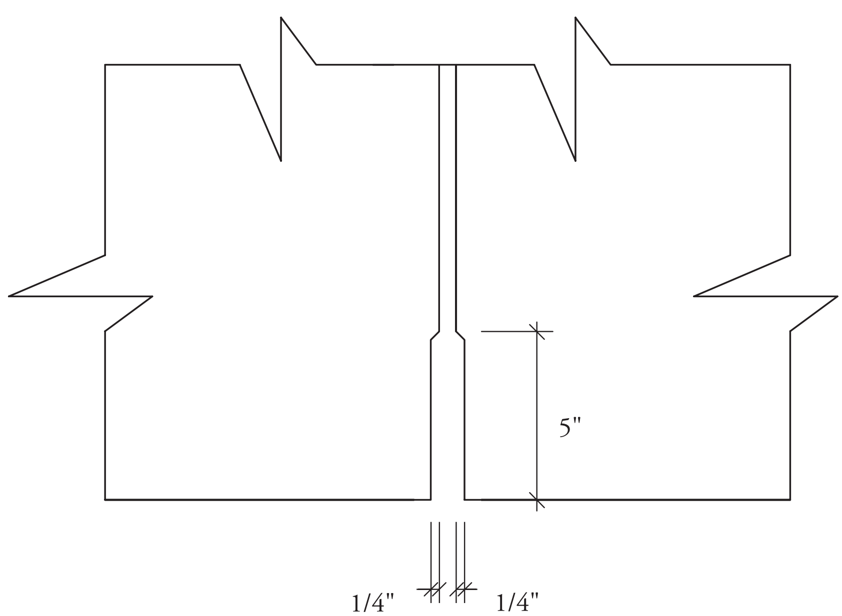
Each member is produced with shear keys as shown in **Figure 8.9.2.2-1**. The shear key shown has a well-defined and bulbous shape with an adequate opening at the top to provide access for installation of grout. The face of the key is sandblasted at the precast plant to remove loose materials and to provide a good bonding surface.

Figure 8.9.2.2-1 Shear Key Detail



It is advisable to provide a recess along the bottom edge of the beam, as shown in **Figure 8.9.2.2-2**, to prevent spalling due to a stress concentration that could result from possible formed surface bulges or protrusions. All beams must have diaphragms at their ends and at tie rod locations. Diaphragm dimensions and locations are determined with consideration of the skew angle. A 3 in. diameter hole is formed at each tie rod location.

Figure 8.9.2.2-2 Joint Detail



8.9.2.3
Installation

Erection of precast units begins at either exterior beam or at the center of the bridge depending upon the width of the bridge and the desired crane placement. After placing the first two adjacent units the tie rods are installed and the nuts are tightened until the ridges on the load indicator washers collapse (see **Figure 8.9.2.3-1**). Prior to installing the nuts on the tie rod, the threads are lubricated with a suitable wax or tension control fluid to allow the required tension on the rods to be developed. The sequence continues by placing a beam and installing the required number of tie rods each time a beam is set. Handholes are provided in the concrete beams at each end of the bolts to provide access to the non-turned nut located on the far side of the previous unit set. After all the units in a span are set, the grout may be installed in the shear keys. The grout should be non-shrink, non-ferrous, non-epoxy grout with a minimum design strength of 5,000 psi. The surface of the keys should be kept damp for twenty-four hours before and after installing the grout. If the space between beams is wide enough to allow the grout to run through, a strip of foam rod stock is installed at the bottom of the shear key to seal it. In order to provide a positive seal, the grout is tooled-down from the top of the deck to provide a recess for the installation of caulking or a poured joint filler. This area must again be cleaned to remove any loose grout before installing the sealant, as shown in **Figure 8.9.2.3-2**.

Figure 8.9.2.3-1
Tie Rod Recess Detail

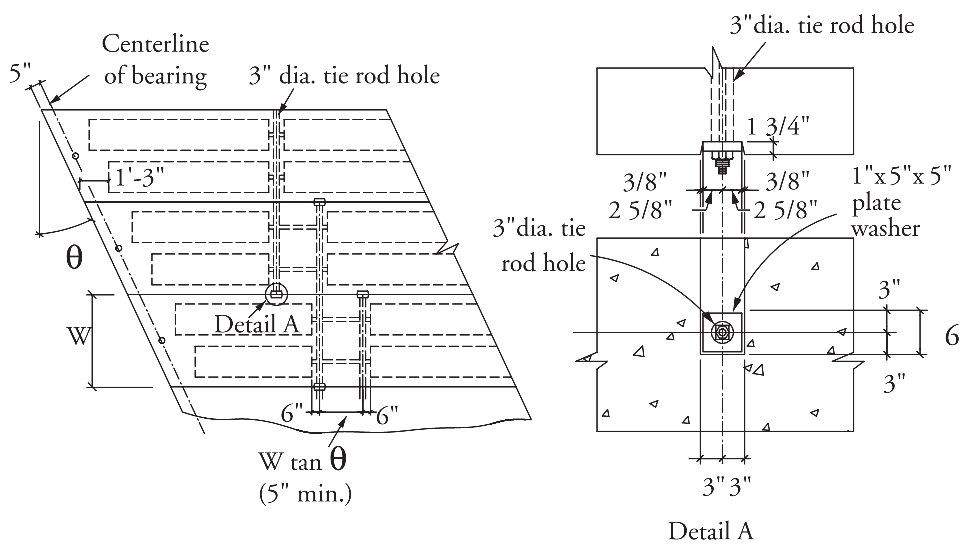
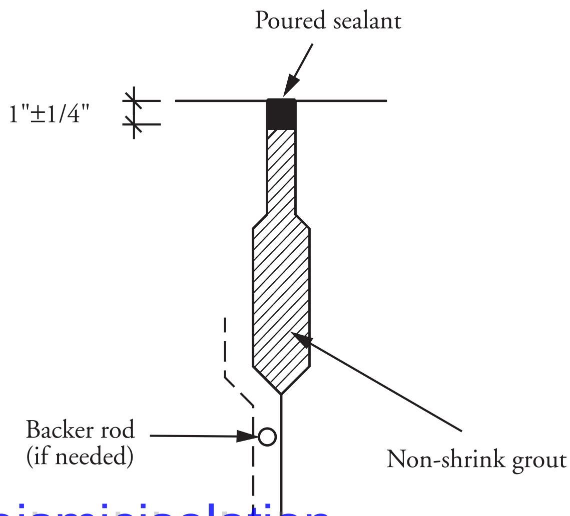


Figure 8.9.2.3-2
Keyway Grout Detail



8.9.3
Suggested Design Procedure

A design procedure has been developed (El-Remaily, 1996) which assumes that post-tensioned transverse diaphragms serve as the primary mechanism for the distribution of wheel loads across the bridge. Five diaphragms are provided in each span: one at each end and one at each quarter-point. A typical detail at a diaphragm is shown in **Figure 8.9.3-1**. The amount of post-tensioning required at each diaphragm depends on the bridge geometry and loading. A chart, **Figure 8.9.3-2**, that has been developed for the determination of the required amount of transverse post-tensioning, will be described in Section 8.9.3.5.

Figure 8.9.3-1
Transverse Post-Tensioning Arrangement

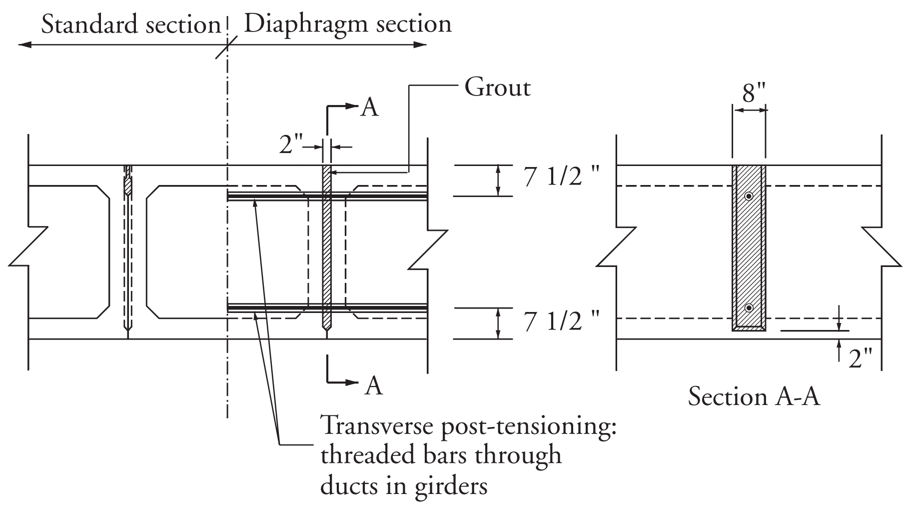
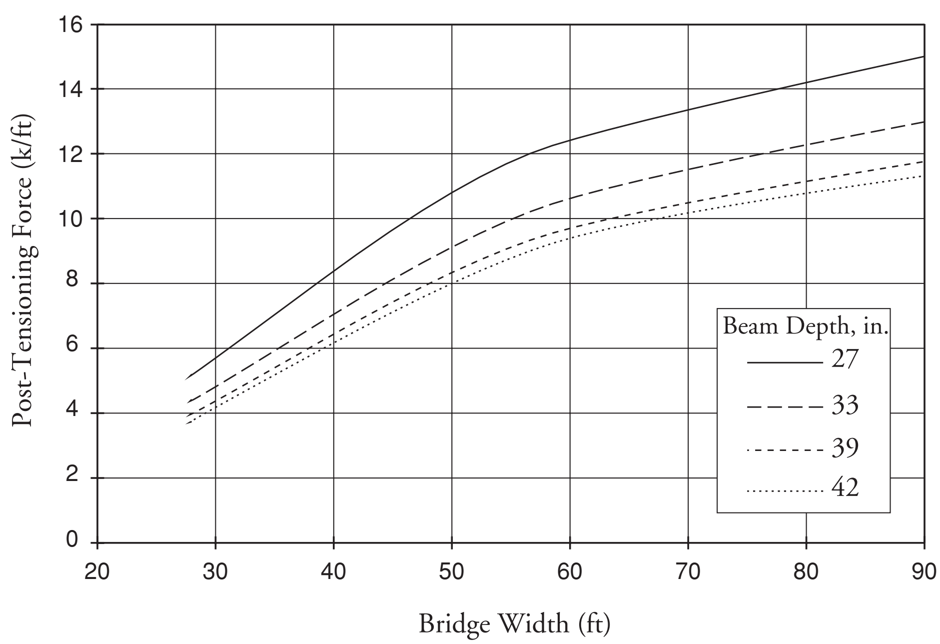


Figure 8.9.3-2
Required Effective Post-Tensioning Force



8.9.3.1
Transverse Diaphragms

The transverse diaphragms are made continuous across the entire width of the bridge by providing grout pockets in the faces of the joints at each diaphragm location. These vertical pockets, which are filled with grout prior to post-tensioning, extend nearly the full depth of the beam (see **Figure 8.9.3-1**). The grout must be installed and cured prior to post-tensioning so that it will be compressed. This precompression of the grout is necessary to avoid cracking in the diaphragm. The 1-in.-deep grout pocket is formed into the side of each box beam by attaching a blockout to the interior of the steel side form. Installation of the blockout is a simple and inexpensive modification to the box beam form.

It has been found that for spans of up to 100 ft, the use of five post-tensioned diaphragms limit differential deflection between adjacent box beams to 0.02 in., which is an acceptable amount. The use of three diaphragms, one at each end and one at midspan, was found to reduce the required quantity of post-tensioning, but the differential deflection between beams increased to an unacceptable level, i.e. higher than 0.02 in. As a guideline, for spans up to 60 ft, three diaphragms, at ends and at midspan, may be used. For spans over 60 ft, five diaphragms, at ends, midspan, and at quarter points, may be used.

Diaphragms are post-tensioned because of the difficulty of providing continuous, conventionally reinforced diaphragms across the width of the bridge. Conventionally reinforced diaphragms would also be subject to cracking, which would reduce their effectiveness and possibly allow water penetration. Post-tensioned diaphragms are precompressed and should not crack.

8.9.3.2 Longitudinal Joints Between Beams

The transverse connection between adjacent box beams is made at the diaphragms, so longitudinal shear keys are not required for the structural performance of the bridge. However, the gap between beams should be sealed with grout or an appropriate non-structural sealant to prevent water leakage between beams. Longitudinal shear keys tend to seal the gap between girders much more effectively when grouted than those without grout.

8.9.3.3 Tendons

In most cases, a pair of post-tensioning tendons is placed in each diaphragm. Each tendon may consist of a single post-tensioning bar or strand. The use of post-tensioning bars may be preferred because they are easier to install, achieve a higher force for a single bar as compared to a single strand, provide greater stiffness across the joint, and generally have a lower anchorage seating loss, which is especially significant for short tendons. Strands may be used if power seating of the anchorage is used to minimize the seating loss. The tendons are placed symmetrically about the mid-height of the section in order to provide equal resistance to the positive and negative moments that are caused by live load. The vertical distance between tendons should as large as possible in order to maximize the flexural resistance of the diaphragm.

Tendons may be either bonded, by grouting after post-tensioning, or left unbonded. Bonded tendons provide higher capacity at ultimate conditions and are protected from corrosion by the grout, but grouting is an additional operation that must be performed in the field. Unbonded tendons are easier to install and can be more easily removed if a damaged box beam must be replaced. However, the tendon must be protected in some way from corrosion and the force in the tendon at ultimate is lower than for bonded tendons. In either case, the tendon anchorages must be protected from corrosion by encasing the anchorage in grout or by using galvanized hardware and sealing anchorages with grease caps.

8.9.3.4 Modeling and Loads for Analysis

The bridge is modeled using grid analysis to determine member forces. A series of beam elements is used to represent the beams. These elements are connected by a series of crossing beam elements, representing the diaphragms. The joints between elements allow the transmission of shear, bending moment and torsion.

Barrier rails and live loads are the main sources of transverse bending moments generated in the diaphragms. The live loads are positioned to produce the maximum positive and negative moments in the diaphragms. To obtain the maximum transverse positive moment in the diaphragm at midspan, the live load is placed over the center of the deck. For maximum transverse negative moment at the same location, the load is placed as close to the barrier rail as possible or as required by the specifications.

8.9.3.5
Post-Tensioning Design Chart

A design chart developed by El-Remaily (1996) using the above procedure is shown in **Figure 8.9.3-2**. This chart provides the required transverse post-tensioning force for the standard box beam depths of 27, 33, 39 and 42 in. To prepare the chart, bridges with widths of 28, 52 and 84 ft were considered. For each combination of section depth and bridge width, three different spans appropriate for the beam size, were considered. This chart may be used for both 3- and 4-ft-wide beams and for bridges with mild skews, i.e. less than about 15°. Use of the chart should satisfy both service load and factored load design criteria. Similar charts can be generated for other box beam depths and for bridges with sharp skews. Use of the chart is illustrated in the design example, which follows in Section 8.9.3.7.

The required transverse post-tensioning force was found to be almost linearly proportional to the span length. The forces shown in the design chart were obtained by dividing the required effective post-tensioning force for the midspan diaphragm by the spacing between diaphragms, and then taking the average of the three span lengths analyzed. The chart was developed assuming that bonded post-tensioning is used. If unbonded post-tensioning is used, the required post-tensioning force would increase about 30%.

The required post-tensioning force for the diaphragms at the quarter points was found to be similar to the midspan diaphragm. It is therefore recommended that the same force be used at all diaphragms within the span. The end diaphragms, however, are subjected to very small bending moments because they are continually supported at the piers or abutments. The same prestressing force may be provided for the end diaphragms or a minimum prestress force of 250 psi on the area of the diaphragm may be provided.

Although the chart was developed for bridges with no skew, it can be used for bridges with mild skews up to 15°. For bridges with high skews, over 15°, grid analysis should be conducted. Grid analysis is relatively simple to conduct with commercially available computer programs.

8.9.3.6
Design Method

Because the transverse diaphragms are post-tensioned, working stress design is the primary approach used to design them. Stresses in the diaphragms due to loads and transverse prestress are computed and compared to allowable stress limits. To prevent cracking of the grout or opening of the cracks along the interface between the grout and box beams, tensile stresses are not permitted.

Flexural strength of the diaphragm must also be checked for factored loads. The available stress in the tendons at ultimate depends upon whether the tendon is bonded or unbonded. When bonded post-tensioning is used, the stress in the tendon at ultimate flexure may be obtained with procedures given in Section 8.2. The tendon on the compression side of the diaphragm should be neglected when computing the stress in the tendon at ultimate and the flexural strength of the diaphragm.

Shear in a vertical plane through the joint between beams may be checked using shear friction theory, similar to horizontal shear design between precast beams and cast-in-place composite decks – see Section 8.5. However, due to the very large clamping force caused by post-tensioning, shear generally does not govern the design.

8.9.3.7 Design Example

The design of the diaphragms for the bridge in Design Example 9.1 is performed here. The bridge is 28 ft wide, has a span of 95 ft and utilizes AASHTO BIII-48 box beams. The strength of the grout used in the grout pocket between girders at each diaphragm is 5,000 psi. Post-tensioning bars with a strength of 160 ksi are used for transverse post-tensioning of the diaphragms.

Five transversely post-tensioned diaphragms are used, located at the ends and quarter points of the span. The spacing between the diaphragms is therefore 95/4 = 23.75 ft. Using the design chart, **Figure 8.9.3-2**, for a section depth of 39 in. and a bridge width of 28 ft, the required effective prestressing force is 4 kips/ft. The force required for each diaphragm is computed by multiplying the distance between diaphragms by the required force per foot of length. Therefore, the required post-tensioning force at each diaphragm is 4 kips/ft (23.75 ft) = 95 kips.

Two tendons will be used at each diaphragm. The required area of post-tensioning for each bar is computed by dividing the required force by the effective prestress, which is assumed to be 55% of the strength of the bar. Therefore:

A_{pt} = 95 kips/[0.55(160 ksi)] = 1.08 in.^{2}

Try (2) 7/8-in.-diameter, 160 ksi smooth bars (*PCI Design Handbook*, 1992). The total area provided is 1.202 in.^{2} > 1.08 in.^{2} OK

The total effective prestress force for the two bars is:

F_{pe} = 0.55(160)(1.202) = 105.8 kips

The grout pockets are 37 in. deep and 8-in. wide. Since the bar positions result in eccentric prestressing of the diaphragm, the stress in the grout due to the effective prestress force is:

$$f_c = 105.8/8(37) \pm \frac{105.8(1)}{(1/6)(8)(37)^2} = 0.415 \text{ and } 0.299 \text{ ksi, top and bottom, respectively.}$$

As discussed in Section 7.5.5, the *LRFD Specifications* requires a prestress of 0.250 ksi for the beams to qualify as rigidly connected for live load distribution. However, the 0.250-ksi value appears to be somewhat arbitrary. The rigorous analysis developed here should provide an acceptable design in compliance with the *LRFD Specifications*. Even though shear keys are recommended to be grouted full length, this grouting is not required for the successful structural performance of this recommended system. Greater than 0.250 ksi is provided across each of the five continuous diaphragms.

8.9.4 Lateral Post-Tensioning Detailing for Skewed Bridges

Diaphragms in skewed bridges can either be skewed or perpendicular to the longitudinal axis of the beam. Diaphragms that are skewed are normally limited to skew angles of 20° or less. Use of skewed diaphragms allows for placement of grout between beams all at one time, then post-tensioning in one operation. For multistage or phased construction, it is possible to use staggered post-tensioning with skewed or perpendicular diaphragms. Refer to Sections 3.6.3.4 and 3.6.3.5 in Chapter 3 for illustrations and Section 8.9.2 for details used in Oregon.

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DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pb} = compressive stress at bottom fiber of the beam due to prestress force

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = the modulus of rupture of concrete [STD]

f_r = modulus of rupture of concrete [LRFD]

f_s = allowable stress in steel

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress

f_{si} = effective initial pretension stress

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

f_t = concrete stress at top fiber of the beam for the non-composite section

f_{tc} = concrete stress at top fiber of the slab for the composite section

f_{tg} = concrete stress at top fiber of the beam for the composite section

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity, percent [LRFD]

H = height of wall [LRFD]

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_c	= total height of composite section	
h_f	= compression flange depth	[LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD]
I	= moment of inertia about the centroid of the non-composite precast beam	[LRFD]
I	= impact fraction (maximum 30%)	[STD]
I_c	= moment of inertia of composite section	
IM	= dynamic load allowance	[LRFD]
J	= St. Venant torsional constant	
K	= longitudinal stiffness parameter	[STD]
K_g	= longitudinal stiffness parameter	[LRFD]
k	= factor used in calculation of distribution factor for multi-beam bridges	[LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
L_c	= critical length of yield line failure pattern	[LRFD]
LL	= vehicular live load	[LRFD]
ℓ_d	= development length	[LRFD]
ℓ_x	= length required to fully develop the strand measured from the end of the strand	
M_a	= negative moment at the end of the span being considered	
M_b	= negative moment at the end of the span being considered	
M_b	= unfactored bending moment due to barrier weight	
M_c	= flexural resistance of cantilevered wall	[LRFD]
M_{CIP}	= unfactored bending moment due to cast-in-place topping slab	
M_{const}	= unfactored bending moment due to construction load	
M_{col}	= bending moment due to horizontal collision force	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads (after dead load)	[STD]
M_{cr}	= cracking moment	[LRFD]
M^*_{cr}	= cracking moment	[STD]
M_D	= unfactored bending moment due to diaphragm weight	
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_f	= unfactored bending moment due to fatigue truck per beam	
M_g	= unfactored bending moment due to beam self-weight	
M_{LL}	= unfactored bending moment due to lane load per beam	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_{LL+I}	= unfactored bending moment due to design vehicular load	
M_{LT}	= unfactored bending moment due to truck load with dynamic allowance per beam	

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT} = total loss in pretensioning steel stress [LRFD]
Δ_D = deflection due to diaphragm weight
Δ_L = deflection due to specified live load
Δ_{LL+I} = deflection due to live load and impact
Δ_{LL} = deflection due to lane load
Δ_{LT} = deflection due to design truck load and impact
Δ_{max} = maximum allowable live load deflection
Δ_p = camber due pretension force at transfer
Δ_{SDL} = deflection due to barrier and wearing surface weights
Δ_{slab} = deflection due to the weights of slab and haunch
ε_x = longitudinal strain in the web reinforcement on the flexural tension side of the member [LRFD]
γ = load factor [STD]
γ^* = factor for type of pretensioning reinforcement, 0.28 for low relaxation strand [STD]
γ_i = load factor [LRFD]
η = load modifier (a factor relating to ductility, redundancy, and operational importance) [LRFD]
ϕ = strength reduction factor for moment = 1.0 [STD]
ϕ = strength reduction factor for shear = 0.90 [STD]
ϕ = resistance factor [LRFD]
λ = parameter used to determine friction coefficient μ [LRFD]
μ = Poisson's ratio for beams [STD]
μ = coefficient of friction [LRFD]
θ = angle of inclination of diagonal compressive stresses [LRFD]
ρ_{actual} = actual ratio of non-pretensioned reinforcement
ρ_b = reinforcement ratio producing balanced strain condition [STD]
$\rho^* = \frac{A_s^*}{bd}$, ratio of pretensioning reinforcement [STD]
ψ = angle of harped pretensioned reinforcement

DESIGN EXAMPLES

9.0 INTRODUCTION

Design examples included in this chapter illustrate the step-by-step procedure used in the design of precast, prestressed concrete bridges. Some of the design examples are for simply supported bridges for dead and live load and some examples are carried out for bridges simply supported for dead load but made continuous for live load and subsequent superimposed dead loads. The examples represent several types of bridges. Each type of bridge in this chapter is designed twice. One design conforms with the provisions of the AASHTO *Standard Specifications for Highway Bridges*, 16th Edition, 1996. The second design conforms with the *AASHTO LRFD Bridge Design Specifications*, 1st Edition, 1994, and 1996 Interim.

Examples 9.5 and 9.6 are designed to be continuous for superimposed dead loads, live loads and impact. Sections at the piers were designed as reinforced concrete members to resist flexure at ultimate. No attempt was made to calculate the positive moment that might develop at the piers, instead the readers are directed to Chapter 8, where forces due to creep and shrinkage and their effects on continuity are discussed in depth.

SIGN CONVENTION

For concrete:

Compression positive (+ ve)

Tension negative (– ve)

For steel:

Compression negative (– ve)

Tension positive (+ ve)

Distance from center of gravity:

Downward positive (+ ve)

Upward negative (– ve)

LEVEL OF ACCURACY:

Item	Units	Accuracy
Concrete Stress	ksi	1/1000
Steel Stress	ksi	1/10
Prestress Force	kips	1/10
Moments	ft-kips	1/10
Shears	kips	1/10
For the beam:		
Cross-Section	in.	1/100
Section Properties	in.	1 in.
Length	ft	1/100
Area of Pretensioning Steel	in.^{2}	1/1000
Area of Mild Reinforcement	in.^{2}	1/100

Some calculations were carried out to a higher number of significant figures than common practice with hand calculation. Depending on available computation resources and designer preference, other levels of accuracy may be used.

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DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e′	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]
f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]
f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pb} = compressive stress at bottom fiber of the beam due to prestress force
f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]
f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]
f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pe} = effective stress in the pretensioning steel after losses [LRFD]
f_{pi} = initial stress immediately before transfer
f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]
f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]
f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]
f_{pu} = specified tensile strength of pretensioning steel [LRFD]
f_{py} = yield strength of pretensioning steel [LRFD]
f_r = the modulus of rupture of concrete [STD]
f_r = modulus of rupture of concrete [LRFD]
f_s = allowable stress in steel
f'_s = ultimate stress of pretensioning reinforcement [STD]
f_{se} = effective final pretension stress
f_{si} = effective initial pretension stress
f^*_{su} = average stress in pretensioning steel at ultimate load [STD]
f_t = concrete stress at top fiber of the beam for the non-composite section
f_{tc} = concrete stress at top fiber of the slab for the composite section
f_{tg} = concrete stress at top fiber of the beam for the composite section
f_y = yield strength of reinforcing bars [STD]
f_y = specified minimum yield strength of reinforcing bars [LRFD]
f_y = yield stress of pretensioning reinforcement [STD]
f'_y = specified minimum yield strength of compression reinforcement [LRFD]
f_{yh} = specified yield strength of transverse reinforcement [LRFD]
H = average annual ambient mean relative humidity, percent [LRFD]
H = height of wall [LRFD]
h = overall depth of precast beam [STD]
h = overall depth of a member [LRFD]

h_c	= total height of composite section	
h_f	= compression flange depth	[LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD]
I	= moment of inertia about the centroid of the non-composite precast beam	[LRFD]
I	= impact fraction (maximum 30%)	[STD]
I_c	= moment of inertia of composite section	
IM	= dynamic load allowance	[LRFD]
J	= St. Venant torsional constant	
K	= longitudinal stiffness parameter	[STD]
K_g	= longitudinal stiffness parameter	[LRFD]
k	= factor used in calculation of distribution factor for multi-beam bridges	[LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
L_c	= critical length of yield line failure pattern	[LRFD]
LL	= vehicular live load	[LRFD]
ℓ_d	= development length	[LRFD]
ℓ_x	= length required to fully develop the strand measured from the end of the strand	
M_a	= negative moment at the end of the span being considered	
M_b	= negative moment at the end of the span being considered	
M_b	= unfactored bending moment due to barrier weight	
M_c	= flexural resistance of cantilevered wall	[LRFD]
M_{CIP}	= unfactored bending moment due to cast-in-place topping slab	
M_{const}	= unfactored bending moment due to construction load	
M_{col}	= bending moment due to horizontal collision force	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads (after dead load)	[STD]
M_{cr}	= cracking moment	[LRFD]
M^*_{cr}	= cracking moment	[STD]
M_D	= unfactored bending moment due to diaphragm weight	
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_f	= unfactored bending moment due to fatigue truck per beam	
M_g	= unfactored bending moment due to beam self-weight	
M_{LL}	= unfactored bending moment due to lane load per beam	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_{LL+I}	= unfactored bending moment due to design vehicular load	
M_{LT}	= unfactored bending moment due to truck load with dynamic allowance per beam	

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	$= \dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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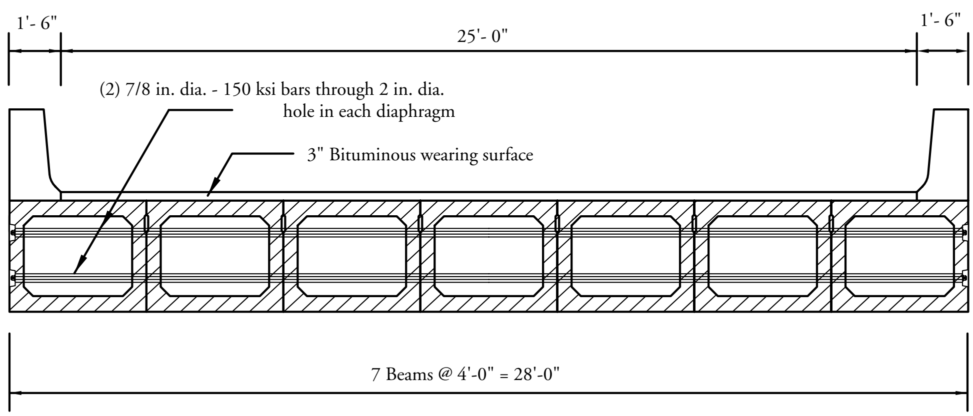
BOX BEAM (BIII-48), SINGLE SPAN, NON-COMPOSITE SURFACE, STANDARD SPECIFICATIONS

Box Beam (BIII-48), Single Span, Non-Composite Surface, Standard Specifications

9.1.1 INTRODUCTION

This design example demonstrates the design of a 95-ft single-span AASHTO Type BIII-48 box beam bridge with no skew. This example illustrates in detail the design of a typical interior beam at the critical sections in positive flexure, shear and deflection due to pretension, dead loads and live load. The superstructure consists of seven beams abutted as shown in **Figure 9.1.1-1**. A 3 in. bituminous surfacing will be placed on the beams as a wearing surface. Beams are transversely post-tensioned through 8 in.-thick full-depth diaphragms located at quarter points. Design live load is AASHTO HS25. The design is carried out in accordance with the AASHTO *"Standard Specifications for Highway Bridges,"* 17th Edition, 2002.

Figure 9.1.1-1
Bridge Cross-Section



9.1.2 MATERIALS

Precast beams: AASHTO Type BIII-48 Box Beams (as shown in **Fig. 9.1.2-1**)

Concrete strength at release, f'_{ci} = 4,000 psi

Concrete strength at 28 days, f'_c = 5,000 psi

Concrete unit weight, w_c = 150 pcf

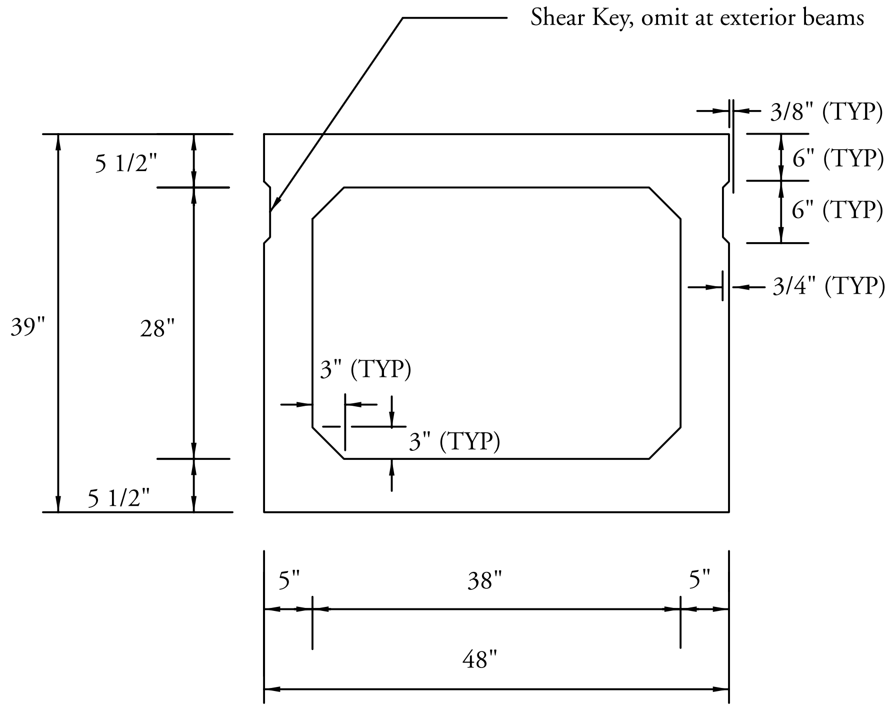
Overall beam length = 96.0 ft

Design span = 95.0 ft

BOX BEAM (BIII-48), SINGLE SPAN, NON-COMPOSITE SURFACE, STANDARD SPECIFICATIONS

9.1.2 Materials

Figure 9.1.2-1
AASHTO Type BIII-48
Box Beam



Pretensioning strands: 1/2 in. diameter, seven wire, low relaxation

Area of one strand, = 0.153 in.^{2}

Ultimate strength, $f_s' = 270{,}000$ psi

Yield strength, $f_y^* = 0.9f_s' = 243{,}000$ psi [STD Art. 9.1.2]

Initial pretension, $f_{si} = 0.75f_s' = 202{,}500$ psi [STD Art. 9.15.1]

Modulus of elasticity, $E_s = 28{,}500$ ksi

The *Standard Specifications* [STD Art. 9.16.2.1.2] indicate that the modulus of elasticity for the strand, E_s is 28,000 ksi. However, the value of 28,500 ksi is a more accurate value, according to the PCI Design Handbook and *LRFD Specifications.*

Reinforcing bars: Yield strength, $f_y = 60{,}000$ psi

Bituminous surfacing: Unit weight = 150 pcf

New Jersey-type barrier: Unit weight = 300 lb/ft/side

9.1.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

The standard precast concrete boxes shown in Appendix B have dimensions such that the entire top flange is effective in resisting flexural loads. This is because 12 times the top flange thickness of the box is greater than the entire width of the top flange.

A = area of cross section of precast beam = 813 in.^{2}

h = overall depth of precast beam = 39 in.

I = moment of inertia about the centroid of the non-composite precast beam = 168,367 in.^{4}

y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam = 19.29 in.

y_t = distance from centroid to the extreme top fiber of the non-composite precast beam = 19.71 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 8,728 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 8,542 in.^{3}

Wt. = 0.847 kip/ft

$E_c = (w_c)^{1.5}(33)\sqrt{f'_c}$ [STD Art. 8.7.1]

where

E_c = modulus of elasticity of concrete, psi

w_c = unit weight of concrete = 150 pcf

Standard Specifications [STD Art. 8.7.1] indicates that the unit weight of normal weight concrete is 145 pcf. However, precast concrete mixes typically have a relatively low water/cement ratio and high density. Therefore, a unit weight of 150 pcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f'_c = specified strength of concrete, psi

Therefore, the modulus of elasticity of the beam at release, using $f'_c = f'_{ci}$ = 4,000 psi, is:

$$E_{ci} = (150)^{1.5}(33)\sqrt{4{,}000}\,/1000 = 3{,}834 \text{ ksi}$$

Modulus of elasticity of the beam at service loads, using f'_c = 5,000 psi is:

$$E_c = (150)^{1.5}(33)\sqrt{5{,}000}\,/1000 = 4{,}287 \text{ ksi}$$

9.1.4 SHEAR FORCES AND BENDING MOMENTS

9.1.4.1 Shear Forces and Bending Moments Due to Dead Loads

[STD Art. 3.3]

9.1.4.1.1 Dead Loads

Beam self-weight = 0.847 kip/ft

Diaphragm weight

$$= \left(\frac{8}{12}\right)\left[\frac{(48-10)}{12} \text{ x } \frac{(39-11)}{12} - 4\left(\frac{1}{2}\right)\left(\frac{3}{12}\right)\left(\frac{3}{12}\right)\right]\left(\frac{150}{1000}\right) = 0.73 \text{ kip/diaphragm}$$

Generally, the unit weight of reinforced concrete should be slightly greater than the unit weight of concrete alone because of the added weight of reinforcement. However, in this example, the difference is negligible.

The weight of diaphragms is considered a concentrated load acting at quarter points, as shown in **Figure 9.1.4.1.1-1**.

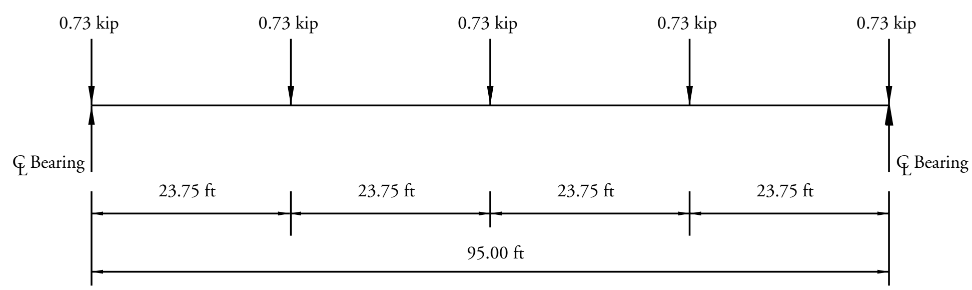


Figure 9.1.4.1.1-1 Diaphragm Loads per Beam

$$\text{Barrier weight} = 2\left(\frac{300}{1,000}\right)\frac{1}{(7 \text{ beams})} = 0.086 \text{ kip/ft}$$

$$\text{3 in. bituminous surface weight} = \left(\frac{3}{12}\right)\left(\frac{150}{1,000}\right)(25)\frac{1}{(7 \text{ beams})} = 0.134 \text{ kip/ft}$$

Note that barriers and wearing surface are placed after the transverse post-tensioning has been applied. Therefore, superimposed dead loads are distributed equally to all beams. [STD Art. 3.23.2.3.2.1]

9.1.4.1.2 Unfactored Shear Forces and Bending Moments

For a simply supported beam with a span length (L) loaded with a uniformly distributed load (w), the shear force (V_x) and the bending moment (M_x) at a distance (x) from the support are given by:

$$V_x = w(0.5L - x) \quad \text{(Eq. 9.1.4.1.2-1)}$$

$$M_x = 0.5wx(L - x) \quad \text{(Eq. 9.1.4.1.2-2)}$$

Using the above equations, values of shear forces and bending moments for a typical interior beam under dead loads (weight of beam, diaphragms, barriers and wearing surface) are computed and given in **Table 9.1.4.2.4-1**. For these calculations, the span length is the design span (95 ft). However, for calculation of stresses and deformations at the time of pretension release, the span length used is the overall length of the precast member (96 ft) as illustrated later in this example.

9.1.4.2 Shear Forces and Bending Moments Due to Live Load

9.1.4.2.1 Live Load

[STD Art. 3.7]

Live load consists of either the standard truck or lane loading. In this example, an HS25 loading is used. For the 95 ft span used in this example, the standard truck loading governs design for both shear and flexure.

9.1.4.2.2 Live Load Distribution Factor for a Typical Interior Beam

The live load bending moments and shear forces in multi-beam precast concrete bridges are determined by applying to each beam the fraction of a wheel load using the following equation: [STD Art. 3.23.4.3]

Distribution Factor:

$$DF = \frac{S}{D} \quad \text{[STD Eq. 3-11]}$$

where

S = width of precast beam in feet = 4.00 ft

$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2\ C)^2$ [STD Eq. 3-12]

where

N_L = number of traffic lanes = integer part of $\frac{25}{12}$ = 2 lanes [STD Art. 3.6]

C = K(W/L) for W/L < 1 [STD Eq. 3-13]

= K for W/L ≥ 1

where

W = overall width of bridge measured perpendicular to the longitudinal beam in feet = 28 ft

L = span length measured parallel to longitudinal beams = 95 ft

$K = \{(1 + \mu)\ I/J\}^{1/2}$ [STD Art. 3.23.4.3]

where

μ = Poisson's ratio for beams = 0.20 [STD Art. 8.7.3]

J = St. Venant torsion constant

For this box beam section:

$$J = \frac{2\,t t_f\,(b - t)^2\,(h - t_f)^2}{bt + h\,t_f - t^2 - t_f^2} \quad \text{[STD Art. 3.23.4.3]}$$

where

b = width of beam = 48.00 in.

h = overall depth of beam = 39.0 in.

t = thickness of web = 5.00 in.

t_f = thickness of flange = 5.50 in.

$$J = \frac{2(5)(5.5)(48-5)^2(39-5.5)^2}{48(5) + 39(5.5) - (5)^2 - (5.5)^2} = 285{,}854 \text{ in.}^4$$

$$I/J = \frac{168{,}367}{285{,}854} = 0.589$$

If $\sqrt{I/J} > 5.0$, a more precise method of determining live load distribution is required. [STD Art. 3.23.4.3]

$$\sqrt{I/J} = \sqrt{0.589} = 0.767 < 5.0 \qquad \text{O.K.}$$

$$K = \{(1+0.20)(0.589)\}^{1/2} = 0.841$$

$$W/L = 28/95 = 0.295 < 1$$

$$C = 0.841\left(\frac{28}{95}\right) = 0.248$$

Therefore,

$$D = [5.75 - 0.5(2)] + 0.7(2)[1 - 0.2(0.248)]^2 = 6.01 \qquad \text{[STD Eq. (3-12)]}$$

$$DF = \frac{4.00}{6.01} = 0.666 \text{ wheel lines/beam} = 0.333 \text{ lanes/beam}$$

However, using a K value of 1.0 as suggested in the *Standard Specifications* [STD Art. 3.23.4.3] for preliminary design, gives a distribution factor of 0.334 lanes/beam. This is generally adequate for design and the detailed calculation is not necessary. Therefore, a distribution factor of 0.334 lanes/beam will be used.

9.1.4.2.3 Live Load Impact

[STD Art. 3.8]

The live load impact is computed using Eq. (3-1) in the *Standard Specifications.*

$$I = \frac{50}{L+125} \qquad \text{[STD Eq. 3-1]}$$

where

I = impact fraction (maximum 30%)

L = the length in feet of the span under consideration = 95 ft [STD Art. 3.8.2.2]

$$I = \frac{50}{95+125} = 0.23$$

Impact for shear varies along the span according to the location of the truck [STD Art. 3.8.2.2 (d)]. For simplicity, the impact factor computed above is used for shear.

9.1.4.2.4 Unfactored Shear Forces and Bending Moments

Shear force and bending moment envelopes on a per-lane-basis are calculated at tenth points of the span using the equations given in Chapter 8. However, this generally can be done by means of commercially available computer software that has the ability to deal with moving loads.

Live load shear force and bending moment per beam are:

V_{LL+I} = (shear force per lane)(Distribution Factor)(1 + I)
= (shear force per lane)(0.334)(1 + 0.23)
= (shear force per lane)(0.411) kips

M_{LL+I} = (bending moment per lane)(Distribution Factor)(1 + I)
= (bending moment per lane)(0.334)(1 + 0.23)
= (bending moment per lane)(0.411) ft-kips

At any section along the span, the maximum bending moment and shear are computed for the standard truck loading and for the lane loading separately. The larger of the two loading types controls the design for the section in question. At each section, the load position must be determined to give the maximum shears and moments. This can be done by means of commercially available programs.

Values of V_{LL+I} and M_{LL+I} at different points are given in **Table 9.1.4.2.4-1**.

Table 9.1.4.2.4-1 Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Distance x	Section x/L	Beam weight		Diaphragm weight		Barrier weight		Wearing surface weight		Live load + Impact	
		Shear V_g	Moment M_g	Shear V_D	Moment M_D	Shear V_b	Moment M_b	Shear V_{ws}	Moment M_{ws}	Shear V_{LL+I}	Moment M_{LL+I}
ft		kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips
0.0	0.000	40.2	0.0	1.1	0.0	4.1	0.0	6.4	0.0	33.3	0.0
1.625[1]	0.017	38.9	64.3	1.1	1.8	3.9	6.5	6.1	10.2	32.7	53.2
9.5	0.100	32.2	344.0	1.1	10.4	3.3	34.9	5.1	54.4	29.6	281.7
19.0	0.200	24.1	611.5	1.1	20.8	2.5	62.1	3.8	96.7	25.9	493.2
28.5	0.300	16.1	802.6	0.4	27.7	1.6	81.5	2.5	127.0	22.2	634.4
38.0	0.400	8.0	917.3	0.4	31.2	0.8	93.1	1.3	145.1	18.5	716.8
47.5	0.500	0.0	955.5	0.4	34.7	0.0	97.0	0.0	151.2	14.8	734.7

[1] Critical section for shear (see section 9.1.11)

9.1.4.3 Load Combinations

[STD Art. 3.22]

For service load design, Group I is 1.00 D + 1.00(L + I) [STD Table 3.22.1A]

where

D = dead load

L = live load

I = impact fraction

For load factor design, Group I is 1.3(1.00D + 1.67(L + I)) [STD Table 3.22.1A]

9.1.5 ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber at the section of maximum moment, i.e., positive moment at midspan.

9.1.5.1 Service Load Stresses at Midspan

Bottom tensile stress due to applied loads is:

$$f_b = \frac{M_g + M_D + M_b + M_{ws} + M_{LL+I}}{S_b}$$

where

f_b = concrete stress at the bottom fiber of the beam

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_D = unfactored bending moment due to diaphragm weight, ft-kips

M_b = unfactored bending moment due to weight of barriers, ft-kips

M_{ws} = unfactored bending moment due to wearing surface, ft-kips

M_{LL+I} = unfactored bending moment due to live load + impact, ft-kips

Using values of bending moments from **Table 9.1.4.2.4-1**, the bottom tensile stress at midspan is:

$$f_b = \frac{955.5 + 34.7 + 97.0 + 151.2 + 734.7}{8{,}728}(12) = 2.713 \text{ ksi}$$

9.1.5.2 Allowable Stress Limit

At service loads, allowable tensile stress in the precompressed tensile zone:

$$F_b = 6\sqrt{f'_c} = -6\sqrt{5{,}000}\left(\frac{1}{1{,}000}\right) = -0.424 \text{ ksi} \qquad \text{[STD Art. 9.15.2.2]}$$

9.1.5.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tension stress at final

2.713 – 0.424 = 2.289 ksi

The location of the strand center of gravity at midspan usually ranges from 5 to 15% of the beam depth, measured from the bottom of the beam. Use a value of 5% for newer, more efficient sections like bulb-tees, and 15% for the less efficient AASHTO-PCI shapes of normal design. Assume the distance from the center of gravity of strands to the bottom fiber of the beam is equal to $y_{bs} = 4.00$ in. (Note: $y_{bs}/h = 4/39 = 10.3\%$) Strand eccentricity at midspan is equal to

$$e_c = y_b - y_{bs} = 19.29 - 4.00 = 15.29 \text{ in.}$$

Bottom fiber stress due to pretension after all losses: $f_b = \dfrac{P_{se}}{A_c} + \dfrac{P_{se}\,e_c}{S_b}$

where P_{se} = effective prestress force after allowing for all losses

Set the required precompression (2.289 ksi) equal to the bottom fiber stress due to pretension and solve for the required minimum, P_{se}:

$$2.289 = \frac{P_{se}}{813} + \frac{P_{se}(15.29)}{8{,}728}$$

$$P_{se} = 767.6 \text{ kips}$$

Assume final losses = 20% of f_{si} = 0.20(202.5) = 40.5 ksi

The available prestress force per strand after all losses

= (cross-sectional area of one strand)(f_{si} – losses)

= 0.153(202.5 – 40.5) = 24.8 kips

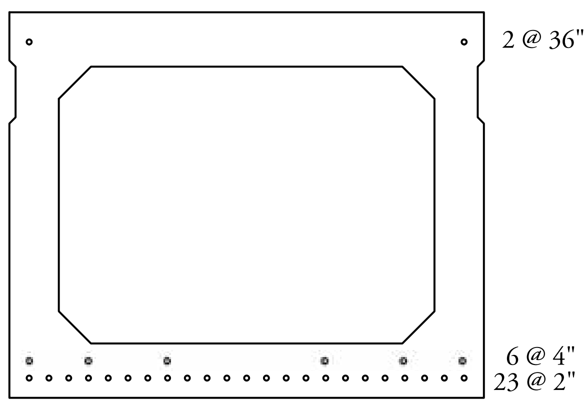
Number of strands required = $\frac{767.6}{24.8}$ = 30.95

Try (31) 1/2-in.-diameter, 270 ksi strands

9.1.5.4 Strand Pattern

Figure 9.1.5.4-1 shows the assumed strand pattern for the 31 strands at midspan with a straight strand pattern.

Figure 9.1.5.4-1
Strand Pattern at Midspan



Calculate the distance from center of gravity of the strands to the bottom fiber of the beam, (y_{bs}):

$$y_{bs} = \frac{23(2) + 6(4) + 2(36)}{31} = 4.58 \text{ in.}$$

Strand eccentricity at midspan

$$e_c = y_b - y_{bs} = 19.29 - 4.58 = 14.71 \text{ in.}$$

9.1.6 FLEXURAL STRENGTH

[STD Art. 9.17]

For box sections, it is common that flexural strength controls the flexural design of the beams. It is therefore recommended that the strength calculation be conducted prior to the stress check. As the applications of high strength concrete continue to grow and emphasis continues to be made on the importance of member strength, it is possible that designs of other precast shapes will be controlled by strength of the members.

Using the Group I load factor design loading combination, as given in Section 9.1.4.3 of the *Standard Specifications,*

$$M_u = 1.3[M_g + M_D + M_b + M_{ws} + 1.67(M_{LL+I})] \quad \text{[STD Table 3.22.1.A]}$$

$$= 1.3[955.5 + 34.7 + 97.0 + 151.2 + 1.67(734.7)] = 3{,}205.0 \text{ ft-kips}$$

Compute average stress in pretensioning steel at ultimate load, f^*_{su}:

$$f^*_{su} = f'_s\left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right) \quad \text{[STD Eq. 9-17]}$$

where

f^*_{su} = average stress in pretensioning steel at ultimate load.

γ^* = 0.28 for low relaxation strand [STD Art. 9.1.2]

$$\beta_1 = 0.85 - 0.05\frac{(f'_c - 4{,}000)}{1{,}000} \geq 0.65 \text{ when } f'_c > 4{,}000 \text{ psi} \quad \text{[STD Art. 8.16.2.7]}$$

$$= 0.85 - 0.05\frac{(5{,}000 - 4{,}000)}{1{,}000} = 0.80$$

b = effective flange width = 48 in.

It should be noted that in computing the flexural strength of members with strands placed near the compression face of the member, it is not correct to use the combined centroid of the entire strand group for establishing the effective depth, d, and the area of pretensioning steel, A^*_s. This is because the top strands will have different strain from that of the bottom strands. An accurate solution can be achieved using the detailed strain compatibility approach which can account for the steel strain at various distances from the neutral axis. However, a reasonable approximation is to ignore all the strands placed on the compression side.

Therefore, area of pretensioning steel, $A^*_s = 29(0.153) = 4.437 \text{ in.}^2$

For the 29 bottom strands, the distance between the center of gravity of the strands and the bottom fiber of the beam, y_{bs}, is $\dfrac{23(2)+6(4)}{29} = 2.41$ in.

Thus, $d = h - y_{bs} = 39 - 2.41 = 36.59$ in.

$$\rho^* = \frac{A^*_s}{bd} = \frac{4.437}{48(36.59)} = 0.00253$$

$$f^*_{su} = 270.0\left(1 - \left(\frac{0.28}{0.80}\right)(0.00253)\left(\frac{270.0}{5.0}\right)\right) = 257.1 \text{ ksi}$$

Compute the depth of the compression block:

$$a = \frac{A_s^* f_{su}^*}{0.85\, f'_c\, b} = \frac{4.437(257.1)}{0.85(5)(48)} = 5.59 \text{ in.} > 5.50 \text{ in.} \qquad \text{[STD Art. 9.17.2]}$$

Therefore, the depth of the compression block is greater than the flange thickness and the section must be considered as a flanged section.

Ultimate moment capacity of flanged section:

$$\phi M_n = \phi \left\{ A_{sr} f_{su}^* d\left(1 - 0.6\frac{A_{sr} f_{su}^*}{b' d f'_c}\right) + 0.85 f'_c (b - b')(t)(d - 0.5t) \right\} \qquad \text{[STD Eq. 9-14]}$$

where

ϕ = strength reduction factor = 1.0

M_n = nominal moment strength of a section.

b' = width of web of the section = 2(5) = 10 in.

A_{sr} = steel area required to develop the compressive strength of the web of a flanged section.

$$= A_s^* - A_{sf} \qquad \text{[STD Eq. 9-15]}$$

A_{sf} = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

$$= 0.85 f'_c (b - b') \frac{t}{f_{su}^*} \qquad \text{[STD Eq. 9-16]}$$

$$= 0.85(5.0)(48 - 10)\left(\frac{5.5}{257.1}\right) = 3.455 \text{ in.}^2$$

A_{sr} = 4.437 – 3.455 = 0.982 in.^{2}

$$\phi M_n = 1.0\left\{0.982(257.1)(36.59)\left(1 - 0.6\frac{0.982(257.1)}{10(36.59)(5.0)}\right) + 0.85(5.0)(48-10)(5.5)[36.59 - 0.5(5.5)]\right\}\left(\frac{1}{12}\right)$$

= 3,211.0 ft-kips > M_u = 3,205.0 ft-kips O.K.

9.1.7 PRESTRESS LOSSES

[STD Art. 9.16.2]

Total losses = $SH + ES + CR_c + CR_s$ [STD Eq. 9-3]

where

SH = loss of prestress due to concrete shrinkage, ksi

ES = loss of prestress due to elastic shortening, ksi

CR_c = loss of prestress due to creep of concrete, ksi

CR_s = loss of prestress due to relaxation of pretensioning steel, ksi

9.1.7.1 Shrinkage

[STD Art. 9.16.2.1.1]

Relative humidity varies significantly from one area of the country to another, see U.S. map, Fig. 9.16.2.1.1 in the *Standard Specifications.*

Assume relative humidity, RH = 70%

$$SH = 17{,}000 - 150RH = [17{,}000 - 150(70)]\frac{1}{1{,}000} = 6.5 \text{ ksi}$$ [STD Eq. 9-4]

9.1.7.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members:

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^{\,2}}{I} - \frac{(M_g + M_D)e_c}{I}$$

f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer

where

P_{si} = pretension force after allowing for the initial losses. The *Standard Specifications* allow that the reduction to initial tendon stress be estimated as $0.69f_s'$ for low relaxation strands

= (number of strands)(area of strands)($0.69f_s'$)

= 31(0.153)(0.69)(270) = 883.6 kips

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_D = unfactored bending moment due to diaphragm weight, ft-kips

M_g and M_D should be calculated based on the overall beam length of 96 ft. However, since the elastic shortening losses will be a part of the total losses, f_{cir} is conservatively computed based on M_g using the design span length of 95 ft.

$$f_{cir} = \frac{883.6}{813} + \frac{883.6(14.71)^2}{168{,}367} - \frac{(955.5 + 34.7)(12)(14.71)}{168{,}367}$$

$$= 1.087 + 1.136 - 1.038 = 1.185 \text{ ksi}$$

$$ES = \frac{E_s}{E_{ci}} f_{cir} = \frac{28{,}500}{3{,}834}(1.185) = 8.8 \text{ ksi}$$

9.1.7.3
Creep of Concrete

[STD Art. 9.16.2.1.3]

$CR_c = 12f_{cir} - 7f_{cds}$ [STD Eq. 9-9]

where

f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied.

$$= \frac{M_{SDL}\, e_c}{I}$$

where

M_{SDL} = super-imposed dead load moment = $M_b + M_{ws}$

= 97.0 + 151.2 = 248.2 ft-kips

$$f_{cds} = \frac{248.2(12)(14.71)}{168{,}367} = 0.260 \text{ ksi}$$

$CR_c = 12(1.185) - 7(0.260) = 12.4$ ksi

9.1.7.4
Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low relaxation strand:

$CR_s = 5{,}000 - 0.10ES - 0.05(SH + CR_c)$ [STD Eq. 9-10A]

$$= [5{,}000 - 0.10(8{,}809) - 0.05(6{,}500 + 12{,}400)]\frac{1}{1{,}000} = 3.2 \text{ ksi}$$

9.1.7.5
Total Losses at Transfer

Losses due to elastic shortening, (ES) = 8.8 ksi

Total initial losses = 8.8 ksi

f_{si} = effective initial pretension stress = 202.5 − 8.8 = 193.7 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 31(0.153)(193.7) = 918.7 kips

9.1.7.6 Total Losses at Service Loads

SH = 6.5 ksi

ES = 8.8 ksi

CR_c = 12.4 ksi

CR_s = 3.2 ksi

Total final losses = 6.5 + 8.8 + 12.4 + 3.2 = 30.9 ksi

or, $\dfrac{30.9}{0.75(270)}(100) = 15.26\%$ losses

f_{se} = effective final pretension stress = 0.75(270) − 30.9 = 171.6 ksi

$f_{se} = 171.6 \text{ ksi} < 0.80\, f_y^* = 0.80(0.9)(270) = 194.4 \text{ ksi}$ O.K. [STD Art. 9.15.1]

$P_{se} = 31(0.153)(171.6) = 814.0$ kips

9.1.8 CONCRETE STRESSES AT TRANSFER

9.1.8.1 Allowable Stress Limits

[STD Art. 9.15.2.1]

Compression: $0.6f'_{ci}$ = +2.400 ksi (compression)

Tension: the maximum tensile stress should not exceed

$7.5\sqrt{f'_{ci}}$ = −0.474 ksi (tension)

If the calculated tensile stress exceeds 200 psi or $3\sqrt{f'_{ci}}$ = 0.190 ksi, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

9.1.8.2 Stresses at Transfer Length Section

This section is located at a distance equal to the transfer length from the end of the beam. Stresses at this location need only be checked at release, because it almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length = 50(strand diameter) [STD Art. 9.20.2.4]

= 50(0.5 in.) = 25 in. = 2.08 ft

The transfer length section is located at 2.08 ft from the end of the beam or at a point 1.58 ft from centerline of the bearing, since the beam extends 6 in. beyond the centerline of bearing. Location on the design span = 1.58/95 = 0.017 L.

Due to the camber of the beam at release, the weight of the beam and diaphragms act on the overall beam length (96 ft). Therefore, the values of bending moment given in **Table 9.1.4.2.4-1** cannot be used because they are based on the design span (95 ft). The values of bending moments at the transfer length due to beam and diaphragm weights, are:

$M_g = 0.5wx(L - x) = 0.5(.847)(2.08)(96 - 2.08) = 82.7$ ft-kips

M_D = 2.3 ft-kips.

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t}$$

$$f_t = \frac{918.7}{813} - \frac{918.7(14.71)}{8{,}542} + \frac{(82.7 + 2.3)(12)}{8{,}542}$$

$$= 1.130 - 1.582 + 0.119 = -0.333 \text{ ksi}$$

Allowable tension with no bonded reinforcement = −0.190 ksi, thus debond some strands or provide bonded reinforcement at the ends.

Compute concrete stress at the bottom fiber of the beam, f_b:

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b}$$

$$= \frac{918.7}{813} + \frac{918.7(14.71)}{8{,}728} - \frac{(82.7 + 2.3)(12)}{8{,}728} = 1.130 + 1.548 + 0.117 = +2.561 \text{ ksi}$$

Allowable compression = +2.400 ksi N.G.

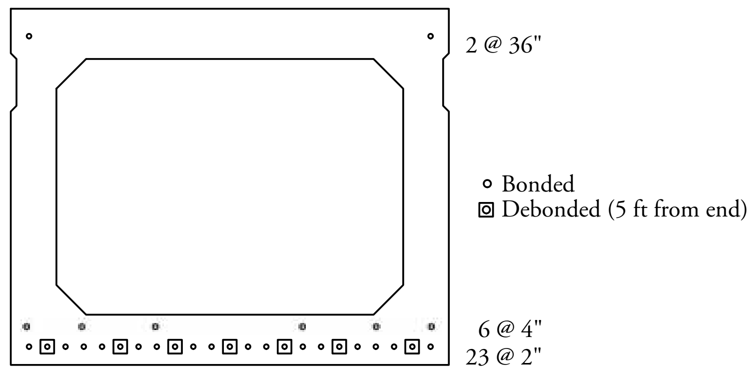
Thus, initial pretension needs to be reduced or the initial concrete strength (f'_{ci}) increased, or debond strands at the end. Try debonding 7 strands from strand group at 2 in. from the bottom for a distance equal to 5'- 0" from the end of beam or 4'- 6" from centerline of bearing.

Although debonding is not addressed in the *Standard Specifications,* Article 5.11.4.2 in the *LRFD Specifications* requires that the following conditions should be satisfied if debonding is used:

- % debonded of total = 7/31 = 22.6% < 25% O.K.
- % debonded of row = 7/23 = 30.4% < 40% O.K.
- All limit states should be satisfied O.K.
- Debonded strands should be symmetrically distributed O.K.
- Exterior strands in each horizontal line are fully bonded O.K.

Even though the *LRFD Specifications* do not address staggering of the debonding length, it is recommended that the maximum number of strands debonded for any given length be kept at 25% and that debonding length can be done in increments of 5 ft. To minimize the shock impact of detensioning and cracks at corners and bottom, assume the strand pattern shown in **Figure 9.1.8.2-1**.

Figure 9.1.8.2-1
Strand Pattern at End of Beam



Recompute the concrete stresses at transfer length section due to the debonded strands.

Compute center of gravity of strand pattern at transfer length section to the bottom fiber of the beam.

$$y_{bs} = \frac{16(2) + 6(4) + 2(36)}{24} = 5.33 \text{ in.}$$

e_e = the eccentricity of the strands at the transfer length section

$$e_c = y_b - y_{bs} = 19.29 - 5.33 = 13.96 \text{ in.}$$

Pretension force at transfer:

$$P_{si} = 24(0.153)(193.7) = 711.3 \text{ kips}$$

$$f_t = \frac{711.3}{813} - \frac{711.3(13.96)}{8{,}542} + \frac{(82.7 + 2.3)(12)}{8{,}542} = 0.875 + 1.162 + 0.119$$

$$= -0.168 \text{ ksi}$$

Allowable tension with no bonded reinforcement = –0.190 ksi O.K.

Note: Since the top fiber stress is smaller than $3\sqrt{f'_{ci}}$, there is no need for additional bonded reinforcement.

$$f_b = \frac{711.3}{813} + \frac{711.3(13.96)}{8{,}728} - \frac{(82.7 + 2.3)(12)}{8{,}728} = 0.875 + 1.138 + 0.117$$

$$= +1.896 \text{ ksi}$$

Allowable compression = +2.400 ksi O.K.

9.1.8.3
Stresses at Transfer Length Section of Debonded Strands

Bending moments due to the self-weight of the beam and diaphragm, at (5.00 + 2.08 = 7.08 ft) from the end of the beam, based on overall length, are 266.6 ft-kips and 7.8 ft-kips respectively. All strands are effective at this location, therefore use full value of P_i.

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t}$$

$$f_t = \frac{918.7}{813} - \frac{918.7(14.71)}{8,542} + \frac{(266.6 + 7.8)(12)}{8,542} = 1.130 - 1.582 + 0.385$$

$$= -0.067 \text{ ksi}$$

Allowable tension with no bonded reinforcement = –0.190 ksi O.K.

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b}$$

$$f_b = \frac{918.7}{813} + \frac{918.7(14.71)}{8,728} - \frac{(266.6 + 7.8)(12)}{8,728} = 1.130 + 1.548 - 0.377$$

$$= +2.301 \text{ ksi}$$

Allowable compression = +2.400 ksi O.K.

9.1.8.4 Stresses at Midspan

Bending moments due to beam self-weight and diaphragm weight at midspan, based on overall length, are 975.7 ft-kips and 35.2 ft-kips respectively.

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t}$$

$$f_t = \frac{918.7}{813} - \frac{918.7(14.71)}{8,542} + \frac{(975.7 + 35.2)(12)}{8,542} = 1.130 - 1.582 + 1.420$$

$$= +0.968 \text{ ksi}$$

Allowable compression = +2.400 ksi O.K.

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b}$$

$$f_b = \frac{918.7}{813} + \frac{918.7(14.71)}{8,728} - \frac{(975.7 + 35.2)(12)}{8,728} = 1.130 + 1.548 - 1.390$$

$$= +1.288 \text{ ksi}$$

Allowable compression = +2.400 ksi O.K.

9.1.8.5 Summary of Stresses at Transfer

	Top of Beam f_t (ksi)	Bottom of Beam f_b (ksi)
At transfer length section	–0.168	+1.896
At end of debonded strands + transfer length	–0.067	+2.301
At midspan	+0.968	+1.288

9.1.9 CONCRETE STRESSES AT SERVICE LOADS

9.1.9.1 Allowable Stress Limits

[STD Art. 9.15.2.2]

Compression:

(Case I): for all load combinations

$$= 0.60f_c' = 0.6(5{,}000)/1{,}000 = +3.000 \text{ ksi}$$

(Case II): for effective pretension force plus permanent dead loads

$$= 0.40f_c' = 0.4(5{,}000)/1{,}000 = +2.000 \text{ ksi}$$

(Case III): for live loads plus 1/2 (pretension + permanent (dead) loads)

$$= 0.40f_c' = 0.4(5{,}000)/1{,}000 = +2.000 \text{ ksi}$$

$$\text{Tension} = 6\sqrt{f_c'} = -6\sqrt{5{,}000}/1{,}000 = -0.424 \text{ ksi}$$

9.1.9.2 Stresses at Midspan

Bending moment values at this section are given in **Table 9.1.4.2.4-1**.
Compute concrete stress at top fiber of beam.

Case I:

$$f_t = \frac{P_{se}}{A} - \frac{P_{se}e_c}{S_t} + \frac{M_g + M_D + M_b + M_{ws}}{S_t} + \frac{M_{LL+I}}{S_t}$$

$$f_t = \frac{814.0}{813} - \frac{814.0(14.71)}{8{,}542} + \frac{(955.5 + 34.7 + 97.0 + 151.2)(12)}{8{,}542} + \frac{734.7(12)}{8{,}542}$$

$$= 1.001 - 1.402 + 1.740 + 1.032 = +2.371 \text{ ksi}$$

Allowable compression = +3.000 ksi O.K.

Case II:

$$f_t = 1.001 - 1.402 + 1.740 = +1.339$$

Allowable compression = +2.000 ksi O.K.

Case III:

$$f_t = 1.032 + 0.5(+1.001 - 1.402 + 1.740) = +1.702 \text{ ksi;}$$

Allowable compression = +2.000 ksi O.K.

Compute concrete stress at bottom of beam:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} - \frac{M_g + M_D + M_b + M_{ws}}{S_b} - \frac{M_{LL+I}}{S_b}$$

$$f_b = \frac{814.0}{813} + \frac{814.0(14.71)}{8{,}728} - \frac{(955.5 + 34.7 + 97.0 + 151.2)(12)}{8{,}728} - \frac{734.7(12)}{8{,}728}$$

$$= 1.001 + 1.372 - 1.703 - 1.010 = -0.340 \text{ ksi}$$

Allowable tension = –0.424 ksi O.K.

9.1.9.3
Summary of Stresses at Service Load

	Top of Beam, f_t, ksi			Bottom of Beam, f_b, ksi
	Case I	Case II	Case III	
At midspan:	+2.371	+1.339	+1.702	–0.340

9.1.10 DUCTILITY LIMITS

9.1.10.1
Maximum Reinforcement

[STD Art. 9.18.1]

Pretensioned concrete members are designed so that the steel is yielding as ultimate capacity is approached.

Reinforcement index for flanged section:

$$\frac{A_{sr} f^*_{su}}{b'df'_c} \le 0.36\,\beta_1 = \frac{0.982(257.1)}{10.0(36.59)(5.0)} = 0.138 < 0.36\,(0.80) = 0.288 \quad \text{O.K.}$$

[STD Eq. 9-21]

9.1.10.2
Minimum Reinforcement

[STD Art. 9.18.2]

The total amount of pretensioned and non-pretensioned reinforcement should be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment, M^*_{cr}:

$$\phi\, M_n \ge 1.2 M^*_{cr}$$

Compute cracking moment:

$$M^*_{cr} = (f_r + f_{pe})S_c - M_{d/nc}\left(\frac{S_c}{S_b} - 1\right)$$ [STD Art. 9.18.2.1]

where

$M_{d/nc}$ = moment due to non-composite dead loads

Since there is no composite action, substitute S_b for the section modulus of a composite section, S_c. Thus, the second term,

$M_{d/nc}\left(\frac{S_c}{S_b} - 1\right)$ is equal to zero.

f_r = modulus of rupture = $7.5\left(\frac{\sqrt{5{,}000}}{1{,}000}\right)$ = 0.530 ksi [STD Art. 9.15.2.3]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} = \left(\frac{814.0}{813} + \frac{814.0(14.71)}{8{,}728}\right) = 2.373 \text{ ksi}$$

$$M^*_{cr} = (0.530+2.373)\frac{8{,}728}{12} = 2{,}111.4 \text{ ft-kips}$$

$1.2M^*_{cr}$ = 2,533.7 ft-kips < ϕ M_n = 3,211.0 ft-kips O.K.

Contrary to the *LRFD Specifications,* the *Standard Specifications* do not specifically require that this criteria be met at all sections along the member.

9.1.11 SHEAR DESIGN

Pretensioned members subject to shear should be designed so that [Art. 9.20]

$$V_u \le \phi(V_c + V_s)$$ [STD Eq. 9-26]

where

V_u = the factored shear force at the section considered

V_c = the nominal shear strength provided by concrete

V_s = the nominal shear strength provided by web reinforcement

ϕ = strength reduction factor for shear = 0.90 [STD Art. 9.14]

The critical section in pretensioned concrete beams is located at a distance h/2 from the face of the support, according to the *Standard Specifications,* Article 9.20.1.4. Since the width of the bearing has not yet been determined, it will be conservatively assumed to be zero.

Therefore, detailed calculations are shown here for the section at h/2 (0.017 L = 1.625 ft) from the center line of support or 2.125 ft from the end of the member. The following calculations demonstrate how to compute V_{ci} and V_{cw} at this location.

Shear forces at this section are given in **Table 9.1.4.2.4-1**

Compute V_{ci}: [STD Art 9.20.2.3]

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$ [STD Eq. 9-27]

where

V_d = shear force at section due to unfactored dead load

= 38.9 + 1.1 + 3.9 + 6.1 = 50.0 kips

V_{LL+I} = unfactored shear force at section due to live load + impact

= 32.7 kips

M_d = bending moment at section due to unfactored dead load

= 64.3 + 1.8 + 6.5 + 10.2 = 82.8 ft-kips

M_{LL+I} = unfactored bending moment at section due to live load + impact

= 53.2 ft-kips

V_u = factored shear force at section

$= 1.3(V_d + 1.67\ V_{LL+I}) = 1.3[50.0 + 1.67(32.7)] = 136.0$ kips

M_u = factored bending moment at section

$= 1.3(M_d + 1.67\ M_{LL+I}) = 1.3[82.8 + 1.67(53.2)] = 223.1$ ft-kips

V_{mu} = factored shear force occurring simultaneously with M_u. Conservatively use the maximum shear load occurring at this section.

= 1.3[50.0 + 1.67(32.7)] = 136.0 kips

M_{max} = maximum factored moment at section due to externally applied loads.

$= M_u - M_d = 223.1 - 82.8 = 140.3$ ft-kips

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}.

$= V_{mu} - V_d = 136.0 - 50.0 = 86.0$ kips

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses), at extreme fiber of section where tensile stress is caused by externally applied loads.

The beam at this section is under positive flexure. Thus, f_{pe} should be evaluated at the bottom of the beam. This section is also within the debonded length, so only 24 strands are effective. Thus,

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}e}{S_b}$$

where

e = eccentricity of the strands at h/2 = 13.96 in.

$P_{se} = 24(0.153)(171.6) = 630.1$ kips

$$f_{pe} = \frac{630.1}{813} + \frac{630.1(13.96)}{8{,}728} = 1.783 \text{ ksi}$$

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads.

$$= \frac{M_d}{S_b} = \frac{82.8(12)}{8{,}728} = 0.114 \text{ ksi}$$

M_{cr} = moment causing flexural cracking at section due to externally applied loads (after dead load).

$$= (6\sqrt{f'_c} + f_{pe} - f_d)S_b \qquad \text{[STD Eq. 9-28]}$$

$$= \left(\frac{6\sqrt{5{,}000}}{1{,}000} + 1.783 - 0.114\right)\frac{8{,}728}{12} = 1{,}522.5 \text{ ft-kips}$$

d = distance from extreme compressive fiber to centroid of the pretensioning force. However, d need not be taken less than 0.8 h = 31.20 in. [STD Art. 9.20.2.2]. The centroid of the pretensioned reinforcement is located at 5.33 in. from the bottom of the beam.

$= 39 - 5.33 = 33.67$ in. > 31.20 in.

Therefore, d = 33.67 in.

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad \text{[STD Eq. 9-27]}$$

$$= \frac{0.6\sqrt{5{,}000}(10)(33.67)}{1{,}000} + 50.0 + \frac{86.0(1{,}522.5)}{140.3} = 997.5 \text{ kips}$$

But this value should not be less than minimum V_{ci}.

$$\text{Minimum } V_{ci} = 1.7\sqrt{f'_c}\,b'd \qquad \text{[STD Art. 9.20.2.2]}$$

$$= 1.7\sqrt{5{,}000}\,\frac{10(33.67)}{1{,}000} = 40.5 \text{ kips} < V_{ci} = 997.5 \text{ kips} \quad \text{O.K.}$$

Therefore, V_{ci} = 997.5 kips

Compute V_{cw}: [STD Art 9.20.2.3]

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3f_{pc})b'd + V_p \qquad \text{[STD Eq. 9-29]}$$

where

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross-section resisting externally applied loads. For a non-composite section

$$f_{pc} = \frac{P_{se}}{A} = \frac{630.1}{813} = 0.775 \text{ ksi}$$

V_p = vertical component of effective pretension force at section

= 0 for straight strands

$$V_{cw} = \left(\frac{3.5\sqrt{5{,}000}}{1{,}000} + 0.3(0.775)\right)(10)(33.67) + 0 = 161.6 \text{ kips}$$

The allowable nominal shear provided by concrete should be the lesser of V_{ci} (997.5 kips) and V_{cw} (161.6 kips). Therefore,

V_c = 161.6 kips

$V_u = 136.0 < \phi V_c = 0.9(161.6) = 145.4$ kips O.K.

Since $V_c > V_u$, the concrete resists the total shear and shear steel is not required to assist in resisting V_u. Therefore, provide minimum reinforcement.

Minimum shear reinforcement [STD Art. 9.20.3.3]

$$A_{v\text{-}min} = \frac{50\,b's}{f_y} = \text{minimum area of web reinforcement} \qquad \text{[STD Eq. 9-31]}$$

where

s = longitudinal spacing of the web reinforcement, in.

Set s = 12 in.

$$A_{v\text{-}min} = \frac{50(10)(12)}{60{,}000} = 0.10 \text{ in.}^2 \text{ (per foot)}$$

Maximum spacing = 0.75h = 0.75(39) = 29.25 in. [STD Art. 9.20.3.2]
or 24.00 in.

Then, maximum s = 24 in.

Use # 4 at 24 in. spacing in each web (A_v = 0.20 in.^{2}/ft)

Note that the above calculations need to be repeated along the entire length of the beam at regular intervals to determine the area and spacing of shear reinforcement.

9.1.12 PRETENSIONED ANCHORAGE ZONE

[STD Art. 9.22]

9.1.12.1 Minimum Vertical Reinforcement

In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4% of the total pretensioning force should be placed within the distance of d/4 of the end of the beam. [STD Art. 9.22.1]

Note that the force in the bonded strands prior to release is used.

Minimum steel at the end of the beam:

P_{si} = 24(0.153)(202.5) = 743.6 kips

4% P_{si} = 0.04(743.6) = 29.7 kips

$$A_v = \frac{29.7}{20.0} = 1.485 \text{ in.}^2$$

$$\frac{d}{4} = \frac{34.42}{4} = 8.61 \text{ in.}$$

Use an orthogonal welded wire reinforcement of #4 bars @ 12 in. both vertically and horizontally. Space layers of WWR @ 3 in. starting @ 2 in. from end of beam. (area of steel provided = 2.40 in.^{2})

9.1.12.2 Confinement Reinforcement

Provide nominal reinforcement to enclose the pretensioning steel for a distance equal to the depth of the beam from the end of the beam.

9.1.13 DEFLECTION AND CAMBER

9.1.13.1 Deflection Due to Pretensioning Force at Transfer

For straight strands,

$$\Delta_p = \frac{P_{si}\ e_c L^2}{8\ E_{ci} I}$$

where

Δ_p = camber due to pretension force at transfer

L = overall span length = 96 ft

$$\Delta_p = \frac{918.7(14.71)[(96)(12)]^2}{8(3{,}834)(168{,}367)} = 3.47 \text{ in.}\uparrow$$

9.1.13.2 Deflection Due to Beam Self-Weight

$$\Delta_{beam} = \frac{5 w_g L^4}{384\ E_{ci}\ I}$$

where w_g = beam self-weight = 0.847 kips/ft

Deflection due to beam self-weight at transfer (L = 96 ft):

$$\Delta_{beam} = \frac{5\left(\frac{0.847}{12}\right)[(96)(12)]^4}{384\,(3{,}834)\,(168{,}367)} = 2.51 \text{ in.}\downarrow$$

Deflection due to beam self-weight used to compute deflection at erection (L = 95 ft):

$$\Delta_{beam} = \frac{5\left(\frac{0.847}{12}\right)[(95)(12)]^4}{384\,(3{,}834)\,(168{,}367)} = 2.40 \text{ in.}\downarrow$$

9.1.13.3 Deflection Due to Diaphragm Weight

$$\Delta_D = \frac{19\ P\ L^3}{384\ E_{ci}\ I}$$

where P = 0.73 kips

Deflection due to diaphragm weight at transfer (L = 96 ft):

$$\Delta_D = \frac{19(0.73)[(96)(12)]^3}{384(3{,}834)(168{,}367)} = 0.09 \text{ in.} \downarrow$$

Deflection due to diaphragm weight used to calculate deflection at erection (L = 95 ft):

$$\Delta_D = \frac{19(0.73)[(95)(12)]^3}{384(3{,}834)(168{,}367)} = 0.08 \text{ in.} \downarrow$$

9.1.13.4
Total Initial Deflection

Total deflection at transfer:

$$3.47 - 2.51 - 0.09 = 0.87 \text{ in.} \uparrow$$

Total deflection at erection, using PCI multipliers (see PCI Design Handbook):

$$= 1.80(3.47) - 1.85(2.40 + 0.08) = 1.66 \text{ in.} \uparrow$$

PCI multipliers used to calculate long term deflection have generally proven to be inaccurate for bridge construction. Therefore, it is recommended that the designer follow the guidelines of the owner agency for whom the bridge is being designed. Frequently, no additional multipliers are used.

9.1.13.5
Deflection Due to Barrier and Wearing Surface Weights

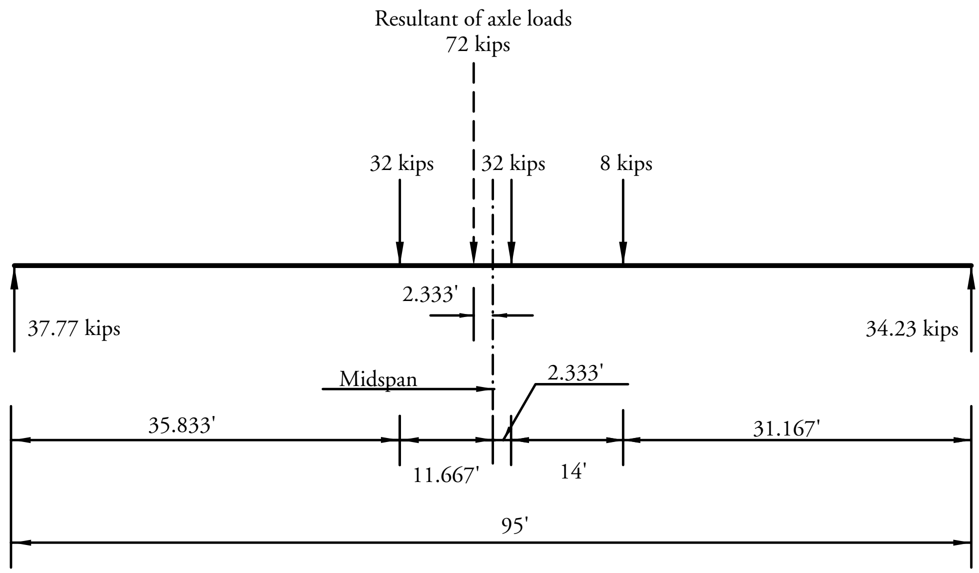
Since these loads are applied to the structure in its final location, L is the design span = 95 ft

$$\Delta_{SDL} = \frac{5(w_b + w_{ws})L^4}{384E_cI} = \frac{5\left(\frac{0.086 + 0.134}{12}\right)[(95)(12)]^4}{384(4{,}287)(168{,}367)} = 0.56 \text{ in.}\downarrow$$

9.1.13.6
Deflection Due to Live Load and Impact

Live load deflection limit (optional) = L/800 = $\frac{95(12)}{800}$ = 1.43 in.

Figure 9.1.13.6-1 Design Truck Axle Loads Positions on the Span for Maximum Moment



To get maximum moment and deflection at midspan section, arrange the wheel loads for the truck as shown in the **Figure 9.1.13.6-1**.

Some state DOTs consider that all beams act together in resisting deflection due to live load and impact. Using that assumption, the distribution factor for deflection is:

$$DF = \frac{\text{number of lanes}}{\text{number of beams}} = \frac{2}{7} = 0.286 \text{ lanes/beam}$$

For all design examples, deflection is conservatively computed based on the distribution factor used to calculate the bending stresses.

Using the elastic moment area method:

$$\Delta_{LL+I} = 2.77(I)(DF_m)$$

$$\Delta_{LL+I} = 2.77(1.230)(0.334) = 1.14 \text{ in.} < 1.43 \text{ in.} \quad \text{O.K.}$$

9.1.14 TRANSVERSE POST-TENSIONING

The *Standard Specifications* do not have any provisions for transverse design of adjacent boxes. Therefore, the transverse design will be done according to *LRFD Specifications.*

Article C4.6.2.2.1 in the *LRFD Specifications,* states that for this type of bridge, the structure acts as a monolithic unit if sufficiently interconnected. To satisfy this requirement, the *LRFD Specifications* recommend that a minimum average transverse pretension of 0.250 ksi be used. However, definition of the contact area for that post-tensioning is unclear as to whether it is the shear key, the diaphragm, or the entire box-side surface. Instead of an empirical minimum, El-Remaily (1996) recommends that the entire deck surface be modeled as a rigid assembly of gridwork with adequate post-tensioning to provide for a continuous transverse member at the diaphragm locations. A design chart based on this theory is given in Chapter 8 for the required transverse post-tensioning per unit length of the span.

According to the chart, for a 28 ft-wide bridge with 39 in.-deep beams, an effective post-tensioning force of 4.0 kips/ft is required. Since diaphragms are provided at quarter-points of the span, the post-tensioning force required is:

4.0 (23.75) = 95 kips/diaphragm

It is recommended that transverse post-tensioning consist of one tendon near the top and another near the bottom in order to provide sufficient flexural strength.
Use 160 ksi pretensioning bars. Assume the effective pretension to be 55 percent of the ultimate strength of the bar:

$P_{eff} = 0.55(160)A_{PT} = 88.0A_{PT}$ kips

Thus, total required $A_{PT} = \dfrac{95.0}{88.0} = 1.08$ in.^{2}

Try (2) 7/8-in. dia, 160 ksi bars.

Total provided area, $A_{PT} = 2(0.601) = 1.202$ in.^{2}

Total provided post-tensioning force = (1.202)(0.55)(160)= 105.8 kips/diaphragm

> 95.0 kips/diaphragm O.K.

If the post-tensioning bars are positioned so that they are concentric with the diaphragm cross-section, concrete stress due to effective pretensioning force is:

105.8/[(8)(39)] = 0.339 ksi

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pb} = compressive stress at bottom fiber of the beam due to prestress force

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = the modulus of rupture of concrete [STD]

f_r = modulus of rupture of concrete [LRFD]

f_s = allowable stress in steel

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress

f_{si} = effective initial pretension stress

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

f_t = concrete stress at top fiber of the beam for the non-composite section

f_{tc} = concrete stress at top fiber of the slab for the composite section

f_{tg} = concrete stress at top fiber of the beam for the composite section

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity, percent [LRFD]

H = height of wall [LRFD]

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_c	= total height of composite section	
h_f	= compression flange depth	[LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD]
I	= moment of inertia about the centroid of the non-composite precast beam	[LRFD]
I	= impact fraction (maximum 30%)	[STD]
I_c	= moment of inertia of composite section	
IM	= dynamic load allowance	[LRFD]
J	= St. Venant torsional constant	
K	= longitudinal stiffness parameter	[STD]
K_g	= longitudinal stiffness parameter	[LRFD]
k	= factor used in calculation of distribution factor for multi-beam bridges	[LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
L_c	= critical length of yield line failure pattern	[LRFD]
LL	= vehicular live load	[LRFD]
ℓ_d	= development length	[LRFD]
ℓ_x	= length required to fully develop the strand measured from the end of the strand	
M_a	= negative moment at the end of the span being considered	
M_b	= negative moment at the end of the span being considered	
M_b	= unfactored bending moment due to barrier weight	
M_c	= flexural resistance of cantilevered wall	[LRFD]
M_{CIP}	= unfactored bending moment due to cast-in-place topping slab	
M_{const}	= unfactored bending moment due to construction load	
M_{col}	= bending moment due to horizontal collision force	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads (after dead load)	[STD]
M_{cr}	= cracking moment	[LRFD]
M^*_{cr}	= cracking moment	[STD]
M_D	= unfactored bending moment due to diaphragm weight	
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_f	= unfactored bending moment due to fatigue truck per beam	
M_g	= unfactored bending moment due to beam self-weight	
M_{LL}	= unfactored bending moment due to lane load per beam	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_{LL+I}	= unfactored bending moment due to design vehicular load	
M_{LT}	= unfactored bending moment due to truck load with dynamic allowance per beam	

M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_s	= maximum positive moment	
M_s	= unfactored bending moment due to slab and haunch weights	
M_{SDL}	= unfactored bending moment due to super-imposed dead loads	
$M_{service}$	= total bending moment for service load combination	
M_{SIP}	= unfactored bending moment due to stay-in-place panel	
M_u	= factored bending moment at section	[STD]
M_u	= factored moment at a section	[LRFD]
M_{ws}	= unfactored bending moment due to wearing surface	
M_x	= bending moment at a distance (x) from the support	
m	= material parameter	
m	= stress ratio = $(f_y/0.85f'_c)$	
N_b	= number of beams	[LRFD]
N_L	= number of traffic lanes	[STD]
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
n	= modular ratio between deck slab and beam materials	
P	= diaphragm weight concentrated at quarter points	
P	= load on one rear wheel of design truck (P_{15} or P_{20})	[STD]
P_c	= permanent net compression force	[LRFD]
P_{eff}	= effective post-tensioning force	
P_i	= total pretensioning force immediately after transfer	
P_{pe}	= total pretensioning force after all losses	
P_r	= factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement	
P_s	= prestress force before initial losses	
P_{se}	= effective pretension force after allowing for all losses	
P_{si}	= effective pretension force after allowing for the initial losses	
P_{20}	= load on one rear wheel of the H20 truck	[STD]
Q	= total factored load	[LRFD]
Q_i	= specified loads	[LRFD]
q	= generalized load	[LRFD]
RH	= relative humidity	[STD]
R_n	= coefficient of resistance	
R_u	= flexural resistance factor	
R_w	= total transverse resistance of the railing or barrier	[LRFD]
S	= width of precast beam	[STD]
S	= average spacing between beams in feet	[STD]
S	= spacing of beams	[LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= $\frac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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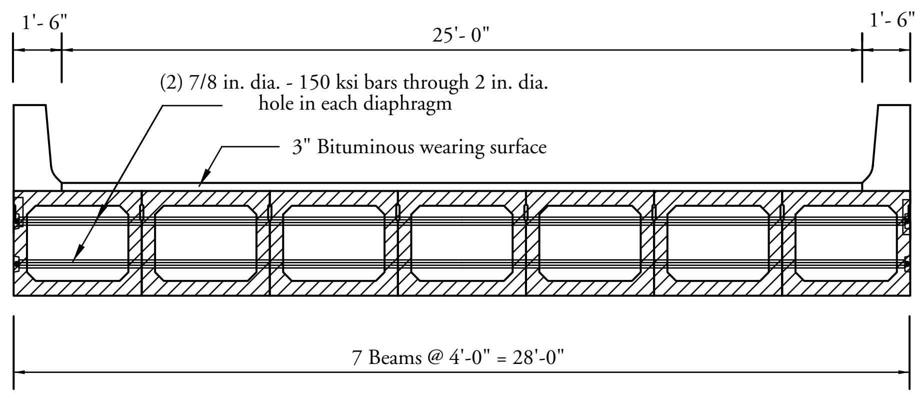
9.2.1 INTRODUCTION

This design example demonstrates the design of a 95-ft single-span AASHTO Type BIII-48 box beam bridge with no skew. This example illustrates in detail the design of a typical interior beam at the critical sections in positive flexure, shear and deflection due to prestress, dead loads and live loads. The superstructure consists of seven beams abutted as shown in **Figure 9.2.1-1**. A 3-in. bituminous surfacing will be placed on the beams as a wearing surface. Beams are transversely post-tensioned through 8-in.-thick full-depth diaphragms located at quarter-points. Design live load is HL-93. The design is accomplished in accordance with the *"AASHTO LRFD Bridge Design Specifications,"* 2nd Edition, 1998, and including through the 2003 Interim Revisions.

Figure 9.2.1-1
Bridge Cross-Section



9.2.2 MATERIALS

Precast beams: AASHTO Box Beam, Type BIII-48, as shown in **Figure 9.2.2-1**

Concrete strength at transfer, $f'_{ci} = 4.0$ ksi

Concrete strength at 28 days, $f'_c = 5.0$ ksi

Concrete unit weight, $w_c = 0.150$ kcf

Overall beam length = 96.0 ft

Design span = 95.0 ft

Prestressing strands: 1/2 in. dia., seven-wire, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate strength $f_{pu} = 270.0$ ksi

Yield strength $f_{py} = 0.9f_{pu} = 243.0$ ksi [LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

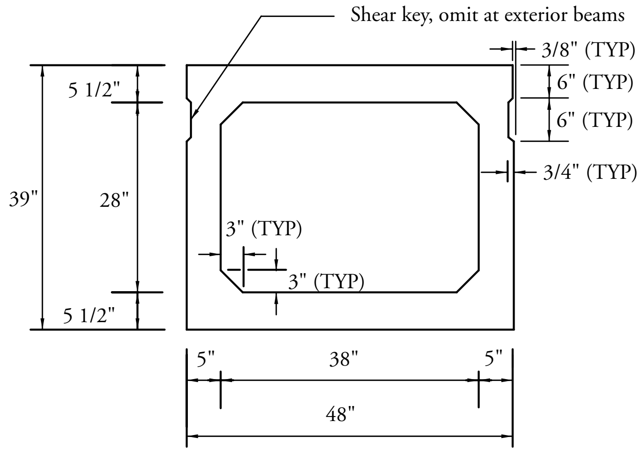
before transfer, $f_{pi} \leq 0.75f_{pu} \leq 202.5$ ksi

at service limit state (after all losses), $f_{pe} \leq 0.8f_{py} \leq 194.4$ ksi

Modulus of elasticity, $E_p = 28{,}500$ ksi [LRFD Art. 5.4.4.2]

BOX BEAM (BIII-48), SINGLE SPAN, NON-COMPOSITE SURFACE, LRFD SPECIFICATIONS

Figure 9.2.2-1
AASHTO Box Beam
Type BIII-48



Reinforcing bars: yield strength, f_y = 60.0 ksi

modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Bituminous surfacing: unit weight = 0.140 kcf [LRFD Table 3.5.1-1]

New Jersey-type barrier: unit weight = 0.300 kip/ft/side

9.2.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

The standard precast concrete box beams shown in Appendix B have dimensions that result in the entire top flange being effective in resisting flexural loads. This is because 12 times the top flange thickness of the box is greater than the entire width of the top flange.

A = area of cross-section of precast beam = 813 in.^{2}

h = overall depth of precast beam = 39 in.

I = moment of inertia about the centroid of the beam = 168,367 in.^{4}

y_b = distance from centroid to the extreme bottom fiber of the precast beam = 19.29 in.

y_t = distance from centroid to the extreme top fiber of the precast beam = 19.71 in.

S_b = section modulus for extreme bottom fiber of the precast beam = I/y_b = 8,728 in.^{3}

S_t = section modulus for extreme top fiber of the precast beam = I/y_t = 8,542 in.^{3}

Wt. = 0.847 kip/ft

$E_c = 33{,}000\,(w_c)^{1.5}\sqrt{f'_c}$ [LRFD Eq. 5.4.2.4-1]

where

E_c = modulus of elasticity of concrete (ksi)

w_c = unit weight of concrete (kcf) = 0.150 kcf

The *LRFD Specifications* Commentary, C5.4.2.4, indicates that the unit weight of normal weight concrete is 0.145 kcf. However, precast concrete mixes typically have a relatively low water/cementitious materials ratio and high density. Therefore, a unit weight of 0.150 kcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f'_c = specified strength of concrete (ksi)

Modulus of elasticity at transfer:

$E_{ci} = 33{,}000(0.150)^{1.5}\sqrt{4.0} = 3{,}834$ ksi

Modulus of elasticity at service loads:

$E_c = 33{,}000(0.150)^{1.5}\sqrt{5.0} = 4{,}287$ ksi

9.2.4 SHEAR FORCES AND BENDING MOMENTS

9.2.4.1. Shear Forces and Bending Moments Due to Dead Loads

9.2.4.1.1 Dead Loads

[LRFD Art. 3.3.2]

DC = dead load of structural components and nonstructural attachments

Beam self-weight = 0.847 kip/ft

$$\text{Diaphragm weight} = \left(\frac{8}{12}\right)\left[\frac{(48-10)}{12}\,\text{x}\,\frac{(39-11)}{12} - 4\left(\frac{1}{2}\right)\left(\frac{3}{12}\right)\left(\frac{3}{12}\right)\right](0.15)$$

= 0.73 kip/diaphragm

Generally, the unit weight of reinforced concrete should be slightly greater than the unit weight of concrete alone because of the added weight of reinforcement. However, in this example, the difference is considered negligible.

The weights of the diaphragms are considered concentrated loads acting at quarter-points as shown in **Figure 9.2.4.1.1-1**.

Barrier Wt. = (2)(0.300 kips)/(7 beams) = 0.086 kip/ft

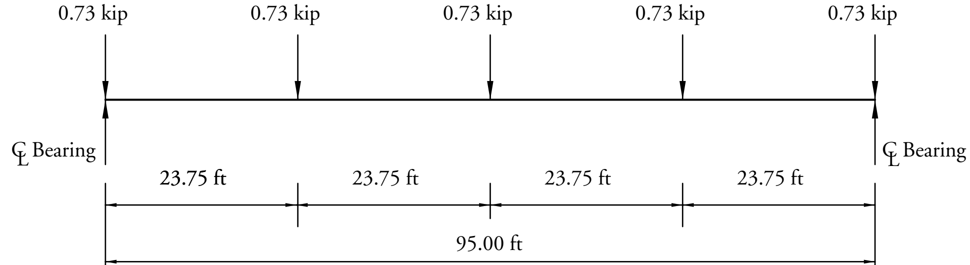
DW = dead load of wearing surface and utilities (weight of 3 in. bituminous wearing surface = 0.140 kcf) [LRFD Table 3.5.1-1]

$$= \left(\frac{3}{12}\text{ft}\right)\frac{(25\text{ft})(0.140\text{kcf})}{(7\text{beams})} = 0.125 \text{ kip/ft}$$

DW load should be kept separately from DC because a higher load factor is applied to it. LRFD Article 4.6.2.2.1 states that permanent loads (barrier and wearing surface loads) may be distributed uniformly among the beams if the following conditions are met:

- Width of the slab is constant O.K.
- Number of beams, N_b, is not less than four ($N_b = 7$) O.K.
- The roadway part of the overhang, $d_e \le 3.0$ ft. ($d_e = 0.0$) O.K.
- Curvature in plan is less than 4° (curvature = 0.0) O.K.

Figure 9.2.4.1.1-1
Diaphragm Loads per Beam



- Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1. O.K.

Since these criteria are satisfied, the barrier and wearing surface loads have been equally distributed among the seven beams.

9.2.4.1.2 Unfactored Shear Forces and Bending Moments

For a simply supported beam with a span length (L) loaded with a uniformly distributed load (w), the shear force (V_x) and the bending moment (M_x) at a distance (x) from the support are given by:

$$V_x = w(0.5L - x) \quad \text{(Eq. 9.2.4.1.2-1)}$$

$$M_x = 0.5wx(L - x) \quad \text{(Eq. 9.2.4.1.2-2)}$$

Using the above equations, values of shear forces and bending moments for a typical interior beam under the self-weight of beam, diaphragms, barriers, and wearing surface are computed and given in **Table 9.2.4.1.2-1**. Using statics, values of shear forces and bending moments due to diaphragm weight are calculated and given in **Table 9.2.4.1.2-1**. For these calculations, the span length is the design span (95 ft). However, for calculation of stresses and deformations at the time of prestress release, the span length should be the overall length of the precast member (96 ft) as illustrated later in this example.

Table 9.2.4.1.2-1 Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Distance x	Section x/L	Beam weight		Diaphragm weight		Barrier weight		Wearing surface	
		Shear	Moment M_g	Shear	Moment M_D	Shear	Moment M_b	Shear	Moment M_{ws}
ft		kip	ft-kip	kip	ft-kip	kip	ft-kip	kip	ft-kip
0.000	0.000	40.2	0.0	1.1	0.0	4.1	0.0	5.9	0.0
3.560 [1]	0.038	37.2	137.8	1.1	3.9	3.8	14.0	5.5	20.3
9.500	0.100	32.2	344.0	1.1	10.4	3.3	34.9	4.8	50.8
19.000	0.200	24.1	611.5	1.1	20.8	2.5	62.1	3.6	90.3
28.500	0.300	16.1	802.6	0.4	27.7	1.6	81.5	2.4	118.5
38.000	0.400	8.0	917.3	0.4	31.2	0.8	93.1	1.2	135.4
47.500	0.500	0.0	955.5	0.4	34.7	0.0	97.0	0.0	141.0

[1] Critical section for shear (see section 9.2.11)

Distance x	Section x/L	Design truck with dynamic allowance		Design lane load		Fatigue truck with dynamic allowance
		Shear V_{LT}	Moment M_{LT}	Shear V_{LL}	Moment M_{LL}	Moment M_f
ft		kip	ft-kip	kip	ft-kip	ft-kip
0.000	0.000	38.2	0.0	13.5	0.0	0.0
3.560 [1]	0.038	36.5	84.2	12.4	29.8	35.0
9.500	0.100	34.0	209.3	10.9	74.6	85.0
19.000	0.200	29.8	366.4	8.6	132.6	145.9
28.500	0.300	25.5	471.3	6.6	174.1	187.7
38.000	0.400	21.3	532.6	4.8	198.9	208.8
47.500	0.500	17.0	545.8	3.4	207.2	205.9

[1] Critical section for shear (see section 9.2.11)

9.2.4.2 Shear Forces and Bending Moments Due to Live Loads

9.2.4.2.1 Live Loads

The *LRFD Specifications* state that vehicular live loading on the roadway of bridges, HL-93, consists of a combination of: [LRFD Art. 3.6.1.2.1]

1. Design truck or design tandem with dynamic allowance. The design truck is the same as the HS-20 design truck specified in the Standard Specifications, [LRFD Art. 3.6.1.2.2]. The design tandem consists of a pair of 25.0 kip axles spaced at 4.0 ft apart, [LRFD Art. 3.6.1.2.3].
2. Design lane load of 0.64 kip/ft without dynamic allowance, [LRFD Art. 3.6.1.2.4].

9.2.4.2.2 Live Load Distribution Factors for a Typical Interior Beam

The live load bending moments and shear forces are determined by using the simplified distribution factor formulas [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions must be met: [LRFD Art. 4.6.2.2.1]

- Width of slab is constant O.K.
- Number of beams, $N_b \geq 4$ ($N_b = 7$) O.K.
- Beams are parallel and approximately of the same stiffness O.K.
- The roadway part of the overhang, $d_e \leq 3.0$ ft. ($d_e = 0.0$) O.K.
- Curvature in plan is less than 5° [LRFD Table. 4.6.1.2.1-1] (Curvature = 0.0°) O.K.

For a precast cellular concrete box with shear keys and with or without transverse post-tensioning, the bridge type is (g). [LRFD Table 4.6.2.2.1-1]

In order to use live load distribution factor formulas, it is necessary to determine the number of lanes.

Number of design lanes = The integer part of the ratio w/12, where w is the clear roadway width, in ft, between the curbs [LRFD Art. 3.6.1.1.1]

From **Figure 9.2.1-1**, w = 25 ft

Number of design lanes = Integer part of (25/12) = 2 lanes

9.2.4.2.2.1 Distribution Factor for Bending Moments

- For all limit states except Fatigue Limit State

For two or more lanes loaded, if members are sufficiently connected to act as a unit:

$$DFM = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$$ [LRFD Table 4.6.2.2.2b-1]

Provided that:	$35 \leq b \leq 60$	$b = 48$ in O.K.
	$20 \leq L \leq 120$	$L = 95$ ft O.K.
	$5 \leq N_b \leq 20$	$N_b = 7$ O.K.

where

DFM = Distribution factor for moment for interior girder

b = Beam width, in.

L = Beam span, ft

I = moment of inertia of the beam, in.^{4}

k $= 2.5(N_b)^{-0.2} \geq 1.5 = 2.5(7)^{-0.2} = 1.694 > 1.5$ O.K.

N_b = Number of beams

J = St. Venant torsional constant, in.^{4}

For closed, thin-walled shapes:

where

$$J \sim \frac{4A_0^2}{\sum \frac{s}{t}}$$ [LRFD Eq. C4.6.2.2.1-3]

where

A_o = area enclosed by centerlines of the elements of the beam
$= (48 - 5)(39 - 5.5) = 1{,}440.5$ in.^{2}

s = length of an element of the beam

t = thickness of an element of the beam

$$J = \frac{4(1{,}440.5)^2}{2\left(\frac{48-5}{5.5}\right)+2\left(\frac{39-5.5}{5}\right)} = 285{,}854 \text{ in.}^4$$

Therefore:

$$DFM = 1.694\left(\frac{48}{305}\right)^{0.6}\left(\frac{48}{12.0\text{x}95}\right)^{0.2}\left(\frac{168{,}367}{285{,}854}\right)^{0.06} = 0.287 \text{ lanes/beam}$$

For one lane loaded, if sufficiently connected to act as a unit:

$$DFM = k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$$ [LRFD Table 4.6.2.2.2b-1]

$$= 1.694\left(\frac{48}{33.3\text{x}95}\right)^{0.5}\left(\frac{168{,}367}{285{,}854}\right)^{0.25} = 0.183 \text{ lanes/beam}$$

Thus, the case of two or more lanes loaded controls, DFM = 0.287 lanes/beam.

• For Fatigue Limit State:

LRFD Article 3.4.1 states that for fatigue limit state, a single design truck should be used. Therefore, the distribution factor for one design lane loaded should be used as previously calculated. However, live load distribution factors given in Article 4.6.2.2 in the *LRFD Specifications* take into consideration the multiple presence factor, m. LRFD Article 3.6.1.1.2 states that the multiple presence factor, m, for one design lane loaded is 1.2. Distribution factor for fatigue limit state is:

0.183/1.2 = 0.153 lanes/beam

9.2.4.2.2.2 Distribution Factor for Shear Forces

For two or more lanes loaded:

$$DFV = \left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12.0L}\right)^{0.1}\left(\frac{I}{J}\right)^{0.05}$$ [LRFD Table 4.6.2.2.3a-1]

where DFV = Distribution factor for shear for interior beam

Provided that:	$35 \le b \le 60$	$b = 48$ in	O.K.
	$20 \le L \le 120$	$L = 95$ ft	O.K.
	$5 \le N_b \le 20$	$N_b = 7$	O.K.
	$25{,}000 \le J \le 610{,}000$	$J = 285{,}854$ in.^{4}	O.K.
	$40{,}000 \le I \le 610{,}000$	$I = 168{,}367$ in.^{4}	O.K.

$$DFV = \left(\frac{48}{156}\right)^{0.4}\left(\frac{48}{12.0\text{x}95}\right)^{0.1}\left(\frac{168{,}367}{285{,}854}\right)^{0.05} = 0.443 \text{ lanes/beam}$$

For one design lane loaded:

$$DFV = \left(\frac{b}{130L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05} = \left(\frac{48}{130 \times 95}\right)^{0.15}\left(\frac{168{,}367}{285{,}854}\right)^{0.05}$$ [LRFD Table 4.6.2.2.3a-1]

$= 0.424$ lanes/beam

Thus, the case of two lanes loaded controls, DFV = 0.443

9.2.4.2.3 Dynamic Allowance

[LRFD Art. 3.6.2]

IM = 15% for fatigue limit state

= 33% for all other limit states [LRFD Table 3.6.2.1-1]

where IM = dynamic load allowance, applied to design truck load only

9.2.4.2.4 Unfactored Shear Forces and Bending Moments

9.2.4.2.4.1 Due to Design Truck Load, V_{LT} and M_{LT}

- For all limit states except for Fatigue Limit State:

Shear force and bending moment envelopes on a per-lane-basis are calculated at tenth-points of the span using the equations given in Chapter 8 of this manual. However, this is generally done with commercially available computer software that has the ability to deal with moving loads.

Therefore, truck load shear forces and bending moments per beam are:

V_{LT} = (shear force per lane)(DFV)(1 + IM)
= (shear force per lane)(0.443)(1 + 0.33)
= (shear force per lane)(0.589) kips

M_{LT} = (bending moment per lane)(DFM)(1 + IM)
= (bending moment per lane)(0.287)(1 + 0.33)
= (bending moment per lane)(0.382) ft-kips

Values of V_{LT} and M_{LT} at different points are given in **Table 9.2.4.1.2-1**.

• For Fatigue Limit State:

LRFD Article 3.6.1.4.1 states that the fatigue load is a single design truck which has the same axle weight used in all other limit states but with a constant spacing of 30.0 ft between the 32.0-kip axles.

The bending moment envelope on a per-lane-basis is calculated using the equation given in Chapter 8 of this manual.

Therefore, bending moment of fatigue truck load is:

M_f = (bending moment per lane)(DFM)(1 + IM)
= (bending moment per lane)(0.153)(1 + 0.15)
= (bending moment per lane)(0.176) ft-kips

Values of M_f at different points are given in **Table 9.2.4.1.2-1**.

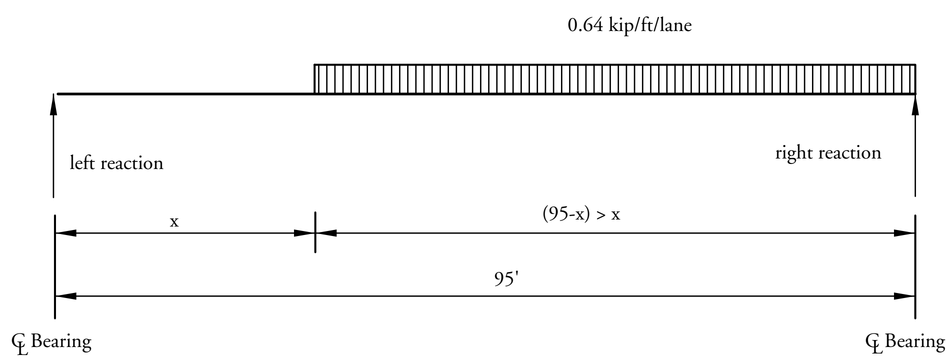
9.2.4.2.4.2 Due to Design Lane Load, V_{LL} and M_{LL}

To obtain the maximum shear force at a section located at a distance (x) from the left support under a uniformly distributed load of 0.64 kip/ft, load the member to the right of the section under consideration as shown in **Figure 9.2.4.2.4.2-1**. Therefore, the maximum shear force per lane is:

$$V_x = \frac{0.32(L-x)^2}{L} \quad \text{for } x \le 0.5L \qquad \text{(Eq. 9.2.4.2.4.2-1)}$$

where V_x is in kip/lane and L and x are in ft.

Figure 9.2.4.2.4.2-1 Maximum Shear Force due to Design Lane Load



In order to get the maximum bending moment at any section, use Eq. (9.2.4.1.2-2).

Therefore, lane load shear forces and bending moments per beam are:

V_{LL} = (lane load shear force)(DFV)
= (lane load shear force)(0.443) kips

For all limit states except for fatigue limit state:

M_{LL} = (lane load bending moment)(DFM)
= (lane load bending moment)(0.287) ft-kips

Note that the dynamic allowance is not applied to the design lane loading.

Values of shear forces and bending moments, V_{LL} and M_{LL}, are given in **Table 9.2.4.1.2-1**.

[LRFD Art. 3.4]

9.2.4.3 Load Combinations

Total factored load, Q, is taken as:

$Q = \eta\Sigma\, \gamma_i\, q_i$ [LRFD Eq. 3.4.1-1]

where η = a factor relating to ductility, redundancy, and operational importance for this example, η is considered to be 1.0 [LRFD Art. 1.3.2.1]

γ_i = load factors [LRFD Table 3.4.1-1]

q_i = specified loads

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

Service I : to check compressive stress in prestressed concrete components

$Q = 1.00(DC + DW) + 1.00(LL + IM)$ [LRFD Table 3.4.1-1]

This load combination is the general combination for service limit state stress and applies to all conditions other than Service III.

Service III: to check tensile stress in prestressed concrete components

$Q = 1.00(DC + DW) + 0.80(LL + IM)$ [LRFD Table 3.4.1-1]

This load combination is a special combination for service limit state stress that applies only to tension in prestressed concrete structures to control cracks.

Strength I: to check ultimate strength [LRFD Tables 3.4.1-1 and 2]

Q maximum $= 1.25(DC) + 1.50(DW) + 1.75(LL + IM)$

Q minimum $= 0.90(DC) + 0.65(DW) + 1.75(LL + IM)$

This load combination is the general load combination for strength limit state design.

Note: For simple span bridges, the maximum load factors produce maximum effects. However, it is necessary to use minimum load factors for dead load (DC), and wearing surface (DW) when dead load and wearing surface stresses are opposite to those of the live load.

Fatigue: to check stress range in strands

$Q = 0.75(LL + IM)$

This is a special load combination to check the tensile stress range in the strands due to live load and dynamic allowance.

Note: The LL used in the above equation results only from a single design truck with a 30 ft constant spacing between the 32.0 kip axles with the special dynamic allowance, (IM) for fatigue.

9.2.5 ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber at the Service Limit State (Service III) and in some cases at the Strength Limit State (Strength I). For simple span structures, the location which governs is typically the section of maximum moment (midspan). In this example, the number of required strands is estimated based on the Service Limit State using the Service III load combination.

9.2.5.1 Service Load Stresses at Midspan

Bottom tensile stresses, according to service load combination III, due to applied loads is:

$$f_b = \frac{M_g + M_D + M_b + M_{ws} + (0.8)(M_{LT} + M_{LL})}{S_b}$$

where

f_b = bottom tensile stresses, ksi

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_D = unfactored bending moment due to diaphragm weight, ft-kips

M_b = unfactored bending moment due to barrier weight, ft-kips

M_{ws} = unfactored bending moment due to wearing surface, ft-kips

M_{LT} = unfactored bending moment due to truck load with dynamic allowance, ft-kips

M_{LL} = unfactored bending moment due to lane load, ft-kips

Using values of bending moments from **Table 9.2.4.1.2-1**, bottom tensile stress at midspan is:

$$f_b = \frac{(955.5 + 34.7) + (97.0 + 141.0) + (0.8)(545.8 + 207.2)}{8{,}728}(12) = 2.517 \text{ ksi}$$

9.2.5.2 Stress Limits for Concrete

Tensile stress limit at service limit state = $0.19\sqrt{f'_c}$ [LRFD Art. 5.9.4.2.2b]

where f'_c = specified concrete strength (ksi)

Therefore, tensile concrete stress limit = $-0.19\sqrt{5.0} = -0.425$ ksi

9.2.5.3 Required Number of Strands

Required precompressive stress at the bottom fiber after all losses is the difference between the bottom tensile stress due to the applied loads and the concrete tensile stress limit

$f_{pb} = 2.517 - 0.425 = 2.092$ ksi

Assume the distance between the center of gravity of strands and the bottom fiber of the beam equals 4.5 in. at midspan.

Strand eccentricity at midspan, $(e_c) = y_b - 4.5 = 19.29 - 4.5 = 14.79$ in.

If P_{pe} is the total prestress force after all losses, the bottom fiber stress due to prestress is:

$$f_{pb} = \frac{P_{pe}}{A} + \frac{P_{pe}\, e_c}{S_b}, \text{ or } 2.092 = \frac{P_{pe}}{813} + \frac{P_{pe}(14.79)}{8{,}728}$$

Solving for P_{pe}, the required P_{pe} = 715.3 kips

Final prestress force per strand = (area of strand)(f_{pi})(1 – final losses, %)

where

f_{pi} = initial stress before transfer = 202.5 ksi see (Sect. 9.2.2)

final losses = assume 20% of f_{pi}

Therefore, final prestress force per strand after all losses:

(0.153)(202.5)(1 – 0.20) = 24.8 kips

Number of strands required = 715.3/24.8 = 28.84 strands

Try (31) 1/2-in.-diameter, 270 ksi strands

9.2.5.4 Strand Pattern

Figure 9.2.5.4-1 shows the assumed strand pattern for the 31 strands at midspan of the beam. All strands are straight.

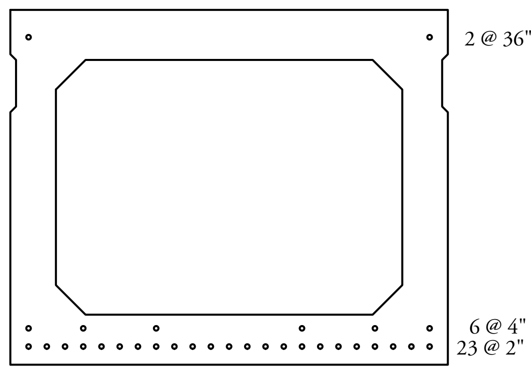
For 31 strands, calculate the distance between the center of gravity of the strands and the bottom fiber of the beam at midspan:

$$y_{bs} = \frac{23(2) + 6(4) + 2(36)}{31} = 4.58 \text{ in.}$$

Strand eccentricity at midspan:

$$e_c = y_b - y_{bs} = 19.29 - 4.58 = 14.71 \text{ in.}$$

Figure 9.2.5.4-1
Strand Pattern at Midspan



9.2.6 STRENGTH LIMIT STATE

For box sections, it is common that the flexural strength controls design. It is therefore recommended that the strength calculations be conducted prior to the stress check. As the state-of-the-art continues to develop into the use of high strength concrete and emphasis continues to be placed on the importance of member strength, it is possible that future designs using other cross-sections will be controlled by the strength limit state.

9.2.6.1 Flexural Resistance

Total ultimate bending moment for Strength I limit state, as given in section 9.2.4.3, is:

$$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$$

Therefore, the ultimate bending moment at midspan is:

$$M_u = 1.25(955.5 + 34.7 + 97) + 1.5(141.0) + 1.75(545.8 + 207.2\ ) = 2{,}888.3 \text{ ft-kips}$$

Average stress in prestressing steel when $f_{pe} \geq 0.5\ f_{pu}$:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{[LRFD Eq. 5.7.3.1.1-1]}$$

where

f_{ps} = average stress in prestressing steel

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{[LRFD Eq. 5.7.3.1.1-2]}$$

For low-relaxation strands, $k = 0.28$ [LRFD Table C5.7.3.1.1-1]

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons

c = distance from extreme compression fiber to the neutral axis

To compute c, assume rectangular section behavior and check if the depth of the equivalent stress block, $a = \beta_1 c$, is less than or equal to compression flange depth, h_f. [LRFD Art. C5.7.3.2.2]

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y'}{0.85 f_c' \beta_1 b + k A_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1-4]

where

A_{ps} = area of prestressing steel

A_s = area of mild steel tension reinforcement = 0 in.^{2}

A_s' = area of compression reinforcement = 0 in.^{2}

β_1 = stress factor of compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f_c' \le 4.0$ ksi

= $0.85 - 0.05(f_c' - 4.0) \ge 0.65$ for $f_c' > 4.0$ ksi

= $0.85 - 0.05(5.0 - 4.0) = 0.80$

b = width of compression flange = 48 in.

a = depth of the equivalent stress block = $\beta_1 c$

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons

Note: In computing the flexural strength of members with strands placed near the compression face of the member, it is not correct to use the combined centroid of the entire strand group for establishing the effective depth, d_p, and the area of prestressing steel, A_{ps}. This is because the top strands will have different strain from that of the bottom strands. An accurate solution can be achieved using the detailed strain compatibility approach which accounts for the steel strain at various distances from the neutral axis. However, a reasonable approximation is to ignore all strands placed on the compression side.

Therefore, the area of prestressing steel, $A_{ps} = 29(0.153) = 4.437$ in.^{2}

For the 29 bottom strands, the distance between the center of gravity of the strands and the bottom fiber of the beam, y_{bs}, is:

$$\frac{23(2) + 6(4)}{29} = 2.41 \text{ in.}$$

Thus, $d_p = 39 - 2.41 = 36.59$ in.

$$c = \frac{4.437(270) + 0 - 0}{(0.85)(5.0)(0.80)(48) + 0.28(4.437)\left(\dfrac{270}{36.59}\right)} = 6.95 \text{ in.}$$

$a = \beta_1 c = 0.80(6.95) = 5.56$ in. $> h_f = 5.5$ in. N.G.

Therefore, compute c using T-section behavior.

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y' - 0.85\beta_1 f_c'(b - b_w)h_f}{0.85 f_c' \beta_1 b_w + k A_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1-3]

where

h_f = depth of compression flange = 5.5 in.

b_w = width of web = 2(5) = 10 in.

$$c = \frac{4.437(270) + 0 - 0 - 0.85(0.80)(5.0)(48 - 10)(5.5)}{0.85(5.0)(0.80)(10) + 0.28(4.437)\left(\dfrac{270}{36.59}\right)} = 11.29 \text{ in.}$$

$a = \beta_1 c = 0.80(11.29) = 9.03$ in. > $h_f = 5.5$ in. O.K.

Therefore, the average stress in the prestressing steel:

$$f_{ps} = 270\left(1 - 0.28\frac{11.29}{36.59}\right) = 246.7 \text{ ksi}$$

Nominal flexural resistance, M_n: [LRFD Art. 5.7.3.2.2]

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85f'_c(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \quad \text{[LRFD Eq. 5.7.3.2.2-1]}$$

$$M_n = (4.437)(246.7)\left(36.59 - \frac{9.03}{2}\right)/12 + 0.85(5)(48 - 10)(0.80)(5.5)\left(\frac{9.03}{2} - \frac{5.5}{2}\right)/12$$

= 3,030.3 ft-kips

Factored flexural resistance, M_r:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where ϕ = resistance factor = 1.00 [LRFD Art. 5.5.4.2.1]

M_r = 3,030.3 ft-kips > Mu = 2,888.3 ft-kips O.K.

9.2.7 PRESTRESS LOSSES

Total prestress loss $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ [LRFD Eq. 5.9.5.1-1]

where

Δf_{pES} = loss due to elastic shortening

Δf_{pCR} = loss due to creep

Δf_{pSR} = loss due to shrinkage

Δf_{pR2} = loss due to relaxation of steel after transfer

9.2.7.1 Elastic Shortening

For pretensioning members:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \quad \text{[LRFD Eq. 5.9.5.2.3a-1]}$$

where

E_p = modulus of elasticity of prestressing steel = 28,500 ksi

E_{ci} = modulus of elasticity of the beam at transfer = 3,834 ksi

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member at the sections of maximum positive moment

Force per strand at transfer = (area of strand)(prestress stress at transfer)

LRFD Article 5.9.5.2.3a states that f_{cgp} can be calculated on the basis of a prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, common practice assumes the initial losses as a percentage of initial prestressing stress before transfer, f_{pi}. In both procedures, assumed initial losses should be checked and if different from the assumed value, a second iteration should be carried out. In this example, 5% f_{pi} initial loss is assumed.

Therefore, prestress stress at transfer = $f_{pi}(1 - 0.05) = 202.5(1 - 0.05) = 192.4$ ksi

Force per strand at transfer = (0.153)(192.4) = 29.4 kips

Total prestressing force at transfer, P_i = (31 strands)(29.4) = 911.4 kips

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{(M_g + M_D)\, e_c}{I}$$

where

M_g = unfactored bending moment due to beam self-weight

M_D = unfactored bending moment due to diaphragm weight

M_g and M_D should be calculated based on the overall beam length of 96 ft. However, since the elastic shortening losses will be a part of the total losses, f_{cgp} is conservatively computed based on M_g using the design span length of 95 ft.

$$f_{cgp} = \frac{911.4}{813} + \frac{(911.4)(14.71)^2}{168{,}367} - \frac{(955.5 + 34.7)(12)(14.71)}{168{,}367}$$

$$= 1.121 + 1.171 - 1.038 = 1.254 \text{ ksi}$$

$$\Delta f_{pES} = \frac{28{,}500}{3{,}834}(1.254) = 9.3 \text{ ksi}$$

9.2.7.2 Shrinkage

$\Delta f_{pSR} = (17 - 0.15H)$ [LRFD Eq. 5.9.5.4.2-1]

where H = relative humidity

Relative humidity varies significantly from one area of the country to another, see LRFD Figure 5.4.2.3.3-1.

Assume relative humidity, H = 70%

$\Delta f_{pSR} = [17 - 0.15(70)] = 6.5$ ksi

9.2.7.3 Creep of Concrete

$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

where

f_{cdp} = change in concrete stress at center of gravity of prestressing steel due to permanent loads, except the load acting at the time the prestressing force is applied, calculated at the same section as f_{cgp}

$$\Delta f_{cdp} = \frac{(M_{ws} + M_b)e_c}{I} = \frac{(97 + 141.0)(12)(14.71)}{168{,}367} = 0.250 \text{ ksi}$$

$$f_{pCR} = 12(1.254) - 7(0.250) = 13.3 \text{ ksi}$$

9.2.7.4 Relaxation of Prestressing Strand

9.2.7.4.1 Relaxation before Transfer

Initial loss due to relaxation of the prestressing steel is accounted for in the beam fabrication process. Therefore, losses due to relaxation of the prestressing steel prior to transfer are not computed, $\Delta f_{pR1} = 0.0$. Recognizing this for pretensioned members, the *LRFD Specifications*, Art. 5.9.5.1, allows the portion of the relaxation loss that occurs prior to transfer to be neglected in computing the final losses.

9.2.7.4.2 Relaxation after Transfer

For low-relaxation strands, loss due to relaxation after transfer is: [LRFD Art. 5.9.5.4.4c]

$\Delta f_{pR2} = 30\%[20.0 - 0.4\ f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$ [LRFD Eq. 5.9.5.4.4c-1]

$= 0.3[20.0 - 0.4(9.3) - 0.2(6.5 + 13.3)] = 3.7$ ksi

9.2.7.5 Total Loss At Transfer

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1} = 9.322 + 0 = 9.3$ ksi

Stress in tendons after transfer, $f_{pt} = f_{pi} - \Delta f_{pi} = 202.5 - 9.3 = 193.2$ ksi

Force per strand = (Stress in tendons after transfer)(area of strand) = 193.2 x 0.153
= 29.6 kips

Therefore total prestressing force after transfer, $P_i = 29.6 \times 31 = 917.6$ kips

Initial loss, % = $\Delta f_{pi}/f_{pi} = 9.3/202.5 = 4.6\%$

The first estimation of loss at transfer, i.e. 5%, is very close to the actual computed initial loss of 4.6%. Thus, there is no need for a second iteration to refine the initial assumption.

9.2.7.6 Total Loss At Service Loads

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} = 9.3 + 6.5 + 13.3 + 3.7 = 32.8$ ksi

Stress in tendon after all losses:

$f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 32.8 = 169.7$ ksi

Check prestressing stress limit at Service Limit State:
[LRFD Table 5.9.3-1]

$f_{pe} \leq 0.8 f_{py} = 0.8(243) = 194.4$ ksi > 169.7 ksi O.K.

Force per strand = (Stress in tendon after all losses)(strand area)

= 169.7 x 0.153 = 26.0 kips

Therefore, the total prestressing force after all losses, $P_{pe} = 31(26.0) = 806.0$ kips

Final losses, % = $\Delta f_{pT}/f_{pi} = 32.8/202.5 = 16.2$ %

The initial estimate of final losses of 20%, which was used to determine the strand pattern, is conservative. Since flexural strength controls design, there is no need to perform a second iteration with the computed total losses.

9.2.8 STRESSES AT TRANSFER

Force per strand after initial losses = 29.6 kips

Total prestressing force after transfer, $P_i = 917.6$ kips

9.2.8.1 Stress Limits for Concrete

[LRFD Art. 5.9.4]

Compression: $0.6f'_{ci} = 0.6(4.0) = 2.4$ ksi

Tension:

- without bonded auxiliary reinforcement

$0.0948\sqrt{f'_{ci}} \leq 0.200 \text{ ksi} = -0.0948\sqrt{4.0} = -0.190 \text{ ksi} \leq -0.200 \text{ ksi}$ O.K.

- with bonded auxiliary reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete

$0.22\sqrt{f'_{ci}} = -0.22\sqrt{4.0} = -0.440$ ksi

9.2.8.2 Stresses at Transfer Length Section

This section is located at a distance equal to the transfer length from the end of the beam. Stresses at this location need only be checked at release, because losses with time will reduce the concrete stresses making them less critical.

Transfer length = 60(strand diameter) [LRFD Art. 5.8.2.3]

= 60(0.5) = 30 in. = 2.5 ft

The transfer length extends to approximately 2.5 ft from the end of the beam or 2.0 ft from centerline of the bearing. Due to the camber of the beam at release, the self-weight of the beam and diaphragm act on the overall beam length, 96 ft. Therefore, the values of bending moment given in **Table 9.2.4.1.2-1** cannot be used at release because they are based on the design span, 95 ft. Using statics, bending moments at the end of the transfer zone due to beam and diaphragm weights, are

M_g = 0.5wx(L − x) = 0.5(.847)(2.5)(96 − 2.5) = 99.0 ft-kips, and

M_D = 2.7 ft-kips.

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t} = \frac{917.6}{813} - \frac{(917.6)(14.71)}{8{,}542} + \frac{(99.0 + 2.7)(12)}{8{,}542}$$

= 1.129 − 1.580 + 0.143 = −0.308 ksi

The tension stress limit for concrete with no bonded reinforcement: −0.190 ksi N.G.

Tension stress limit for concrete with bonded reinforcement: −0.440 ksi O.K.

Compute the concrete stress at the bottom fiber of the beam, f_b:

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b} = \frac{917.6}{813} + \frac{917.6(14.71)}{8{,}728} - \frac{(99.0 + 2.7)(12)}{8{,}728}$$

= 1.129 + 1.547 − 0.140 = 2.536 ksi

Compression stress limit for concrete: 2.400 ksi N.G.

Therefore, try debonding 7 strands from the strand group at 2 in. from bottom for a distance of 5'-0" from the end of the beam or 4'-6" from centerline of bearing.

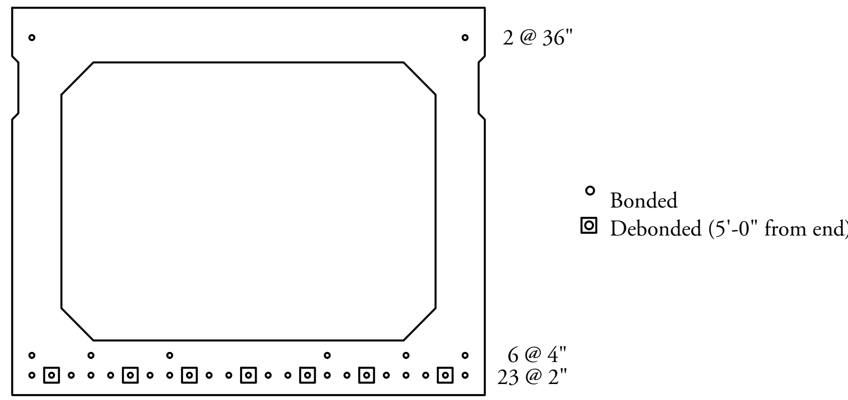
To minimize the shock impact of detensioning and cracks at corners and bottom, assume the strand pattern shown in **Figure 9.2.8.2-1**. LRFD Article 5.11.4.2 requires that the following conditions be satisfied if debonding is used:

- % debonded of total = 7/31 = 22.6% < 25% O.K.
- % debonded of row = 7/23 = 30.4% < 40% O.K.
- All limit states should be satisfied O.K.
- Debonded strands should be symmetrically distributed O.K.
- Exterior strands in each horizontal line are fully bonded O.K.

Compute new strand pattern properties:

Distance from the center of gravity of strands to the bottom fiber of the beam is:

Figure 9.2.8.2-1
Strand Pattern at End of Beam



y_{bs} = [16(2) + 6(4) + 2(36)]/(24) = 5.33 in.

and the strand eccentricity at end of beam is:

e_e = 19.29 − 5.33 = 13.96 in.

Total prestressing force at release at end section = 24(29.6) = 710.4 kips

$$f_t = \frac{710.4}{813} - \frac{710.4(13.96)}{8{,}542} + \frac{(99.0 + 2.7)(12)}{8{,}542} = 0.874 - 1.161 + 0.143 = -0.144 \text{ ksi}$$

Tension stress limit for concrete with no bonded reinforcement: −0.190 ksi O.K.

Thus, there is no need for additional bonded reinforcement; and,

$$f_b = \frac{710.4}{813} + \frac{710.4(13.96)}{8{,}728} - \frac{(99.0 + 2.7)(12)}{8{,}728} = 0.874 + 1.136 - 0.140 = +1.870 \text{ ksi}$$

Compression stress limit for concrete: +2.400 ksi O.K.

9.2.8.3 Stresses at Transfer Length Section of Debonded Strands

Bending moments due to the self-weight of the beam and diaphragm, at (5' + 2.5' = 7.5') from the end of the beam, based on overall length, are 281.1 ft-kips and 8.2 ft-kips respectively. All strands are effective at this location, therefore use the full value of P_i.

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t} = \frac{917.6}{813} - \frac{917.6(14.71)}{8{,}542} + \frac{(281.1 + 8.2)(12)}{8{,}542}$$

$$= 1.129 - 1.547 + 0.406 = -0.045 \text{ ksi}$$

Tension stress limit for concrete with no bonded reinforcement: −0.190 ksi O.K.

and,

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b} = \frac{917.6}{813} + \frac{917.6(14.71)}{8{,}728} - \frac{(281.1 + 8.2)(12)}{8{,}728}$$

$$= 1.129 + 1.547 - 0.398 = +2.278 \text{ ksi}$$

Compression stress limit for concrete: +2.400 ksi O.K.

9.2.8.4 Stresses at Midspan

Bending moments due to beam self-weight and diaphragm weight at midspan, based on overall length, are 975.7 ft-kips and 35.2 ft-kips respectively.

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{(M_g + M_D)}{S_t} = \frac{917.6}{813} - \frac{917.6(14.71)}{8{,}542} + \frac{(975.7 + 35.2)(12)}{8{,}542}$$

$$= 1.129 - 1.547 + 1.420 = +0.969 \text{ ksi}$$

Tension stress limit for concrete with no bonded reinforcement: −0.190 ksi O.K.

and,

$$f_b = \frac{P_i}{A} + \frac{P_i e_c}{S_b} - \frac{(M_g + M_D)}{S_b} = \frac{917.6}{813} + \frac{917.6(14.71)}{8{,}728} - \frac{(975.7 + 35.2)(12)}{8{,}728}$$

$$= 1.129 + 1.547 - 1.390 = +1.286 \text{ ksi}$$

Compression stress limit for concrete: +2.400 ksi O.K.

9.2.8.5 Summary of Stresses at Transfer

	Top Stress f_t(ksi)	Bottom Stress f_b(ksi)
At transfer length section	−0.144	+1.870
At end of debonded strands + transfer length	−0.045	+2.278
At midspan	+0.969	+1.286

9.2.9 STRESSES AT SERVICE LOADS

[LRFD Art. 5.9.4.2]

Total prestressing force after all losses, P_{pe} = 806.0 kips

9.2.9.1 Stress Limits for Concrete

Compression:

Due to permanent loads, i.e., girder self-weight, diaphragm weight, wearing surface, and barrier load, for load combination Service I: $0.45f'_c = 0.45(5.0) = 2.25$ ksi

Due to permanent and transient loads, i.e., all dead loads and live loads, for load combination Service I: $0.60f'_c = 0.60(5.0) = 3.00$ ksi

Tension:

For components with bonded prestressing tendons:

for load combination Service III: $0.19\sqrt{f'_c} = -0.19\sqrt{5.0} = -0.425$ ksi

9.2.9.2 Stresses at Midspan

To review compressive stresses at the bottom of the beam, two cases are checked.

1. Under permanent load, load combination Service I, using values in **Table 9.2.4.1.2-1**:

$$f_t = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{(M_g + M_D + M_{ws} + M_b)}{S_t}$$

$$= \frac{806}{813} - \frac{806(14.71)}{8{,}542} + \frac{(955.5 + 34.7 + 97.0 + 141.0)(12)}{8{,}542}$$

$$= 0.991 - 1.388 + 1.725 = +1.328 \text{ ksi}$$

Compression stress limit for concrete: +2.250 ksi O.K.

2. Under permanent and transient loads, load combination Service I:

$$f_t = +1.328 + \frac{(M_{LT} + M_{LL})(12)}{S_t} = +1.328 + \frac{(545.8 + 207.2)(12)}{8542}$$

$$= +1.328 + 1.058 = +2.386 \text{ ksi}$$

Compression stress limit for concrete: +3.000 ksi O.K.

Check bottom tension stress, load combination Service III:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}\,e_c}{S_b} - \frac{(M_g + M_D + M_{ws} + M_b) + 0.8(M_{LT} + M_{LL})}{S_b}$$

$$= \frac{806}{813} + \frac{806\,(14.71)}{8{,}728} - \frac{[(955.5 + 34.7 + 97.0 + 141.0) + 0.8(545.8 + 207.2)](12)}{8{,}728}$$

$= +0.991 + 1.358 - 2.517 = -0.168$ ksi

Tension stress limit for concrete: -0.425 ksi O.K.

9.2.9.3 Fatigue Limit State

LRFD Article 5.5.3.1 states that in regions of compressive stress due to permanent loads and prestress, fatigue is considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination. However, the *LRFD Specifications* does not state where this stress is computed: at the bottom fiber of concrete or at the lower strand level. In this example, for convenience, the calculations for this limit are done at the bottom fiber of concrete.

At midspan, the bottom compressive stress due to permanent loads and prestress is:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}\,e_c}{S_b} - \frac{(M_g + M_D + M_{ws} + M_b)}{S_b}$$

$$= \frac{806}{813} + \frac{806(14.71)}{8{,}728} - \frac{[(955.5 + 34.7 + 97.0 + 141.0)](12)}{8{,}728}$$

$= +0.991 + 1.358 - 1.689 = +0.660$ ksi

From **Table 9.2.4.1.2-1**, the unfactored fatigue bending moment at midspan, M_f, is 205.9 ft-kips. Therefore, bottom stress due to the fatigue load combination:

$$= -\frac{0.75(M_f)}{S_b} = -\frac{0.75(205.9)(12)}{8{,}728} = -0.212 \text{ ksi}$$

Since 2(0.212) = 0.424 ksi < 0.660 ksi, the fatigue check is not necessary. However, for illustration purposes, the fatigue check is completed below.

Since the precast section is not permitted to crack under service loads, the uncracked section properties are used. The fatigue stress range, SR, caused by the fatigue load combination at the bottom row of strands is calculated by transforming the concrete stress to steel stress using the modular ratio.

$$SR = \frac{M_f(y_b - 2)}{I}\frac{E_p}{E_c}$$

where SR = fatigue Stress Range

$$SR = \frac{(0.75)(205.9)(12)(19.29 - 2)}{168{,}367}\frac{28{,}500}{4{,}287} = 1.3 \text{ ksi}$$

LRFD Article 5.5.3.3 states that the stress range in prestressing tendons should not exceed 18.0 ksi for a radius of curvature in excess of 30.0 ft, which includes straight strand. O.K.

Discussion:

1. Current practice with precast pretensioned concrete members does not permit cracking of concrete due to service loads. Accordingly, the tensile stress range in the strands due to live loads is only (n) times the corresponding concrete stress, which is always lower than the limiting stress range of 10 or 18 ksi for fatigue. Thus, the need to check for fatigue limit state under those conditions is questionable.
2. Bridges built with adjacent precast members with no cast-in-place concrete topping generally have a large bottom compressive stress at midspan due to permanent loads plus prestressing. This is because there is no tensile stress resulting from the cast-in-place topping. Therefore, the exception in LRFD Article 5.5.3.1 results in a fatigue check to be unnecessary for this type of bridge member.

9.2.9.4 Summary of Stresses at Service Loads

	Top Stresses (Service I)		Bottom Stress
	Permanent loads	Total loads	Service III
At midspan	+1.328 ksi	+2.386 ksi	−0.168 ksi

9.2.10 LIMITS OF REINFORCEMENT

9.2.10.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The maximum amount of prestressed and non-prestressed reinforcement must be such that

$$\frac{c}{d_e} \leq 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

where

$$d_e = \frac{A_{ps}f_{ps}d_p + A_sf_yd_s}{A_{ps}f_{ps} + A_sf_y} \qquad \text{[LRFD Eq. 5.7.3.3.1-2]}$$

$$= \frac{4.743(241.8)34.42 + 0.0}{4.743(241.8) + 0.0} = 34.42 \text{ in}$$

$$\frac{c}{d_e} = \frac{12.83}{34.42} = 0.37 < 0.42 \quad \text{O.K.}$$

9.2.10.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

At any section, the amount of prestressed and non-prestressed tensile reinforcement should be adequate to developed a factored flexural resistance, M_r, equal to the lesser of:

- 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and
- 1.33 times the factored moment required by the applicable strength load combination.

Check at midspan:

The *LRFD Specifications* do not give any procedure for computing the cracking moment. Therefore, the equation in the Standard Specifications, Art. 9.18.2.1, is used.

$$M_{cr} = (f_r + f_{pb})S_{bc} - M_{d/nc}(S_{bc}/S_b - 1) \qquad \text{(Eq. 9.2.10.2-1)}$$

where

f_r = modulus of rupture [LRFD Art. 5.4.2.6]

$= 0.24\sqrt{f'_c} = 0.24\sqrt{5.0} = 0.537$ ksi

f_{pb} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} = \frac{806.0}{813} + \frac{806.0(14.71)}{8{,}728} = 2.350 \text{ ksi}$$

$M_{d/nc}$ = non-composite dead load moment at the section

S_{bc} = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads

S_b = non-composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 8,728 in.^{3}

Since there is no cast-in-place topping, $S_{bc} = S_b$, therefore, the second part of the equation is zero.

M_{cr} = (0.537 + 2.350)8,728/12 = 2,099.8 ft-kips

1.2M_{cr} = 1.2(2,099.8) = 2,519.8 ft-kips

At midspan, the factored moment required by Strength I load combination is:

M_u = 2,888.3 ft-kips (as calculated in Sect. 9.2.6)

Thus, 1.33M_u = 1.33(2,888.3) = 3,841.4 ft-kips

Since 1.2M_{cr} < 1.33M_u, the 1.2M_{cr} requirement controls.

M_r = 3,030.3 ft-kips > 1.2M_{cr} O.K.

Note: Contrary to the *Standard Specifications*, the *LRFD Specifications* require that this criterion be met at all sections.

9.2.11 SHEAR DESIGN

The area and spacing of shear reinforcement must be determined at regular intervals along the entire lenth of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement shall be provided when:

$V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

where

V_u = total factored shear force, kips

V_c = shear strength provided by concrete, kips

V_p = component of the effective prestressing force in the direction of the applied shear, kips

ϕ = resistance factor = 0.9 [LRFD Art. 5.5.4.2.1]

9.2.11.1 Critical Section

Critical section near the supports is the greater of: [LRFD Art. 5.8.3.2]

$0.5d_v\cot\theta$

or, d_v from the face of the support

where

d_v = effective shear depth

= distance between resultants of tensile and compressive forces, (d_e – a/2 for a rectangular section)

but not less than 0.9d_e or 0.72h [LRFD Art. 5.8.2.7]

where

d_e = the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement

= d_p, since only prestressing steel is used

= 39 − 2.55 = 36.45 in.

Note: Only 22 strands (16 @ 2 in. and 6 @ 4 in.) are effective at the critical section for shear, because 7 strands are debonded for a distance equal to 5 ft from the end of the beam and the top level of strands is ignored.

a = equivalent compressive block depth = 9.03 in. (at midspan, conservative estimate)

h = total height of the section = 39.00 in.

θ = angle of inclination of diagonal compressive stresses, assume = 21° (slope of compression field)

9.2.11.1.1 Angle of Diagonal Compressive Stresses

The shear design at any section depends on the angle of diagonal compressive stresses at the section. Shear design is an iterative process that begins with assuming a value for θ. For this example, only the final cycle of calculations is shown. As a guide, for areas which have high shear forces and low bending moments, the angle θ ranges from 20° to 30°. For areas of low shear forces and high bending moments, the angle θ goes up to 45°. Using these guidelines, two iterations are adequate in most cases.

9.2.11.1.2 Effective Shear Depth

Since the beam has been identified as a flanged section, the effective shear depth, d_v, should be determined using LRFD Eq. C5.8.2.9-1. However, d_v can be conservatively approximated as d_e − a/2 using 'a' as determined in the midspan flexural analysis in Section 9.2.6.1.

d_v = d_e − a/2 = [36.45 − 0.5(9.03)] = 31.94 in.

$0.9d_e$ = 0.9(36.45) = 32.81 in. (Controls)

0.72h = 0.72(39) = 28.08 in.

Therefore, d_v = 32.81 in.

9.2.11.1.3 Calculation of Critical Section

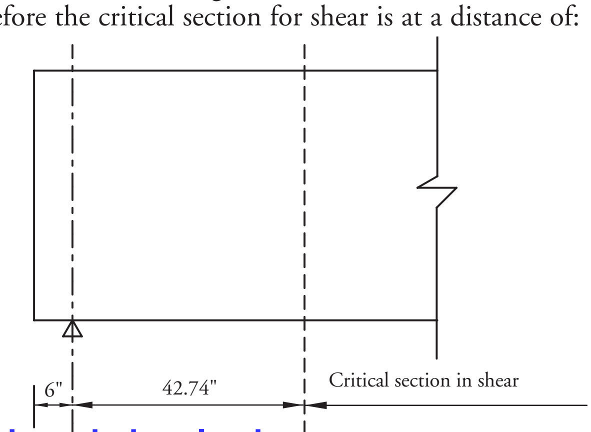
Critical section near the supports is the greater of:

$0.5d_v\cot\theta$ = [0.5(32.81)]cot21° = 42.74 in. (Controls)

and, d_v = 32.81 in.

Since the width of the bearing is not yet determined, it was conservatively assumed to be equal to zero for determining the critical section for shear, as shown in **Figure 9.2.11-1**. Therefore the critical section for shear is at a distance of:

Figure 9.2.11-1 Critical Section in Shear



42.74/12 ft = 3.56 ft from centerline of support

x/L = 3.56/95 = 0.038L

9.2.11.2
Contribution of Concrete to Nominal Shear Resistance

The contribution of the concrete to the nominal shear resistance is:

$$V_c = 0.0316\beta\sqrt{f_c'}\,b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

Several quantities must be determined before this expression can be evaluated.

9.2.11.2.1
Strain in Flexural Tension Reinforcement

Calculate the strain in the reinforcement on the flexural tension side, ε_x:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})} \le 0.001 \qquad \text{[LRFD Eq. 5.8.3.4.2-1]}$$

where

M_u = applied factored bending moment at the specified section which occurs simultaneously with V_u, or, conservatively taken as the maximum M_u.

Using load combination Strength I

= 1.25(137.8 + 3.9 + 14.0) + 1.50(20.3) + 1.75(84.2 + 29.8)

= 424.6 ft-kips

V_u = applied factored shear force at the specified section

= 1.25(37.2 + 1.1 + 3.8) + 1.50(5.5) + 1.75(36.5 + 12.4)

= 146.5 kips

V_p = component in the direction of the applied shear of the effective prestressing force = 0 (since strand pattern is straight)

N_u = applied factored normal force at the specified section = 0

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For pretensioned members, LRFD Article C5.8.3.4.2 indicates that f_{po} can be taken as the stress in the strands when the concrete is cast around them, which is the jacking stress, f_{pj}, or $0.75f_{pu}$.

= 0.75(270.0) = 202.5 ksi

θ = angle of inclination of diagonal compressive stresses = 21° (assumed)

A_{ps} = area of prestressing steel on the flexural tension side of the member

= 22(0.153) = 3.366 in.^{2} (Only 22 strands are effective of the 29 strands in the flexural tension side because 7 strands are debonded.)

A_s = area of nonprestressing steel on the flexural tension side of the member = 0.0

$$\varepsilon_x = \frac{\dfrac{424.6(12)}{32.81} + 0 + 0.5(146.5)(\cot 21°) - 3.366(202.5)}{2\left[0 + 28{,}500(3.366)\right]}$$

$$= \frac{-335.5}{191{,}862} = -1.749\times10^{-3}$$

9.2.11.2.1.1 Strain in Flexural Tension Reinforcement when ε_x is Negative

Since the value of ε_x is negative, a different equation must be used:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})} \quad \text{[LRFD Eq. 5.8.3.4.2-3]}$$

where A_c = area of concrete (in.^{2}) on the flexural tension side, as shown in **Figure 9.2.11-2**
= (0.5)(813) = 406.5 in.^{2} (because the section is symmetrical about the longitudinal axis)

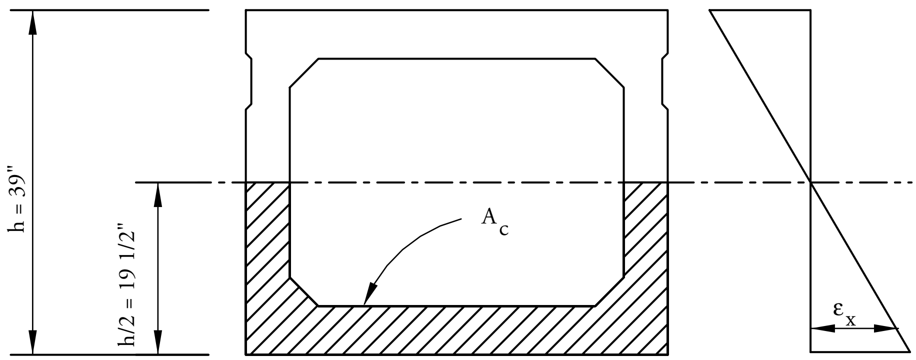


Figure 9.2.11-2 Illustration of A_c

$$\varepsilon_x = \frac{-335.5}{2\left[(4{,}287)(406.5)+(28{,}500)(3.366)\right]} = -0.0912\times10^{-3}$$

Note that the sign of ε_x should be maintained.

9.2.11.2.1.2 Compute Shear Stress

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \quad \text{[LRFD Eq. 5.8.3.4.2-1]}$$

where

v_u = factored shear stress in concrete

V_p = component of the effective prestressing force in the direction of the applied shear
= 0 (because no harped strands are used)

ϕ = resistance factor [LRFD Art. 5.5.4.2.1]

b_v = effective web width = 2 x 5 = 10 in.

$$v_u = \frac{146.5 - 0}{(0.9)(10)(32.81)} = 0.4961 \text{ ksi}$$

$(v_u/f'_c) = (0.4961/5.0) = 0.0992$

9.2.11.2.2
Values of β and θ

Having computed ε_x and v_u/f'_c, find a better estimate of θ from LRFD Table 5.8.3.4.2-1. Since the computed value of v_u/f'_c is likely to fall between two rows in the table, a linear interpolation may be performed. However, for hand calculations, interpolation is not recommended (LRFD Art. C5.8.3.4.2). Values of θ in the lower row that bounds the computed value may be used. Similarly, values of θ in the first column to the right of the computed value may be used. For this example, the applicable row and column are the ones labeled "≤0.100" and "≤–0.05", respectively. The values of θ and β contained in the cell of intersection of that row and column are:

$\theta = 21.4° \approx$ the assumed value O.K.

If the assumed value of θ differs greatly from the calculated value, repeat the calculations using the new value of θ until the answer converges.

$\beta = 3.24$

where β is a factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution).

9.2.11.2.3
Concrete Contribution

The nominal shear resisted by the concrete is:

$$V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

$$= 0.0316(3.24)\sqrt{5.0}\,(10)(32.81) = 75.1 \text{ kips}$$

9.2.11.3
Contribution of Reinforcement to Nominal Shear Resistance

9.2.11.3.1
Requirement for Reinforcement

Check if $V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

$V_u = 146.5 \text{ kips} > 0.5\phi(V_c + V_p) = [0.5(0.9)](75.1 + 0) = 33.8 \text{ kips}$

Therefore, transverse shear reinforcement should be provided.

9.2.11.3.2
Required Area of Reinforcement

$V_u/\phi \le V_n = V_c + V_s + V_p$ [LRFD Eq. 5.8.3.3-1]

where

V_s = shear force carried by transverse reinforcement

$= (V_u/\phi) - V_c - V_p = (146.5/0.9) - 75.1 - 0 = 87.7 \text{ kips}$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \text{[LRFD Eq. 5.8.3.3-4]}$$

where

A_v = area of shear reinforcement within a distance s, in.^{2}

s = spacing of stirrups, in.

f_y = yield strength of shear reinforcement

α = angle of inclination of transverse reinforcement to longitudinal axis = 90°

Therefore, area of shear reinforcement (in.^{2}) within a spacing (s), is:

req'd $A_v = (s\,V_s)/(f_y d_v \cot\theta)$

$= s(87.7)/[(60)(32.81)\cot 21°] = 0.0171(s)$

If s = 12 in., then $A_v = 0.21$ in.^{2}/ft

9.2.11.3.3
Spacing of Reinforcement

Maximum spacing of transverse reinforcement may not exceed the following:

[LRFD Art. 5.8.2.7]

If $v_u = 0.496$ ksi $< 0.125f'_c = 0.125(5.0) = 0.625$ ksi (yes), then:

s and $s_{max} \leq 24$ in. (Controls) [LRFD Eq. 5.8.2.7-1]

$\leq 0.8d_v = (0.8)(32.81) = 26.25$ in.

s provided = 12 in. < 24 in. O.K.

Use # 3 single leg in each web at 12 in. spacing

A_v provided = 0.22 in.^{2}/ft > A_v required = 0.21 in.^{2} O.K.

$$V_s = \frac{0.22(60)(32.81)(\cot 21° + \cot 90°)(\sin 90°)}{12} = 94.0 \text{ kips}$$

9.2.11.3.4
Minimum Reinforcement Requirement

[LRFD Art. 5.8.2.5]

The area of transverse reinforcement should not be less than:

$$0.0316\sqrt{f'_c}\frac{b_v s}{f_y} = 0.0316\sqrt{5}\frac{(10)(12)}{60} = 0.14 \text{ in.}^2\text{/ft} < 0.22 \text{ in.}^2\text{/ft} \quad \text{O.K.}$$

[LRFD Eq. 5.8.2.5-1]

9.2.11.4
Maximum Nominal Shear Resistance

In order to assure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement, the *LRFD Specifications* give an upper limit for V_n as follows:

$V_n = 0.25 f'_c b_v d_v + V_p$ [LRFD Eq. 5.8.3.3-2]

Comparing this equation with [LRFD Eq. 5.8.3.3-2], it can be concluded that

$V_c + V_s \leq 0.25 f'_c b_v d_v$

$75.1 + 94.0 = 169.1$ kips $\leq 0.25(5.0)(10)(32.81) = 410.1$ kips O.K.

Using the outlined procedures above, shear design was carried out at tenth points along the span. The results are shown below in **Table 9.2.11.4-1**. Note that two iterations were done for each tenth-point beginning with assuming the angle θ.

9.2.12 MINIMUM LONGITUDINAL REINFORCEMENT REQUIREMENT

[LRFD ART. 5.8.3.5]

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied:

$$A_s f_y + A_{ps} f_{ps} \geq \frac{M_u}{d_v \phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta \quad \text{[LRFD Eq. 5.8.3.5-1]}$$

where

- A_s = area of non-prestressed tension reinforcement, in.^{2}
- f_y = specified minimum yield strength of reinforcing bars, ksi
- A_{ps} = area of prestressing steel at the tension side of the section, in.^{2}
- f_{ps} = average stress in prestressing steel, ksi
- M_u = factored moment at the section corresponding to the factored shear force, ft-kips
- N_u = applied factored axial force = 0, kips
- V_u = factored shear force at section, kips

Table 9.2.11.4-1
Design of Vertical Shear

Distance, x ft.	Section x/L	Shear, V_u kips	Moment, M_u kip-ft	d_v in.	A_{ps} in.^{2}	Strain, ε_x in./in. x 1000	v ksi	v/f'_c	Actual θ deg	β	V_c kips	V_s kips	Max. Spac. in	A_v in.^{2}/ft	Min A_v in.^{2}/ft
3.56[1]	0.038	146.5	425	32.81	3.366	-0.0912	0.496	0.0992	21.4	3.2	75.1	87.7	24	0.21	0.14
9.50	0.100	131.0	1,060	33.59	4.437	-0.0928	0.433	0.0867	20.5	4.0	96.2	49.3	24	0.11	0.14
19.00	0.200	106.7	1,877	33.41	4.437	-0.0236	0.355	0.071	21.4	3.9	92.4	26.1	24	0.06	0.14
28.50	0.300	82.0	2,448	33.41	4.437	0.2472	0.273	0.0546	26.6	3.0	69.6	21.6	24	0.06	0.14
38.00	0.400	58.7	2,771	33.41	4.437	0.5728	0.195	0.0391	31.4	2.5	59.7	5.6	24	0.02	0.14
47.50	0.500	35.9	2,889	33.41	4.437	0.6613	0.119	0.0239	32.6	2.5	58.0	-	24	-	0.14

[1] Critical section for shear (see Section 9.2.11)

V_s = shear resistance provided by shear reinforcement, kips

V_p = component in the direction of the applied shear of the effective, prestressing force, kips

d_v = effective shear depth, in.

ϕ = resistance factor as appropriate for moment, shear and axial resistance. Therefore, different ϕ factors will be used for the terms in Eq. 5.8.3.5-1, depending on the type of action considered in that term, see the *LRFD Specifications*, Art. 5.5.4.2

θ = angle of inclination of diagonal compressive stresses.

9.2.12.1 Required Reinforcement at Face of Bearing

[LRFD Art. 5.8.3.5]

Since the width of the bearing is not yet determined, it was assumed to be equal to zero. This assumption is more conservative for these calculations. From **Table 9.2.4.1.2-1**, using load combination Strength I, the factored shear force and bending moment at centerline of bearing:

V_u = 1.25(40.2 + 1.1 + 4.1) + 1.5(5.9) + 1.75(38.2 + 13.5) = 156.1 kips

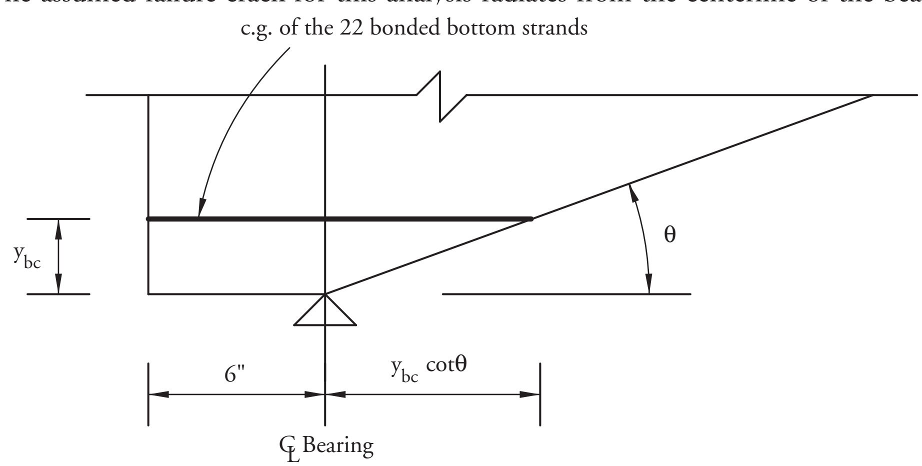
M_u = 0 ft-kips

$$\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

$$= 0 + 0 + \left(\frac{156.1}{0.9} - (0.5)(94.0) - 0\right)\cot 21° = 329.4 \text{ kips}$$

The assumed failure crack for this analysis radiates from the centerline of the bear-

Figure 9.2.12-1 Assumed Failure Crack



ing, which is 6 in. from the end of the beam, while the transfer length is 60 times the strand diameter = 30 in. This crack crosses the centroid of the bottom bonded strands at (6 + y_{bc}cotθ), which is still within the strand transfer length of 30 in. as computed earlier and shown in **Figure 9.2.12-1**.

Note: This crack is quite unlikely because it would form in the end block, which is a large solid section of concrete. However, the analysis does not account for the area of concrete involved. It simply assumes a crack.

For the 22 bonded bottom strands, $y_{bc} = \dfrac{2(16) + 6(4)}{22} = 2.55$ in.

Therefore, 6 + y_{bc} cot θ = 6 + 2.55cot21° = 12.64 in. < 30 in.

Since the location is within the transfer length, the available prestress is less than the effective prestress. The prestressing force at the center line of bearing is:

$$A_{ps}f_{ps} = (22)(0.153)\left(\frac{12.64}{30}(169.7)\right) = 240.7 \text{ kips} < 329.4 \text{ kips}$$

The strands are not adequate to resist the required longitudinal force. Therefore, provide additional mild reinforcement to carry the difference.

Force to be resisted by additional reinforcement = 329.4 – 240.7 = 88.7 kips

Additional mild steel reinforcement required = (88.7 kips)/(60 ksi) = 1.48 in.^{2}

Use (5) #5. The area of steel provided = 5 x 0.31 = 1.55 in.^{2}

9.2.13 PRETENSIONED ANCHORAGE ZONE

[LRFD Art. 5.10.10]

Design of the anchorage zone reinforcement is computed using the force in the strands just before transfer. Since 7 strands are debonded at the ends of the beam, the force in the remaining strands before transfer is:

F_{pi} = 24(0.153)(202.5) = 743.6 kips

The bursting resistance, P_r, should not be less than 4.0% of the prestress force at transfer. [LRFD Arts. 5.10.10.1 and C3.4.3]

$P_r = f_sA_s \geq 0.04F_{pi} = 0.04(743.6) = 29.4$ kips

where

A_s = total area of vertical reinforcement located within the distance h/4 from the end of the beam, in.^{2}

f_s = stress in steel, but not taken greater than 20 ksi

Solving for the required area of steel, A_s = 29.4/(20) = 1.47 in.^{2}

At least 1.47 in.^{2} of vertical transverse reinforcement should be provided within a distance from the end of the beam equal to one-fourth of the precast beam depth, (h/4 = 39/4 = 9.75 in.).

Use an orthogonal welded wire reinforcement of W20 or D20 wires @ 12 in. centers vertically and horizontally. Space two layers of WWR @ 3 in. starting @ 2 in. from, and parallel to the end of the beam in the diaphragm. The layers of WWR each provide 4 vertical and 3 horizontal wires. Area of steel provided is 2(4 + 3)(0.2) = 2.80 in.^{2} A reinforcing bar cage could be used as well. Provide adequate embeddment for bars.

9.2.14 DEFLECTION AND CAMBER

[LRFD Art. 5.7.3.6.2]

Deflections are calculated at midspan based on the moment of inertia shown and modulus of elasticity of concrete calculated in Section 9.2.3.

9.2.14.1 Deflection Due to Prestressing Force at Transfer

$$\Delta_p = \frac{P_i e L^2}{8E_{ci}I}$$

where

P_i = total prestressing force at transfer = 917.6 kips

e = eccentricity of prestressing force = 14.71 in.

L = overall span length = 96.0 ft

E_{ci} = modulus of elasticity at transfer = 3,834 ksi

$$\Delta_p = \frac{(917.6)(14.71)(96\text{x}12)^2}{(8)(3{,}834)(168{,}367)} = 3.47 \text{ in.} \uparrow$$

9.2.14.2 Deflection Due to Beam Self-Weight

$$\Delta_{beam} = \frac{5wL^4}{384E_{ci}I}$$

where w = beam self-weight = 0.847 kip/ft

Deflection due to beam self-weight at transfer:

L = overall beam length = 96.0 ft

$$\Delta_{beam} = \frac{5\left(\frac{0.847}{12}\right)(96 \text{ x } 12)^4}{(384)(3{,}834)(168{,}367)} = 2.51 \text{ in.} \downarrow$$

Deflection due to beam self-weight used to compute deflection at erection:

L = span length = 95.0 ft

$$\Delta_{beam} = \frac{5\left(\frac{0.847}{12}\right)(95\text{x}12)^4}{(384)(3{,}834)(168{,}367)} = 2.40 \text{ in.} \downarrow$$

9.2.14.3 Deflection Due to Diaphragm Weight

$$\Delta_D = \frac{19PL^3}{384E_{ci}I}$$

where P = diaphragm weight concentrated at quarter points = 0.73 kip

Deflection due to diaphragm weight at transfer:

L = span length = 96.0 ft

$$\Delta_D = \frac{19(0.73)(96\text{x}12)^3}{(384)(3{,}834)(168{,}367)} = 0.09 \text{ in.} \downarrow$$

Deflection due to diaphragm weight used to compute deflection at erection:

L = span length = 95.0 ft

$$\Delta_D = \frac{19(0.73)(95\text{x}12)^3}{(384)(3{,}834)(168{,}367)} = 0.08 \text{ in.} \downarrow$$

9.2.14.4 Deflection Due to Barrier and Wearing Surface Weights

$$\Delta_{b+ws} = \frac{5wL^4}{384E_cI}$$

where

w = barrier and wearing surface weight = (0.086 + 0.125) = 0.211 kip/ft

E_c = modulus of elasticity of precast girder at 28 days = 4,287 ksi

L = span length = 95.0 ft because these loads are applied to the structure in its final location

$$\Delta_{b+ws} = \frac{5\left(\frac{0.211}{12}\right)(95 \text{ x } 12)^4}{(384)(4{,}287)(168{,}367)} = 0.54 \text{ in.} \downarrow$$

9.2.14.5 Deflection and Camber Summary

At transfer: $(\Delta_p + \Delta_{beam} + \Delta_D) = 3.47 - 2.51 - 0.09 = 0.87$ in. ↑

Total deflection at erection using PCI multipliers (see PCI Design Handbook):

$1.8(3.47) - 1.85(2.51 + 0.09) = 1.44$ in. ↑

Long-Term Deflection:

The *LRFD Specifications*, Art. 5.7.3.6.2, states that the long-term deflection may be taken as the instantaneous deflection multiplied by a factor, 4.0, if the instantaneous deflection is based on gross moment of inertia. However, a factor of 4.0 is not appropriate for this type of precast construction. It is recommended that the designer follow the guidelines of the owner agency for whom the bridge is being designed, or undertake a more rigorous time-dependent analysis.

9.2.14.6 Deflection Due to Live Load and Impact

Live load deflection limit (optional) [LRFD Art. 2.5.2.6.2]

Span/800 = (95)(12)/800 = 1.43 in.

If the owner invokes the optional live load deflection criteria specified in Art. 2.5.2.6.2, the deflection is the greater of [LRFD Art. 3.6.1.3.2]

- that resulting from the design truck alone, or
- that resulting from 25% of the design truck taken together with the design lane load

The *LRFD Specifications* state that all the beams should be assumed to deflect equally under the applied live load and impact (Art. 2.5.2.6.2). Therefore, the distribution factor for deflection, DFD, is calculated as follows:

DFD = (number of lanes/number of girders) [LRFD Art. C2.5.2.6.2]

= (2/7) = 0.286 lanes/beam

However, it is more conservative to use the distribution factor for moment, DFM.

Therefore, design lane load, w = 0.64DFM = 0.64(0.287) = 0.184 kips/ft

$$\Delta_{LL} = \frac{5wL^4}{384E_cI} = \frac{5\left(\frac{0.184}{12}\right)(95 \text{ x } 12)^4}{(384)(4,287)(168,367)} = 0.46 \text{ in} \downarrow$$

Deflection due to design truck load with impact:

To derive maximum moment and deflection at midspan, let the centerline of the beam coincide with the mid-point of the distance between the inner 32-kip axle and the resultant of the truck load, as shown in **Figure 9.2.14.6-1**.

Using the elastic moment area, deflection at midspan is:

$\Delta_{LT} = 2.90(IM)(DFM) = 2.90(1.33)(0.287) = 1.10$ in. ↓

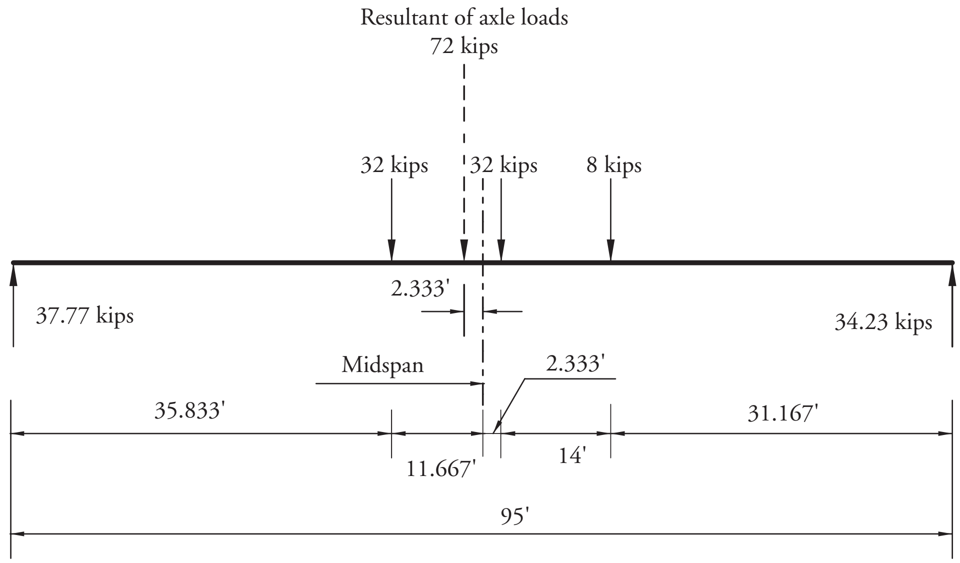
Total live load deflection is the greater of:

$\Delta_{LT} = 1.10$ in. ↓ (Controls)

$0.25\ \Delta_{LT} + D_{LL} = 0.25(1.10) + 0.46 = 0.74$ in. ↓

Allowable live load deflection: 1.43 > 1.10 in. O.K.

Figure 9.2.14.6-1
Design Truck Axle Load Positions on the Span for Maximum Moment



9.2.15 TRANSVERSE POST-TENSIONING

Article C4.6.2.2.1 in the *LRFD Specifications* states that for bridge type (g), the structure acts as a monolithic unit if sufficiently interconnected. To satisfy this requirement, the *LRFD Specifications* recommend that a minimum average transverse prestress of 0.250 ksi be used. However, definition of the contact area for that post-tensioning is unclear as to whether it is the shear key, the diaphragm, or the entire box side surface. Instead of an empirical minimum, El-Remaily (1996) recommends that the entire deck surface be modeled as a rigid assembly of gridwork with adequate post-tensioning to provide for a continuous transverse member at the diaphragm locations. A design chart based on this theory is given in Chapter 8 for the required transverse post-tensioning per unit length of span.

According to the chart, for a 28 ft-wide bridge with 39 in. deep beams, an effective post-tensioning force of 4.0 kips/ft is required. Since diaphragms are provided at quarter-points of the span, the post-tensioning force required is:

4.0(23.75) = 95 kips/diaphragm

It is recommended that transverse post-tensioning consist of one tendon near the top and another near the bottom in order to provide sufficient flexural strength.

Use 160-ksi prestressing bars. Assume the effective prestress to be 55 percent of the ultimate strength of the bar.

$P_{eff} = 0.55(160)A_{PT} = 88.0A_{PT}$ kips

Thus, total required $A_{PT} = \dfrac{95.0}{88.0} = 1.08$ in.^{2}/diaphragm

Try (2) 7/8 in. diameter, 160 ksi, bars.

The total area provided is $A_{PT} = 2(0.601) = 1.202$ in.^{2}

Total provided post-tensioning force = (1.202)(0.55)(160) = 105.8 kips/diaphragm

> 95.0 kips/diaphragm O.K.

If the post-tensioning bars are positioned so that they are concentric with the diaphragm cross section, concrete stress due to the effective prestressing force is:

105.8/(8)(39) = 0.339 ksi

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

NOTATION

DESIGN EXAMPLES

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pb} = compressive stress at bottom fiber of the beam due to prestress force

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = the modulus of rupture of concrete [STD]

f_r = modulus of rupture of concrete [LRFD]

f_s = allowable stress in steel

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress

f_{si} = effective initial pretension stress

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

f_t = concrete stress at top fiber of the beam for the non-composite section

f_{tc} = concrete stress at top fiber of the slab for the composite section

f_{tg} = concrete stress at top fiber of the beam for the composite section

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity, percent [LRFD]

H = height of wall [LRFD]

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_c = total height of composite section

h_f = compression flange depth [LRFD]

I = moment of inertia about the centroid of the non-composite precast beam [STD]

I = moment of inertia about the centroid of the non-composite precast beam [LRFD]

I = impact fraction (maximum 30%) [STD]

I_c = moment of inertia of composite section

IM = dynamic load allowance [LRFD]

J = St. Venant torsional constant

K = longitudinal stiffness parameter [STD]

K_g = longitudinal stiffness parameter [LRFD]

k = factor used in calculation of distribution factor for multi-beam bridges [LRFD]

k = factor used in calculation of average stress in pretensioning steel for Strength Limit State

L = live load [STD]

L = length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment [STD]

L = overall beam length or design span

L = span length measured parallel to longitudinal beams [STD]

L = span length [LRFD]

L_c = critical length of yield line failure pattern [LRFD]

LL = vehicular live load [LRFD]

ℓ_d = development length [LRFD]

ℓ_x = length required to fully develop the strand measured from the end of the strand

M_a = negative moment at the end of the span being considered

M_b = negative moment at the end of the span being considered

M_b = unfactored bending moment due to barrier weight

M_c = flexural resistance of cantilevered wall [LRFD]

M_{CIP} = unfactored bending moment due to cast-in-place topping slab

M_{const} = unfactored bending moment due to construction load

M_{col} = bending moment due to horizontal collision force

M_{cr} = moment causing flexural cracking at section due to externally applied loads (after dead load) [STD]

M_{cr} = cracking moment [LRFD]

M^*_{cr} = cracking moment [STD]

M_D = unfactored bending moment due to diaphragm weight

M_d = bending moment at section due to unfactored dead load

$M_{d/nc}$ = moment due to non-composite dead loads [STD]

M_f = unfactored bending moment due to fatigue truck per beam

M_g = unfactored bending moment due to beam self-weight

M_{LL} = unfactored bending moment due to lane load per beam

M_{LL+I} = unfactored bending moment due to live load + impact

M_{LL+I} = unfactored bending moment due to design vehicular load

M_{LT} = unfactored bending moment due to truck load with dynamic allowance per beam

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S	= span length of deck slab	[STD]
S	= effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web	LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	[STD]
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)	
SH	= loss of pretension due to concrete shrinkage	[STD]
SR	= fatigue stress range	
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	
S_{tc}	= composite section modulus for top fiber of the deck slab	
S_{tg}	= composite section modulus for top fiber of the precast beam	
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
T	= collision force at deck slab level	
t	= thickness of web	
t	= thickness of an element of the beam	
t_f	= thickness of flange	
t_s	= cast-in-place deck thickness	
t_s	= depth of concrete deck	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_{LL}	= unfactored shear force due to lane load per beam	
V_{LL+I}	= unfactored shear force due to live load plus impact	
V_{LL+I}	= unfactored shear force due design vehicular live load	
V_{LT}	= unfactored shear force due to truck load with dynamic allowance per beam	
V_{mu}	= ultimate shear force occurring simultaneously with M_u	
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= $\dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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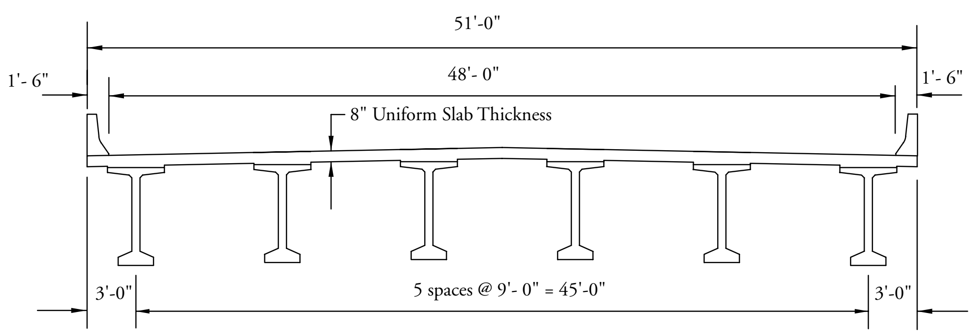
BULB-TEE (BT-72), SINGLE SPAN, COMPOSITE DECK, STANDARD SPECIFICATIONS

Bulb-Tee (BT-72), Single Span, Composite Deck, Standard Specifications

9.3.1 INTRODUCTION

This design example demonstrates the design of a 120-ft single-span AASHTO-PCI bulb-tee beam bridge. This example illustrates in detail the design of a typical interior beam at the critical sections in positive flexure, shear and deflection. The superstructure consists of six beams spaced at 9 ft on-center. Beams are designed to act compositely with the 8 in. thick cast-in-place concrete slab to resist all superimposed dead loads, live loads and impact. The top 1/2 in. of the slab is considered to be a wearing surface. Design live load is AASHTO HS20. The design is carried out in accordance with the AASHTO *Standard Specification for Highway Bridges,* 17th Edition, 2002.

Figure 9.3.1-1
Bridge Cross-section



9.3.2 MATERIALS

Cast-in-place slab: Thickness, actual, t_s = 8.0 in.

Structural = 7.5 in.

Note that a 1/2 in. wearing surface is considered to be an integral part of the 8 in. slab.

Concrete strength at 28 days, f'_c = 4,000 psi

Precast beams: AASHTO-PCI BT-72 Bulb-tee (as shown in **Fig. 9.3.2-1**)

Concrete strength at release, f'_{ci} = 5,500 psi

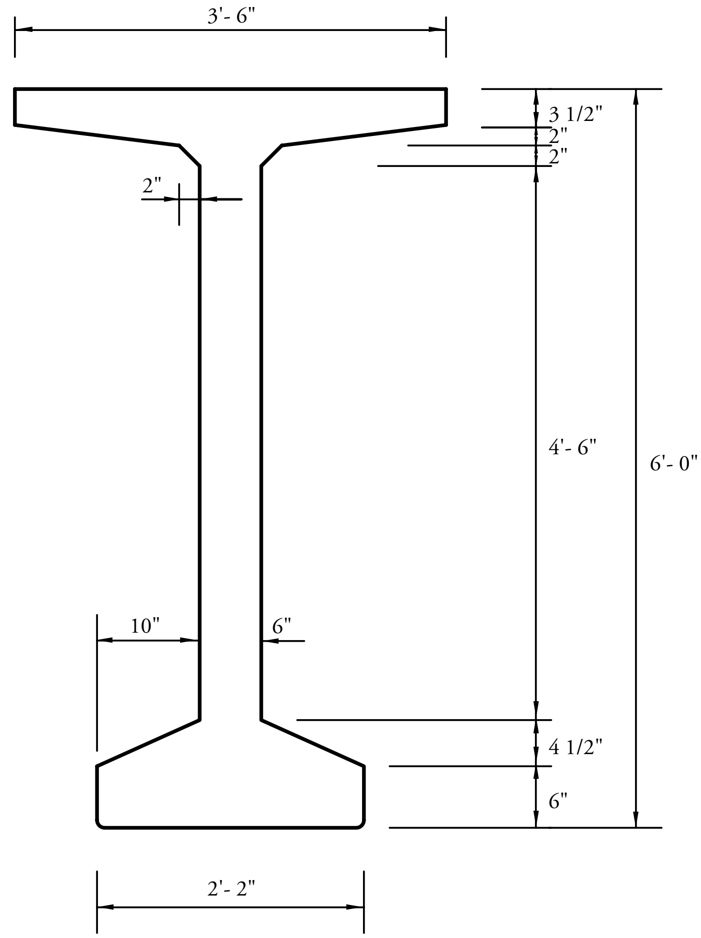
Concrete strength at 28 days, f'_c = 6,500 psi

Concrete unit weight = 150 pcf

Overall beam length = 121.0 ft

Design span = 120.0 ft

Figure 9.3.2-1
AASHTO-PCI BT-72 Bulb-Tee



Pretensioning strands: 1/2 in. diameter, seven wire, low relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, $f_s' = 270{,}000$ psi

Yield strength, $f_y^* = 0.9\ f_s' = 243{,}000$ psi [STD Art. 9.1.2]

Initial pretensioning, $f_{si} = 0.75\ f_s' = 202{,}500$ psi [STD Art. 9.15.1]

Modulus of elasticity, $E_s = 28{,}500$ ksi

Although the *Standard Specifications,* [Art. 9.16.2.1.2] indicates that the modulus of elasticity, E_s is 28,000 ksi, a value of 28,500 ksi is a more accurate value, according to the PCI Design Handbook and the *LRFD Specifications.*

Reinforcing bars: Yield strength, $f_y = 60{,}000$ psi

Future wearing surface: additional 2 in. with unit weight = 150 pcf

New Jersey-type barrier weight = 300 lbs/ft/side

9.3.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

9.3.3.1 Non-Composite Section

A = area of cross section of precast beam = 767 in.^{2}

h = overall depth of precast beam = 72 in.

I = moment of inertia about the centroid of the non-composite precast beam = 545,894 in.^{4}

y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam = 36.60 in.

y_t = distance from centroid to the extreme top fiber of the non-composite precast beam = 35.40 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 14,915 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 15,421 in.^{3}

Wt. = 0.799 k/ft

E_c = modulus of elasticity of concrete, psi

where

$E_c = (w_c)^{1.5}(33)\sqrt{f'_c}$ [STD Art. 8.7.1]

w_c = unit weight of concrete = 150 pcf [STD Art. 3.3.6]

The *Standard Specifications* [STD Art. 8.7.1] indicates that the unit weight of normal weight concrete is 145 pcf. However, precast concrete mixes typically have a relatively low water/cement ratio and high density. Therefore, a unit weight of 150 pcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f'_c = specified strength of concrete, psi

Modulus of elasticity for the cast-in-place slab, using f'_c = 4,000 psi, is:

$$E_c = (150)^{1.5}(33)\sqrt{4{,}000}/1000 = 3{,}834 \text{ ksi}$$

Modulus of elasticity for the beam at release, using $f'_c = f'_{ci}$ = 5,500 psi, is:

$$E_{ci} = (150)^{1.5}(33)\sqrt{5{,}500}/1000 = 4{,}496 \text{ ksi}$$

Modulus of elasticity of the beam at service loads, using f'_c = 6,500 psi is:

$$E_c = (150)^{1.5}(33)\sqrt{6{,}500}/1000 = 4{,}888 \text{ ksi}$$

9.3.3.2 Composite Section

9.3.3.2.1 Effective Flange Width

[STD Art. 9.8.3]

Effective web width of the precast beam is the lesser of: [STD Art. 9.8.3.1]

b_e = top flange width = 42 in. (controls)

or, b_e =2(6)(5.5) + 6 + 2(2) = 76 in.

Effective web width, b_e = 42 in.

The effective flange width is the lesser of: [STD Art. 9.8.3.2]

1/4 span length: $\dfrac{120(12)}{4} = 360$ in.

Distance center-to-center of beams: 9(12) = 108 in. (controls)

12 (effective slab thickness) plus effective beam web width

12(7.5) + 42 = 132 in.

Effective flange width = 108 in.

9.3.3.2.2 Modular Ratio Between Slab and Beam Materials

Modular ratio between slab and beam materials:

$$n = \frac{E_c(\text{slab})}{E_c(\text{beam})} = \frac{3{,}834}{4{,}888} = 0.7845$$

9.3.3.2.3 Transformed Section Properties

Note: Only the structural thickness of the deck (7.5 in.) will be used in these computations.

Transformed flange width = n (effective flange width) = 0.7845 (108) = 84.73 in.

Transformed flange area = n (effective flange width)(t_s) = 0.7845(108)(7.5) = 635.45 in^2

Due to camber of the pretensioned precast beam, a minimum haunch thickness of 1/2 in. at midspan is considered in the structural properties of the composite section. The haunch width must also be transformed.

Transformed haunch width = (0.7845)(42) = 32.95 in.

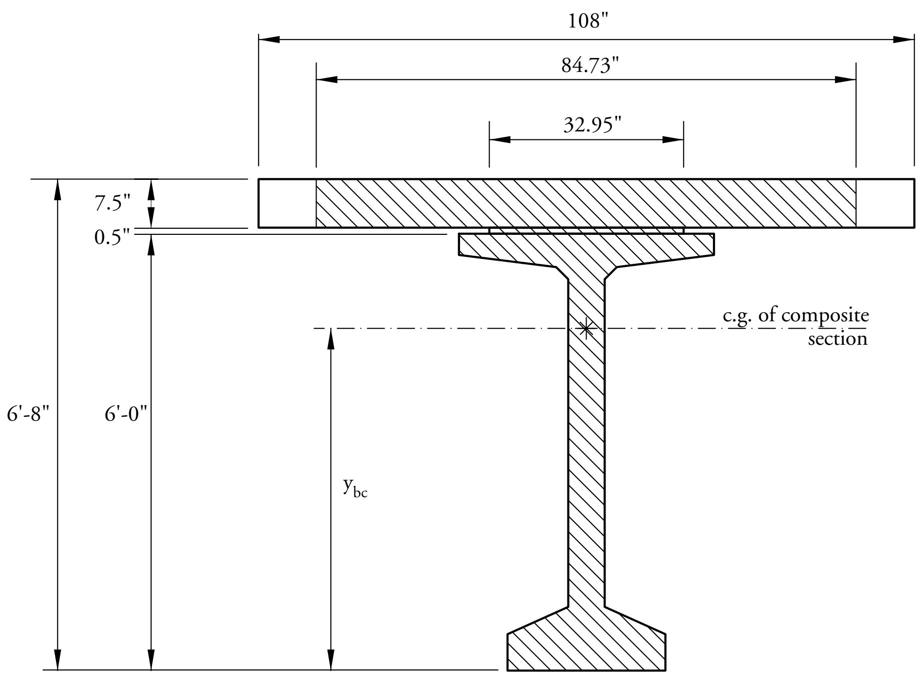
Transformed area of haunch = (0.7845)(42)(0.5) = 16.47 in.^{2}

Note that the haunch should only be considered to contribute to section properties if it is required to be provided in the completed structure. Therefore, some designers neglect its contribution to the section properties.

Table 9.3.3.2.2-1
Properties of Composite Section

	Transformed Area in.^{2}	y_b in.	$A y_b$ in.^{3}	$A(y_{bc}-y_b)^2$	I in.^{4}	$I + A(y_{bc}-y_b)^2$ in.^{4}
Beam	767.00	36.60	28,072.20	253,224.21	545,894.00	799,118.21
1/2" Haunch	16.47	72.25	1,189.96	5,032.42	0.34	5,032.76
Slab	635.45	76.25	48,453.06	293,190.53	2,978.79	296,169.32
Σ	1,418.92		77,715.22			1,100,320.20

Figure 9.3.3.2.3-1
Composite Section



A_c = total area of composite section = 1,419 in.^{2}

h_c = total height of composite section = 80.00 in.

I_c = moment of inertia of composite section = 1,100,320 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = 77,715/1,419 = 54.77 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 72 − 54.77 = 17.23 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab = 80 − 54.77 = 25.23 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam

$$= I_c/y_{bc} = \frac{1{,}100{,}320}{54.77} = 20{,}090 \text{ in.}^3$$

S_{tg} = composite section modulus for top fiber of the precast beam

$$= I_c/y_{tg} = \frac{1{,}100{,}320}{17.23} = 63{,}861 \text{ in.}^3$$

S_{tc} = composite section modulus for top fiber of the slab

$$= \frac{I_c}{n\ y_{tc}} = \frac{1{,}100{,}320}{0.7845\,(25.23)} = 55{,}592 \text{ in.}^3$$

9.3.4 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the beam and the weight of the slab and haunch act on the non-composite simple-span structure, while the weight of barriers, future wearing surface, and live load plus impact act on the composite simple-span structure.

9.3.4.1 Shear Forces and Bending Moments Due to Dead Loads

[STD Art. 3.3]

9.3.4.1.1 Dead Loads

Beam weight = 0.799 kip/ft

$$\text{8 in. slab weight} = 0.150\left(\frac{8}{12}\right)(9.0) = 0.900 \text{ kip/ft}$$

$$\text{Haunch weight} = \frac{1/2}{12}(3.5 \text{ ft})(0.150) = 0.022 \text{ kip/ft}$$

Note:

1. Actual slab thickness (8 in.) is used for computing slab dead load
2. A 1/2 in. minimum haunch is assumed in the computation of forces. If a deeper haunch will be used because of final beam camber, the weight of the actual haunch should be used.
3. Diaphragms: Many state agencies are moving away from using cast-in-place concrete diaphragms in favor of lighter weight steel diaphragms. Therefore, the weight of diaphragms will be ignored.
4. Dead loads placed on the composite structure are distributed equally among all beams. [STD Art. 3.23.2.3.1.1]

$$\text{Barriers: (2 barriers)}\ \frac{300}{6 \text{ beams}}/1{,}000 = 0.100 \text{ kip/ft/beam}$$

Weight of future wearing surface: $\frac{2}{12}(150) = 25$ psf which is applied over the entire width of the bridge between curbs (48 ft)

$$\text{Future wearing surface: } \frac{25(48.0)}{6}/1{,}000 = 0.200 \text{ kip/ft/beam}$$

9.3.4.1.2
Unfactored Shear Forces and Bending Moments

For a simply supported beam, simple span (L), loaded with a uniformly distributed load (w), the shear force (V_x) and the bending moment (M_x) at a distance (x) from the support are given by:

$$V_x = w(0.5L - x) \quad \text{(Eq. 9.3.4.1.2-1)}$$

$$M_x = 0.5wx(L - x) \quad \text{(Eq. 9.3.4.1.2-2)}$$

Using the above equations, values of shear forces and bending moments for a typical interior beam under dead loads (weight of beam, slab, haunch, barriers and future wearing surface) are computed and given in **Table 9.3.4.1.2-1**. For these calculations, the span length is the design span (120 ft). However, for calculations of stresses and deformations at the time the prestress is released, the overall length of the precast beam (121 ft) is used, as illustrated later in this example.

9.3.4.2
Shear Forces and Bending Moments Due to Live Load

9.3.4.2.1
Live Load

Live load is either the standard truck or lane loading corresponding to HS20. The standard truck load will govern the design for this 120-ft simple-span example.

[STD Art. 3.7.1.1]

9.3.4.2.2
Live Load Distribution Factor for a Typical Interior Beam

Using the live load distribution factor for moment for a precast pretensioned concrete beam, the fraction of the wheel load carried by the interior beam:

$$DF_m = \frac{S}{5.5} = \frac{9.0}{5.5} = 1.636 \text{ wheels/beam} \quad \text{[STD Table 3.23.1]}$$

where S = average spacing between beams in feet

$DF_m/2 = 0.818$ lanes/beam

[STD Art. 3.8]

9.3.4.2.3
Live Load Impact

The live load impact factor is computed using in the following equation:

$$I = \frac{50}{L + 125} \quad \text{[STD Eq. 3-1]}$$

where

I = impact fraction (maximum 30%)

L = length in feet of the span under consideration = 120 ft [STD Art. 3.8.2.2]

$$I = \frac{50}{120 + 125} = 0.204$$

Impact for shear varies along the span according to the location of the truck [STD Art. 3.8.2.2 (d)]. For simplicity, the impact factor computed above is used for shear.

9.3.4.2.4
Unfactored Shear Forces and Bending Moments

Shear force and bending moment envelopes on a per-lane-basis are calculated at tenth-points of the span using the equations given in Chapter 8. However, this generally is done by means of commercially available computer software that has the ability to deal with moving loads.

Therefore, live load shear force and bending moment per beam are:

V_{LL+I} = (shear force per lane)(Distribution Factor)(1+I)

= (shear force per lane)(0.818)(1+0.204)

= (shear force per lane)(0.985), kips

M_{LL+I} = (bending moment per lane)(Distribution Factor)(1+I)

= (bending moment per lane)(0.818)(1+0.204)

= (bending moment per lane)(0.985), ft-kips

At any section along the span, the maximum bending moment and shear are computed for the standard truck loading and for the lane loading separately. The larger of the two loading types controls the design for the section in question. At each section, the load position must be determined to give the maximum shears and moments. This can be done by means of commercially available software programs.

Values of V_{LL+I} and M_{LL+I} at different points are given in **Table 9.3.4.1.2-1.**

9.3.4.3 *Load Combinations*

[STD Art. 3.22]

For service load design (Group I): 1.00 D + 1.00(L+I) [STD Table 3.22.1A]

where

D = dead load

L = live load

I = impact fraction

For load factor design (Group I): 1.3[1.00 D + 1.67(L + I)] [STD Table 3.22.1A]

9.3.5 ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber at the section of maximum moment or at the harp points. For estimating the number of strands, only the stresses at midspan need to be considered.

9.3.5.1 *Service Load Stresses at Midspan*

Bottom tensile stresses due to applied loads:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + M_{LL+I}}{S_{bc}}$$

where

f_b = concrete stress at the bottom fiber of the beam

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_s = unfactored bending moment due to slab and haunch weights, ft-kips

M_b = unfactored bending moment due to barrier weight, ft-kips

M_{ws} = unfactored bending moment due to wearing surface, ft-kips

M_{LL+I} = unfactored bending moment due to live load + impact, ft-kips

Using values of bending moments from **Table 9.3.4.1.2-1**, the bottom tensile stress at midspan is:

$$f_b = \frac{(1{,}438.2+1{,}659.6)(12)}{14{,}915} + \frac{(180.0+360.0+1{,}851.6)(12)}{20{,}090} = 3.921 \text{ ksi}$$

Table 9.3.4.1.2-1 Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Distance	Section	Beam weight		Slab + Haunch weights		Barrier weight		Wearing Surface weight		Live load + impact	
x	x/L	Shear V_g	Moment M_g	Shear V_s	Moment M_s	Shear V_b	Moment M_b	Shear V_{ws}	Moment M_{ws}	Shear V_{LL+I}	Moment M_{LL+I}
ft		kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips
0	0.000	47.9	0.0	55.3	0.0	6.0	0.0	12.0	0.0	65.4	0.0
3.333[1]	0.028	45.3	155.4	52.2	179.3	5.7	19.4	11.3	38.9	63.6	211.5
12	0.100	38.4	517.8	44.3	597.5	4.8	64.8	9.6	129.6	58.3	699.7
24	0.200	28.8	920.4	33.2	1,062.1	3.6	115.2	7.2	230.4	51.2	1,229.1
36	0.300	19.2	1,208.1	22.1	1,394.1	2.4	151.2	4.8	302.4	44.1	1,588.4
48[2]	0.400	9.6	1,380.7	11.1	1,593.2	1.2	172.8	2.4	345.6	37.0	1,799.6
60	0.500	0.0	1,438.2	0.0	1,659.6	0.0	180.0	0.0	360.0	29.9	1,851.6

[1] Critical section for shear (see section 9.3.11 of this example)
[2] Harp point

9.3.5.2 Allowable Stress Limit

At service loads, allowable tensile stress in the precompressed tensile zone:

$$F_b = 6\sqrt{f'_c} = 6\sqrt{6{,}500}\left(\frac{1}{1{,}000}\right) = -0.484\text{ksi} \qquad \text{[STD Art. 9.15.2.2]}$$

9.3.5.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress - allowable tension stress at final = $f_b - F_b$

$= 3.921 - 0.484 = 3.437$ ksi

The location of the center of gravity of strands at midspan usually ranges from 5 to 15% of the beam depth, measured from the bottom of the beam. A value of 5% is appropriate for newer efficient sections like bulb-tees and 15% for the less efficient AASHTO shapes of normal design. Assume the distance from the center of gravity of strands to the bottom fiber of the beam is equal to y_{bs} = 4.00 in.
(Note: y_{bs}/h = 4/72 = 5.5%)

Strand eccentricity at midspan:

$e_c = y_b - y_{bs} = 36.60 - 4.00 = 32.60$ in.

Bottom fiber stress due to prestress after all losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}\, e_c}{S_b}$$

where P_{se} = effective pretension force after allowing for all losses

Set the required precompression (3.437 ksi) equal to the bottom fiber stress due to prestress, solve for the required minimum P_{se}.

Then, $3.437 = \dfrac{P_{se}}{767} + \dfrac{32.60\ P_{se}}{14{,}915}$

$P_{se} = 985.0$ kips

Assume final losses = 25% of f_{si}

Assumed final losses = 0.25(202.5) = 50.6 ksi

The available prestress force per strand after all losses

$= (\text{cross-sectional area of one strand})[f_{si} - \text{losses}]$

$= 0.153(202.5 - 50.6) = 23.24$ kips

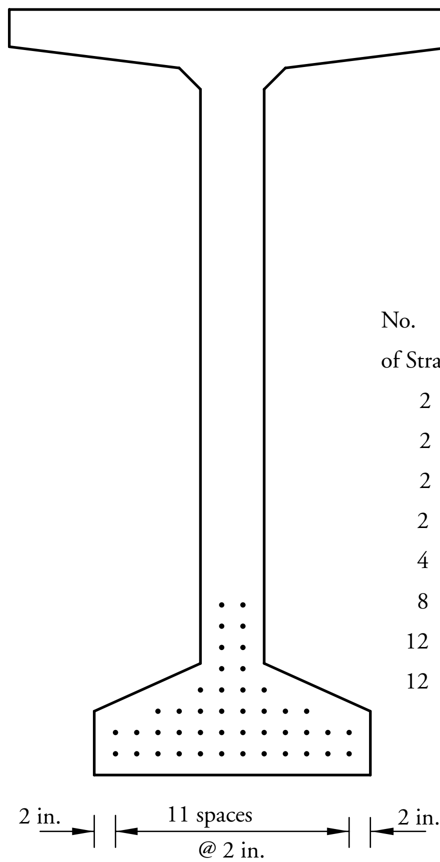
$$\text{Number of strands required} = \frac{985.0}{23.24} = 42.38$$

Try (44) 1/2-in.-diameter, 270 ksi strands

9.3.5.4 Strand Pattern

The assumed strand pattern for the 44 strands at the midspan section is shown in **Figure 9.3.5.4-1**. Each available position was filled beginning with the bottom row.

Figure 9.3.5.4-1 Assumed Strand Pattern at Midspan



No. of Strands	Distance from bottom (in.)
2	16
2	14
2	12
2	10
4	8
8	6
12	4
12	2

Calculate the distance from center of gravity of the strand to the bottom fiber of the beam, y_{bs}.

$$y_{bs} = \frac{12(2)+12(4)+8(6)+4(8)+2(10)+2(12)+2(14)+2(16)}{44} = 5.82 \text{ in.}$$

Strand eccentricity at midspan:

$$e_c = y_b - y_{bs} = 36.60 - 5.82 = 30.78 \text{ in.}$$

9.3.6 PRESTRESS LOSSES

[STD Art. 9.16.2]

Total losses = SH + ES + CR_c + CR_s [STD Eq. 9-3]

where

SH = loss of prestress due to concrete shrinkage, ksi

ES = loss of prestress due to elastic shortening, ksi

Cr_c = loss of prestress due to creep of concrete, ksi

Cr_s = loss of prestress due to relaxation of pretensioning steel, ksi

9.3.6.1 Shrinkage

[STD Art. 9.16.2.1.1]

Relative humidity varies significantly from one area of the country to another. Refer to the U.S. map, Figure 9.16.7.1.1, in the *Standard Specifications.*

Assume relative humidity, RH = 70%

$$SH = 17{,}000 - 150\,RH = [17{,}000 - 150(70)]\frac{1}{1{,}000} = 6.5 \text{ ksi}$$ [STD Eq. 9-4]

9.3.6.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

For pretensioned members

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e_c^{\,2}}{I} - \frac{(M_g + M_D)e_c}{I}$$

f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer

P_{si} = pretension force after allowing for the initial losses. *The Standard Specifications* allow that the reduction to initial tendon stress be estimated as 0.69 f_s' for low-relaxation strands

= (number of strands)(area of strands)(0.69f_s')

= (44)(0.153)[(0.69)(270)] = 1,254.2 kips

M_g = unfactored bending moment due to beam self-weight = 1,438.2 ft-kips

M_D = unfactored bending moment due to diaphragm weight

e_c = eccentricity of the strand at the midspan = 30.78 in.

M_g should be calculated based on the overall beam length of 121 ft. However, since the elastic shortening losses will be a part of the total losses, f_{cir} is conservatively computed based on using the design span length of 120 ft.

$$f_{cir} = \frac{1{,}254.2}{767} + \frac{1{,}254.2(30.78)^2}{545{,}894} - \frac{1{,}438.2(12)(30.78)}{545{,}894}$$

$$= 1.635 + 2.177 - 0.973 = 2.839 \text{ ksi}$$

$$ES = \frac{28{,}500}{4{,}496}(2.839) = 18.0 \text{ ksi}$$

9.3.6.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

$CR_c = 12f_{cir} - 7f_{cds}$ [STD Eq. 9-9]

where

f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied.

$$= \frac{M_s e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

where

M_s = slab and haunch moment = 1,659.6 ft-kips

M_{SDL} = superimposed dead load moment = $M_b + M_{ws}$ = 540.0 ft-kips

y_{bc} = 54.77 in.

y_{bs} = the distance from center of gravity of the strand at midspan to the bottom of the beam = 5.82 in.

I = moment of inertia for the non-composite section = 545,894 in.^{4}

I_c = moment of inertia for the composite section = 1,100,320 in.^{4}

$$f_{cds} = \frac{1{,}659.6(12)(30.78)}{545{,}894} + \frac{(540)(12)(54.77 - 5.82)}{1{,}100{,}320}$$

$= 1.123 + 0.288 = 1.411$ ksi

$CR_c = 12(2.839) - 7(1.411) = 24.2$ ksi

9.3.6.4 Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strand:

$CR_s = 5{,}000 - 0.10\ ES - 0.05(SH + CR_c)$ [STD Eq. 9-10A]

$$= [5{,}000 - 0.10(17{,}996) - 0.05(6{,}500 + 24{,}200)]\left(\frac{1}{1{,}000}\right) = 1.7 \text{ ksi}$$

9.3.6.5 Total Losses at Transfer

Total initial losses = 18.0 ksi

f_{si} = effective initial pretension stress = 202.5 − 18.0 = 184.5 ksi

P_{si} = effective pretension force after allowing for the initial losses

= 44(0.153)(184.5) = 1,242.1 kips

9.3.6.6 Total Losses at Service Loads

SH = 6.5 ksi

ES = 18.0 ksi

CR_c = 24.2 ksi

CR_s = 1.7 ksi

Total final losses = 6.5 + 18.0 + 24.2 + 1.7 = 50.4 ksi

or $\dfrac{50.4}{0.75(270)}(100) = 24.9\%$ losses

f_{se} = effective final prestress = 0.75(270) − 50.4 = 152.1 ksi

P_{se} = 44 (0.153)(152.1) = 1,023.9 kips

9.3.7 CONCRETE STRESSES AT TRANSFER

9.3.7.1 Allowable Stress Limits

[STD Art. 9.15.2.1]

Compression: $0.6\,f'_{ci} = +3.300$ ksi (compression)

Tension: The maximum tensile stress shall not exceed:

- $7.5\sqrt{f'_{ci}} = -0.556$ ksi (tension)
- If the calculated tensile stress exceeds 200 psi or $3\sqrt{f'_{ci}} = 0.222$ ksi, whichever is smaller, bonded reinforcement should be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

9.3.7.2 Stresses at Transfer Length Section

This section is located at a distance equal to the transfer length from the end of the beam. Stresses at this location need only be checked at release, because it almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length = 50(strand diameter) [STD Art. 9.20.2.4]

= 50(0.50) = 25 in. = 2.08 ft

The transfer length section is located at 2.08 ft from the end of the beam or at a point 1.58 ft from centerline of the bearing. This is assuming the beam extends 6 in. beyond the bearing centerline. This point on the design span = 1.58/120.0 = 0.013.

Due to the camber of the beam at release, the beam self-weight is acting on the overall beam length (121.0 ft). Therefore, the values of bending moment given in **Table 9.3.4.1.2-1** cannot be used because they are based on the design span (120.0 ft). The value of bending moment at the transfer length due to beam self-weight is calculated using Equation (9.3.4.1.2-2) based on overall length.

Therefore, $M_g = 0.5(0.799)(2.08)(121 - 2.08) = 98.8$ ft-kips.

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = +\frac{P_{si}}{A} - \frac{P_{si}\,e}{S_t} + \frac{M_g}{S_t}$$

$$f_t = +\frac{1,242.1}{767} - \frac{1,242.1(30.78)}{15,421} + \frac{98.8(12)}{15,421}$$

$$= +1.619 - 2.479 + 0.077 = -0.783 \text{ ksi}$$

Allowable tension: -0.556 ksi N.G.

Compute concrete stress at the bottom fiber of the beam, f_b:

$$f_b = +\frac{P_{si}}{A} + \frac{P_{si}\,e}{S_b} - \frac{M_g}{S_b}$$

$$f_b = +\frac{1,242.1}{767} + \frac{1,242.1(30.78)}{14,915} - \frac{98.8(12)}{14,915}$$

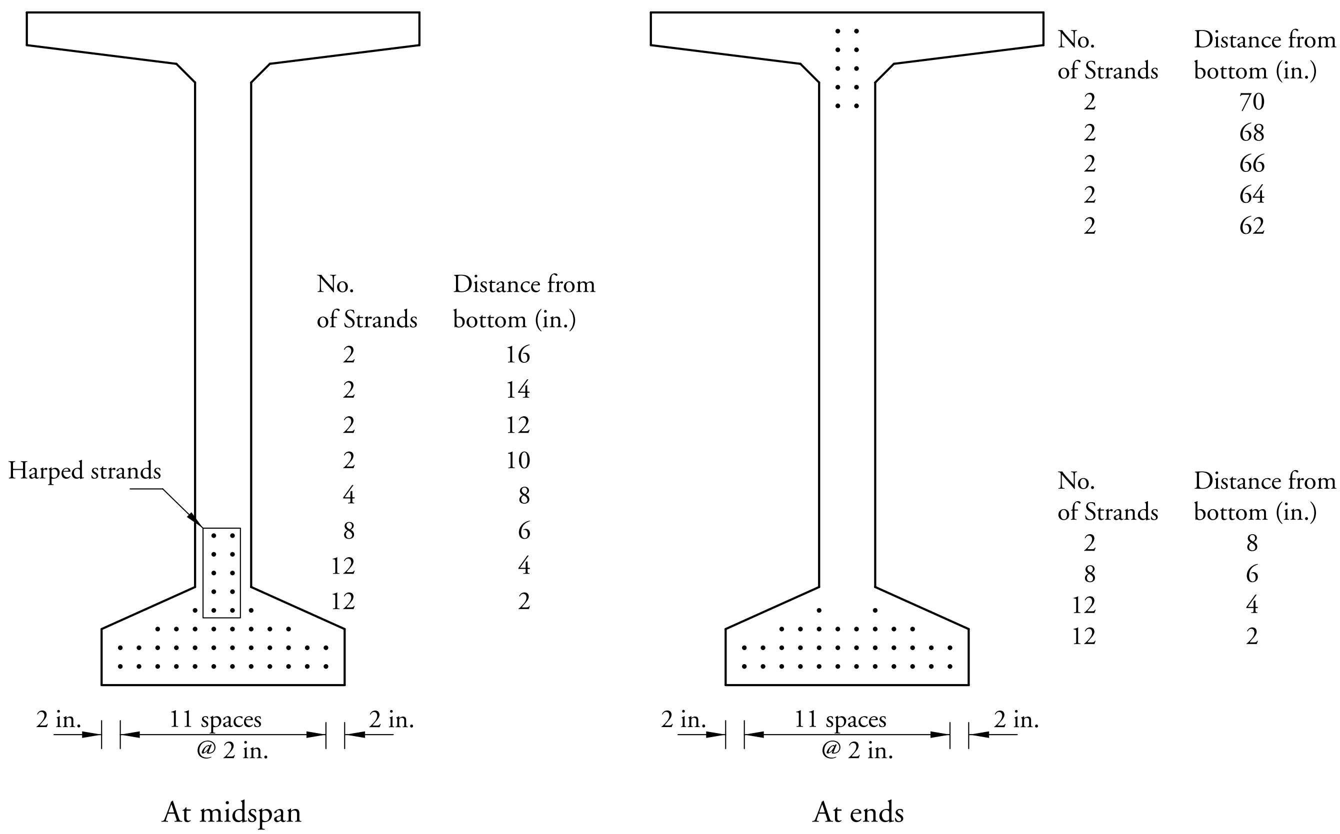
$$= +1.619 + 2.563 - 0.079 = +4.103 \text{ ksi}$$

Allowable compression: +3,300 ksi N.G.

Since the top and bottom fiber stresses exceed those allowed, harped and/or debonded strands must be used.

In this example, a harped strand pattern will be used with harp points at 0.40 L = 48.0 ft from the centerline bearing or 48.5 ft from the end of the beam. Try harping 10 strands as shown in **Figs. 9.3.7.2-1** and **9.3.7.2-2**.

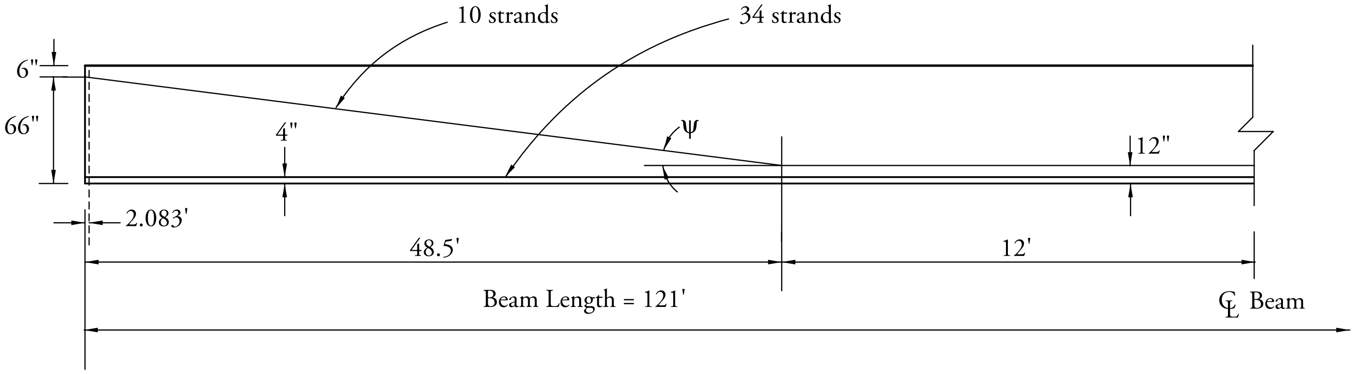
Figure 9.3.7.2-1 Strand Pattern



The distance between the center of gravity of the 10 harped strands and the top of the beam at the end of the beam $= \dfrac{2(2)+2(4)+2(6)+2(8)+2(10)}{10} = 6.00$ in.

The distance between the center of gravity of the 10 harped strands and the bottom of the beam at the harp points $= \dfrac{2(8)+2(10)+2(12)+2(14)+2(16)}{10} = 12.00.$

Figure 9.3.7.2-2 Longitudinal Strand Profile



The distance between the center of gravity of the 10 harped strands and the top of the beam at the transfer length section = $6\text{in.} + \dfrac{(72\text{in.} - 12\text{in.} - 6\text{in.})}{48.5\text{ft}}(2.083\text{ft})$ = 8.32 in.

The distance between the center of gravity of the 34 straight strands and the bottom of the beam at all locations = $\dfrac{12(2) + 12(4) + 8(6) + 2(8)}{34}$ = 4.00 in.

The distance between the center of gravity of all strands and the bottom of the beam:

at end of beam = $\dfrac{34(4) + 10(72 - 6)}{44}$ = 18.09 in.

at transfer length = $\dfrac{34(4) + 10(72 - 8.32)}{44}$ = 17.56 in.

Eccentricity of strands at transfer length, e = 36.60 − 17.56 = 19.04 in.

Recompute the concrete stresses at transfer length section with the harped strands:

$$f_t = +\frac{1,242.1}{767} - \frac{1,242.1(19.04)}{15,421} + \frac{98.8(12)}{15,421} = +1.619 - 1.534 + 0.077 = +0.162 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

Note: Since the top fiber stress is smaller than $3\sqrt{f'_{ci}}$, there is no need for additional bonded reinforcement.

$$f_b = +\frac{1,242.1}{767} + \frac{1,242.1(19.04)}{14,915} - \frac{98.8(12)}{14,915} = +1.619 + 1.586 - 0.079 = +3.126 \text{ ksi}$$

Allowable compression: = +3.300 ksi O.K.

9.3.7.3 Stresses at Harp Points

Eccentricity of strands is the same as at midspan.

Bending moment at the harp point (0.4 L) due to beam self-weight is calculated using Equation (9.3.4.1.2-2) based on overall length.

Therefore, $M_g = 0.5(0.799)(48.5)(121 - 48.5) = 1{,}404.7$ ft-kips

$$f_t = +\frac{1{,}242.1}{767} - \frac{1{,}242.1(30.78)}{15{,}421} + \frac{1{,}404.7(12)}{15{,}421} = +1.619 - 2.479 + 1.093$$

$= +0.233$ ksi

Allowable compression: = +3.300 ksi O.K.

$$f_b = +\frac{1{,}242.1}{767} + \frac{1{,}242.1(30.78)}{14{,}915} - \frac{1{,}404.7(12)}{14{,}915} = +1.619 + 2.563 - 1.130$$

$= +3.052$ ksi

Allowable compression: +3.300 ksi O.K.

9.3.7.4
Stresses at Midspan

Bending moment at midspan due to beam self-weight is calculated using Equation (9.3.4.1.2-2) based on overall length.

Therefore, $M_g = 0.5\,(0.799)(60.5)(121 - 60.5) = 1{,}462.3$ ft-kips

$$f_t = +\frac{1{,}242.1}{767} - \frac{1{,}242.1(30.78)}{15{,}421} + \frac{1{,}462.3(12)}{15{,}421} = +1.619 - 2.479 + 1.138$$

$= +0.278$ ksi

Allowable compression: +3.300 ksi O.K.

$$f_b = +\frac{1{,}242.1}{767} + \frac{1{,}242.1(30.78)}{14{,}915} - \frac{1{,}462.3(12)}{14{,}915} = +1.619 + 2.563 - 1.177$$

$= +3.005$ ksi

Allowable compression: = +3.300 ksi O.K.

Note: Stresses at harp points are more critical than stresses at midspan.

9.3.7.5
Hold-Down Force

Assume the maximum initial pretensioning stress before allowing for any losses:

$0.80f_{pu} = 0.80(270) = 216$ ksi

Initial pretension force per strand before losses = 0.153(216) = 33.0 kips

Hold-down force per strand = $\dfrac{54}{48.5(12)}$(33.0 kips/strand)(1.05) = 3.21 kips/strand

Note: The factor, 1.05, is applied to account for friction.

Total force = 10 strands (3.21) = 32.1 kips

$$\psi = \tan^{-1}\left(\frac{54}{48.5(12)}\right) = 5.30°$$

Where 54 in. is the vertical strand shift in a horizontal distance of 48.5 ft.

The hold-down force and the harp angle should be checked against maximum limits for local practices. Refer to Chapter 3, Fabrication and Construction, Section 3.3.2.2 and Chapter 8, Design Theory and Procedures for additional details.

9.3.7.6
Summary of Stresses at Transfer

	Top of beam f_t (ksi)	Bottom of beam f_b (ksi)
At transfer length section	+0.162	+3.126
At harp points	+0.233	+3.052
At midspan	+0.278	+3.005

9.3.8 CONCRETE STRESSES AT SERVICE LOADS

9.3.8.1 Allowable Stress Limits

[STD Art. 9.15.2.2]

Compression:

Case (I): for all load combinations

$0.60f_c' = 0.60(6{,}500)/1{,}000 = +3.900$ ksi (for precast beam)

$0.60f_c' = 0.60(4{,}000)/1{,}000 = +2.400$ ksi (for slab)

Case (II): for effective pretension force + permanent dead loads

$0.40f_c' = 0.40(6{,}500)/1{,}000 = +2.600$ ksi

$0.40f_c' = 0.40(4{,}000)/1{,}000 = +1.600$ ksi (for slab)

Case (III): live load + 1/2 (pretensioning force + dead loads)

$0.40f_c' = 0.40(6{,}500)/1{,}000 = +2.600$ ksi (for precast beam)

$0.40f_c' = 0.40(4{,}000)/1{,}000 = +1.600$ ksi (for slab)

Tension: $6\sqrt{f_c'} = 6\sqrt{6{,}500}\left(\dfrac{1}{1{,}000}\right) = -0.484$ ksi

9.3.8.2 Stresses at Midspan

Bending moment values at this section are given in **Table 9.3.4.1.2-1**.

Compute concrete stress at top fiber of beam:

$$f_t = +\frac{P_{se}}{A} - \frac{P_{se}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_b + M_{ws}}{S_{tg}} + \frac{M_{LL+I}}{S_{tg}}$$

Case (I):

$$f_t = \frac{1{,}023.9}{767} - \frac{1{,}023.9(30.78)}{15{,}421} + \frac{(1{,}438.2 + 1{,}659.6)(12)}{15{,}421} + \frac{(180 + 360)(12)}{63{,}861} + \frac{1{,}851.6(12)}{63{,}861}$$

$= +1.335 - 2.044 + 2.411 + 0.101 + 0.348 = +2.151$ ksi

Allowable compression: +3.900 ksi O.K.

Case (II):

$f_t = +1.335 - 2.044 + 2.411 + 0.101 = +1.803$ ksi

Allowable compression: +2.600 ksi O.K.

Case (III):

$f_t = 0.5(+1.335 - 2.044 + 2.411 + 0.101) + 0.348 = +1.250$ ksi

Allowable compression: +2.600 ksi O.K.

Compute concrete stresses at bottom of beam:

$$f_b = +\frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_b + M_{ws}}{S_{bc}} - \frac{M_{LL+I}}{S_{bc}}$$

$$f_b = +\frac{1{,}023.9}{767} + \frac{1{,}023.9(30.78)}{14{,}915} - \frac{(1{,}438.2 + 1{,}659.6)(12)}{14{,}915} - \frac{(180 + 360)(12)}{20{,}090} - \frac{1{,}851.6(12)}{20{,}090}$$

$$= +1.335 + 2.113 - 2.492 - 0.323 - 1.106 = -0.473 \text{ ksi}$$

Allowable tension: −0.484 ksi O.K.

Compute stresses at the top of the slab:

Case (I):

$$f_t = \frac{M_b + M_{ws}}{S_{tc}} + \frac{M_{LL+I}}{S_{tc}} = \frac{(180 + 360)(12)}{55{,}592} + \frac{(1{,}851.6)(12)}{55{,}592} = 0.117 + 0.400$$

$$= 0.517 \text{ ksi}$$

Allowable compression: +2.400 ksi O.K.

Case (II):

$f_t = +0.117$ ksi

Allowable compression: +1.600 ksi O.K.

Case (III):

$f_t = 0.5(+0.117) + 0.400 = +0.459$ ksi

Allowable compression: +1.600 ksi O.K.

9.3.8.3 Summary of Stresses at Service Loads

	Top of Slab f_t (ksi)	Top of Beam f_t (ksi)	Bottom of Beam f_t (ksi)
At midspan	+0.517	+2.151	−0.473

9.3.9 FLEXURAL STRENGTH

[STD Art. 9.17]

Using Group I load factor design loading combination, given earlier in Section 9.3.4.3 of the *Standard Specifications:*

$$M_u = 1.3[M_g + M_s + M_b + M_{ws} + 1.67(M_{LL+I})] \quad \text{[STD Table 3.22.1.A]}$$

$$= 1.3[1{,}438.2 + 1{,}659.6 + 180.0 + 360.0 + 1.67(1{,}851.6)] = 8{,}749 \text{ ft-kips}$$

Compute average stress in pretensioning steel at ultimate load, f^*_{su}:

$$f^*_{su} = f'_s\left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right) \quad \text{[STD Eq. 9-17]}$$

where

f^*_{su} = average stress in pretensioning steel at ultimate load.

γ^* = 0.28 for low-relaxation strand [STD Art. 9.1.2]

$$\beta_1 = 0.85 - 0.05\frac{(f'_c - 4{,}000)}{1{,}000} \geq 0.65 \text{ when } f'_c > 4{,}000 \text{ psi} \quad \text{[STD Art. 8.16.2.7]}$$

$$= 0.85 - 0.05\frac{(4{,}000 - 4{,}000)}{1{,}000} = 0.85$$

$$\rho^* = \frac{A^*_s}{bd}$$

where

A^*_s = area of pretensioned reinforcement = 44(0.153) = 6.732 in.^{2}

b = effective flange width = 108 in.

y_{bs} = distance from center of gravity of the strands to the bottom fiber of the beam = 5.82 in.

d = distance from top of slab to centroid of pretensioning strands

= beam depth (h) + haunch + slab thickness − y_{bs}

= 72 + 0.5 + 7.5 − 5.82 = 74.18 in.

$$\rho^* = \frac{6.732}{108\,(74.18)} = 0.000840$$

$$f^*_{su} = 270\left[1-\left(\frac{0.28}{0.85}\right)(0.000840)\left(\frac{270}{4}\right)\right] = 265.0 \text{ ksi}$$

Compute the depth of the compression block:

$$a = \frac{A^*_s f^*_{su}}{0.85\, f'_c\, b} = \frac{6.732(265.0)}{(0.85)(4.0)(108)} = 4.86 \text{ in.} < 7.5 \text{ in.} \quad \text{[STD Art. 9.17.2]}$$

Therefore, the depth of the compression block is less than flange thickness and the section must be considered as a rectangular section.

Design flexural strength of a rectangular section:

$$\phi M_n = \phi A^*_s f^*_{su} d\left(1 - 0.6\frac{\rho^* f^*_{su}}{f'_c}\right) \quad \text{[STD Eq. 9-13]}$$

where

ϕ = strength reduction factor = 1.0 [STD Art. 9.14]

M_n = nominal moment strength of a section

$$\phi M_n = 1.0(6.732)(265.0)(74.18)\left(1-0.6\frac{0.00084(265.0)}{4}\right)/12$$

$$= 10{,}660 \text{ ft-kips} > M_u = 8{,}749 \text{ ft-kips} \quad \text{O.K.}$$

9.3.10 DUCTILITY LIMITS

9.3.10.1 Maximum Reinforcement

[STD Art. 9.18.1]

Pretensioned concrete members are designed so that the steel is yielding as ultimate capacity is approached.

Reinforcement index for rectangular section:

$$\rho^* \frac{f^*_{su}}{f'_c} < 0.36\beta_1 = 0.00084\left(\frac{265.0}{4}\right) = 0.0557 < 0.36(0.85) = 0.306 \text{ O.K. [STD Eq. 9-20]}$$

9.3.10.2 Minimum Reinforcement

[STD Art. 9.18.2]

The total amount of pretensioned and non-pretensioned reinforcement should be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment, M^*_{cr}:

$$\phi M_n \geq 1.2 M^*_{cr}$$

Compute cracking moment:

$$M^*_{cr} = (f_r + f_{pe})S_{bc} - M_{d/nc}\left(\frac{S_{bc}}{S_b} - 1\right) \qquad \text{[STD Art. 9.18.2.1]}$$

where

f_r = modulus of rupture [STD Art. 9.15.2.3]

$$= 7.5\sqrt{f'_c} = 7.5\sqrt{6{,}500}\left(\frac{1}{1{,}000}\right) = 0.605 \text{ ksi}$$

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

where

P_{se} = effective prestress force after losses = 1,023.9 kips

e_c = 30.78 in.

$$f_{pe} = \frac{1{,}023.9}{767} + \frac{1{,}023.9(30.78)}{14{,}915} = 1.335 + 2.113 = 3.448 \text{ ksi}$$

$M_{d/nc}$ = non-composite dead load moment at midspan due to self-weight of beam and weight of slab = 1,438.2 + 1,659.6 = 3,097.8 ft-kips

$$M^*_{cr} = (0.605 + 3.448)(20{,}090)\left(\frac{1}{12}\right) - 3{,}097.8\left(\frac{20{,}090}{14{,}915} - 1\right) = 5{,}711 \text{ ft-kips}$$

$$1.2M^*_{cr} = 6{,}853 \text{ ft-kips} < \phi M_n = 10{,}660 \text{ ft-kips} \quad \text{O.K.}$$

Contrary to *LRFD Specifications,* the *Standard Specifications* indicate that this requirement must be satisfied only at the critical sections.

9.3.11 SHEAR DESIGN

[STD Art. 9.20]

Pretensioned members subject to shear should be designed so that:

$V_u \leq \phi\,(V_c + V_s)$ [STD Eq. 9-26]

where

V_u = the factored shear force at the section considered

V_c = the nominal shear strength provided by concrete

V_s = the nominal shear strength provided by web reinforcement

ϕ = strength reduction factor for shear = 0.90 [STD Art. 9.14]

The critical section in pretensioned concrete beams is located at a distance h/2 from the face of the support, according to the *Standard Specifications,* Article 9.20.1.4. In this example, the critical section for shear will be conservatively calculated from the centerline of support. The width of the bearing has not yet been determined, it will be conservatively assumed to be zero. Therefore, the detailed calculations are shown here for the section at (h_c/2= 80/2 = 40 in.) from the centerline of support. The following calculations demonstrate how to compute V_{ci} and V_{cw} at this location.

Compute V_{ci} :

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$ [STD Eq. 9-27]

where

b' = width of web of a flanged member = 6.00 in.

f'_c = compressive strength of beam concrete at 28 days = 6,500 psi

V_d = total dead load at the section under consideration, (from **Table 9.3.4.1.2-1**)

= $V_g + V_S + V_b + V_{ws}$ = 45.3 + 52.2 + 5.7 + 11.3 = 114.5 kips

V_{LL+I} = unfactored shear force at section due to live load + impact = 63.6 kips

M_d = bending moment at section due to unfactored dead load

= 155.4 + 179.3 + 19.4 + 38.9 = 393.0 ft-kips

M_{LL+I} = live load bending moment plus impact = 211.5 ft-kips

V_u = factored shear force at the section

= $1.3(V_d + 1.67V_{LL+I})$ = 1.3[114.5 + 1.67 (63.6)] = 286.9 kips

M_u = factored bending moment at the section

= $1.3(M_d + 1.67M_{LL+I})$ = 1.3[393.0 + 1.67(211.5)] = 970.1 ft-kips

V_{mu} = factored shear force occurring simultaneously with M_u. Conservatively, use the maximum shear load occurring at this section.

= 1.3[114.5 + 1.67(63.6)] = 286.9 kips

M_{max} = maximum factored moment at section due to externally applied loads

= $M_u - M_d$ = 970.1 – 393.0 = 577.1 ft-kips

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

= $V_{mu} - V_d$ = 286.9 – 114.5 = 172.4 kips

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses), at extreme fiber of section where tensile stress is caused by externally applied loads.

The beam at this section is under positive flexure. Thus, f_{pe} should be evaluated at the bottom of the beam. Thus,

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}e}{S_b}$$

Compute eccentricity of the strands at $h_c/2$:

Center of gravity of 10 harped strands from the top of the beam:

$$6\text{ in.} + \frac{(72\text{ in.} - 12\text{ in.} - 6\text{ in.})}{48.5\text{ ft}}(3.33\text{ ft} + 0.5\text{ ft}) = 10.26\text{ in.}$$

Center of gravity of the 34 straight strands from the bottom of the beam = 4.00 in.

Center of gravity of all strands from the bottom of the beam at h/2:

$$\frac{10(72 - 10.26) + 34(4)}{44} = 17.12\text{ in.}$$

Therefore, the eccentricity of strand at h/2 = 36.60 – 17.12 = 19.48 in.

P_{se} = 1,023.9 kips

$$f_{pe} = \frac{1{,}023.9}{767} + \frac{1{,}023.9(19.48)}{14{,}915} = 1.335 + 1.337 = 2.672\text{ ksi}$$

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \left[\frac{(155.4 + 179.3)(12)}{14{,}915} + \frac{(19.4 + 38.9)(12)}{20{,}090}\right] = 0.304\text{ ksi}$$

M_{cr} = moment causing flexural cracking of section due to externally applied loads (after dead load)

$$= (6\sqrt{f'_c} + f_{pe} - f_d)S_{bc} \qquad \text{[STD Eq. 9-28]}$$

$$= \left(\frac{6\sqrt{6{,}500}}{1{,}000} + 2.673 - 0.304\right)\frac{20{,}090}{12} = 4{,}776\text{ ft-kips}$$

d = distance from extreme compressive fiber to centroid of pretensioned reinforcement. But d need not be taken less than 0.8 h_c = 64.00 in. [STD Art. 9.20.2.2]. The center of gravity of the pretensioned reinforcement is located at 17.12 in. from the bottom of the beam.

= 80 – 17.12 = 62.88 in. < 64.00 in.

Therefore, use d = 64.00 in.

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad \text{[STD Eq. 9-27]}$$

$$= \frac{0.6\sqrt{6{,}500}(6.00)(64.00)}{1{,}000} + 114.5 + \frac{172.4(4{,}776)}{577.1} = 1{,}559.8\text{ kips}$$

This value should not be less than:

Minimum $V_{ci} = 1.7\sqrt{f'_c}\, b'd$ [STD Art. 9.20.2.2]

$$= \frac{1.7\sqrt{6{,}500}\,(6.00)(64.00)}{1{,}000} = 52.6 \text{ kips} < V_{ci} = 1{,}559.8 \text{ kips} \quad \text{O.K.}$$

Compute V_{cw}: [STD Art 9.20.2.3]

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3\, f_{pc}\right) b'd + V_p$$ [STD Eq. 9-29]

where

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross-section resisting externally applied loads. For a non composite section

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se}e(y_{bc} - y_b)}{I} + \frac{(M_g + M_s)(y_{bc} - y_b)}{I}$$

$$= \frac{1{,}023.9}{767} - \frac{1{,}023.9(19.48)(54.77 - 36.60)}{545{,}894} + \frac{334.7(12)(54.77 - 36.60)}{545{,}894}$$

$= 1.335 - 0.664 + 0.134 = 0.805$ ksi

V_p = vertical component of prestress force for harped strands

P_{se} = the effective prestress force for the harped strands

$= 10(0.153)(152.1) = 232.7$ kips

$V_p = P_{se} \sin \psi$ (see Section 9.3.7.5 for calculations of ψ)

$= 232.7\sin(5.30°) = 21.5$ kips

$$V_{cw} = \left(\frac{3.5\sqrt{6{,}500}}{1{,}000} + 0.3(0.805)\right)(6.00)(64.00) + 21.5 = 222.6 \text{ kips}$$

The allowable nominal shear strength provided by concrete should be the lesser of V_{ci} (1,559.8 kips) and V_{cw} (222.6 kips). V_{cw} governs, so:

$V_c = 222.6$ kips

$V_u < \phi(V_c + V_s)$ (Eq. 9-26)

V_s = nominal shear strength provided by shear reinforcement

ϕ = strength reduction factor for shear = 0.90 [STD Art. 9.14]

$$\text{Required } V_s = \frac{V_u}{\phi} - V_c = \frac{286.9}{0.90} - 222.6 = 96.2 \text{ kips}$$

Calculate the maximum shear force that may be carried by reinforcement:

Maximum $V_s = 8\sqrt{f'_c}\, b'd$ [STD Art. 9.20.3.1]

$$= 8\sqrt{6{,}500}\,\frac{(6.00)(64.00)}{1{,}000} = 247.7 \text{ kips} > \text{required } V_s = 96.2 \text{ kips} \quad \text{O.K.}$$

Compute area of shear steel required [STD Art. 9.20.3.1]

$$V_s = \frac{A_v f_y d}{s} \quad \text{[STD Eq. 9-30]}$$

Solving for A_v:

$$A_v = \frac{V_s\, s}{f_y\, d}$$

where

A_v = area of web reinforcement, in.^{2}

s = longitudinal spacing of the web reinforcement, in.

Set s = 12 in. to have units of in.^{2}/ft for A_v.

$$A_v = \frac{(96.2)(12)}{(60)(64.00)} = 0.301 \text{ in.}^2/\text{ft}$$

Minimum shear reinforcement [STD Art. 9.20.3.3]

$A_{v\text{-}min}$ = minimum area of web reinforcement

$$A_{v\text{-}min} = \frac{50b's}{f_y} = \frac{50(6)(12)}{60{,}000} = 0.06 \text{ in.}^2/\text{ft} \quad \text{[STD Eq. 9-31]}$$

The required shear reinforcement is the maximum of A_v (0.301 in.^{2}/ft) and $A_{v\text{-}min}$ (0.06 in.^{2}/ft).

Maximum spacing of web reinforcement is 0.75 h_c or 24 in., unless V_s = 96.2 kips >

$$4\sqrt{f_c'}\, b'd = \frac{4\sqrt{6{,}500}\,(6.00)(64.00)}{1{,}000} = 123.8 \text{ kips} \quad \text{[STD Art. 9.20.3.2]}$$

Since V_s is less than the limit,

Maximum spacing = 0.75 h = 0.75 (72 + 7.5 + 0.5) = 60 in.

or = 24.00 in.

Therefore, maximum s = 24 in.

Use # 4, two-legged stirrups at 12 in. spacing (A_v = 0.40 in.^{2}/ft)

Note that the above calculations need to be repeated at regular intervals along the entire length of the beam to determine the area and spacing of shear reinforcement.

9.3.12 HORIZONTAL SHEAR DESIGN

[STD Art. 9.20.4]

The computation will be carried out for the section at a distance of h_c/2 from the centerline of support.

V_u = 286.9 kips

$$V_u \le \phi V_{nh} \quad \text{[STD Eq. 9-31a]}$$

where V_{nh} = nominal horizontal shear strength, kips

$$V_{nh} \ge \frac{V_u}{\phi} = \frac{286.9}{0.9} = 318.8 \text{ kips}$$

Case (a & b): Contact surface is roughened, or when minimum ties are used

[STD Art. 9.20.4.3]

Allowable shear force:

$V_{nh} = 80b_v d$

where

b_v = width of cross-section at the contact surface being investigated for horizontal shear = 42.00 in.

d = distance from extreme compressive fiber to centroid of the pretensioning force = 62.88 in.

Note: The full 'd' is used because the minimum value for d of 0.8 h_c does not apply to this calculation.

$$V_{nh} = \frac{80(42.00)(62.88)}{1{,}000} = 211.3 \text{ kips} < 318.8 \text{ kips} \quad \text{N.G.}$$

Case (c): Minimum ties provided, and contact surface roughened [STD Art. 9.20.4.3]

Allowable shear force:

$V_{nh} = 350b_v d$

$$= \frac{350(42.00)(62.88)}{1{,}000} = 924.3 \text{ kips} > 318.8 \text{ kips} \quad \text{O.K.}$$

Determine required stirrups for horizontal shear: [STD Art. 9.20.4.5]

$$\text{minimum } A_{vh} = 50\frac{b_v s}{f_y} = 50\frac{42(12)}{60{,}000} = 0.42 \text{ in.}^2/\text{ft}$$

The required minimum horizontal shear reinforcement, A_{vh} = 0.42 in.^{2}/ft is approximately equal to the vertical shear reinforcement provided, A_v = 0.40 in.^{2}/ft. O.K.

Maximum spacing = 4b = 4(6) = 24.00 in [STD Art. 9.20.4.5.a]

or = 24.00 in.

Therefore, maximum s = 24.00 in. > the provided s = 12 in.

9.3.13 PRETENSIONED ANCHORAGE ZONE

[STD Art. 9.22]

9.3.13.1 Minimum Vertical Reinforcement

In a pretensioned beam, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4% of the total pretensioning force, must be placed within the distance of d/4 of the beam end. [STD Art. 9.22.1]

Minimum stirrups at the each end of the beam:

P_s = prestress force before initial losses = 44(0.153)[(0.75)(270)] = 1,363.2 kips

4% P_s = 0.04(1,363.2) = 54.5 kips

$$\text{Required } A_v = \frac{54.5}{20} = 2.73 \text{ in.}^2$$

$$\frac{d}{4} = \frac{62.88}{4} = 15.75 \text{ in.}$$

Use 5 pairs of #5 @ 3 in. spacing at each end of the beam (A_v = 3.10 in.^{2})

9.3.13.2 Confinement Reinforcement

[STD Art. 9.22.2]

Provide nominal reinforcement to enclose the pretensioning steel for a distance from the end of the beam equal to the depth of the beam.

9.3.14 DEFLECTION AND CAMBER

9.3.14.1 Deflection Due to Pretensioning Force at Transfer

For harped strands:

$$\Delta_p = \frac{P_{si}}{E_{ci} I}\left(\frac{e_c L^2}{8} - \frac{e' a^2}{6}\right)$$

where

Δ_p = camber due prestress force at transfer, in.

P_{si} = total pretensioning force = 1,242.1 kips

I = moment of inertia of non-composite section = 545,894 in.^{4}

L = overall beam length = 121.0 ft

E_{ci} = modulus of elasticity of the beam concrete at release = 4,496 ksi

e_c = eccentricity of pretensioning force at the midspan = 30.78 in.

e' = difference between eccentricity of pretensioning steel at midspan and end = $e_c - e_e$ = 30.78 – (36.60 – 18.09) = 12.27 in.

a = distance from the end of beam to harp point. = 48.50 ft

$$\Delta_p = \frac{1{,}242.1}{(4{,}496)(545{,}894)}\left(\frac{(30.78)[(121)(12)]^2}{8} - \frac{(12.27)[(48.5)(12)]^2}{6}\right) = 3.75 \text{ in.}\uparrow$$

9.3.14.2 Deflection Due to Beam Self-Weight

$$\Delta_{beam} = \frac{5 w_g L^4}{384 E_{ci} I}$$

where w_g = beam weight = 0.799 kips/ft

Deflection due to beam self-weight at transfer (L = 121 ft):

$$\Delta_{beam} = \frac{5(0.799/12)[(121)(12)]^4}{(384)(4{,}496)(545{,}894)} = 1.57 \text{ in.}\downarrow$$

Deflection due to beam self-weight used to compute deflection at erection (L = 120 ft):

$$\Delta_{beam} = \frac{5(0.799/12)[(120)(12)]^4}{(384)(4{,}496)(545{,}894)} = 1.52 \text{ in.}\downarrow$$

9.3.14.3 Total Initial Deflection

Total deflection at transfer: 3.75 − 1.57 = 2.18 in.↑

Total deflection at erection, using PCI multipliers (see PCI Design Handbook)

$$1.80(3.75) - 1.85(1.520) = 3.94 \text{ in.}\uparrow$$

PCI multipliers used to calculate long-term deflection in building products have not proven to be accurate for bridge construction. Therefore, it is recommended that the designer follow the guidelines of the owner agency for whom the bridge is being designed. Frequently, no additional multipliers are used.

9.3.14.4 Deflection Due to Slab and Haunch Weights

$$\Delta_{slab} = \frac{5 w_s L^4}{384 E_c I}$$

where

w_s = slab weight + haunch weight = 0.922 kips/ft

E_c = modulus of elasticity of the beam concrete at service = 4,888 ksi

$$\Delta_{slab} = \frac{5(0.922/12)[(120)(12)]^4}{(384)(4{,}888)(545{,}894)} = 1.61 \text{ in.}\downarrow$$

9.3.14.5 Deflection Due to Barrier and Wearing Surface Weights

Since these loads are applied to the structure in its final location, L is the design span = 120 ft.

$$\Delta_{SDL} = \frac{5(w_b + w_{ws})L^4}{384 E_c I_c}$$

where

w_b = weight of barriers = 0.100 kips/ft

w_{ws} = weight of future wearing surface = 0.200 kips/ft

I_c = moment of inertia of composite section = 1,100,320 in.^{4}

$$\Delta_{SDL} = \frac{5\left(\dfrac{0.100 + 0.200}{12}\right)[(120)(12)]^4}{(384)(4{,}888)(1{,}100{,}320)} = 0.26 \text{ in.}\downarrow$$

9.3.14.6 Deflection Due to Live Load and Impact

Live load deflection limit (optional) = L/800

$$\Delta_{max} = 120(12)/800 = 1.80 \text{ in.}$$

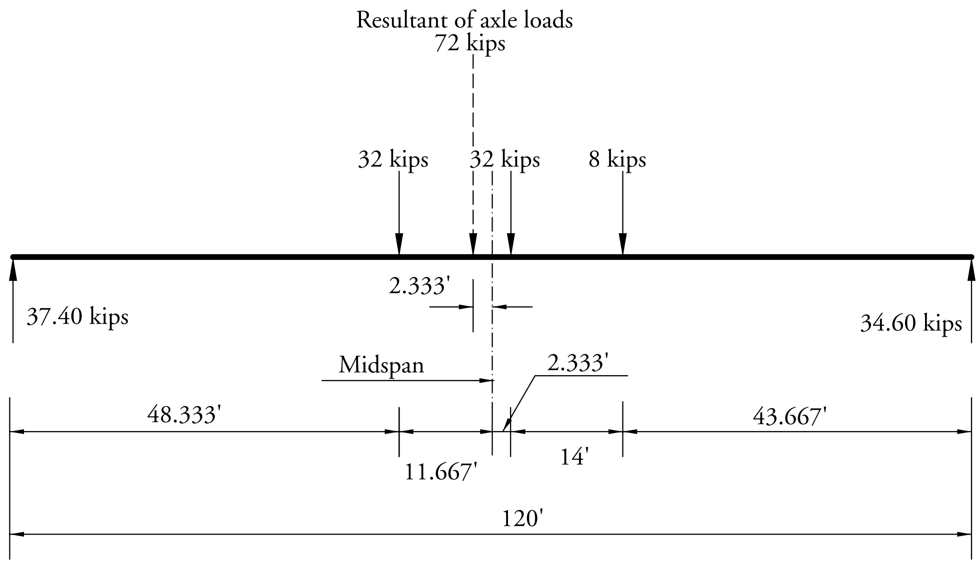
Some state DOTs consider that all beams act together in resisting deflection due to live load and impact. Using that assumption, the distribution factor for deflection is:

$$\frac{\text{number of lanes}}{\text{number of beams}} = \frac{4}{6} = 0.667 \text{ lanes / beam}$$

For all the design examples, live load deflection is conservatively computed based on the distribution factor used to calculate bending stresses.

To derive maximum moment and deflection at midspan due to the truck load, let the centerline of the beam coincide with the midpoint of the distance between the inner 32-kip axle and the resultant of the truck load (as shown in the **Figure 9.3.14.6-1**).

Figure 9.3.14.6-1
Design Truck Axle Load Positions on the Span for Maximum Moment



Beam analysis shows live load deflection at midspan is 0.80 in./lane.

$\Delta_{LL+I} = 0.80(1 + I)(DF_m)$

$\Delta_{LL+I} = 0.80(1.204)(0.818) = 0.79$ in. < 1.80 in. O.K.

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC = dead load of structural components and non structural attachments [LRFD]

DFD = distribution factor for deflection

DFM = distribution factor for bending moment

DF_m = live load distribution factor for moment

DFV = distribution factor for shear force

DW = load of wearing surfaces and utilities [LRFD]

d = distance from extreme compressive fiber to centroid of the pretensioning force [STD]

d_b = nominal strand diameter [LRFD]

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar [STD], [LRFD]

d_e = distance from exterior web of exterior beam and the interior side of curb or traffic barrier [LRFD]

d_e = effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement [LRFD]

d_p = distance from extreme compression fiber to the centroid of the pretensioning tendons [LRFD]

d_v = effective shear depth [LRFD]

E = width of slab over which a wheel load is distributed [STD]

E_c = modulus of elasticity of concrete [STD]

E_c = modulus of elasticity of concrete [LRFD]

E_{ci} = modulus of elasticity of the beam concrete at transfer

E_p = modulus of elasticity of pretensioning tendons [LRFD]

ES = loss of pretension due to elastic shortening [STD]

E_s = modulus of elasticity of pretensioning reinforcement [STD]

E_s = modulus of elasticity of reinforcing bars [LRFD]

e = eccentricity of the strands at h/2

e = eccentricity of strands at transfer length

e' = difference between eccentricity of pretensioning steel at midspan and end span

e_c = eccentricity of the strand at the midspan

e_e = eccentricity of pretensioning force at end of beam

e_g = distance between the centers of gravity of the beam and the slab [LRFD]

F_b = allowable tensile stress in the precompressed tensile zone at service loads

F_{pi} = total force in strands before release

F_ε = reduction factor [LRFD]

f_b = concrete stress at the bottom fiber of the beam

f'_c = specified concrete strength at 28 days [STD]

f'_c = specified compressive strength at 28 days [LRFD]

f_{cdp} = change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp} [LRFD]

f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied [STD]

f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer [STD]

f'_{ci} = concrete strength at release [STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]
f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]
f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pb} = compressive stress at bottom fiber of the beam due to prestress force
f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]
f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]
f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pe} = effective stress in the pretensioning steel after losses [LRFD]
f_{pi} = initial stress immediately before transfer
f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]
f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]
f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]
f_{pu} = specified tensile strength of pretensioning steel [LRFD]
f_{py} = yield strength of pretensioning steel [LRFD]
f_r = the modulus of rupture of concrete [STD]
f_r = modulus of rupture of concrete [LRFD]
f_s = allowable stress in steel
f'_s = ultimate stress of pretensioning reinforcement [STD]
f_{se} = effective final pretension stress
f_{si} = effective initial pretension stress
f^*_{su} = average stress in pretensioning steel at ultimate load [STD]
f_t = concrete stress at top fiber of the beam for the non-composite section
f_{tc} = concrete stress at top fiber of the slab for the composite section
f_{tg} = concrete stress at top fiber of the beam for the composite section
f_y = yield strength of reinforcing bars [STD]
f_y = specified minimum yield strength of reinforcing bars [LRFD]
f_y = yield stress of pretensioning reinforcement [STD]
f'_y = specified minimum yield strength of compression reinforcement [LRFD]
f_{yh} = specified yield strength of transverse reinforcement [LRFD]
H = average annual ambient mean relative humidity, percent [LRFD]
H = height of wall [LRFD]
h = overall depth of precast beam [STD]
h = overall depth of a member [LRFD]

h_c = total height of composite section
h_f = compression flange depth [LRFD]
I = moment of inertia about the centroid of the non-composite precast beam [STD]
I = moment of inertia about the centroid of the non-composite precast beam [LRFD]
I = impact fraction (maximum 30%) [STD]
I_c = moment of inertia of composite section
IM = dynamic load allowance [LRFD]
J = St. Venant torsional constant
K = longitudinal stiffness parameter [STD]
K_g = longitudinal stiffness parameter [LRFD]
k = factor used in calculation of distribution factor for multi-beam bridges [LRFD]
k = factor used in calculation of average stress in pretensioning steel for Strength Limit State
L = live load [STD]
L = length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment [STD]
L = overall beam length or design span
L = span length measured parallel to longitudinal beams [STD]
L = span length [LRFD]
L_c = critical length of yield line failure pattern [LRFD]
LL = vehicular live load [LRFD]
ℓ_d = development length [LRFD]
ℓ_x = length required to fully develop the strand measured from the end of the strand
M_a = negative moment at the end of the span being considered
M_b = negative moment at the end of the span being considered
M_b = unfactored bending moment due to barrier weight
M_c = flexural resistance of cantilevered wall [LRFD]
M_{CIP} = unfactored bending moment due to cast-in-place topping slab
M_{const} = unfactored bending moment due to construction load
M_{col} = bending moment due to horizontal collision force
M_{cr} = moment causing flexural cracking at section due to externally applied loads (after dead load) [STD]
M_{cr} = cracking moment [LRFD]
M^*_{cr} = cracking moment [STD]
M_D = unfactored bending moment due to diaphragm weight
M_d = bending moment at section due to unfactored dead load
$M_{d/nc}$ = moment due to non-composite dead loads [STD]
M_f = unfactored bending moment due to fatigue truck per beam
M_g = unfactored bending moment due to beam self-weight
M_{LL} = unfactored bending moment due to lane load per beam
M_{LL+I} = unfactored bending moment due to live load + impact
M_{LL+I} = unfactored bending moment due to design vehicular load
M_{LT} = unfactored bending moment due to truck load with dynamic allowance per beam

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	$= \dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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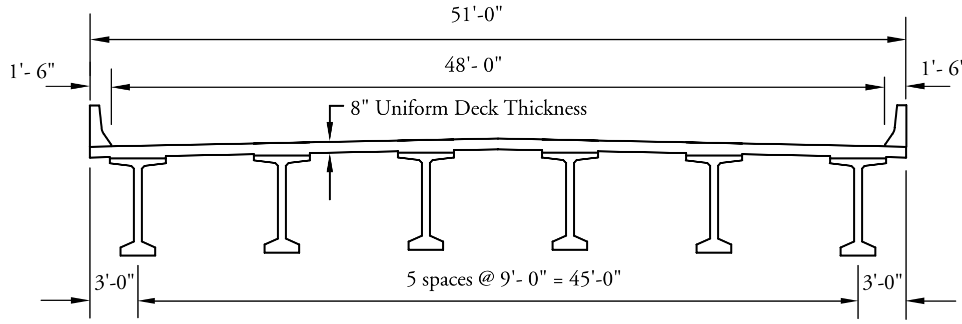
Bulb-Tee (BT-72), Single Span, Composite Deck, LRFD Specifications

9.4.1 INTRODUCTION

This design example demonstrates the design of a 120-ft single span AASHTO-PCI bulb-tee beam bridge with no skew. This example illustrates in detail the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection due to prestress, dead loads and live load. The superstructure consists of six beams spaced at 9'- 0" centers, as shown in **Figure 9.4.1-1.** Beams are designed to act compositely with the 8-in. cast-in-place concrete deck to resist all superimposed dead loads, live loads and impact. A 1/2 in. wearing surface is considered to be an integral part of the 8-in. deck. Design live load is HL-93. The design is accomplished in accordance with the *AASHTO LRFD Bridge Design Specifications*, 2nd Edition, 1998, and including through the 2003 Interim Revisions.

Figure 9.4.1-1
Bridge Cross-Section



9.4.2 MATERIALS

Cast-in-place slab: Actual thickness, t_s = 8.0 in.

Structural thickness = 7.5 in.

Note that a 1/2 in. wearing surface is considered to be an integral part of the 8-in. deck.

Concrete strength at 28 days, f'_c = 4.0 ksi

Precast beams: AASHTO-PCI bulb-tee as shown in **Figure 9.4.2-1**

Concrete strength at transfer, f'_{ci} = 5.8 ksi

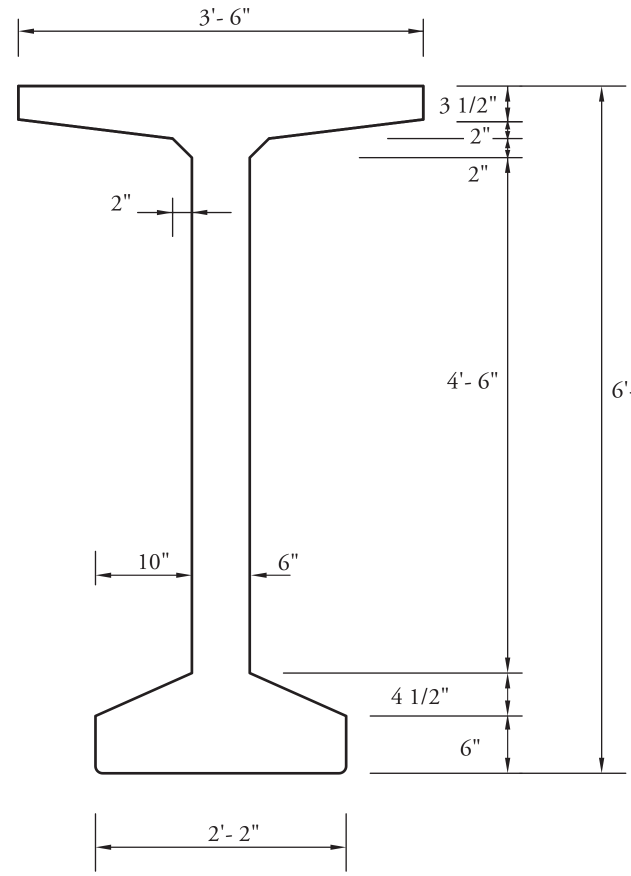
Concrete strength at 28 days, f'_c = 6.5 ksi

Concrete unit weight, w_c = 0.150 kcf

Overall beam length = 121.0 ft

Design span = 120.0 ft

Figure 9.4.2-1
AASHTO-PCI BT-72
Dimensions



Prestressing strands: 1/2 in. dia., seven-wire, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

- before transfer, $f_{pi} \leq 0.75f_{pu}$ = 202.5 ksi
- at service limit state (after all losses) $f_{pe} \leq 0.80f_{py}$ = 194.4 ksi

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Reinforcing bars: Yield strength, f_y = 60 ksi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Future wearing surface: 2 in. additional concrete, unit weight = 0.150 kcf

New Jersey-type barrier: Unit weight = 0.300 kip/ft/side

9.4.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

9.4.3.1 Non-Composite Section

A = area of cross-section of beam = 767 in.^{2}

h = overall depth of beam = 72 in.

I = moment of inertia about the centroid of the non-composite precast beam = 545,894 in.^{4}

y_b = distance from centroid to extreme bottom fiber of the non-composite precast beam = 36.60 in.

y_t = distance from centroid to extreme top fiber of the non-composite precast beam = 35.40 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 14,915 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 15,421 in.^{3}

Wt. = 0.799 kip/ft

[LRFD Eq. 5.4.2.4-1]

E_c = modulus of elasticity, ksi = $33{,}000(w_c)^{1.5}\sqrt{f_c'}$

where

w_c = unit weight of concrete, kcf = 0.150 kcf

LRFD Article C5.4.2.4 indicates that the unit weight of normal weight concrete is 0.145 kcf. However, precast concrete mixes typically have a relatively low water/cement ratio and high density. Therefore, a unit weight of 0.150 kcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f_c' = specified strength of concrete, ksi

Therefore, the modulus of elasticity for:

cast-in-place slab, $E_c = 33{,}000(0.150)^{1.5}\sqrt{4.0} = 3{,}834$ ksi

precast beam at transfer, $E_{ci} = 33{,}000(0.150)^{1.5}\sqrt{5.80} = 4{,}617$ ksi

precast beam at service loads, $E_c = 33{,}000(0.150)^{1.5}\sqrt{6.50} = 4{,}888$ ksi

9.4.3.2 Composite Section

9.4.3.2.1 Effective Flange Width

[LRFD Art. 4.6.2.6.1]

Effective flange width shall be the lesser of:

(1/4) span = (120)(12/4) = 360 in.;

$12t_s$ plus greater of web thickness or 1/2 beam top flange width

= (12 x 7.5) + (0.5 x 42) = 111 in.; or,

average spacing between beams = (9 x 12) = 108 in.

Therefore, the effective flange width is = 108 in.

9.4.3.2.2 Modular Ratio between Slab and Beam Materials

Modular ratio between slab and beam materials, $n = \dfrac{E_c(\text{slab})}{E_c(\text{beam})} = \dfrac{3{,}834}{4{,}888} = 0.7845$

9.4.3.2.3 Transformed Section Properties

Transformed flange width = n(Effective flange width) = (0.7845)(108) = 84.73 in.

Transformed flange area = n(Effective flange width)(t_s)

= (0.7845)(108)(7.5) = 635.45 in.^{2}

Note: Only the structural thickness of the deck, 7.5 in., is considered.

Due to camber of the precast, prestressed beam, a minimum haunch thickness of 1/2 in. at midspan is considered in the structural properties of the composite section. Also, the width of haunch must be transformed.

Transformed width of haunch = (0.7845)(42) = 32.95 in.

Transformed area of haunch = (0.7845)(42)(0.5) = 16.47 in.^{2}

Note that the haunch should only be considered to contribute to section properties if it is required to be provided in the completed structure. Therefore, some designers neglect its contribution to the section properties.

Figure 9.4.3.2.3-1 Dimensions of the Composite Section.

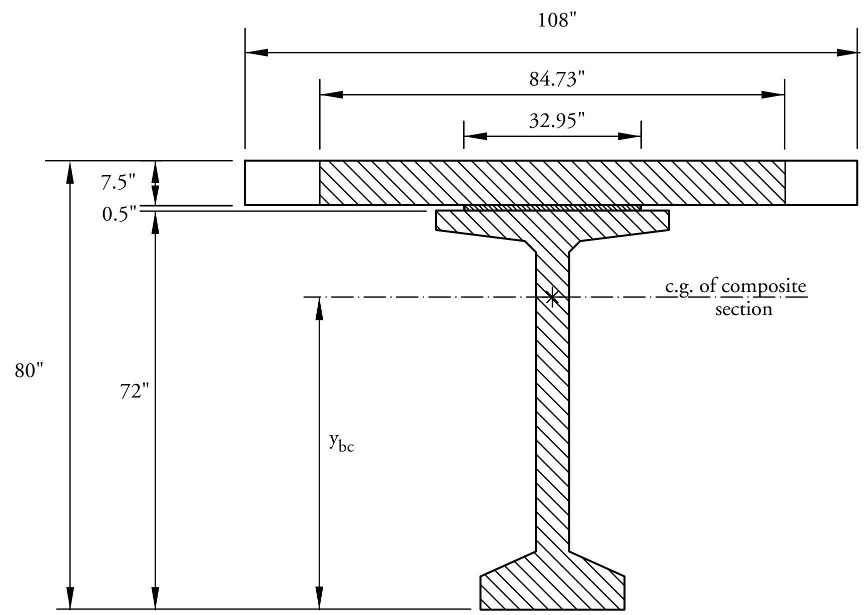


Table 9.4.3.2.3-1 Properties of Composite Section

	Area, in.^{2}	y_b, in.	Ay_b, in.^{3}	$A(y_{bc}-y_b)^2$, in.^{4}	I, in.^{4}	$I + A(Y_b-y_b)^2$, in.^{4}
Beam	767.00	36.60	28,072.20	253,224.21	545,894.00	799,118.21
Haunch	16.47	72.25	1,189.96	5,032.42	0.34	5,032.76
Deck	634.45	76.25	448,453.06	293,190.53	2,978.79	296,169.32
Σ	1,418.92		77,715.22			1,100,320.29

A_c = total area of the composite section = 1,419 in.^{2}

h_c = overall depth of the composite section = 80 in.

I_c = moment of inertia of the composite section = 1,100,320 in.^{4}

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = 77,715/1,419 = 54.77 in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 72 − 54.77 = 17.23 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the deck = 80 − 54.77 = 25.23 in.

S_{bc} = composite section modulus for the extreme bottom fiber of the precast beam

$$= (I_c/y_{bc}) = \frac{1{,}100{,}320}{54.77} = 20{,}090 \text{ in.}^3$$

S_{tg} = composite section modulus for the top fiber of the precast beam

$$= (I_c/y_{tg}) = \frac{1{,}100{,}320}{17.23} = 63{,}861 \text{ in.}^3$$

S_{tc} = composite section modulus for extreme top fiber of the deck slab

$$= \left(\frac{1}{n}\right)\left(I_c / y_{tc}\right) = \left(\frac{1}{0.7845}\right)\left(\frac{1{,}100{,}320}{25.23}\right) = 55{,}592 \text{ in.}^3$$

9.4.4 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the beam and the weight of the deck and haunch act on the non-composite, simple-span structure, while the weight of barriers, future wearing surface, and live loads with impact act on the composite, simple-span structure. Refer to **Table 9.4.4-1** which follows Section 9.4.4 for a summary of unfactored values calculated below.

9.4.4.1 Shear Forces and Bending Moments Due to Dead Loads

9.4.4.1.1 Dead Loads

[LRFD Art. 3.3.2]

DC = Dead load of structural components and non-structural attachments

Dead loads acting on the non-composite structure:

Beam self-weight = 0.799 kip/ft

8 in. deck weight = (8/12 ft)(9 ft)(0.150 kcf) = 0.900 kip/ft

1/2 in. haunch weight = (0.5)(42/144)(0.150) = 0.022 kip/ft

Notes:

1. Actual deck thickness (8 in.) is used for computing dead load.
2. A 1/2 in. minimum haunch thickness is assumed in the computations of dead load. If a deeper haunch will be used because of final beam camber, the weight of the actual haunch should be used.
3. The weight of cross-diaphragms is ignored since most agencies are changing from cast-in-place concrete diaphragms to lightweight steel diaphragms.

Dead loads placed on the composite structure:

LRFD Article 4.6.2.2.1 states that permanent loads (curbs and future wearing surface) may be distributed uniformly among all beams if the following conditions are met:

- Width of the deck is constant O.K.
- Number of beams, N_b, is not less than four ($N_b = 4$) O.K.
- The roadway part of the overhang, $d_e \leq 3.0$ ft

$d_e = 3.0 - 1.5 = 1.5$ ft O.K.

- Curvature in plan is less than 4° (curvature = 0.0) O.K.
- Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1 O.K.

Since these criteria are satisfied, the barrier and wearing surface loads are equally distributed among the 6 beams.

Barrier weight = (2 barriers)(0.300 kip/ft)/(6 beams) = 0.100 kip/ft

DW = Dead load of future wearing surface

= (2/12)(0.15) = 0.025 ksf

= (0.025 ksf)(48.0 ft)/(6 beams) = 0.200 kip/ft

9.4.4.1.2 Unfactored Shear Forces and Bending Moments

For a simply supported beam with span (L) loaded with a uniformly distributed load (w), the shear force (V_x) and bending moment (M_x) at any distance (x) from the support are given by:

$V_x = w(0.5L - x)$ (Eq. 9.4.4.1.2-1)

$M_x = 0.5wx(L - x)$ (Eq. 9.4.4.1.2-2)

Using the above equations, values of shear forces and bending moments for a typical interior beam, under self-weight of beam, weight of slab and haunch, weight of barriers and future wearing surface are computed and shown in **Table 9.4.4-1**. For these calculations, the span length (L) is the design span, 120 ft. However, for calculations of stresses and deformation at the time prestress is released, the overall length of the precast member, 121 ft, is used as illustrated later in this example.

9.4.4.2 Shear Forces and Bending Moments Due to Live Loads

9.4.4.2.1 Live Loads

Design live load is HL-93 which consists of a combination of: [LRFD Art. 3.6.1.2.1]

1. Design truck or design tandem with dynamic allowance [LRFD Art. 3.6.1.2.2]

   The design truck is the same as the HS20 design truck specified by the *Standard Specifications*, [STD Art. 3.6.1.2.2]. The design tandem consists of a pair of 25.0-kip axles spaced at 4.0 ft apart. [LRFD Art. 3.6.1.2.3]

2. Design lane load of 0.64 kip/ft without dynamic allowance [LRFD Art. 3.6.1.2.4]

9.4.4.2.2 Live Load Distribution Factor for Typical Interior Beam

The live load bending moments and shear forces are determined by using the simplified distribution factor formulas, [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions must be met: [LRFD Art. 4.6.2.2.1]

- Width of slab is constant O.K.
- Number of beams, $N_b \geq 4$ ($N_b = 6$) O.K.
- Beams are parallel and of the same stiffness O.K.
- The roadway part of the overhang, $d_e \leq 3.0$ ft ($d_e = 1.5$ ft) O.K.
- Curvature is less than 4° [LRFD Table. 4.6.1.2.1-1] (Curvature = 0.0°) O.K.

For precast concrete I- or bulb-tee beams with cast-in-place concrete deck, the bridge type is (k). [LRFD Table 4.6.2.2.1-1]

The number of design lanes is computed as:

Number of design lanes = the integer part of the ratio of (w/12), where (w) is the clear roadway width, in ft, between the curbs [LRFD Art. 3.6.1.1.1]

From **Figure 9.4.1-1**, w = 48 ft

Number of design lanes = integer part of (48/12) = 4 lanes

9.4.4.2.2.1 Distribution Factor for Bending Moment

- For all limit states except fatigue limit state:

For two or more lanes loaded:

$$\text{DFM} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$ [LRFD Table 4.6.2.2.2b-1]

Provided that: $3.5 \le S \le 16$; $S = 9.0$ ft O.K.

$4.5 \le t_s \le 12$; $t_s = 7.5$ in. O.K.

$20 \le L \le 240$; $L = 120$ ft O.K.

$N_b \ge 4$; $N_b = 6$ O.K.

$10{,}000 \le K_g \le 7{,}000{,}000$ O.K. (see below)

where

DFM = distribution factor for moment for interior beam

S = beam spacing, ft

L = beam span, ft

t_s = depth of concrete slab, in.

K_g = longitudinal stiffness parameter, in.^{4} = $n(I + Ae_g^2)$ [LRFD Eq. 4.6.2.2.1-1]

where

n = modular ratio between beam and slab materials

$$= \frac{E_c(\text{beam})}{E_c(\text{beam})} = \frac{4{,}888}{3{,}834} = 1.2749$$

A = cross-sectional area of the beam (non-composite section), in.^{2}

I = moment of inertia of the beam (non-composite section), in.^{4}

e_g = distance between the centers of gravity of the beam and slab, in.

= (7.5/2 + 0.5 + 35.4) = 39.65 in.

Therefore,

K_g = 1.2749[545,894 + 767(39.65)2] = 2,233,257 in.^{4}

$$\text{DFM} = 0.075 + \left(\frac{9}{9.5}\right)^{0.6}\left(\frac{9}{120}\right)^{0.2}\left(\frac{2,233,257.6}{12.0(120)(7.5)^3}\right)^{0.1}$$

= 0.075 + (0.968)(0.596)(1.139) = 0.732 lanes/beam

For one design lane loaded:

$$\text{DFM} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$ [LRFD Table 4.6.2.2.2b-1]

$$= 0.06 + \left(\frac{9}{14}\right)^{0.4}\left(\frac{9}{120}\right)^{0.3}\left(\frac{2,233,257}{12.0(120)(7.5)^3}\right)^{0.1} = 0.499 \text{ lanes/beam}$$

Thus, the case of two or more lanes loaded controls and DFM = 0.732 lanes/beam.

- For fatigue limit state:

  The *LRFD Specifications*, Art. 3.4.1, states that for fatigue limit state, a single design truck should be used. However, live load distribution factors given in LRFD Article 4.6.2.2 take into consideration the multiple presence factor, m. LRFD Article 3.6.1.1.2 states that the multiple presence factor, m, for one design lane loaded is 1.2. Therefore, the distribution factor for one design lane loaded with the multiple presence factor removed, should be used. The distribution factor for fatigue limit state is: 0.499/1.2 = 0.416 lanes/beam.

9.4.4.2.2.2
Distribution Factor for Shear Force

For two or more lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 \qquad \text{[LRFD Table 4.6.2.2.3a-1]}$$

Provided that: $3.5 \le S \le 16$; $S = 9.0$ ft O.K.
$20 \le L \le 240$; $L = 120$ ft O.K.
$4.5 \le t_s \le 12$; $t_s = 7.5$ in. O.K.
$N_b \ge 4$; $N_b = 6$ O.K.

where

DFV = Distribution factor for shear for interior beam

S = Beam spacing, ft

Therefore, the distribution factor for shear force is:

$$DFV = 0.2 + \left(\frac{9}{12}\right) - \left(\frac{9}{35}\right)^2 = 0.884 \text{ lanes/beam}$$

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25.0}\right) = 0.36 + \left(\frac{9}{25.0}\right) = 0.720 \text{ lanes/beam} \quad \text{[LRFD Table 4.6.2.2.3a-1]}$$

Thus, the case of two or more lanes loaded controls and DFV = 0.884 lanes/beam.

9.4.4.2.3
Dynamic Allowance

IM = 33% [LRFD Table 3.6.2.1-1]

where IM = dynamic load allowance, applied to truck load only

9.4.4.2.4 Unfactored Shear Forces and Bending Moments

9.4.4.2.4.1 Due To Truck Load; V_{LT} and M_{LT}

- For all limit states except for fatigue limit state:

Shear force and bending moment envelopes on a per-lane-basis are calculated at tenth-points of the span using the equations given in Chapter 8 of this manual. However, this is generally done by means of commercially available computer software that has the ability to deal with moving loads. Therefore, truck load shear force and bending moments per beam are:

V_{LT} = (shear force per lane)(DFV)(1 + IM)
= (shear force per lane)(0.884)(1 + 0.33)
= (shear force per lane)(1.176) kips

M_{LT} = (bending moment per lane)(DFM)(1 + IM)
= (bending moment per lane)(0.732)(1 + 0.33)
= (bending moment per lane)(0.974) ft-kips

Values of V_{LT} and M_{LT} at different points are given in **Table 9.4.4-1**.

- For fatigue limit state:

Art. 3.6.1.4.1 in the *LRFD Specifications* states that the fatigue load is a single design truck which has the same axle weight used in all other limit states but with a constant spacing of 30.0 ft between the 32.0-kip axles. Bending moment envelope on a per-lane-basis is calculated using the equation given in Chapter 8 of this manual.

Therefore, bending moment of fatigue truck load is:

M_f = (bending moment per lane)(DFM)(1 + IM)
= (bending moment per lane)(0.416)(1 + 0.15)
= (bending moment per lane)(0.478) ft-kips

Values of M_f at different points are given in **Table 9.4.4-1**.

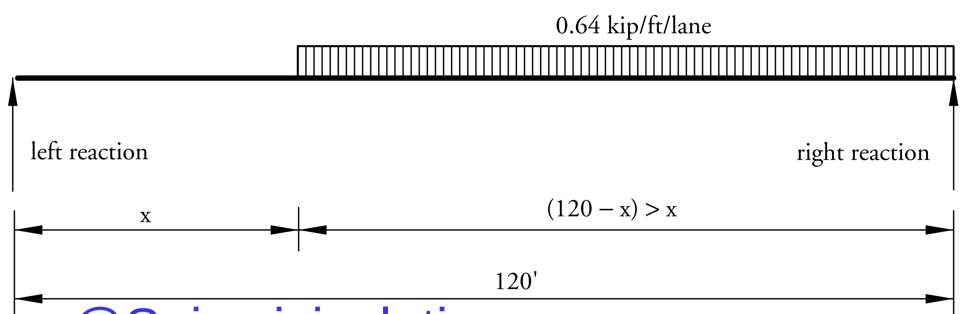
9.4.4.2.4.2 Due To Design Lane Load; V_{LL} And M_{LL}

To obtain the maximum shear force at a section located at a distance (x) from the left support under a uniformly distributed load of 0.64 kip/ft, load the member to the right of section under consideration as shown in **Figure 9.4.4.2.4.2-1**. Therefore, the maximum shear force per lane is:

$$V_x = \frac{0.32(L-x)^2}{L} \quad \text{for } x \le 0.5L \qquad \text{(Eq. 9.4.4.2.4.2-1)}$$

where V_x is in kip/lane and L and x are in ft.

Figure 9.4.4.2.4.2-1
Maximum Shear Force Due to Design Lane Load



To calculate the maximum bending moment at any section, use Eq. (9.4.4.1.2-2).

Lane load shear force and bending moment per typical interior beam are as follows:

V_{LL} = (lane load shear force)(DFV)

= (lane load shear force)(0.884) kips

For all limit states except for fatigue limit state:

M_{LL} = (lane load bending moment)(DFM)

= (lane load bending moment)(0.732) ft-kips

Note that the dynamic allowance is not applied to the design lane loading.

Values of shear forces and bending moments, V_{LL} and M_{LL}, are given in **Table 9.4.4-1.**

9.4.4.3 Load Combinations

[LRFD Art. 3.4]

Total factored load shall be taken as:

$Q = \eta \Sigma \gamma_i q_i$ [LRFD Eq. 3.4.1-1]

where

η = a factor relating to ductility, redundancy and operational importance (Here, η is considered to be 1.0) [LRFD Art. 1.3.2]

γ_i = load factors [LRFD Table 3.4.1-1]

q_i = specified loads

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

Service I: check compressive stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 1.00(LL + IM) [LRFD Table 3.4.1-1]

This load combination is the general combination for service limit state stress checks and applies to all conditions other than Service III.

Service III: check tensile stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 0.80(LL + IM) [LRFD Table 3.4.1-1]

This load combination is a special combination for service limit state stress checks that applies only to tension in prestressed concrete structures to control cracks.

Strength I: check ultimate strength: [LRFD Tables 3.4.1-1 and 2]

Maximum Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)

Minimum Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)

This load combination is the general load combination for strength limit state design.

Note: For simple-span bridges, the maximum load factors produce maximum effects. However, use minimum load factors for dead load (DC), and wearing surface (DW) when dead load and wearing surface stresses are opposite to those of live load.

Fatigue: check stress range in strands: [LRFD Table 3.4.1-1]

Q = 0.75(LL + IM)

This load combination is a special load combination to check the tensile stress range in the strands due to live load and dynamic allowance.

Table 9.4.4-1
Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Distance x	Section x/L	Beam weight		(Slab+Haunch) weight		Barrier weight		Wearing surface	
		Shear	Moment M_g	Shear	Moment M_s	Shear	Moment M_b	Shear	Moment M_{ws}
ft		kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips
0	0.0	47.9	0.0	55.3	0.0	6.0	0.0	12.0	0.0
*7.18	0.06	42.2	323.6	48.7	373.3	5.3	40.5	10.6	81.0
12	0.1	38.4	517.8	44.3	597.5	4.8	64.8	9.6	129.6
24	0.2	28.8	920.4	33.2	1,062.1	3.6	115.2	7.2	230.4
36	0.3	19.2	1,208.1	22.1	1,394.1	2.4	151.2	4.8	302.4
48	0.4	9.6	1,380.7	11.1	1,593.2	1.2	172.8	2.4	345.6
60	0.5	0.0	1,438.2	0.0	1,659.6	0.0	180.0	0.0	360.0

* Critical section for shear (see Section 9.4.11)

Distance x	Section x/L	Truck load with impact		Lane load		Fatigue truck with impact
		Shear V_{LT}	Moment M_{LT}	Shear V_{LL}	Moment M_{LL}	Moment M_f
ft		kips	ft-kips	kips	ft-kips	ft-kips
0	0.0	78.1	0.0	33.9	0.0	0.0
*7.18	0.06	72.9	433.9	30.0	189.7	165.0
12	0.1	69.6	691.6	27.5	303.6	309.2
24	0.2	61.1	1,215.0	21.7	539.7	535.8
36	0.3	52.7	1,570.2	16.6	708.3	692.7
48	0.4	44.2	1,778.9	12.2	809.5	776.2
60	0.5	35.7	1,830.2	8.5	843.3	776.9

* Critical section for shear (see Section 9.4.11)

9.4.5 ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber for load combination at Service III at the section of maximum moment or at the harp points. For estimating the number of strands, only the stresses at mid-span are considered.

9.4.5.1 Service Load Stresses at Midspan

Bottom tensile stress due to applied dead and live loads using load combination Service III is:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + (0.8)(M_{LT} + M_{LL})}{S_{bc}}$$

where

f_b = bottom tensile stress, ksi

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_s = unfactored bending moment due to slab and haunch weights, ft-kips

M_b = unfactored bending moment due to barrier weight, ft-kips

M_{ws} = unfactored bending moment due to future wearing surface, ft-kips

M_{LT} = unfactored bending moment due to truck load, ft-kips

M_{LL} = unfactored bending moment due to lane load, ft-kips

Using values of bending moments from **Table 9.4.4-1**, bottom tensile stress at midspan is:

$$f_{bc} = \frac{(1{,}438.2 + 1{,}659.6)}{14{,}915}(12) + \frac{(180 + 360) + (0.8)(1{,}830.3 + 843.3)}{20{,}090}(12)$$

$$= (2.492 + 1.600) = 4.092 \text{ ksi}$$

9.4.5.2 Stress Limits for Concrete

Tensile stress limit at service loads = $0.19\sqrt{f'_c}$ [LRFD Art. 5.9.4.2b]

where f'_c = specified 28-day concrete strength of beam, ksi

Tensile concrete stress limit in concrete = $0.19\sqrt{6.50} = -0.484$ ksi

9.4.5.3 Required Number of Strands

The required precompressive stress at the bottom fiber of the beam is the difference between bottom tensile stress due to the applied loads and the concrete tensile stress limit:

$f_{pb} = (4.092 - 0.484) = 3.608$ ksi

The location of the strand center of gravity at midspan, ranges from 5 to 15% of the beam depth, measured from the bottom of the beam. A value of 5% is appropriate for newer efficient sections like the bulb-tee beams and 15% for less efficient AASHTO standard shapes.

Assume the distance between the center of gravity of bottom strands and the bottom fiber of the beam:

$y_{bs} = 0.05h = 0.05(72) = 3.60$ in., use $y_{bs} = 4.0$ in.

Therefore, strand eccentricity at midspan, $e_c = (y_b - y_{bs}) = (36.6 - 4.0) = 32.6$ in.

If P_{pe} is the total prestressing force, the stress at the bottom fiber due to prestress is:

$$f_{pb} = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b}, \quad \text{or, } 3.608 = \frac{P_{pe}}{767} + \frac{P_{pe}(32.6)}{14{,}915}$$

Solving for P_{pe}, the required $P_{pe} = 1{,}034.4$ kips.

Final prestress force per strand = (area of strand)(f_{pi})(1 − losses, %)

where f_{pi} = initial stress before transfer, ksi (see Section 9.4.2) = 202.5 ksi

Assuming final loss of 25% of f_{pi}, the prestress force per strand after all losses

= (0.153)(202.5)(1 − 0.25) = 23.2 kips

Number of strands required = (1,034.4/23.2) = 44.6 strands

As an initial trial, (46) 1/2 in., 270 ksi strands were selected. The center of gravity of the 46 strands at midspan is 6.35 in. from the bottom of the concrete, which is higher than the assumed value, 4.0 in. Thus, a second iteration using the new value of strand eccentricity indicates that 48 strands are required. The strand pattern at midspan for the 48 strands is shown in **Figure 9.4.5.3-1**. Each available position is filled beginning with the bottom row.

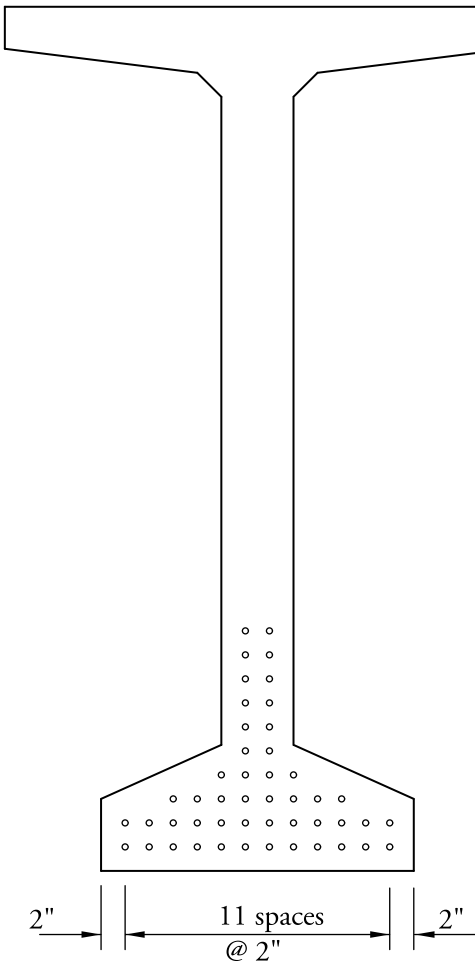
9.4.5.4 Strand Pattern

The distance between the center of gravity of bottom strands and the bottom concrete fiber of the beam is:

$$y_{bs} = [12(2) + 12(4) + 8(6) + 4(8) + 2(10) + 2(12) + 2(14) + 2(16) + 2(18) + 2(20)]/(48)$$
$$= 6.92 \text{ in.}$$

Strand eccentricity at midspan, $e_c = y_b - y_{bs} = 36.60 - 6.92 = 29.68$ in.

Figure 9.4.5.3-1
Assumed Strand Pattern at Midspan



No. Strands	Distance from bottom (in.)
2	20
2	18
2	16
2	14
2	12
2	10
4	8
8	6
12	4
12	2

9.4.6 PRESTRESS LOSSES

Total prestress loss:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ [LRFD Eq. 5.9.5.1-1]

where

Δf_{pES} = loss due to elastic shortening

Δf_{pSR} = loss due to shrinkage

Δf_{pCR} = loss due to creep

Δf_{pR2} = loss due to relaxation of steel after transfer

9.4.6.1 Elastic Shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Art. 5.9.5.2.3a]

where

E_p = modulus of elasticity of prestressing reinforcement = 28,500 ksi

E_{ci} = modulus of elasticity of beam at release = 4,617 ksi

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member at sections of maximum moment.

Force per strand at transfer = (area of strand)(prestress stress at transfer)

LRFD Article 5.9.5.2.3a states that f_{cgp} can be calculated on the basis of prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, common practice assumes the initial losses as a percentage of initial prestressing stress before release, f_{pi}. In both procedures, assumed initial losses should be checked and if different from the assumed value, a second iteration should be carried out. In this example, 9% f_{pi} initial loss is used.

Force per strand at transfer = (0.153)(202.5)(1 − 0.09) = 28.2 kips

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{M_g e_c}{I}$$

where

e_c = eccentricity of strands measured from the center of gravity of the precast beam at midspan

P_i = total prestressing force at release = (48 strands)(28.2) = 1,353.6 kips

M_g should be calculated based on the overall beam length of 121 ft. Since the elastic shortening loss is a part of the total loss, f_{cgp} will be conservatively computed based on M_g using the design span length of 120 ft.

$$f_{cgp} = \frac{1{,}353.6}{767} + \frac{(1{,}353.6)(29.68)^2}{545{,}894} - \frac{[(1{,}438.2)(12)](29.68)}{545{,}894}$$

$$= 1.765 + 2.184 - 0.938 = 3.011 \text{ ksi}$$

Therefore, loss due to elastic shortening:

$$\Delta f_{pES} = \frac{28{,}500}{4{,}617}(3.011) = 18.6 \text{ ksi}$$

$$\text{Percent actual loss due to elastic shortening} = \frac{18.6}{202.5}(100) = 9.2\%$$

Since the calculated loss of 9.2% is approximately equal to the initial loss assumption of 9%, a second iteration is not necessary. Note that this loss is equivalent to a stress after initial losses of 0.68 f_{pu}. This stress is lower than the estimate of 0.70 f_{pu}, provided in Article 5.9.5.2.3a. If the elastic shortening loss was calculated using a stress of 0.70 f_{pu}, a second iteration would be required to arrive at a steel stress of 0.68 f_{pu}.

9.4.6.2 Shrinkage

$\Delta f_{pSR} = (17 - 0.15H)$ [LRFD Eq. 5.9.5.4.2-1]

where H = relative humidity (assume 70%)

Relative humidity varies significantly from one area of the country to another, see Figure 5.4.2.3.3-1 in the *LRFD Specifications*.

$\Delta f_{pSR} = 17 - 0.15(70) = 6.5$ ksi

9.4.6.3 Creep of Concrete

$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

where Δf_{cdp} = change of stresses at center of gravity of prestressing due to permanent loads, except dead load acting at time the prestress force is applied calculated at the same section as f_{cgp}

$$= \frac{M_s e_c}{I} + \frac{(M_{ws} + M_b)(y_{bc} - y_{bs})}{I_c}$$

$$= \frac{(1{,}659.6)(12)(29.68)}{545{,}894} + \frac{(180 + 360)(12)(54.77 - 6.92)}{1{,}100{,}320}$$

$$= 1.083 + 0.282 = 1.365 \text{ ksi}$$

$\Delta f_{pCR} = 12(3.011) - 7(1.365) = 26.6$ ksi

9.4.6.4 Relaxation of Prestressing Strands

9.4.6.4.1 Relaxation before Transfer

Initial loss due to relaxation of prestressing steel is accounted for in the beam fabrication process. Therefore, loss due to relaxation of the prestressing steel prior to transfer is not be computed, i.e. $\Delta f_{pR1} = 0$. Recognizing this for pretensioned members, LRFD Article 5.9.5.1 allows the portion of the relaxation loss that occurs prior to transfer to be neglected in computing the final loss.

9.4.6.4.2 Relaxation after Transfer

For low-relaxation strands, loss due to relaxation after transfer: [LRFD Art. 5.9.5.4.4c]

$\Delta f_{pR2} = 30\%[20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$ [LRFD Art. 5.9.5.4.4c-1]

$= 0.3[20.0 - 0.4(18.6) - 0.2(6.5 + 26.6)] = 1.8$ ksi

9.4.6.5 Total Losses at Transfer

$\Delta f_{pi} = \Delta f_{pES} = 18.6$ ksi

Stress in tendons after transfer, $f_{pt} = f_{pi} - \Delta f_{pi} = (202.5 - 18.6) = 183.9$ ksi

Force per strand = (f_{pt})(area of strand) = 183.9(0.153) = 28.1 kips

Initial loss, % = (Total losses at transfer)/(f_{pi}) = 18.6/(202.5) = 9.2 %

The first estimation of loss at transfer, 9%, is very close to the actual computed initial loss of 9.2%. Thus, there is no need to perform a second iteration to refine the initial losses.

9.4.6.6 Total Losses at Service Loads

Total loss of prestress at service loads is:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} = 18.6 + 6.5 + 26.6 + 1.8 = 53.5$ ksi

Stress in tendons after all losses, $f_{pe} = f_{pi} - \Delta f_{pt} = 202.5 - 53.5 = 149.0$ ksi

Check prestressing stress limit at service limit state: [LRFD Table 5.9.3-1]

$f_{pe} \leq 0.8f_{py} = 0.8(243) = 194.4 \text{ ksi} > 149.0$ ksi O.K.

Force per strand = (f_{pe})(area of strand) = 149.0(0.153) = 22.8 kips

Therefore, the total prestressing force after all losses, $P_{pe} = 22.8(48) = 1{,}094.4$ kips

Final loss, % = (total losses at transfer)/(f_{pi}) = 53.5/(202.5) = 26.4%

9.4.7 STRESSES AT TRANSFER

Force per strand after initial losses = 28.1 kips

Therefore, the total prestressing force after transfer = 1,348.8 kips

9.4.7.1 Stress Limits for Concrete

[LRFD Art. 5.9.4]

Compression: $0.6f'_{ci} = 0.6(5.8) = +3.480$ ksi

where f'_{ci} = concrete strength at release = 5.80 ksi

Tension:

- without bonded reinforcement

$$0.0948\sqrt{f'_{ci}} \leq 0.200 \text{ ksi}; -0.0948\sqrt{5.8} = -0.228 \text{ ksi}$$

Therefore, − 0.200 ksi (Controls)

- with bonded auxiliary reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete

$$-0.22\sqrt{f'_{ci}} = -0.22\sqrt{5.8} = -0.530 \text{ ksi}$$

9.4.7.2 Stresses at Transfer Length Section

Stresses at this location need only be checked at release since this stage almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length = 60(strand diameter) = 60(0.5) = 30 in. = 2.5 ft [LRFD Art. 5.8.2.3]

Due to camber of the beam at release, the beam self-weight acts on the overall beam length, 121 ft. Therefore, values for bending moment given in **Table 9.4.4-1** cannot be used because they are based on the design span of 120 ft. Using statics, bending moment at transfer length due to beam self-weight is:

$0.5wx(L - x) = (0.5)(0.799)(2.5)(121 - 2.5) = 118.4$ ft-kips

Compute stress in the top of beam:

$$f_t = \frac{P_i}{A} - \frac{P_i e}{S_t} + \frac{M_g}{S_t} = \frac{1{,}348.8}{767} - \frac{(1{,}348.8)(29.68)}{15{,}421} + \frac{(118.4)(12)}{15{,}421}$$

$$= 1.759 - 2.605 + 0.092 = -0.754 \text{ ksi}$$

Tensile stress limit for concrete with bonded reinforcement: –0.530 ksi N.G.

Compute stress in the bottom of beam:

$$f_b = \frac{P_i}{A} + \frac{P_i e}{S_b} - \frac{M_g}{S_b} = \frac{1{,}348.8}{767} + \frac{(1{,}348.8)(29.68)}{14{,}915} - \frac{(118.4)(12)}{14{,}915}$$

$$= 1.759 + 2.684 - 0.095 = +4.348 \text{ ksi}$$

Compressive stress limit for concrete: + 3.480 ksi N.G.

Since stresses at the top and bottom exceed the stress limits, harp strands to satisfy the specified limits. Harp 12 strands at the 0.4L points, as shown in **Figures 9.4.7.2-1** and **9.4.7.2-2**.

Figure 9.4.7.2-1 Strand Pattern

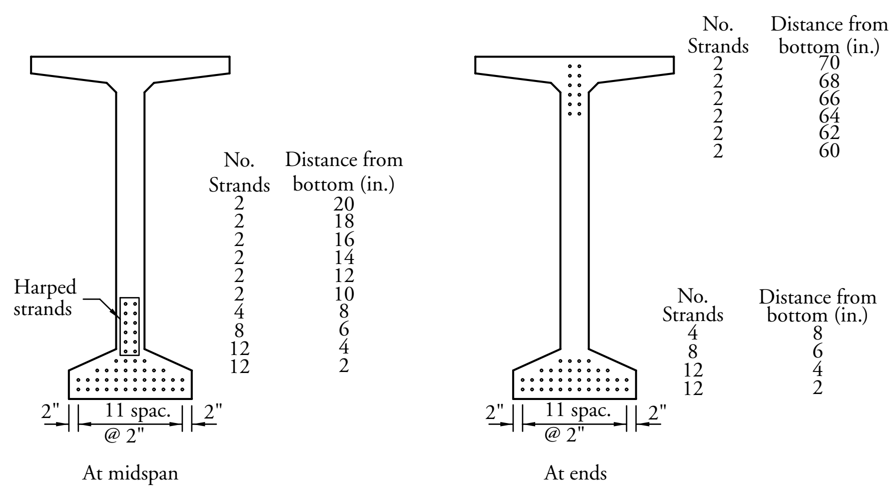
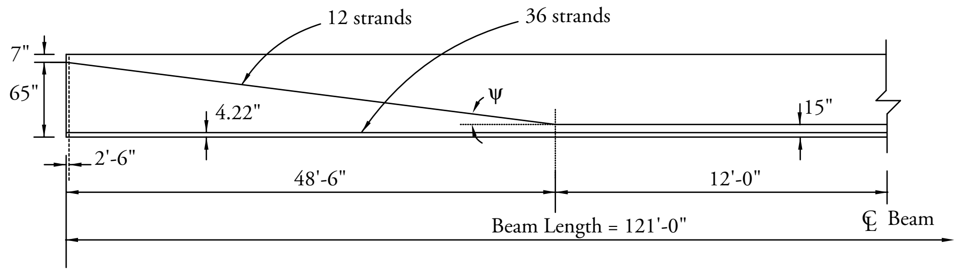


Figure 9.4.7.2-2
Longitudinal Strand Profile



Compute the center of gravity of the prestressing strands at the transfer length section using the harped pattern.

The distance between the center of gravity of the 12 harped strands at the end of the beam and the top fiber of the precast beam is:

$$\frac{2(2)+2(4)+2(6)+2(8)+2(10)+2(12)}{12} = 7.0 \text{ in.}$$

The distance between the center of gravity of the 12 harped stands at the harp point and the bottom fiber of the beam is:

$$\frac{2(10)+2(12)+2(14)+2(16)+2(18)+2(20)}{12} = 15.0 \text{ in.}$$

The distance between the center of gravity of the 12 harped strands and the top fiber of the beam at the transfer length section:

$$7 \text{ in.} + \frac{(72-15-7)\text{ in.}}{48.5 \text{ ft}}(2.5 \text{ ft}) = 9.58 \text{ in.}$$

The distance between the center of gravity of the bottom straight 36 strands and the extreme bottom fiber of the beam is:

$$\frac{12(2)+12(4)+8(6)+4(8)}{36} = 4.22 \text{ in.}$$

Therefore, the distance between the center of gravity of the total number of strands measured to the bottom of the precast beam at transfer length:

$$\frac{36(4.22)+12(72-9.58)}{48} = 18.77 \text{ in.}$$

Eccentricity of the strand group at transfer length, e, is: 36.6 – 18.77 = 17.83 in.

The center of gravity of all prestressing strand with respect to the extreme bottom fiber at the end of the beam, y_{bs}, is:

$$\frac{(36)(4.22)+12(72-7)}{48} = 19.42 \text{ in.}$$

Recompute the stresses at the transfer length section with harped strands:

Concrete stress in top of beam:

$$f_t = \frac{1{,}348.8}{767} - \frac{(1{,}348.8)(17.83)}{15{,}421} + \frac{(118.4)(12)}{15{,}421} = 1.759 - 1.560 + 0.092 = +0.291 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

Concrete stress in bottom of beam:

$$f_b = \frac{1,348.8}{767} + \frac{(1,348.8)(17.83)}{14,915} - \frac{(118.4)(12)}{14,915}$$

$$= 1.759 + 1.612 - 0.095 = +3.276 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

9.4.7.3
Stresses at Harp Points

The strand eccentricity at the harp points is the same as at midspan, e_c = 29.68 in.

Bending moment at the harp points (0.4L) due to the self-weight of the beam is:

(0.5)(0.799)(48.5)(121 – 48.5) = 1,404.7 ft-kips.

Therefore, top and bottom stresses are:

Concrete stress in top of beam:

$$f_t = \frac{1,348.8}{767} - \frac{(1,348.8)(29.68)}{15,421} + \frac{(1,404.7)(12)}{15,421}$$

$$= 1.759 - 2.596 + 1.093 = +0.256 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

Concrete stress in bottom of beam:

$$f_b = \frac{1,348.8}{767} + \frac{(1,348.8)(29.68)}{14,915} - \frac{(1,404.7)(12)}{14,915}$$

$$= 1.759 + 2.684 - 1.130 = +3.313 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

9.4.7.4
Stresses at Midspan

Bending moment at midspan due to the beam self-weight is:

M_g= 0.5(0.799)(60.5)(121 – 60.5) = 1,462.3 ft-kips.

$$f_t = \frac{1,348.8}{767} - \frac{(1,348.8)(29.68)}{15,421} + \frac{(1,462.3)(12)}{15,421}$$

$$= 1.759 - 2.596 + 1.138 = +0.301 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

$$f_b = \frac{1,348.8}{767} + \frac{(1,348.8)(29.68)}{14,915} - \frac{(1,462.3)(12)}{14,915}$$

$$= 1.759 + 2.684 - 1.177 = +3.266 \text{ ksi}$$

Compressive stress limit: +3.480 ksi O.K.

9.4.7.5
Hold-Down Forces

Assume that the stress in the strand at the time of prestressing, before seating losses, is: 0.80 f_{pu} = 0.80(270) = 216 ksi

Thus, the prestress force per strand before seating losses is: 0.153(216) = 33.0 kips

From **Figure 9.4.7.2-2**, the harp angle,

$$\psi = \tan^{-1}\left(\frac{50}{48.5(12)}\right) = 4.91°$$

Therefore, hold-down force/strand = 1.05(force per strand)(sin ψ)

= 1.05(33.0)sin 4.91° = 2.97 kips/strand

Note the factor 1.05 is applied to account for friction.

Total hold-down force = 12 strands(2.97) = 35.6 kips

The hold-down force and the harp angle should be checked against maximum limits for local practices. Refer to Chapter 3, Fabrication and Construction, Section 3.3.2.2 and Chapter 8, Design Theory and Procedures for additional details.

9.4.7.6 Summary of Stresses at Transfer

	Top Stresses f_t (ksi)	Bottom Stresses f_b (ksi)
At transfer length section	+0.291	+3.276
At harp points	+0.256	+3.313
At midspan	+0.301	+3.266

9.4.8 STRESSES AT SERVICE LOADS

The total prestressing force after all losses, P_{pe} = 1,094.4 kips

9.4.8.1 Stress Limits For Concrete

[LRFD Art. 5.9.4.2]

Compression:

Due to permanent loads, (i.e. beam self-weight, weight of slab and haunch, weight of future wearing surface, and weight of barriers), for load combination Service I:

for precast beam: $0.45f'_c = 0.45(6.5) = +2.925$ ksi

for deck: $0.45f'_c = 0.45(4.0) = +1.800$ ksi

Due to permanent and transient loads (i.e. all dead loads and live loads), for load combination Service I:

for precast beam: $0.60f'_c = 0.60(6.5) = +3.900$ ksi

for deck: $0.60f'_c = 0.60(4.0) = +2.400$ ksi

Tension:

For components with bonded prestressing tendons:

for load combination Service III: $-0.19\sqrt{f'_c}$

for precast beam: $-0.19\sqrt{6.5} = -0.484$ ksi

9.4.8.2 Stresses at Midspan

• Concrete stress at top fiber of the beam:

To check top compressive stresses, two cases are checked:

1. Under permanent loads, Service I:

Using bending moment values given in **Table 9.4.4-1**, compute the top fiber stresses:

$$f_{tg} = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{(M_g + M_s)}{S_t} + \frac{(M_{ws} + M_b)}{S_{tg}}$$

$$= \frac{1,094.4}{767} - \frac{(1,094.4)(29.68)}{15,421} + \frac{(1,438.2 + 1,659.6)(12)}{15,421} + \frac{(360 + 180)(12)}{63,861}$$

$= 1.427 - 2.106 + 2.411 + 0.101 = +1.833$ ksi

Compressive stress limit: +2.925 ksi O.K.

2. Under permanent and transient loads, Service I:

$$f_{tg} = +1.833 + \frac{(M_{LT} + M_{LL})}{S_{tc}}$$

$$= +1.833 + \frac{(1,830.3 + 843.3)(12)}{63,861}$$

$= +1.833 + 0.502 = +2.335$ ksi

Compressive stress limit: +3.900 ksi O.K.

• Concrete stress at the top fiber of the deck:

Note: Compressive stress in the deck slab at service loads never controls the design for typical applications. The calculations shown below are for illustration purposes and may not be necessary in most practical applications.

1. Under permanent loads, Service I:

$$f_{tc} = \frac{M_{ws} + M_b}{S_{tc}} = \frac{(360 + 180)(12)}{55,592} = +0.117 \text{ ksi}$$

Compressive stress limit: +1.800 ksi O.K.

2. Under permanent and transient loads, Service I:

$$f_{tc} = \frac{M_{ws} + M_b}{S_{tc}} + \frac{M_{LT} + M_{LL}}{S_{tc}} = 0.117 + \frac{(1,830.3 + 843.3)}{55,592} = +0.694 \text{ ksi}$$

Compressive stress limit: +2.400 ksi O.K.

• Concrete stress in bottom of beam, Service III:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{(M_g + M_s)}{S_b} - \frac{(M_{ws} + M_b) + 0.8(M_{LT} + M_{LL})}{S_{bc}}$$

$$= \frac{1,094.4}{767} + \frac{(1,094.4)(29.68)}{14,915}$$

$$- \frac{(1,438.2 + 1,659.6)(12)}{14,915} - \frac{[(360 + 180) + (0.8)(1,830.3 + 843.3)](12)}{20,090}$$

$= 1.427 + 2.178 - 2.492 - 1.600 = -0.487$ ksi

Tensile stress limit: −0.484 ksi

Stress is within 1% of stress limit. O.K..

9.4.8.3
Fatigue Stress Limit

LRFD Article 5.5.3.1 states that in regions of compressive stress due to permanent loads and prestress, fatigue is considered only if the compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination. However, the *LRFD Specifications* do not state where this stress is computed: at the bottom fiber of concrete or at the lower strand level. In this example, for convenience, the calculations are done at the bottom fiber of concrete.

At midspan, the bottom compressive stress due to permanent loads and prestress is:

$$\frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{(M_g + M_s)}{S_b} - \frac{(M_{ws} + M_b)}{S_{bc}}$$

$$= \frac{1,094.4}{767} + \frac{(1,094.4)(29.68)}{14,915} - \frac{(1,438.2 + 1,659.6)}{14,915} - \frac{(360.0 + 180.0)}{20,090}$$

$$= 1.427 + 2.178 - 2.492 - 0.323 = +0.790 \text{ ksi}$$

From **Table 9.4.4-1**, the unfactored fatigue bending moment at midspan, M_f, is 776.9 ft-kips. Thus, stress at the bottom fiber of the beam due to fatigue load combination is:

$$-\frac{0.75(M_f)}{S_{bc}} = -\frac{0.75(776.9)(12)}{20,090} = -0.348 \text{ ksi}$$

Since 2 (0.348) = 0.696 ksi < 0.790 ksi, the fatigue check does not need to be performed. See Discussion in Section 9.2.9.3 in Example 9.2.

9.4.8.4 Summary of Stresses at Service Loads

	Top of Deck (ksi) Service I		Top of Beam (ksi) Service I		Bottom of Beam (ksi)
	Permanent Loads	Total Loads	Permanent Loads	Total Loads	Service III
At midspan	+0.117	+0.694	+1.833	+2.335	–0.487

9.4.9 STRENGTH LIMIT STATE

Total ultimate bending moment for Strength I is:

M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM)

Using the values of unfactored bending moment given in **Table 9.4.4-1**, the ultimate bending moment at midspan is:

M_u = 1.25(1,438.2 + 1,659.6 + 180) + 1.5(360) + 1.75(1,830.3 + 843.3) = 9,316.1 ft-kips

Average stress in prestressing steel when $f_{pe} \geq 0.5\, f_{pu}$:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{[LRFD Eq. 5.7.3.1.1-1]}$$

where

f_{ps} = average stress in prestressing steel

f_{pu} = specified tensile strength of prestressing steel = 270 ksi

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad \text{[LRFD Eq. 5.7.3.1.1-2]}$$

= 0.28 for low-relaxation strand [LRFD Table C5.7.3.1.1-1]

d_p = distance from extreme compression fiber to the centroid of the prestressing

tendons = $h - y_{bs} = 80.00 - 6.92 = 73.08$ in.

c = distance between the neutral axis and the compressive face, in.

To compute c, assume rectangular section behavior and check if the depth of the equivalent compression stress block, c, is less than or equal to t_s:

[LRFD C5.7.3.2.2]

where $a = \beta_1 c$

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{0.85 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1.-4]

where

A_{ps} = area of prestressing steel = 48(0.153) = 7.344 in.^{2}

A_s = area of mild steel tension reinforcement = 0 in.^{2}

A'_s = area of compression reinforcement = 0 in.^{2}

f'_c = compressive stress of deck concrete = 4.0 ksi

f_y = yield strength of tension reinforcement, ksi

f'_y = yield strength of compression reinforcement, ksi

β_1 = stress factor of compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f'_c \le 4.0$ ksi

= $0.85 - 0.05\,(f'_c - 4.0) \ge 0.65$ for $f'_c > 4.0$ ksi

= 0.85

b = effective width of compression flange = 108 in.

$$c = \frac{7.344(270) + 0 - 0}{0.85(4.0)(0.85)(108) + 0.28(7.344)\left(\dfrac{270}{73.08}\right)} = 6.20 \text{ in.} < t_s = 7.5 \text{ in.}$$ O.K.

$a = \beta_1 c = 0.85(6.2) = 5.27$ in.

Therefore, the rectangular section behavior assumption is valid.

The average stress in prestressing steel is:

$$f_{ps} = 270\left(1 - 0.28\frac{6.20}{73.08}\right) = 263.6 \text{ ksi}$$

Nominal flexural resistance: [LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right)$$ [LRFD Eq. 5.7.3.2.2-1]

The above equation is a simplified form of LRFD Equation 5.7.3.2.2-1 because no compression reinforcement or mild tension reinforcement is considered and the section behaves as a rectangular section.

$$M_n = (7.344)(263.6)\left(73.08 - \frac{5.27}{2}\right)/12 = 11{,}364.4 \text{ ft-kips}$$

Factored flexural resistance:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where

ϕ = resistance factor [LRFD Art. 5.5.4.2.1]

= 1.00, for flexure and tension of prestressed concrete

M_r = 11,364.4 ft-kips > M_u = 9,316.1 ft-kips O.K.

9.4.10 LIMITS OF REINFORCEMENT

9.4.10.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The amount of prestressed and non-prestressed reinforcement should be such that

$$\frac{c}{d_e} \leq 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

where $d_e = \dfrac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$ [LRFD Eq. 5.7.3.3.1-2]

Since $A_s = 0$, $d_e = d_p$ = 73.08 in.

$$\frac{c}{d_e} = \frac{6.20}{73.08} = 0.085 \leq 0.42 \quad \text{O.K.}$$

9.4.10.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

At any section, the amount of prestressed and nonprestressed tensile reinforcement should be adequate to developed a factored flexural resistance, M_r, equal to the lesser of:

- 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and,
- 1.33 times the factored moment required by the applicable strength load combination.

Check at midspan:

The *LRFD Specifications* do not give any procedure for computing the cracking moment. Therefore, the following equation adapted from the *Standard Specifications*, Art. 9.18.2.1, is used.

$M_{cr} = (f_r + f_{pb})S_{bc} - M_{d/nc}(S_{bc}/S_b - 1)$

where

f_r = modulus of rupture [LRFD Art. 5.4.2.6]

$= 0.24\sqrt{f'_c} = 0.24\sqrt{6.5} = 0.612$ ksi

f_{pb} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} = \frac{1{,}094.4}{767} + \frac{1{,}094.4(29.68)}{14{,}915} = 3.605 \text{ ksi}$$

$M_{d/nc}$ = non-composite dead load moment at the section

$= M_g + M_s$ = 1,438.2 + 1,659.6 = 3,097.8 ft-kips

S_{bc} = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 20,090 in.^{4}

S_b = non-composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 14,915 in.^{4}

$$M_{cr} = (0.612 + 3.605)\frac{20{,}090}{12} - (3{,}097.8)\left(\frac{20{,}090}{14{,}915} - 1\right) = 5{,}985.1 \text{ ft-kips}$$

$1.2\ M_{cr} = 1.2(5{,}985.1) = 7{,}182.1$ ft-kips

At midspan, the factored moment required by the Strength I load combination is:

$M_u = 9{,}316.1$ ft-kips (as calculated in Section 9.4.9)

Thus, $1.33M_u = 1.33(9{,}316.1) = 12{,}390.4$ ft-kips

Since $1.2M_{cr} < 1.33M_u$, the $1.2M_{cr}$ requirement controls.

$M_r = 11{,}364.4$ ft-kips $> 1.2M_{cr} = 7{,}182.1$ ft-kips O.K.

Note: Contrary to the *Standard Specifications*, the *LRFD Specifications* requires that this criterion be met at every section.

9.4.11 SHEAR DESIGN

The area and spacing of shear reinforcement must be determined at regular intervals along the entire length of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement is provided when:

$V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

where

V_u = total factored shear force, kips

V_c = shear strength provided by concrete, kips

V_p = component of the effective prestressing force in the direction of the applied shear, kips

ϕ = resistance factor = 0.9 [LRFD Art. 5.5.4.2.1]

9.4.11.1 Critical Section

Critical section near the supports is the greater of: [LRFD Art. 5.8.3.2]

$0.5d_v\cot\theta$, or, d_v

where

d_v = effective shear depth

= distance between resultants of tensile and compressive forces, $(d_e - a/2)$, but not less than $(0.9d_e)$ or $(0.72h)$ [LRFD Art. 5.8.2.7]

where

d_e = the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement [LRFD Art. 5.7.3.3.1]

a = depth of compression block = 5.27 in. at midspan (assumed adequate)

h = total height of the section = 80 in.

θ = angle of inclination of diagonal compressive stresses, assume θ is 23° (slope of compression field)

9.4.11.1.1 Angle of Diagonal Compressive Stresses

The shear design at any section depends on the angle of diagonal compressive stresses at the section. Shear design is an iterative process that begins with assuming a value for θ. For this example, only the final cycle of calculations is shown. As a guide, for areas which have high shear forces and low bending moments, the angle θ ranges from 20° to 30°. For areas of low shear forces and high bending moments, the angle θ ranges to 45°. Using the previously stated guidelines, two iterations are enough in most cases.

9.4.11.1.2 Effective Shear Depth

Since some of the strands are harped, the effective depth, d_e, varies from point-to-point. However, d_e must be calculated at the critical section in shear which is not yet determined; therefore, for the first iteration, d_e is calculated based on the center of gravity of the strand group at the end of the beam, y_{bs}.

$$d_e = h_c - y_{bs} = 80.0 - 4.22 = 75.78 \text{ in.}$$

$$d_v = 75.78 - (0.5)(5.27) = 73.14 \text{ in.}$$

$$\geq 0.9\,d_e = 0.9(75.78) = 68.20 \text{ in.}$$

$$\geq 0.72\,h = 0.72(80) = 57.60 \text{ in.} \quad \text{O.K.}$$

Therefore, $d_v = 73.14$ in.

9.4.11.1.3 Calculation of Critical Section

The critical section near the support is the greater of:

$d_v = 73.14$ in.

and

$0.5d_v\cot\theta = (0.5)(73.14)\cot 23° = 86.15$ in. (Controls)

Since the width of the bearing is not yet determined, it was conservatively assumed to be zero. Therefore the critical section in shear is located at a distance of:

86.15 in. = 7.18 ft from centerline of support

$(x/L) = 7.18/120 = 0.06L$

The effective depth, d_e, and the position of the critical section in shear may be refined based on the position of the critical section calculated above. However, the difference is small and on the conservative side. Thus, no more refinement is performed.

9.4.11.2 Contribution of Concrete to Nominal Shear Resistance

The contribution of the concrete to the nominal shear resistance is:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

Several quantities must be determined before this expression can be evaluated.

9.4.11.2.1 Strain in Flexural Tension Reinforcement

Calculate the strain in the reinforcement, ε_x:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \leq 0.001 \qquad \text{[LRFD Eq. 5.8.3.4.2-2]}$$

where

M_u = applied factored bending moment at the specified section, 0.05L
= 1.25(323.6 + 373.3 + 40.5) + 1.50(81.0) + 1.75(433.9 + 189.7)
= 2,134.0 ft-kips

V_u = applied factored shear force at the specified section, 0.05L

= 1.25(42.2 + 48.7 + 5.3) + 1.50(10.6) + 1.75(72.9 + 30.0) = 316.2 kips

N_u = applied factored normal force at the specified section, 0.06L = 0

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For pretensioned members, LRFD Article C5.8.3.4.2 indicates that f_{po} can be taken as the stress in the strands when the concrete is cast around them, which is the jacking stress, f_{pj}, or $0.75f_{pu}$.

= 0.75(270.0) = 202.5 ksi.

V_p = component of the effective prestressing force in the direction of the applied shear

= (Force per strand)(Number of harped strands)(sin ψ)

V_p = (22.8)(12)sin4.91° = 23.4 kips

$$\varepsilon_x = \frac{\dfrac{2{,}134.0(12)}{73.14} + 0 + 0.5(316.2 - 23.4)(\cot 23°) - 5.508(202.5)}{2[0 + 28{,}500(5.508)]} = \frac{-420.3}{313{,}956}$$

$= -1.339 \times 10^{-3}$

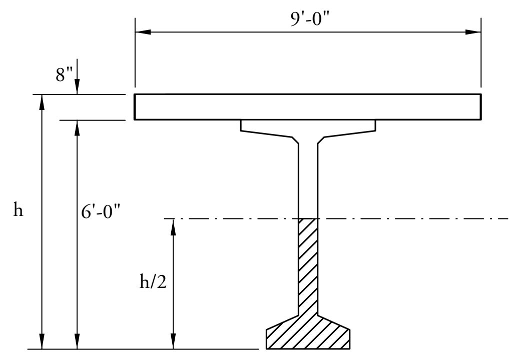
9.4.11.2.1.1 Calculation for Negative Strain

Since the value of ε_x is negative, a different equation for ε_x must be used.

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$ [LRFD Eq. 5.8.3.4.2-3]

where A_c = area of concrete (in.^{2}) on the flexural tension side, as shown in **Figure 9.4.11-1** (area of concrete below h/2 = 80/2 = 40 in.)

Figure 9.4.11-1 Illustration of A_c



A_c = [26(6) + (2)(0.5)(10)(4.5) + 6(34)] = 405 in.^{2}

Therefore, $\varepsilon_x = \dfrac{-420.3}{2[(4{,}888)(405) + 0 + (28{,}500)(5.508)]} = -0.098 \times 10^{-3}$

Note that the sign of ε_x should be maintained.

9.4.11.2.1.2 Compute Shear Stress

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v}$$ [LRFD Eq. 5.8.3.4.2-1]

where

v_u = shear stress in concrete

ϕ = resistance factor = 0.9 [LRFD Art. 5.5.4.2.1]

V_p = component of the effective prestressing force in the direction of the applied shear (calculated in Sect. 9.4.11.2.1)

$$v_u = \frac{316.2 - (0.9)(23.4)}{(0.9)(6)(73.14)} = 0.7473 \text{ ksi}$$

$(v_u/f'_c) = (0.7473/6.5) = 0.115$

9.4.11.2.2
Determine Values of β and θ

Having computed ε_x and v_u/f'_c, find a better estimate of θ from LRFD Table 5.8.3.4.2-1. Since the computed value of v_u/f'_c is likely to fall between two rows in the table, a linear interpolation may be performed. However, for hand calculations, interpolation is not recommended (LRFD Art. C5.8.3.4.2). The values of θ in the lower row that bounds the computed value may be used. Similarly, the values of θ in the first column to the right of the computed value may be used. For this example, the applicable row and column are the ones labeled "≤ 0.125" and "≤ −0.05", respectively. The values of θ and β contained in the cell of intersection of that row and column are:

$\theta = 22.8° \cong$ assumed value of 23° O.K.

Therefore, no further iteration is needed. However, if the designer desires to go through further iteration, it should be kept in mind that the position of the critical section of shear, and consequently the values of V_u and M_u, will need to be based on the new value of θ, 22.8°.

$\beta = 2.94$

where β = a factor indicating the ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution).

9.4.11.2.3
Compute Concrete Contribution

The nominal shear resisted by the concrete is:

$V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v d_v$ [LRFD Eq. 5.8.3.3-3]

$= 0.0316(2.94)\sqrt{6.5}(6)(73.14) = 103.9$ kips

9.4.11.3
Contribution of Reinforcement to Nominal Shear Resistance

9.4.11.3.1
Requirement for Reinforcement

Check if $V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

$0.5\phi(V_c + V_p) = 0.5(0.9)(103.9 + 23.4) = 57.3$ kips < 316.2 kips

Therefore, transverse shear reinforcement should be provided.

9.4.11.3.2
Required Area of Reinforcement

$V_u/\phi \le V_n = V_c + V_s + V_p$ [LRFD Eq. 5.8.3.3-1]

where

V_s = shear force carried by transverse reinforcement

$= (V_u/\phi) - V_c - V_p = (316.2/0.9) - 103.9 - 23.4 = 188.9$ kips

$$V_s = \frac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s}$$ [LRFD Eq. 5.8.3.3-4]

where

A_v = area of shear reinforcement within a distance s, in.^{2}

s = spacing of stirrups, in.

f_y = yield strength of shear reinforcement, ksi

α = angle of inclination of transverse reinforcement to longitudinal axis
= 90° for vertical stirrups

Therefore, area of shear reinforcement within a distance s, is:

$A_v = (sV_s)/(f_y d_v \cot\theta)$

$= [(s)(188.9)]/[60(73.14)\cot 23°] = 0.018(s)$

If s = 12 in., required A_v = 0.22 in.^{2}/ft

9.4.11.3.3
Determine Spacing of Reinforcement

Check maximum spacing of transverse reinforcement. [LRFD Art 5.8.2.7]

Check if $v_u < 0.125f'_c$ [LRFD Eqs. 5.8.2.7-1]

or if $v_u \geq 0.125f'_c$ [LRFD Eqs. 5.8.2.7-2]

$0.125f'_c = (0.125)(6.5) = 0.813$ ksi

$v_u = 0.7473$ ksi

Since $v_u < 0.125f'_c$ [LRFD Eq. 5.8.2.7.-2]

then, $s \leq 24$ in.

$s \leq 0.8d_v = 0.4(73.14) = 58.5$ in.

Therefore, maximum s = 24 in. > s provided O.K..

Use # 4 bar double legs @ 12 in., A_v = 0.40 in.^{2}/ft > 0.22 in.^{2}/ft.

$$V_s = \frac{0.4(60)73.14\cot 23°}{12} = 344.6 \text{ kips}$$

9.4.11.3.4
Minimum Reinforcement Requirement

The area of transverse reinforcement should not be less than: [LRFD Eq. 5.8.2.5-1]

$$0.0316\sqrt{f'_c}\frac{b_v s}{f_y} = 0.0316\sqrt{6.5}\frac{(6)(12)}{60} = 0.10 \text{ in.}^2\text{/ft} < A_v \text{ provided} \quad \text{O.K.}$$

9.4.11.4
Maximum Nominal Shear Resistance

In order to ensure that the concrete in the web of the beam will not crush prior to yielding of the transverse reinforcement, the *LRFD Specifications* give an upper limit of V_n as follows:

$V_n = 0.25f'_c b_v d_v + V_p$ [LRFD Eq. 5.8.3.3-2]

Comparing this equation with Eq. 5.8.3.3-1, it can be concluded that

$V_c + V_s \leq 0.25f'_c b_v d_v$

103.9 + 344.6 = 448.5 kips ≤ 0.25(6.5)(6)(73.14) = 713.1 kips O.K.

Using the foregoing procedures, the transverse reinforcement can be determined at increments along the entire length of the beam.

9.4.12 INTERFACE SHEAR TRANSFER

9.4.12.1 Factored Horizontal Shear

[LRFD Art. 5.8.4]

At the strength limit state, the horizontal shear at a section on a per unit basis can be taken as:

$$V_h = \frac{V_u}{d_v} \qquad \text{(LRFD Eq. C5.8.4.1-1)}$$

where

V_h = horizontal factored shear force per unit length of the beam, kips/in.

V_u = factored shear force at specified section due to superimposed loads, kips

d_v = the distance between resultants of tensile and compressive forces, $(d_e - a/2)$

The *LRFD Specifications* does not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, at point 0.06L.

Using load combination Strength I:

V_u = 1.25(5.3) + 1.5(10.6) + 1.75(72.9+30.0) = 202.6 kips

d_v = 73.14 in.

Therefore, the applied factored horizontal shear is:

$$V_h = \frac{202.6}{73.14} = 2.77 \text{ kips/in.}$$

9.4.12.2 Required Nominal Resistance

Required $V_n = V_h/\phi = 2.77/0.9 = 3.08$ kips/in.

9.4.12.3 Required Interface Shear Reinforcement

The nominal shear resistance of the interface surface is:

$$V_n = cA_{cv} + \mu[A_{vf} f_y + P_c] \qquad \text{[LRFD Eq. 5.8.4.1-1]}$$

where

c = cohesion factor, ksi [LRFD Art. 5.8.4.2]

μ = friction factor [LRFD Art. 5.8.4.2]

A_{cv} = area of concrete engaged in shear transfer, in.^{2}

A_{vf} = area of shear reinforcement crossing the shear plane, in.^{2}

P_c = permanent net compressive force normal to the shear plane, kips

f_y = shear reinforcement yield strength, ksi

For concrete placed against clean, hardened concrete but not an intentionally roughened surface: [LRFD Art. 5.8.4.2]

c = 0.075 ksi

μ = 0.6λ, where λ =1.0 for normal weight concrete, and therefore,

μ = 0.6

The actual contact width, b_v, between the slab and the beam is 42 in.

A_{cv} = (42.0 in.)(1.0 in.) = 42.0 in.^{2}

LRFD Eq. 5.8.4.1-1 can be solved for A_{vf} as follows:

3.08 = (0.075 x 42.0) + 0.6[A_{vf}(60) + 0]

solving for A_{vf},

A_{vf} (req'd) < 0

Since the resistance provided by cohesion is higher than the applied force, provide the minimum required interface reinforcement.

9.4.12.3.1 Minimum Interface Shear Reinforcement

Minimum $A_{vf} \geq (0.05b_v)/f_y$ [LRFD Eq. 5.8.4.1-4]

where b_v = width of the interface

From the design of vertical shear reinforcement, a #4 double-leg bar at 12-in. spacing is provided from the beam extending into the deck. Therefore, A_{vf} = 0.40 in.^{2}/ft.

A_{vf} = (0.40 in.^{2}/ft) < $(0.05b_v)/f_y$ = 0.05(42)/60 = 0.035 in.^{2}/in. = 0.42 in.^{2}/ft N.G.

However, LRFD Article 5.8.4.1 states that the minimum reinforcement requirement may be waived if V_n/A_{cv} < 0.100 ksi.

3.08 kips/in./42.0 in. = 0.073 ksi < 0.100 ksi

Therefore, the minimum reinforcement requirement is waived.

9.4.12.4 Maximum Nominal Shear Resistance

$$V_n \text{ provided} = (0.075)(42) + 0.6\left(\frac{0.40}{12}(60) + 0\right) = 4.35 \text{ kips/in.}$$

$0.2f'_cA_{cv}$ = (0.2)(4.0)(42) = 33.6 kips/in.

$0.8A_{cv}$ = 0.8(42) = 33.6 kips/in.

Since provided $V_n \leq 0.2f'_cA_{cv}$ O.K. [LRFD Eq. 5.8.4.1-2]

$\leq 0.8A_{cv}$ O.K. [LRFD Eq. 5.8.4.1-3]

9.4.13 MINIMUM LONGITUDINAL REINFORCEMENT REQUIREMENT

[LRFD Art. 5.8.3.5]

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied:

$$A_sf_y + A_{ps}f_{ps} \geq \frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta \qquad \text{[LRFD Eq. 5.8.3.5-1]}$$

where

A_s = area of nonprestressed tension reinforcement, in.^{2}

f_y = specified minimum yield strength of reinforcing bars, ksi

A_{ps} = area of prestressing steel at the tension side of the section, in.^{2}

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance is required, ksi

M_u = factored moment at the section corresponding to the factored shear force, ft-kips

N_u = applied factored axial force, kips

V_u = factored shear force at section, kips

V_s = shear resistance provided by shear reinforcement, kips

V_p = component in the direction of the applied shear of the effective prestressing force, kips

d_v = effective shear depth, in.

ϕ = resistance factor as appropriate for moment, shear and axial resistance. Therefore, different ϕ factors will be used for the terms in Equation (5.8.3.5-1), depending on the type of action being considered

[LRFD Art. 5.5.4.2]

θ = angle of inclination of diagonal compressive stresses

[LRFD Art. 5.8.3.5]

9.4.13.1 Required Reinforcement at Face of Bearing

Because the width of the bearing is not yet determined, it is assumed to be zero. This assumption is more conservative for these calculations. Thus, the failure crack assumed for this analysis radiates from the centerline of the bearing, 6 in. from the end of the beam.

From **Table 9.4.4-1**, the factored shear force and bending moment at this section are:

V_u = 1.25(47.9 + 55.3 + 6.0) + 1.5(12.0) + 1.75(78.1 + 33.9) = 350.5 kips

M_u = 0 ft-kips

$$\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

$$= 0 + 0 + \left(\frac{350.5}{0.9} - 0.5(344.6) - 23.4\right)\cot 23° = 456.4 \text{ kips}$$

As shown in **Figure 9.4.13.1-1**, the assumed crack plane crosses the centroid of the 36-straight strands at a distance of (6 + 4.22cot 23° = 15.94 in.) from the end of the beam. Since the transfer length is 30 in. from the end of the beam (60 times the strand diameter), the available prestress from the 36-straight strands is a fraction of the effective prestress, f_{pe}, in these strands. The 12 harped strands do not contribute to the tensile capacity since they are not on the flexural tension side of the member.

Therefore, the available prestress force is:

$$A_s f_y + A_{ps} f_{ps} = 0 + [(36)(0.153)\left((149.0)\frac{15.94}{30}\right)$$

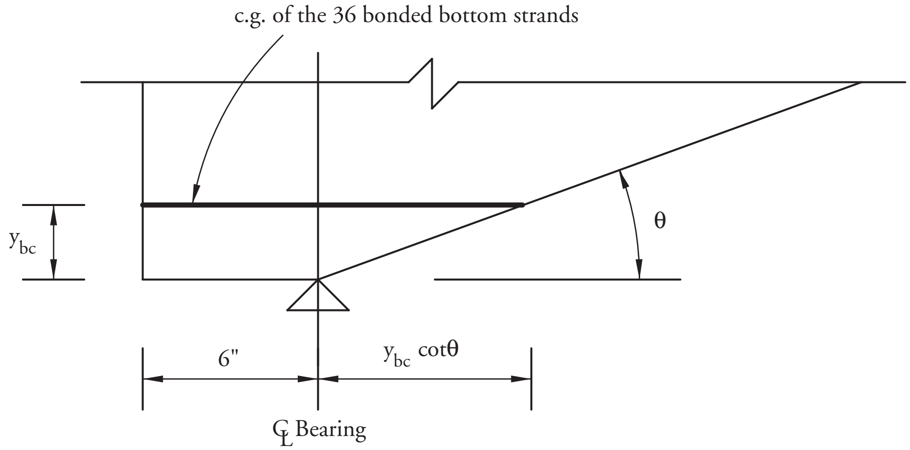
$$= 0 + 436.1 = 436.1 \text{ kips}$$

Additional force required:

(456.4 – 436.1)/60.0 = 0.34 in.^{2}

Provide (2)#4 bars (0.40 in.^{2})

Figure 9.4.13.1-1
Assumed Failure Crack



9.4.14 PRETENSIONED ANCHORAGE ZONE

[LRFD Art. 5.10.10]

9.4.14.1 Anchorage Zone Reinforcement

[LRFD Art. 5.10.10.1]

Design of the anchorage zone reinforcement is computed using the force in the strands just prior to transfer:

Force in the strands before transfer = F_{pi} = 48(0.153)(202.5) = 1,487.2 kips

The bursting resistance, P_r, should not be less than 4.0% of F_{pi}.

[LRFD Arts. 5.10.10.1 and C3.4.3]

$P_r = f_s A_s \geq 0.04F_{pi} = 0.04(1{,}487.2) = 59.5$ kips

where

A_s = total area of vertical reinforcement located within a distance h/4 from the end of the beam, in.^{2}

f_s = stress in steel, but not taken greater than 20 ksi

Solving for the required area of steel, A_s = 59.5/20 = 2.98 in.^{2}

At least 2.98 in.^{2} of vertical transverse reinforcement should be provided within a distance of (h/4 = 72/4 = 18.0 in.) from the end of the beam.

Use (5) #5 double leg bars at 4 in. spacing starting at 2 in. from the end of the beam. The provided A_s = 5(2)(0.31) = 3.10 in.^{2} > 2.98 in.^{2}. O.K.

9.4.14.2 Confinement Reinforcement

[LRFD Art. 5.10.10.2]

For a distance of 1.5d = 1.5(72) = 108 in., from the end of the beam, reinforcement is placed to confine the prestressing steel in the bottom flange. The reinforcement may not be less than #3 deformed bars with spacing not exceeding 6 in. The reinforcement should be of a shape which will confine (enclose) the strands.

9.4.15 DEFLECTION AND CAMBER

[LRFD Art. 5.7.3.6.2]

Deflections are calculated using the modulus of elasticity of concrete calculated in Section 9.4.3.1, and the moment of inertia of the non-composite precast beam.

9.4.15.1 Deflection Due to Prestressing Force at Transfer

Force per strand at transfer = 28.1 kips

$$\Delta_p = \frac{P_i}{E_{ci}I}\left(\frac{e_c L^2}{8} - \frac{e' a^2}{6}\right)$$

where

P_i = total prestressing force after transfer = 48 x 28.1 = 1,348.8 kips

e_c = eccentricity of prestressing force at midspan = 29.68 in.

e' = difference between eccentricity of prestressing steel at midspan and at end of the beam

$= e_c - e_e = 29.68 - (36.6 - 19.42) = 12.50$ in.

a = distance from end of the beam to the harp point = 48.5 ft

L = overall beam length = 121.0 ft

E_{ci} = modulus of elasticity at transfer = 4,617 ksi

I = gross moment of inertia of the precast beam = 545,894 in.^{4}

$$\Delta_p = \frac{1{,}348.8}{(4{,}617)(545{,}894)}\left(\frac{29.68(121 \times 12)^2}{8} - \frac{12.50(48.512)^2}{6}\right) = 3.81 \text{ in.}\uparrow$$

9.4.15.2 Deflection Due to Beam Self-Weight

$$\Delta_g = \frac{5wL^4}{384E_{ci}I}$$

where

w = beam self-weight = 0.799 kip/ft

E_{ci} = modulus of elasticity of precast beam at transfer = 4,617 ksi

I = gross moment of inertia of the precast beam = 545,894 in.^{4}

L = beam length = 121.0 ft at transfer = 120.0 ft at erection

Deflection due to beam self-weight at transfer:

$$\Delta_g = \frac{5\left(\frac{0.799}{12}\right)(121 \times 12)^4}{(384)(4{,}617)(545{,}894)} = 1.53 \text{ in.}\downarrow$$

Deflection due to beam self-weight used to compute deflection at erection:

$$\Delta_g = \frac{5\left(\frac{0.799}{12}\right)(121 \times 12)^4}{(384)(4{,}617)(545{,}894)} = 1.48 \text{ in.}\downarrow$$

9.4.15.3 Deflection Due to Slab and Haunch Weights

$$\Delta_s = \frac{5wL^4}{384E_cI}$$

where

w = slab and haunch weight = 0.922 kip/ft

L = design span = 120.0 ft

E_c = modulus of elasticity of precast beam at service loads = 4,888 ksi

I = gross moment of inertia of the precast beam = 545,894 in.^{4}

$$\Delta_s = \frac{5\left(\frac{0.922}{12}\right)(120 \times 12)^4}{(384)(4{,}888)(545{,}894)} = 1.61 \text{ in.}\downarrow$$

9.4.15.4 Deflection Due to Barrier and Future Wearing Surface

$$\Delta_{b+ws} = \frac{5wL^4}{384E_cI_c}$$

where

w = barrier and future wearing surface weight = 0.300 kip/ft

L = design span = 120.0 ft

E_c = modulus of elasticity of precast beam at service loads = 4,888 ksi

I_c = gross moment of inertia of the composite section = 1,073,931 in.^{4}

$$\Delta_{b+ws} = \frac{(5)\left(\frac{0.300}{12}\right)(120 \text{ x } 12)^4}{(384)(4{,}888)(1{,}100{,}320)} = 0.26 \text{ in.} \downarrow$$

9.4.15.5 Deflection and Camber Summary

At transfer, $(\Delta_p + \Delta_g) = 3.81 - 1.53 = 2.28$ in. ↑

Total deflection at erection, using PCI multipliers (see *PCI Design Handbook*)

$= 1.8(3.81) - 1.85(1.48) = 4.12$ in. ↑

Long-Term Deflection:

LRFD Article 5.7.3.6.2 states that the long time deflection may be taken as the instantaneous deflection multiplied by a factor 4.0, if the instantaneous deflection is based on the gross moment of inertia. However, a factor of 4.0 is not appropriate for this type of precast construction. Therefore, it is recommended that the designer follow the guidelines of the owner agency for whom the bridge is being designed, or undertake a more rigorous, time-dependent analysis.

9.4.15.6 Deflection Due to Live Load and Impact

Live load deflection limit (optional) = Span/800 [LRFD Art. 2.5.2.6.2]

$$= \left(120 \text{ x } \frac{12}{800}\right) = 1.80 \text{ in.}$$

If the owner invokes the optional live load deflection criteria specified in Art. 2.5.2.6.2, the deflection is the greater of:

- that resulting from the design truck alone, or
- that resulting from 25% of the design truck taken together with the design lane load.

The *LRFD Specifications* state that all the beams should be assumed to deflect equally under the applied live load and impact. [LRFD Art. 2.5.2.6.2]

Therefore, the distribution factor for deflection, DFD, is calculated as follows:

DFD = (number of lanes/number of beams) [LRFD Art. C2.5.2.6.2]

= 4/7 = 0.571 lanes/beam

However, it is more conservative to use the distribution factor for moment, DFM.

Deflection due to lane load

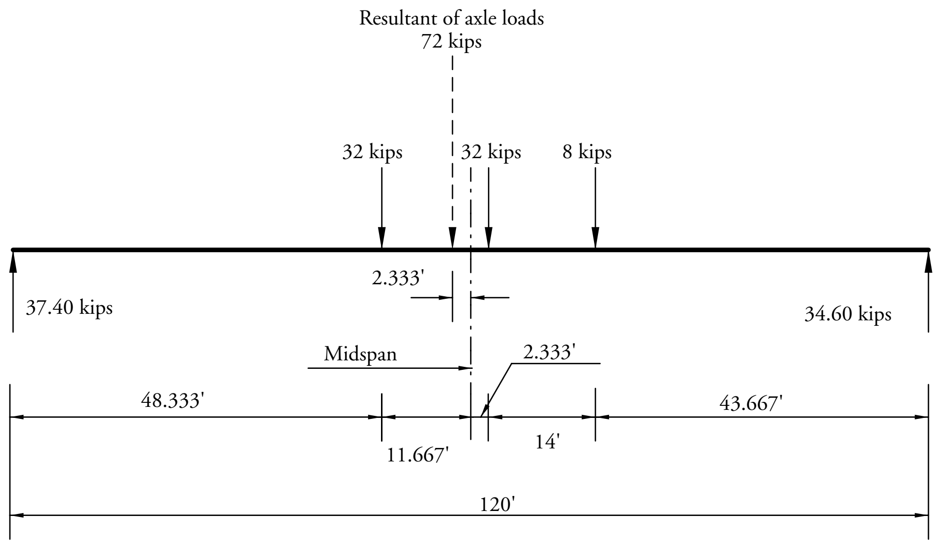
Design lane load, w = 0.64DFM = 0.64(0.732) = 0.469 kips/ft/beam

$$\Delta_{LL} = \frac{5wL^4}{384E_cI} = \frac{5\left(\frac{0.469}{12}\right)(120 \text{ x } 12)^4}{(384)(4{,}888)(1{,}100{,}320)} = 0.41 \text{ in.}$$

Deflection due to Design Truck Load and Impact

To obtain maximum moment and deflection at midspan due to the truck load, let the centerline of the beam coincide with the middle point of the distance between the inner 32 kip axle and the resultant of the truck load, as shown in **Figure 9.4.15.6-1**.

Figure 9.4.15.6-1
Design Truck Axle Load Position for Maximum Bending Moment



Using the elastic moment area or influence lines, deflection at midspan is:

$\Delta_{LT} = (0.803)(IM)(DFM) = (0.803)(1.33)(0.732) = 0.78$ in. ↓

Therefore, live load deflection is the greater of:

$\Delta_{LT} = 0.78$ in. (Controls)

$0.25\Delta_{LT} + \Delta_{LL} = 0.25(0.78) + 0.41 = 0.60$ in.

Therefore, live load deflection = 0.78 in. < allowable deflection = 1.8 in. O.K.

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC = dead load of structural components and non structural attachments [LRFD]
DFD = distribution factor for deflection
DFM = distribution factor for bending moment
DF_m = live load distribution factor for moment
DFV = distribution factor for shear force
DW = load of wearing surfaces and utilities [LRFD]
d = distance from extreme compressive fiber to centroid of the pretensioning force [STD]
d_b = nominal strand diameter [LRFD]
d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar [STD], [LRFD]
d_e = distance from exterior web of exterior beam and the interior side of curb or traffic barrier [LRFD]
d_e = effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement [LRFD]
d_p = distance from extreme compression fiber to the centroid of the pretensioning tendons [LRFD]
d_v = effective shear depth [LRFD]
E = width of slab over which a wheel load is distributed [STD]
E_c = modulus of elasticity of concrete [STD]
E_c = modulus of elasticity of concrete [LRFD]
E_{ci} = modulus of elasticity of the beam concrete at transfer
E_p = modulus of elasticity of pretensioning tendons [LRFD]
ES = loss of pretension due to elastic shortening [STD]
E_s = modulus of elasticity of pretensioning reinforcement [STD]
E_s = modulus of elasticity of reinforcing bars [LRFD]
e = eccentricity of the strands at h/2
e = eccentricity of strands at transfer length
e' = difference between eccentricity of pretensioning steel at midspan and end span
e_c = eccentricity of the strand at the midspan
e_e = eccentricity of pretensioning force at end of beam
e_g = distance between the centers of gravity of the beam and the slab [LRFD]
F_b = allowable tensile stress in the precompressed tensile zone at service loads
F_{pi} = total force in strands before release
F_ε = reduction factor [LRFD]
f_b = concrete stress at the bottom fiber of the beam
f'_c = specified concrete strength at 28 days [STD]
f'_c = specified compressive strength at 28 days [LRFD]
f_{cdp} = change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp} [LRFD]
f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied [STD]
f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer [STD]
f'_{ci} = concrete strength at release [STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]
f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]
f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pb} = compressive stress at bottom fiber of the beam due to prestress force
f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]
f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]
f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]
f_{pe} = effective stress in the pretensioning steel after losses [LRFD]
f_{pi} = initial stress immediately before transfer
f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]
f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]
f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]
f_{pu} = specified tensile strength of pretensioning steel [LRFD]
f_{py} = yield strength of pretensioning steel [LRFD]
f_r = the modulus of rupture of concrete [STD]
f_r = modulus of rupture of concrete [LRFD]
f_s = allowable stress in steel
f'_s = ultimate stress of pretensioning reinforcement [STD]
f_{se} = effective final pretension stress
f_{si} = effective initial pretension stress
f^*_{su} = average stress in pretensioning steel at ultimate load [STD]
f_t = concrete stress at top fiber of the beam for the non-composite section
f_{tc} = concrete stress at top fiber of the slab for the composite section
f_{tg} = concrete stress at top fiber of the beam for the composite section
f_y = yield strength of reinforcing bars [STD]
f_y = specified minimum yield strength of reinforcing bars [LRFD]
f_y = yield stress of pretensioning reinforcement [STD]
f'_y = specified minimum yield strength of compression reinforcement [LRFD]
f_{yh} = specified yield strength of transverse reinforcement [LRFD]
H = average annual ambient mean relative humidity, percent [LRFD]
H = height of wall [LRFD]
h = overall depth of precast beam [STD]
h = overall depth of a member [LRFD]

h_c = total height of composite section
h_f = compression flange depth [LRFD]
I = moment of inertia about the centroid of the non-composite precast beam [STD]
I = moment of inertia about the centroid of the non-composite precast beam [LRFD]
I = impact fraction (maximum 30%) [STD]
I_c = moment of inertia of composite section
IM = dynamic load allowance [LRFD]
J = St. Venant torsional constant
K = longitudinal stiffness parameter [STD]
K_g = longitudinal stiffness parameter [LRFD]
k = factor used in calculation of distribution factor for multi-beam bridges [LRFD]
k = factor used in calculation of average stress in pretensioning steel for Strength Limit State
L = live load [STD]
L = length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment [STD]
L = overall beam length or design span
L = span length measured parallel to longitudinal beams [STD]
L = span length [LRFD]
L_c = critical length of yield line failure pattern [LRFD]
LL = vehicular live load [LRFD]
ℓ_d = development length [LRFD]
ℓ_x = length required to fully develop the strand measured from the end of the strand
M_a = negative moment at the end of the span being considered
M_b = negative moment at the end of the span being considered
M_b = unfactored bending moment due to barrier weight
M_c = flexural resistance of cantilevered wall [LRFD]
M_{CIP} = unfactored bending moment due to cast-in-place topping slab
M_{const} = unfactored bending moment due to construction load
M_{col} = bending moment due to horizontal collision force
M_{cr} = moment causing flexural cracking at section due to externally applied loads (after dead load) [STD]
M_{cr} = cracking moment [LRFD]
M^*_{cr} = cracking moment [STD]
M_D = unfactored bending moment due to diaphragm weight
M_d = bending moment at section due to unfactored dead load
$M_{d/nc}$ = moment due to non-composite dead loads [STD]
M_f = unfactored bending moment due to fatigue truck per beam
M_g = unfactored bending moment due to beam self-weight
M_{LL} = unfactored bending moment due to lane load per beam
M_{LL+I} = unfactored bending moment due to live load + impact
M_{LL+I} = unfactored bending moment due to design vehicular load
M_{LT} = unfactored bending moment due to truck load with dynamic allowance per beam

M_{max}	= maximum factored moment at section due to externally applied loads	[STD]
M_n	= nominal moment strength of a section	[STD]
M_n	= nominal flexural resistance	[LRFD]
$M_{n/dc}$	= non-composite dead load moment at the section	
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_s	= maximum positive moment	
M_s	= unfactored bending moment due to slab and haunch weights	
M_{SDL}	= unfactored bending moment due to super-imposed dead loads	
$M_{service}$	= total bending moment for service load combination	
M_{SIP}	= unfactored bending moment due to stay-in-place panel	
M_u	= factored bending moment at section	[STD]
M_u	= factored moment at a section	[LRFD]
M_{ws}	= unfactored bending moment due to wearing surface	
M_x	= bending moment at a distance (x) from the support	
m	= material parameter	
m	= stress ratio = $(f_y/0.85f'_c)$	
N_b	= number of beams	[LRFD]
N_L	= number of traffic lanes	[STD]
N_u	= applied factored axial force taken as positive if tensile	[LRFD]
n	= modular ratio between deck slab and beam materials	
P	= diaphragm weight concentrated at quarter points	
P	= load on one rear wheel of design truck (P_{15} or P_{20})	[STD]
P_c	= permanent net compression force	[LRFD]
P_{eff}	= effective post-tensioning force	
P_i	= total pretensioning force immediately after transfer	
P_{pe}	= total pretensioning force after all losses	
P_r	= factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement	
P_s	= prestress force before initial losses	
P_{se}	= effective pretension force after allowing for all losses	
P_{si}	= effective pretension force after allowing for the initial losses	
P_{20}	= load on one rear wheel of the H20 truck	[STD]
Q	= total factored load	[LRFD]
Q_i	= specified loads	[LRFD]
q	= generalized load	[LRFD]
RH	= relative humidity	[STD]
R_n	= coefficient of resistance	
R_u	= flexural resistance factor	
R_w	= total transverse resistance of the railing or barrier	[LRFD]
S	= width of precast beam	[STD]
S	= average spacing between beams in feet	[STD]
S	= spacing of beams	[LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u	= factored shear force at section	[LRFD]
V_{uh}	= factored horizontal shear force per unit length of the beam	[LRFD]
V_x	= shear force at a distance (x) from the support	
v	= factored shear stress	[LRFD]
W	= overall width of bridge measured perpendicular to the longitudinal beams	[STD]
w	= a uniformly distributed load	[LRFD]
w	= width of clear roadway	[LRFD]
w_b	= weight of barriers	
w_c	= unit weight of concrete	[STD]
w_c	= unit weight of concrete	[LRFD]
w_g	= beam self-weight	
w_s	= slab and haunch weights	
w_{ws}	= weight of future wearing surface	
X	= distance from load to point of support	[STD]
x	= the distance from the support to the section under question	
y_b	= distance from centroid to the extreme bottom fiber of the non-composite precast beam	
y_{bc}	= distance from the centroid of the composite section to extreme bottom fiber of the precast beam	
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	
y_t	= distance from centroid to the extreme top fiber of the non-composite precast beam	
y_{tc}	= distance from the centroid of the composite section to extreme top fiber of the slab	
y_{tg}	= distance from the centroid of the composite section to extreme top fiber of the precast beam	
Z (or z)	= factor reflecting exposure conditions	[LRFD], [STD]
α	= angle of inclination of transverse reinforcement to longitudinal axis	
β	= factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	[LRFD]
β_D	= load combination coefficient for dead loads	[STD]
β_L	= load combination coefficient for live loads	[STD]
β_1	= factor for concrete strength	[STD]
β_1	= ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone	[LRFD]
Δ_{beam}	= deflection due to beam self-weight	
Δ_{b+ws}	= deflection due to barrier and wearing surface weights	
Δf_{cdp}	= change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied	[LRFD]
Δf_{pCR}	= loss in pretensioning steel stress due to creep	[LRFD]
Δf_{pES}	= loss in pretensioning steel stress due to elastic shortening	[LRFD]
Δf_{pi}	= total loss in pretensioning steel stress immediately after transfer	
Δf_{pR}	= loss in pretensioning steel stress due to relaxation of steel	[LRFD]
Δf_{pR1}	= loss in pretensioning steel stress due to relaxation of steel at transfer	[LRFD]
Δf_{pR2}	= loss in pretensioning steel stress due to relaxation of steel after transfer	[LRFD]
Δf_{pSR}	= loss in pretensioning steel stress due to shrinkage	[LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	$= \dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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BULB-TEE (BT-72), THREE SPANS, COMPOSITE DECK, STANDARD SPECIFICATIONS

Bulb-Tee (BT-72), Three Spans, Composite Deck, Standard Specifications

9.5.1 INTRODUCTION

This design example demonstrates the design of a three-span (110-120-110 ft) AASHTO-PCI bulb-tee beam bridge with no skew, as shown in **Figure 9.5.1-1**. This example illustrates in detail, the design of a typical interior beam of the center span at the critical sections in positive flexure, negative flexure, shear, and deflection due to prestress, dead loads and live load. The superstructure consists of four beams spaced at 12-ft centers (**Figure 9.5.1-2**). Beams are designed to act compositely with the 8 in. thick cast-in-place concrete deck slab to resist all superimposed dead loads, live loads and impact under full continuity. A 1/2 in. wearing surface is considered to be an integral part of the 8 in. deck slab. Design live load is AASHTO HS20. The design is carried out in accordance with the AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, 2002.

Figure 9.5.1-1
Longitudinal Section

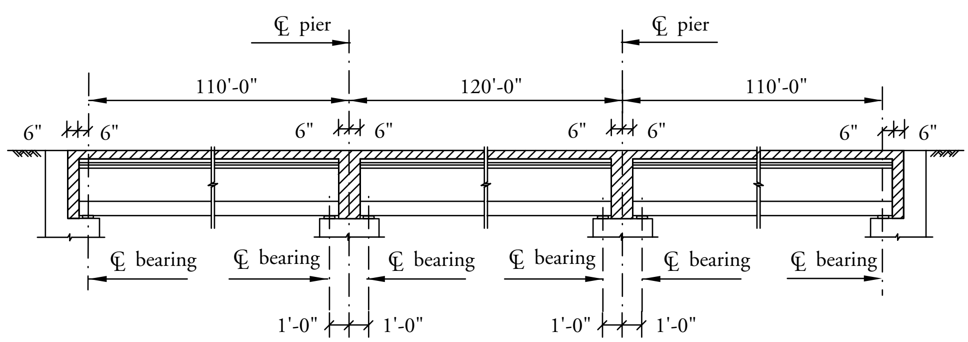
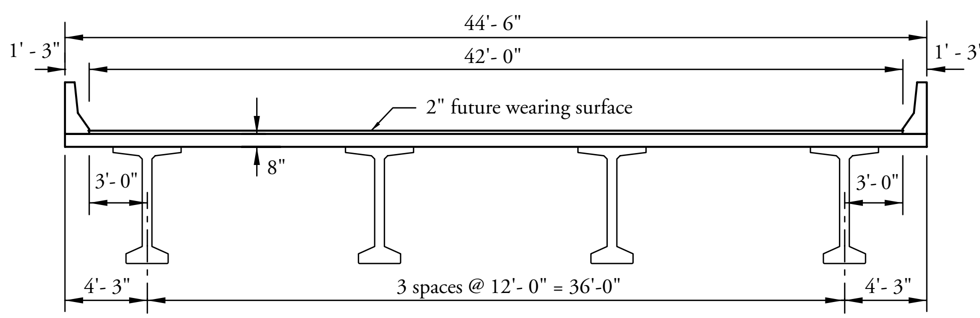


Figure 9.5.1-2
Cross-Section



9.5.2 MATERIALS

Cast-in-place slab: Actual thickness, t_s = 8.0 in.

Structural thickness = 7.5 in.

Note that a 1/2 in. wearing surface is considered to be an integral part of the 8 in. deck.

Concrete strength at 28 days, f'_c = 4,000 psi

Precast beams: AASHTO-PCI Bulb-tee shown in **Figure 9.5.2-1**

Concrete strength at release, f'_{ci} = 5,500 psi

Concrete strength at 28 days, f'_c = 7,000 psi

Concrete unit weight, w_c = 150 pcf

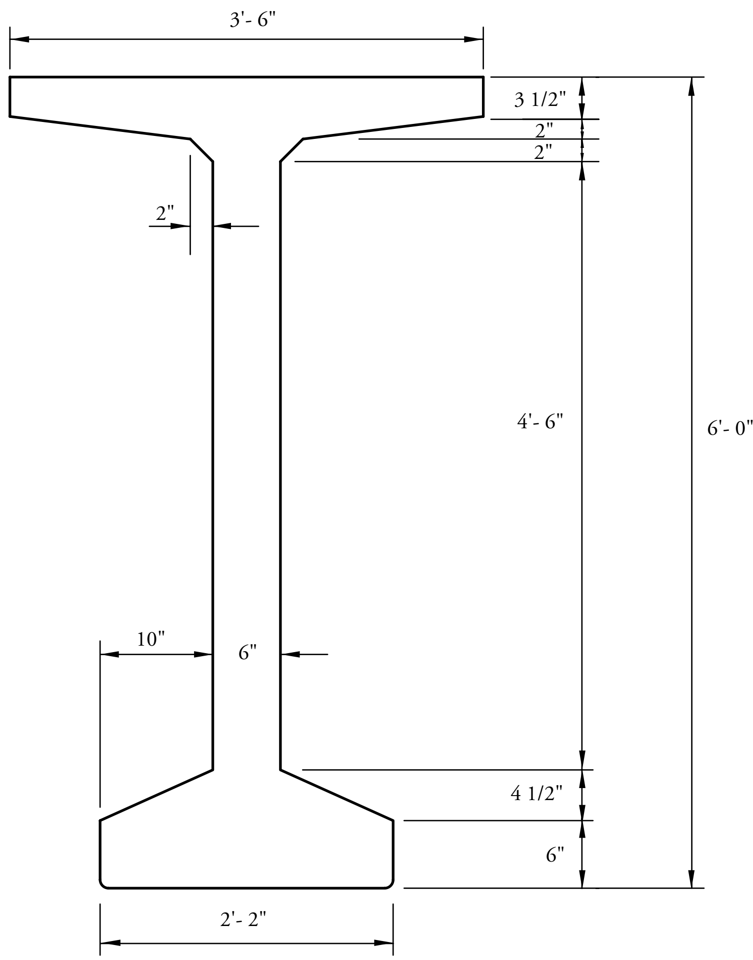
Overall beam length (**Figure 9.5.1-1**) = 110.0 ft (end spans) and 119.0 ft (center span)

Design spans (**Figure 9.5.1-1**):

For non-composite beam: = 109.0 ft (end spans)
= 118.0 ft (center span)

For composite beam: = 110.0 ft (end spans)
= 120.0 ft (center span)

Figure 9.5.2-1
AASHTO-PCI Bulb-Tee BT-72



Pretensioning strands: 1/2 in. dia., seven wire, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, $f_s' = 270{,}000$ psi

Yield strength, $f_y^* = 0.9f_s' = 243{,}000$ psi [STD Art. 9.1.2]

Initial pretensioning, $f_{si} = 0.75f_s'$ [STD Art. 9.15.1]

$= 202{,}500$ psi

Modulus of elasticity, $E_s = 28{,}500$ ksi

Although the *Standard Specifications,* Article 9.16.2.1.2, indicates that the modulus of elasticity, $E_s = 28{,}000$ ksi, a value of 28,500 ksi is a more accurate value, according to the PCI Design Handbook and the *LRFD Specifications.*

Reinforcing bars: yield strength, $f_y = 60{,}000$ psi

Future wearing surface: additional 2 in. with unit weight equal to 150 pcf

New Jersey-type barrier: Unit weight equals 300 lbs/ft/side

9.5.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM IN THE CENTER SPAN

9.5.3.1 Non-Composite Section

A = area of cross section of precast beam = 767 in.^{2}

h = overall depth of precast beam = 72 in.

I = moment of inertia about the centroid of the non-composite precast beam = 545,894 in.^{4}

y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam = 36.60 in.

y_t = distance from centroid to the extreme top fiber of the non-composite precast beam = 35.40 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 14,915 in.$_3$

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 15,421 in.^{3}

Wt = 0.799 k/ft

[STD Art. 8.7.1]

E_c = modulus of elasticity of concrete = $(w_c)^{1.5}(33)\sqrt{f_c'}\,/1{,}000$, ksi

where

w_c = unit weight of concrete = 150 pcf [STD Art. 3.3.6]

The *Standard Specifications,* Article 8.7.1, indicates that the unit weight of normal weight concrete is 145 pcf. However, precast concrete mixes typically have a relatively low water/cementitious materials ratio and high density. Therefore, a unit weight of 150 pcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f'_c = specified strength of concrete (psi)

Therefore, the modulus of elasticity for the:

- cast-in-place slab, using f'_c = 4,000 psi, is:

$$E_c = (150)^{1.5}(33)\sqrt{4{,}000}/1000 = 3{,}834 \text{ ksi}$$

- beam at release, using $f'_c = f'_{ci}$ = 5,500 psi, is:

$$E_{ci} = (150)^{1.5}(33)\sqrt{5{,}500}/1000 = 4{,}496 \text{ ksi}$$

- beam at service loads, using f'_c = 7,000 psi is:

$$E_c = (150)^{1.5}(33)\sqrt{7{,}000}/1000 = 5{,}072 \text{ ksi}$$

9.5.3.2 Composite Section

9.5.3.2.1 Effective Flange Width

[STD Art. 9.8.3]

Effective web width of the precast beam is the lesser of: [STD Art. 9.8.3.1]

b_e = top flange width = 42 in. (Controls)

or, 2(6)(5.5) + 6 + 2(2) = 76 in.

Use b_e = 42 in.

The effective flange width is the lesser of: [STD Art. 9.8.3.2]

1/4 span length: $\dfrac{120(12)}{4}$ = 360 in.

Distance center-to-center of beams = 12 ft (12) = 144 in.

12(effective slab thickness plus effective web width of the beam)

= 12(7.5) + 42 = 132 in. (Controls)

Use effective flange width = 132 in.

9.5.3.2.2 Modular Ratio Between Slab and Beam Materials

Modular ratio between slab and beam materials:

$$n = \frac{E_c(\text{slab})}{E_c(\text{beam})} = \frac{3{,}834}{5{,}072} = 0.7559$$

9.5.3.2.3 Transformed Section Properties

Note: Only the structural thickness of the deck (7.5 in.) will be used in these computations.

Transformed flange width = n(effective flange width) = 0.7559(132) = 99.78 in.

Transformed flange area = n(effective flange width)(t_s) = 0.7559(132)(7.5) = 748.34 in.^{2}

Due to camber of the pretensioned precast beam, a minimum haunch thickness of 1/2 in. at midspan is considered in the structural properties of the composite section. The haunch width must also be transformed.

Transformed haunch width = (0.7559)(42) = 31.75 in.

Transformed area of haunch = (0.7559)(42)(0.5) = 15.87 in.^{2}

Note: The haunch should only be considered to contribute to section properties if it is required to be provided in the completed structure. Therefore, some designers neglect its contribution to the section properties.

Figure 9.5.3.2.3-1 Composite Section

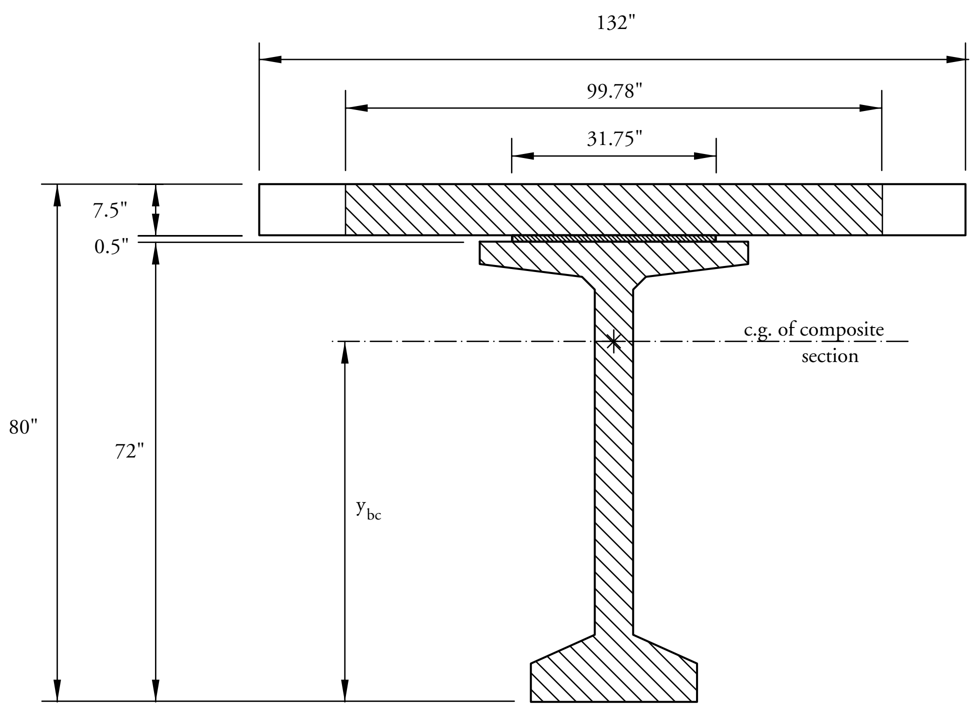


Table 9.5.3.2.3-1 Properties of Composite Section

	Transformed Area, in.^{2}	y_b, in.	Ay_b, in.^{3}	$A(y_{bc}-y_b)^2$	I, in.^{4}	$I + A(y_{bc} - y_b^2)$ in.^{4}
Beam	767.00	36.60	28,072.20	299,177.94	545,894.00	845,071.94
Haunch	15.87	72.25	1,146.61	4,012.09	0.33	4,012.42
Deck	748.34	76.25	57,060.93	296,350.12	3,507.89	299,858.01
Σ	1,531.21		86,279.74			1,148,942.37

A_c = transformed area of composite section = 1,531 in.^{2}

h_c = total height of composite section = 80.00 in.

I_c = moment of inertia of composite section = 1,148,942 in.^{4}

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam = $\dfrac{86{,}279.74}{1{,}531.21}$ = 56.35 in.

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam = 72 − 56.35 = 15.65 in.

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the deck = 80 − 56.35 = 23.65 in.

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam

$$= I_c/y_{bc} = \frac{1{,}148{,}942}{56.35} = 20{,}389 \text{ in.}^3$$

S_{tg} = composite section modulus for top fiber of the precast beam

$$= I_c/y_{tg} = \frac{1{,}148{,}942}{15.65} = 73{,}415 \text{ in.}^3$$

S_{tc} = transformed composite section modulus for top fiber of the slab

$$= I_c/ny_{tc} = \frac{1{,}148{,}942}{(0.7559)(23.65)} = 64{,}269 \text{ in.}^3$$

9.5.4 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the beam, the weight of deck and haunch act on the non-composite simple-span structure, while the weight of barriers, future wearing surface, live load plus impact act on the composite, three-span continuous structure.

9.5.4.1 Shear Forces and Bending Moments Due to Dead Loads

[STD Art. 3.3]

9.5.4.1.1 Dead Loads

Beam weight = 0.799 kip/ft

$$8 \text{ in. deck slab weight} = 0.150\left(\frac{8.0}{12}\right)(12.0) = 1.200 \text{ kip/ft}$$

$$\text{Haunch} = \frac{1/2}{12}(3.5 \text{ ft})(0.150) = 0.022 \text{ kip/ft}$$

Notes:

1. Actual slab thickness (8 in.) is used for computing deck slab dead load
2. A 1/2 in. minimum haunch is assumed in the computation of forces. If a deeper haunch will be used because of final beam camber, the weight of the actual haunch should be used.
3. Diaphragms – many state agencies are moving away from using cast-in-place concrete diaphragms and are using lighter weight steel diaphragms. Therefore, the weight of diaphragms will be ignored.
4. Dead loads placed on the composite structure are distributed equally among all beams. [STD Art. 3.23.2.3.1.1]

Barriers = (2 barriers) $\dfrac{300/1{,}000}{4 \text{ beams}}$ = 0.150 kip/ft/beam

Weight of future wearing surface = $\dfrac{2}{12}(150)$ = 25 psf which is applied over the entire width of the bridge between curbs (42 ft)

Future wearing surface = $\dfrac{25(42.0)}{4 \text{ beams}}/1{,}000$ = 0.263 kips/ft/beam

9.5.4.1.2 Unfactored Shear Forces and Bending Moments

For a simply supported beam, simple span (L), loaded with a uniformly distributed load (w), the shear force (V_x) and the bending moment (M_x) at a distance (x) from the support are given by:

$V_x = w(0.5L - x)$ (Eq. 9.5.4.1.2-1)

$M_x = 0.5wx(L - x)$ (Eq. 9.5.4.1.2-2)

Using the above equations, values of shear forces and bending moments for a typical interior beam due to the weight of beam, slab and haunch are computed and given in **Table 9.5.4.1.2-1**. The span length for each span considered depends on the construction stage:

- overall length immediately after prestress release
- centerline-to-centerline distance between beam bearings at the time of deck placement
- centerline-to-centerline distance between supports after beam is made continuous

The shear forces and bending moments due to barrier and future wearing surface weights are calculated based on the continuous span lengths, i.e., 110, 120 and 110 ft. The three-span structure was analyzed using a continuous beam computer program. Shear forces and bending moments are given in **Table 9.5.4.1.2-1**.

9.5.4.2 Shear Forces and Bending Moments Due to Live Load

[STD Art. 3.7.1.1]

9.5.4.2.1 Live Load

Live load is either the standard truck or lane loading corresponding to HS20.

[STD Art. 3.7.1.1]

9.5.4.2.2 Live Load Distribution Factor for a Typical Interior Beam

Using the live load distribution factor for moment for a precast, prestressed concrete beam, the fraction of the wheel load carried by the interior beam is:

$DF_m = \dfrac{S}{5.5} = \dfrac{12.0}{5.5}$ = 2.182 wheels/beam [STD Table 3.23.1]

where S = average spacing between beams in feet

$DF_m/2$ = 1.091 lanes/beam

9.5.4.2.3 Live Load Impact

[STD Art. 3.8]

From the discussion in Section 9.5.4.1.2, the design spans for the composite continuous bridge are 110 and 120 ft.

The live load impact factor is computed using the following equation:

$$I = \frac{50}{L+125}$$ [STD Eq. 3-1]

where I = impact fraction (maximum 30%).

For continuous spans:

L = the length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment.

[STD Art. 3.8.2.2]

For positive moment: center span, $I = \frac{50}{120+125} = 0.204$

For negative moment: $I = \frac{50}{\left(\frac{110+120}{2}\right)+125} = 0.208$

The impact for shear is calculated according to the *Standard Specifications,* Article 3.8.2. The software used to calculate the live load shear computes the impact which varies along the span length.

9.5.4.2.4 Unfactored Shear Forces and Bending Moments

Live load shear force and bending moment per beam are:

V_{LL+I} = (shear force per lane)(distribution factor)(1 + I)

M_{LL+I} = (bending moment per lane)(distribution factor)(1 + I)

The results of the analysis using HS20 live load is shown in **Table 9.5.4.1.2-1**. Tabulated values for shear and moment are maximum values per beam and include impact.

9.5.4.3 Load Combinations

[STD Art. 3.22]

For service load design (Group I):

1.00 D + 1.00(L + I) [STD Table 3.22.1A]

where

D = dead load

L = live load

I = impact fraction

For load factor design (Group I):

1.3[1.00 D + 1.67(L + I)] [STD Table 3.22.1A]

Table 9.5.4.1.2-1 Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Location		Beam weight [simple span]		Deck plus Haunch weight [simple span]		Barrier weight [continuous span]		Future wearing surface [continuous span]		Live load envelope plus impact [continuous span]		
Distance x ft	Section x/L	Shear V_g kips	Moment M_g ft-kips	Shear V_s kips	Moment M_s ft-kips	Shear V_b kips	Moment M_b ft-kips	Shear V_{ws} kips	Moment M_{ws} ft-kips	Shear V_{LL+I} kips	Moment positive ft-kips	Moment negative ft-kips
0.0 [1]	0.000	43.5	0.0	66.6	0.0	6.0	0.0	11.0	0.0	85.0	0.0	0.0
11.0	0.100	34.8	430.7	53.2	658.7	5.0	62.0	8.0	109.0	73.0	808.0	-92.0
22.0	0.200	26.0	764.6	39.7	1,169.5	3.0	106.0	6.0	186.0	62.0	1,362.0	-183.0
33.0	0.300	17.2	1,001.9	26.3	1,532.4	2.0	132.0	3.0	231.0	51.0	1,682.0	-275.0
44.0	0.400	8.4	1,142.6	12.8	1,747.5	0.0	139.0	0.0	244.0	40.0	1,818.0	-367.0
55.0	0.500	0.4	1,186.5	0.6	1,814.7	2.0	128.0	3.0	225.0	48.0	1,778.0	-458.0
66.0	0.600	9.2	1,133.8	14.1	1,734.0	3.0	100.0	6.0	175.0	58.0	1,592.0	-550.0
77.0	0.700	18.0	984.4	27.5	1,505.5	5.0	53.0	9.0	93.0	68.0	1,248.0	-642.0
88.0	0.800	26.8	738.3	40.9	1,129.1	7.0	-12.0	12.0	-21.0	76.0	783.0	-731.0
99.0	0.900	35.6	395.5	54.4	604.9	8.0	-95.0	15.0	-167.0	84.0	349.0	-1,060.0
109.0 [1]	0.991	43.5	0.0	66.6	0.0	9.8*	-187.7*	17.7*	-328.8*	92.2*	267.2*	-1,711.8*
110.0	0.000	0.0	0.0	0.0	0.0	10.0	-197.0	18.0	-345.0	93.0	259.0	-1,777.0
0.0 [2]	0.000	0.0	0.0	0.0	0.0	9.0	-197.0	16.0	-345.0	91.0	259.0	-1,777.0
1.0 [1]	0.008	47.1	0.0	72.1	0.0	9.3*	-188.9*	15.8*	-330.8*	90.1*	264.1*	-1,714.7
4.3 [3]	0.036	44.5	152.7	68.0	233.5	8.3*	-162.0*	14.9*	-283.6*	87.0*	281.0*	-1,506.9*
12.0	0.100	38.4	470.2	58.7	719.1	7.0	-100.0	13.0	-175.0	80.0	320.0	-1,029.0
24.0	0.200	28.8	872.9	44.0	1,335.0	5.0	-24.0	10.0	-42.0	71.0	866.0	-742.0
36.0	0.300	19.2	1,160.5	29.3	1,775.0	4.0	30.0	6.0	53.0	61.0	1,286.0	-667.0[4]
48.0	0.400	9.6	1,333.1	14.7	2,038.9	2.0	62.0	3.0	109.0	50.0	1,546.0	-667.0[4]
60.0	0.500	0.4	1,390.7	0.0	2,126.9	0.0	73.0	0.0	128.0	39.0	1,618.0	-667.0[4]

[1] Centerline of bearing
[2] Centerline of pier
[3] Critical section for shear (see Section 9.5.11)
[4] Identical values caused by loading of a remote span
Note: Values shown for shear are absolute values.
* Values were calculated using linear interpolation.

9.5.5 ESTIMATE REQUIRED PRESTRESS

The estimation of the required number of strands is usually governed by concrete tensile stresses at bottom fiber at the section of maximum moment, or at the harp points. For estimating purposes, only the stresses at midspan need to be considered.

9.5.5.1 Service Load Stresses at Midspan

Bottom tensile stresses due to applied loads is:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + M_{LL+I}}{S_{bc}}$$

where

f_b = bottom tensile stresses, ksi

M_g = unfactored bending moment due to weight of the beam, ft-kips

M_s = unfactored bending moment due to weights of deck and haunch, ft-kips

M_b = unfactored bending moment due to weight of barriers, ft-kips

M_{ws} = unfactored bending moment due to weight of future wearing surface, ft-kips

M_{LL+I} = unfactored bending moment due to live load plus impact, ft-kips

Using values of bending moments from **Table 9.5.4.1.2-1**, the bottom tensile stress at midspan of the center span (point 0.5):

$$f_b = \frac{(1{,}390.7 + 2{,}126.9)(12)}{14{,}915} + \frac{(73.0 + 128.0 + 1{,}618.0)(12)}{20{,}389}$$

$= 2.830 + 1.071 = 3.901$ ksi

9.5.5.2 Allowable Stress Limit

At service loads, allowable tensile stress in the precompressed tensile zone:

$$F_b = 6\sqrt{f_c'} = 6\sqrt{7{,}000}\left(\frac{1}{1{,}000}\right) = 0.502 \text{ ksi}$$ [STD Art. 9.15.2.2]

9.5.5.3 Required Number of Strands

Required precompressive stress in the bottom fiber after losses:

Bottom tensile stress – allowable tension stress at final $= f_b - F_b = 3.901 - 0.502 = 3.399$ ksi

The location of the center of gravity of strands at midspan usually ranges from 5 to 15% of the beam depth, measured from the bottom of the beam. A value of 5% is appropriate for the newer efficient sections like bulb-tees and 15% for the less efficient AASHTO shapes of normal design. Assume the distance from the center of gravity of strands to the bottom fiber of the beam is equal to $y_{bs} = 5.60$ in. (Note: $y_{bs}/h = 5.6/72 = 7.8\%$)

Strand eccentricity at midspan is equal to:

$e_c = y_b - y_{bs} = 36.60 - 5.60 = 31.00$ in.

Bottom fiber stress due to pretension after all losses: $f_b = \dfrac{P_{se}}{A} + \dfrac{P_{se}\,e_c}{S_b}$

where P_{se} = effective pretension force after allowing for all losses

Set the required precompression (3.399 ksi) equal to the bottom fiber stress due to prestress and solve for the required minimum P_{se}:

$$3.399 = \frac{P_{se}}{767} + \frac{31.00\,P_{se}}{14{,}915}, \text{ and } P_{se} = 1{,}005.0 \text{ kips}$$

Assume final losses of 25% of $f_{si} = 0.25(202.5) = 50.6$ ksi

The available pretension force per strand after all losses:

(cross-sectional area of one strand)$[f_{si} - \text{losses}]$

$= 0.153(202.5 - 50.6) = 23.2$ kips

Number of strands required $= \dfrac{1{,}005.0}{23.2} = 43.3$

Try (44) 1/2-in.-dia, 270 ksi strands

9.5.5.4 Strand Pattern

The assumed strand pattern for 44 strands at the midspan section is shown in **Figure 9.5.5.4-1**. Each available position was filled beginning with the bottom row.

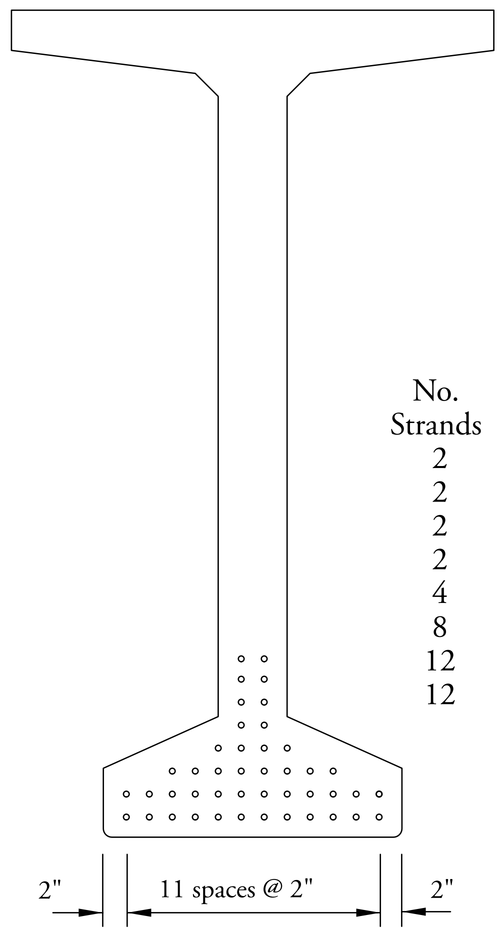
Calculate the distance from the center of gravity of the strands to the bottom fiber of the beam, y_{bs}:

$$y_{bs} = \frac{12(2)+12(4)+8(6)+4(8)+2(10)+2(12)+2(14)+2(16)}{44} = 5.82 \text{ in.}$$

Strand eccentricity at midspan:

$e_c = 36.60 - 5.82 = 30.78$ in.

Figure 9.5.5.4-1
Strand Pattern at Midspan



No. Strands	Distance from bottom (in.)
2	16
2	14
2	12
2	10
4	8
8	6
12	4
12	2

9.5.6 PRESTRESS LOSSES

[STD Art. 9.16.2]

Total losses = SH + ES + CR_c + CR_s [STD Eq. 9-3]

where

SH = loss of prestress due to concrete shrinkage, ksi

ES = loss of prestress due to elastic shortening, ksi

CR_c = loss of prestress due to creep of concrete, ksi

CR_s = loss of prestress due to relaxation of prestressing steel, ksi

9.5.6.1 Shrinkage

[STD Art. 9.16.2.1.1]

Relative humidity varies significantly from one area of the country to another. Refer to the U.S. map, Figure 9.16.7.1.1 in the *Standard Specifications.*

Assume relative humidity, RH = 70%

$$SH = 17{,}000 - 150RH = [17{,}000 - 150(70)]\frac{1}{1{,}000} = 6.5 \text{ ksi} \qquad \text{[STD Eq. 9-4]}$$

[STD Art. 9.16.2.1.2]

9.5.6.2 Elastic Shortening

For pretensioned members:

$$ES = \frac{E_s}{E_{ci}} f_{cir} \qquad \text{[STD Eq. 9-6]}$$

where

f_{cir} = average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of the beam immediately after transfer

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si} e_c^2}{I} - \frac{(M_g + M_D) e_c}{I}$$

where

P_{si} = pretension force after allowing for the initial losses. *The Standard Specifications* allow that the reduction to initial tendon stress be estimated as $0.69f_s'$ for low-relaxation strands

= (number of strands)(area of strands)($0.69f_s'$)

= (44)(0.153)[(0.69)(270)] = 1,254.2 kips

M_g = unfactored bending moment due to beam self-weight at midspan of the center span = 1,390.7 ft-kips

M_D = unfactored bending moment due to diaphragm weight

e_c = eccentricity of the strand at midspan = 30.78 in.

M_g should be calculated based on the overall beam length of 119 ft. However, since the elastic shortening losses will be a part of the total losses, f_{cir} is con servatively computed using the design span length of 118 ft.

$$f_{cir} = \frac{1{,}254.2}{767} + \frac{1{,}254.2(30.78)^2}{545{,}894} - \frac{1{,}390.7(12)(30.78)}{545{,}894}$$

$$= 1.635 + 2.177 - 0.941 = 2.871 \text{ ksi}$$

$$ES = \frac{28{,}500}{4{,}496}(2.871) = 18.2 \text{ ksi}$$

9.5.6.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

$$CR_c = 12f_{cir} - 7f_{cds} \qquad \text{[STD Eq. 9-9]}$$

where

f_{cds} = concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied.

$$= \frac{M_s e_c}{I} + \frac{M_{SDL}(y_{bc} - y_{bs})}{I_c}$$

M_s = unfactored bending moment due to weight of slab and haunch

= 2,126.9 ft-kips

M_{SDL} = superimposed dead load moment = $M_b + M_{ws}$ = 73 + 128 = 201.0 ft-kips

y_{bc} = 56.35 in.

y_{bs} = the distance from center of gravity of the strand at midspan to the bottom of the beam = 5.82 in.

I = moment of inertia for the non-composite section = 545,894 in.^{4}

I_c = moment of inertia for the composite section = 1,148,942 in.^{4}

$$f_{cds} = \frac{2{,}126.9(12)(30.78)}{545{,}894} + \frac{(201.0)(12)(56.35 - 5.82)}{1{,}148{,}942}$$

= 1.439 + 0.106 = 1.545 ksi

CR_c = 12(2.871) – 7(1.545) = 23.6 ksi

9.5.6.4 Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

For pretensioned members with 270 ksi low-relaxation strand:

$CR_s = 5{,}000 - 0.10ES - 0.05(SH + CR_c)$ [STD Eq. 9-10A]

$$= [5000 - 0.1(18{,}200) - 0.05(6{,}500 + 23{,}600)]\left(\frac{1}{1{,}000}\right) = 1.7 \text{ ksi}$$

9.5.6.5 Total Losses at Transfer

Losses due to elastic shortening, ES = 18.2 psi

Total initial losses = 18.2 ksi

f_{si} = effective initial prestress after allowing initial losses:

= 202.5 – 18.2 = 184.3 ksi

P_{si} = 44(0.153)(184.3) = 1,240.7 kips

9.5.6.6 Total Losses at Service Loads

SH = 6.5 ksi

ES = 18.2 ksi

CR_c = 23.6 ksi

CR_s = 1.7 ksi

Total final losses = 6.5 + 18.2 + 23.6 + 1.7 = 50.0 ksi

or, $\frac{50.0}{202.5}(100)$ = 24.7% losses

f_{se} = effective final prestress = 202.5 – 50.0 = 152.5 ksi

P_{se} = 44(0.153)(152.5) = 1,026.6 kips

9.5.7 CONCRETE STRESSES AT TRANSFER

9.5.7.1 Allowable Stress Limits

[STD Art. 9.15.2.1]

Compression: $0.6f'_{ci}$ = +3.300 ksi (compression)

Tension: The maximum tensile stress shall not exceed:

- $7.5\sqrt{f'_{ci}} = -0.556$ ksi (tension)
- If the calculated tensile stress exceeds −200 psi or $3\sqrt{f'_{ci}} = -0.222$ ksi, whichever is smaller, provide bonded reinforcement to resist the total tension force in the concrete computed on the assumption of an uncracked section.

9.5.7.2 Stresses at Transfer Length Section

This section is located at a distance equal to the transfer length from the end of the beam. Stresses at this location need only be checked at release, since this stage almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length equals 50(strand diameter) [STD Art. 9.20.2.4]

= 50(0.50) = 25 in. = 2.08 ft

The transfer length section is located at 2.08 ft. from the end of the beam or at 1.58 ft from centerline of the pier. This is assuming that the distance between the centerline of pier and beam end is 6 in., as shown in **Figure 9.5.1-1**.

Due to the camber of the beam at release, the beam self-weight acts on the overall beam length (119 ft). Therefore, the values of bending moment given in **Table 9.5.4.1.2-1** cannot be used because they are based on the beam span between centerlines of bearings (118 ft). The value of bending moment at the transfer length due to beam self-weight is calculated using Equation (9.5.4.1.2-2) based on overall length.

Therefore, $M_g = 0.5(0.799)(2.08)(119 - 2.08) = 97.3$ ft-kips.

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = +\frac{P_{si}}{A} - \frac{P_{si}\,e}{S_t} + \frac{M_g}{S_t}$$

$$f_t = +\frac{1{,}240.7}{767} - \frac{1{,}240.7\,(30.78)}{15{,}421} + \frac{97.3\,(12)}{15{,}421} = +1.618 - 2.476 + 0.076 = -0.782 \text{ ksi}$$

Allowable tension: −0.556 ksi N.G.

Compute concrete stress at the bottom fiber of the beam, f_b:

$$f_b = +\frac{P_{si}}{A} + \frac{P_{si}\,e}{S_b} - \frac{M_g}{S_b} = \frac{1{,}240.7}{767} + \frac{1{,}240.7(30.78)}{14{,}915} - \frac{97.3(12)}{14{,}915}$$

$$= +1.618 + 2.560 - 0.078 = +4.100 \text{ ksi}$$

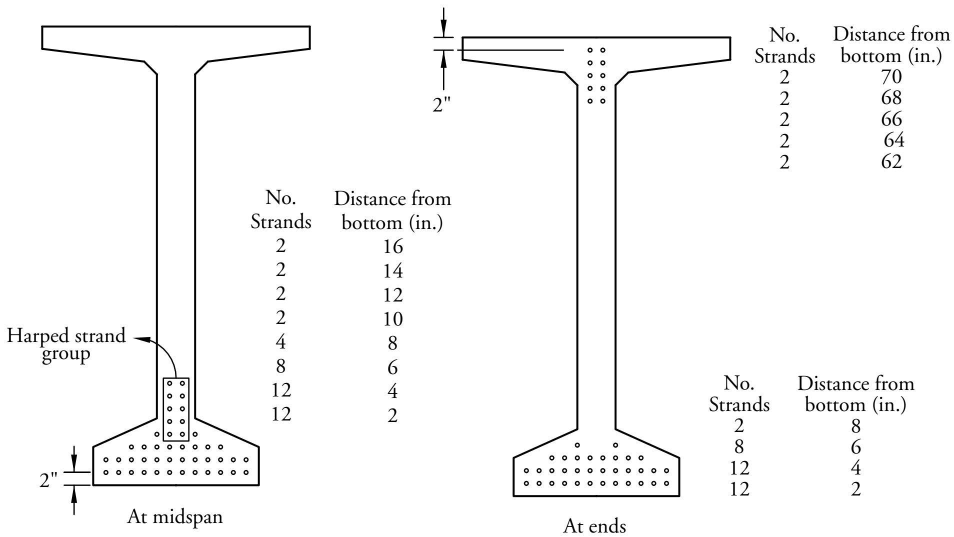
Allowable compression: +3.300 ksi N.G.

Since the top and bottom fiber stresses exceed the allowable, harped and/or debonded strands must used.

In this example, a harped strand pattern will be used with harp points at 0.3L = 35.5 ft from end of beam. Try harping 10 strands as shown in **Figures 9.5.7.2-1** and **9.5.7.2-2.**

The selection of this harp point is more appropriate for exterior spans of multispan continuous bridges since the location of the maximum positive moment is closer to the abutment than in the center span sections. For simple spans, it is desireable to use a harp point at least 0.4L from the supports.

Figure 9.5.7.2-1
Strand Pattern



The distance between the center of gravity of the 10 harped strands and the top of the beam at the end of the beam is: $\dfrac{2(2)+2(4)+2(6)+2(8)+2(10)}{10} = 6.00$ in.

The distance between the center of gravity of the 10 harped strands and the bottom of the beam at the harp points is: $\dfrac{2(8)+2(10)+2(12)+2(14)+2(16)}{10} = 12.00$ in.

The distance between the center of gravity of the 10 harped strands and the top of the beam at the transfer length section is: $6 + \dfrac{(72-12-6)}{35.5}(2.08) = 9.17$ in.

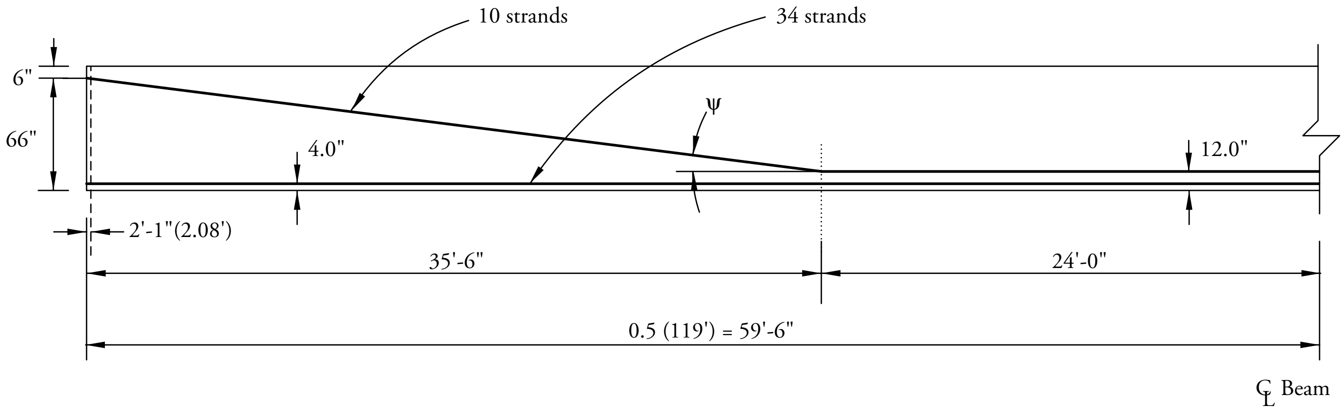
The distance between the center of gravity of the 34 straight strands and the bottom of the beam at all locations is: $\dfrac{12(2)+12(4)+8(6)+2(8)}{34} = 4.00$ in.

The distance between the center of gravity of all the strands and the bottom of the beam:

at end of beam, is: $\dfrac{34(4)+10(72-6)}{44} = 18.09$ in.

at transfer length section, is: $\dfrac{34(4)+10(72-9.17)}{44} = 17.37$ in.

Figure 9.5.7.2-2 Longitudinal Strand Pattern



Eccentricity at the transfer length section, e = 36.60 − 17.37 = 19.23 in.

Recompute the concrete stresses at transfer length section with harped strands:

$$f_t = +\frac{1,240.7}{767} - \frac{1,240.7(19.23)}{15,421} + \frac{97.3(12)}{15,421}$$

$$= +1.618 - 1.547 + 0.076 = +0.147 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

Note: Since the top fiber stress is smaller than $3\sqrt{f'_{ci}}$, there is no need for additional bonded reinforcement.

$$f_b = +\frac{1,240.7}{767} + \frac{1,240.7(19.23)}{14,915} - \frac{97.3\,(12)}{14,915}$$

$$= +1.618 + 1.600 - 0.078 = +3.140 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

9.5.7.3 Stresses at Harp Points

The eccentricity of strands at the harp points is the same as at midspan.

Bending moment at the harp point (0.3L) due to beam self-weight, is calculated using Equation (9.5.4.1.2-2) based on overall length.

Therefore, M_g = 0.5(0.799)(35.5)(119 − 35.5) = 1,184.2 ft-kips

$$f_t = +\frac{1,240.7}{767} - \frac{1,240.7(30.78)}{15,421} + \frac{1,184.2(12)}{15,421}$$

$$= +1.618 - 2.476 + 0.921 = +0.063 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

$$f_b = +\frac{1,240.7}{767} + \frac{1,240.7(30.78)}{14,915} - \frac{1,184.2\,(12)}{14,915}$$

$$= +1.618 + 2.560 - 0.953 = +3.225 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

9.5.7.4 Stresses at Midspan

The bending moment at midspan due to beam self-weight is calculated using Equation (9.5.4.1.2-2) based on overall length.

Therefore, $M_g = 0.5(0.799)(59.5)(119 - 59.5) = 1{,}414.3$ ft-kips

$$f_t = +\frac{1{,}240.7}{767} - \frac{1{,}240.7(30.78)}{15{,}421} + \frac{1{,}414.3(12)}{15{,}421}$$

$$= +1.618 - 2.476 + 1.101 = +0.243 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

$$f_b = +\frac{1{,}240.7}{767} + \frac{1{,}240.7(30.78)}{14{,}915} - \frac{1{,}414.3(12)}{14{,}915}$$

$$= +1.618 + 2.560 - 1.138 = +3.040 \text{ ksi}$$

Allowable compression: +3.300 ksi O.K.

Note: Stresses at harp points are more critical than stresses at midspan.

9.5.7.5 Hold-Down Forces

Assume the maximum initial prestress before allowing for any losses:

$0.80f_s^* = 0.80(270) = 216$ ksi

Initial prestress force per strand before losses = 0.153(216) = 33.0 kips

Hold-down force/strand equals $\frac{54}{35.5(12)}$(33.0 kips/strand)(1.05) = 4.39 kips/strand

Note: The factor 1.05 is applied to account for friction.

Total force = 10 strands(4.39) = 43.9 kips

$$\psi = \tan^{-1}\left(\frac{54}{35.5\,(12)}\right) = 7.22°$$

where 54 in. is the vertical strand shift in a horizontal distance of 35.5 ft.

The hold-down force and the harp angle should be checked against maximum limits for local practices. Refer to Chapter 3, Fabrication and Construction, Section 3.3.2.2 and Chapter 8, Design Theory and Procedures, for additional details.

9.5.7.6 Summary of Stresses at Transfer

	Top of Beam f_t (ksi)	Bottom of Beam f_b (ksi)
At transfer length section	+0.147	+3.140
At harp points	+0.063	+3.225
At midspan	+0.243	+3.040

9.5.8 CONCRETE STRESSES AT SERVICE LOADS

9.5.8.1 Allowable Stress Limits

[STD Art. 9.15.2.2]

Compression:

Case (I): for all load combinations

$0.60f'_c = 0.60(7{,}000)/1{,}000 = +4.200$ ksi (for precast beam)

$0.60f'_c = 0.60(4{,}000)/1{,}000 = +2.400$ ksi (for deck)

Case (II): for prestressing force + permanent dead loads

Case (III): live load plus 1/2 (prestressing force + dead loads)

$0.40f'_c = 0.40(7{,}000)/1{,}000 = +2.800$ ksi

Tension: $6\sqrt{f'_c} = 6\sqrt{7{,}000}/1{,}000 = -0.502$ ksi

9.5.8.2 Stresses at Midspan

Bending moment values at this section are given in **Table 9.5.4.1.2-1**.

Compute concrete stresses at the top of the beam:

$$f_t = +\frac{P_{se}}{A} - \frac{P_{se}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_b + M_{ws}}{S_{tg}} + \frac{M_{LL+I}}{S_{tg}}$$

Case (I):

$$f_t = +\frac{1{,}026.6}{767} - \frac{1{,}026.6(30.78)}{15{,}421} + \frac{(1{,}390.7 + 2{,}126.9)(12)}{15{,}421} + \frac{(73.0 + 128.0)(12)}{73{,}415}$$

$$+\frac{1{,}618.0(12)}{73{,}415} = +1.338 - 2.049 + 2.737 + 0.033 + 0.264 = +2.323 \text{ ksi}$$

Allowable compression: +4.200 ksi O.K.

Case (II):

$f_t = +1.338 - 2.049 + 2.737 + 0.033 = +2.059$

Allowable compression: +2.800 ksi O.K.

Case (III):

$f_t = +0.264 + 0.5(+1.338 - 2.049 + 2.737 + 0.033) = +1.294$ ksi

Allowable compression: +2.800 ksi O.K.

Compute concrete stresses at the bottom of the beam:

$$f_b = +\frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_b + M_{ws}}{S_{bc}} - \frac{M_{LL+I}}{S_{bc}}$$

$$= +\frac{1{,}026.6}{767} + \frac{1{,}026.6(30.78)}{14{,}915} - \frac{(1{,}390.7 + 2{,}126.9)(12)}{14{,}915} - \frac{(73 + 128)(12)}{20{,}389}$$

$$-\frac{1{,}618.0(12)}{20{,}389} = +1.338 + 2.119 - 2.830 - 0.118 - 0.952 = -0.443 \text{ ksi}$$

Allowable tension: −0.502 ksi O.K.

Compute stresses at the top of the deck slab.

Compressive stress in the deck slab at service loads never controls the design for typical applications. The calculations shown below are for illustrative purposes and may not be necessary in most practical applications.

Stresses in the deck slab result from the loads applied on the composite section, such as wearing surface, barrier weight and live loads. Since section properties are calculated based on beam strength, calculated stresses are transformed using the modular ratio between deck slab and beam materials, n, which is included in S_{tc}.

$$f_t = \frac{M_b + M_{ws}}{S_{tc}} + \frac{M_{LL+I}}{S_{tc}} = \frac{(73.0+128.0)(12)}{64{,}269} + \frac{(1{,}168.0)(12)}{64{,}629}$$

$= 0.038 + 0.217 = + 0.255$ ksi;

Allowable compression: + 2.400 ksi O.K.

Case (II):

$f_t = + 0.038$ ksi

Allowable compression: +1.600 ksi O.K.

Case (III):

$f_t = 0.5(0.038) + 0.217 = + 0.236$ ksi

Allowable compression: +1.600 ksi O.K.

9.5.8.3 Summary of Stresses at Service Loads

	Top of Deck f_t (ksi)	Top of Beam f_t (ksi)	Bottom of Beam f_b (ksi)
At midspan	+0.255	+2.323	–0.443

9.5.9 FLEXURAL STRENGTH

[STD Art. 9.17]

9.5.9.1 Positive Moment Section

Using the Group I loading combination given in Section 9.5.4.3, the factored moment at midspan of the center span is:

$M_u = 1.3[M_g + M_s + M_b + M_{ws} + 1.67(M_{LL+I})]$ [STD Table 3.22.1A]

$= 1.3[1{,}390.7 + 2{,}126.9 + 73.0 + 128.0 + 1.67(1{,}618.0)] = 8{,}346.9$ ft-kips

Compute the average stress in the prestressing steel at ultimate load, f^*_{su}:

$$f^*_{su} = \left(1 - \frac{\gamma^*}{\beta_1}\rho^*\frac{f'_s}{f'_c}\right)$$ [STD Eq. 9-17]

where

f^*_{su} = average stress in prestressing steel at ultimate load

γ^* = 0.28 for low-relaxation strand [STD Art. 9.1.2]

$\beta_1 = 0.85 - 0.05\dfrac{(f'_c - 4{,}000)}{1{,}000} \geq 0.65$ when $f'_c > 4{,}000$ psi [STD Art. 8.16.2.7]

$= 0.85 - 0.05\dfrac{(4{,}000 - 4{,}000)}{1{,}000} = 0.85$

$$\rho^* = \frac{A_s^*}{bd}$$

where

A^* = area of prestressed reinforcement = 44(0.153) = 6.732 in.^{2}

b = effective flange width = 132.0 in.

d = distance from top of deck to centroid of prestressing strands

= beam depth (h) + haunch + deck slab thickness − y_{bs}

= 72 + 0.5 + 7.5 − 5.82 = 74.18 in.

where y_{bs} = distance from center of gravity of the strands to the bottom fiber of the beam = 5.82 in.

$$\rho^* = \frac{6.732}{132(74.18)} = 0.000688$$

$$f_{su}^* = 270\left[1-\left(\frac{0.28}{0.85}\right)(0.000688)\left(\frac{270}{4}\right)\right] = 265.9 \text{ ksi}$$

Compute the depth of the compression block:

$$a = \frac{A_s^* f_{su}^*}{0.85 f_c' b} = \frac{(6.732)(265.9)}{0.85(4.0)132} = 3.99 \text{ in.} < 7.5 \text{ in.}$$ [STD Art. 9.17.2]

Since the depth of the compression block is within the deck thickness, this section is considered a rectangular section.

Design flexural strength of a rectangular section:

$$\phi M_n = \phi A_s^* f_{su}^* d\left(1 - 0.6\frac{\rho^* f_{su}^*}{f_c'}\right)$$ [STD Eq. 9-13]

where

ϕ = strength reduction factor = 1.0 [STD Art. 9.14]

M_n = nominal moment strength of a section

$$\phi M_n = 1.0(6.732)(265.9)(74.18)\left(1-0.6\,\frac{0.000688\,(265.9)}{4}\right)/12$$

$$= 10{,}761.8 \text{ ft-kips} > M_u = 8{,}346.9 \text{ ft-kips}$$

9.5.9.2 Negative Moment Section

[STD Art. 9.7.2.3]

For live load continuity at interior supports, provide negative moment reinforcement within the cast-in-place deck. Use Grade 60 deformed reinforcing bars. The negative moment section at the centerline of the pier is considered a non-pretensioned, reinforced concrete section using the compressive strength of the beam concrete. [STD Art. 9.7.2.3.2]

Tensile stresses in the top fibers of the cast-in-place deck need not be limited because the deck is being treated as a conventionally reinforced member which may be cracked under full service load. However, crack control and fatigue criteria must be satisfied.

$$M_u = 1.3[M_g + M_s + M_b + M_{ws} + 1.67(M_{LL+I})]$$ [STD Table 3.22.1A]

$$= 1.3[0 + 0 - 197 - 345.0 + 1.67(-1{,}777.0)] = -4{,}562.5 \text{ ft-kips}$$

Assume center of gravity of the negative moment tension steel occurs at mid-depth of slab.

$$d = 72 + 0.5(\text{haunch}) + \frac{7.5}{2} = 76.25 \text{ in.}$$

Provide steel in the deck to resist the total factored negative moment.

$$R_u = \frac{M_u}{\phi b d^2} = \frac{4{,}562.5(12)(1{,}000)}{0.90(26)(76.25)^2} = 402 \text{ psi}$$

where

M_u = total factored negative moment = 4,562.5 ft-kips

b = width of bottom flange of the beam = 26.00 in.

ϕ = 0.90 for reinforced concrete [STD Art 8.16.1.2.2]

d = distance from extreme compression fiber to centroid of the negative moment reinforcing = 76.25 in.

$$\rho = \frac{1}{m}\left[1 - \sqrt{(1 - \frac{2mR_u}{f_{sy}}}\right] = \frac{1}{10.08}\left[1 - \sqrt{(1 - \frac{2(10.08)(402)}{60{,}000}}\right] = 0.00694$$

where

f_{sy} = yield stress of nonprestressed conventional reinforcement in tension = 60 ksi

f'_c = compressive concrete strength at 28 days for the beam = 7.0 ksi

$$m = \frac{f_s}{0.85f'_c} = \frac{60}{0.85(7.0)} = 10.08$$

Required A_s = area of nonpretensioned tension reinforcement = ρbd
= 0.00694(26)(76.25) = 13.76 in.^{2}

This is the total amount of mild reinforcement required in the slab to resist the negative moment. Assume that the typical slab reinforcement consists of a bottom mat of #5 bars @ 12 in. on-center and a top mat of #4 bars @ 12 in. on-center for total A_s = 0.31 + 0.20 = 0.51 in.^{2}/ft.

Since the *Standard Specifications* do not provide guidance on the width over which this reinforcement is to be distributed, it is assumed here to be the same as the effective compression flange width which was determined earlier to be 132 in.

The typical reinforcement provided over this width is equal to (132)(0.51)/12 = 5.61 in.^{2}. Therefore, the required additional reinforcement at the negative moment section equals 13.78 − 5.61 = 8.17 in.^{2}.

Add 2 # 6 bars between the top # 4 bars

A_s = 2(0.44)(132)/12 = 9.68 in.^{2}

Therefore, total A_s provided = 9.68 + 5.61 = 15.29 in.^{2} > required A_s = 13.76 in.^{2}

Compute the capacity of the section in flexure at the pier: [STD Art. 8.16.3.2]

Compute the depth of the compression block:

$$a = \frac{A_s f_{sy}}{0.85 f'_c b} \quad \text{[STD Eq. 8-17]}$$

$$= \frac{15.29(60)}{0.85(7)(26)} = 5.93 \text{ in.} < \text{thickness of bottom flange} = 6.0 \text{ in.}$$

Since the depth of the compression block is within the bottom flange thickness, the section is considered a rectangular section.

The design moment strength, ϕM_n, may be computed by:

$$\phi M_n = \phi A_s f_s\left(d - \frac{a}{2}\right) \quad \text{[STD Eq. 8-16]}$$

$$\phi M_n = 0.90(15.29)(60)\left(76.25 - \frac{5.93}{2}\right)/12$$

$$= 5{,}042.4 \text{ ft-kips} > M_u = 4{,}562.4 \text{ ft-kips} \quad \text{O.K.}$$

With time, concrete members that are heavily pretensioned will creep. This may cause camber growth. Because this bridge is designed to have rigid connections between beams at the piers, camber growth is restrained. As a result, time-dependent positive moments will develop. Therefore, it is recommended that a nominal amount of positive moment continuity reinforcement be used over the piers to control potential cracking in this region. A common way to provide this reinforcement is to extend approximately 25 percent of the strands from the bottom flange and bend them up into the diaphragm. Another common detail is the addition of a quantity of mild steel reinforcement required to resist a moment equal to $1.2M_{cr}$. This reinforcement is also extended from the ends of the beam and bent up into the diaphragm.

9.5.10 DUCTILITY LIMITS

9.5.10.1 Positive Moment Section

9.5.10.1.1 Maximum Reinforcement

[STD Art. 9.18.1]

Pretensioned concrete members are designed so that the steel yields as ultimate capacity is approached.

Reinforcement index for rectangular section:

$$\rho^* \frac{f^*_{su}}{f'_c} < 0.36\beta_1 \quad \text{[STD Eq. 9-20]}$$

$$= 0.000688\left(\frac{265.9}{4}\right) = 0.0457 < 0.36(0.85) = 0.306 \quad \text{O.K.}$$

9.5.10.1.2 Minimum Reinforcement

[STD Art. 9.18.2]

The total amount of pretensioned reinforcement should be adequate to develop an ultimate moment at the critical section of at least 1.2 times the cracking moment, M^*_{cr}:

$$\phi M_n \geq 1.2M^*_{cr}$$

Compute the cracking moment:

$$M^*_{cr} = (f_r + f_{pe})S_{bc} - M_{d/nc}\left(\frac{S_{bc}}{S_b} - 1\right) \quad \text{[STD Art. 9.18.2.1]}$$

Note: The variable S_{bc} used here is equivalent to S_c in the *Standard Specifications*.

where

f_r = modulus of rupture [STD Art. 9.15.2.3]

$$= 7.5\sqrt{f'_c} = 7.5\sqrt{7{,}000}\left(\frac{1}{1{,}000}\right) = 0.627 \text{ ksi}$$

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

where

P_{se} = effective pretension forces after allowing all losses = 1,026.6 kips

e_c = 30.78 in.

$$f_{pe} = \frac{1{,}026.6}{767} + \frac{1{,}026.6(30.78)}{14{,}915} = 1.338 + 2.119 = 3.457 \text{ ksi}$$

$M_{d/nc}$ = non-composite dead load moment at midspan due to self-weight of beam and weight of slab = 1,390.7 + 2,126.9 = 3,517.6 ft-kips

$$M^*_{cr} = (0.627 + 3.457)\left(\frac{20{,}389}{12}\right) - 3{,}517.6\left(\frac{20{,}389}{14{,}915} - 1\right) = 5{,}648 \text{ ft-kips}$$

$1.2M^*_{cr}$ = 6,777.7 ft-kips < ϕM_n = 10,761.8 ft-kips O.K.

Contrary to the *LRFD Specifications*, the *Standard Specifications* indicate that this requirement must be satisfied only at the critical sections.

9.5.10.2 Negative Moment Section

9.5.10.2.1 Maximum Reinforcement

[STD Art. 8.16.3.1]

$$\rho_{actual} = \frac{15.29}{26(76.25)} = 0.00771$$

$$\rho_b = \frac{0.85\,\beta_1 f'_c}{f_s}\left(\frac{87{,}000}{87{,}000 + f_s}\right)$$ [STD Eq. 8-18]

where $\beta_1 = 0.85 - 0.05\dfrac{(7{,}000 - 4{,}000)}{1{,}000} = 0.70$ [STD Art. 8.16.2.7]

$$\rho_b = \frac{0.85(0.70)(7)}{60}\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right) = 0.04108$$

ρ_{max} = 0.75(0.04108) = 0.03081 > ρ_{actual} = 0.00771 O.K.

9.5.10.2.2 Minimum Reinforcement

The total amount of non-pretensioned reinforcement should be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment, M^*_{cr}:

$\phi M_n \geq 1.2M^*_{cr}$

Calculate M^*_{cr}: [STD Art. 8.17.1.1]

Modulus of rupture: [STD Art. 8.15.2.1.1]

$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4{,}000}\left(\dfrac{1}{1{,}000}\right) = 0.474$ ksi

$M^*_{cr} = S_{tc}f_r = \dfrac{64{,}269(0.474)}{12} = 2{,}538.6$ ft-kips

$1.2M^*_{cr} = 3{,}046.4$ ft-kips $< M_u = 4{,}562.5$ ft-kips

9.5.11 SHEAR DESIGN

[STD Art. 9.20]

Pretensioned members subject to shear should be designed so that:

$V_u \le \phi(V_c + V_s)$ [STD Eq. 9-26]

where

V_u = the factored shear force at the section considered

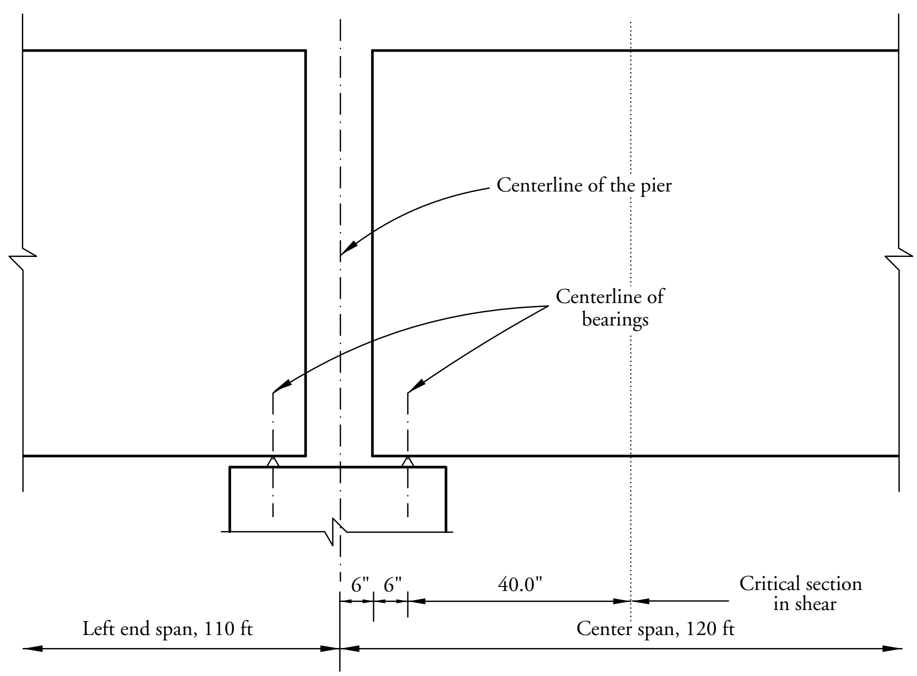
V_c = the nominal shear strength provided by concrete

V_s = the nominal shear strength provided by web reinforcement

ϕ = strength reduction factor for shear = 0.90 [STD Art. 9.14]

The critical section in pretensioned concrete beams, according to the *Standard Specifications*, Article 9.20.1.4, is located at a distance h/2 from the face of the support. In this example, the critical section for shear will be conservatively calculated from the centerline of support, as shown in **Figure 9.5.11-1**. The width of the bearing has not yet been determined and will be conservatively assumed to be zero. Therefore, the detailed calculations shown here are for the section at h/2 at a point 12 + 80/2 = 52 in. (4.33 ft.) from the centerline of the pier. The following calculations demonstrate the procedures used in common practice to compute V_{ci} and V_{cw} at this location.

Figure 9.5.11-1
Critical Section in Shear of the Center Span



Compute V_{ci} at the critical section in the center span:

$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \dfrac{V_i M_{cr}}{M_{max}}$ [STD Eq. 9-27]

where

b' = width of the web of a flanged member = 6.00 in.

f'_c = compressive strength of beam concrete at service loads = 7,000 psi

V_d = total dead load at the section under consideration (**Table 9.5.4.1.2-1**)

$= V_g + V_S + V_b + V_{ws} = 44.5 + 68.0 + 8.3 + 14.9 = 135.7$ kips

V_{LL+I} = unfactored shear force at section due to live load + impact = 87.0 kips

M_d = bending moment at section under consideration due to unfactored dead loads = 152.7 + 233.5 − 162.0 − 283.6 = −59.4 ft-kips

M_{LL+I} = live load bending moment plus impact = −1,506.9 ft-kips

V_u = factored shear force at the section = $1.3(V_d + 1.67V_{LL+I})$

$= 1.3[135.7 + 1.67(87.0)] = 365.3$ kips

M_u = factored bending moment at the section = $1.3(M_d + 1.67M_{LL+I})$

$= 1.3[-59.4 + 1.67(-1{,}506.9)] = -3{,}348.7$ ft-kips

V_{mu} = ultimate shear force occurring simultaneously with M_u. Conservatively, use the maximum shear load occurring at this section = 365.3 kips

M_{max} = maximum factored moment at section due to externally applied loads

$= M_u - M_d = -3{,}348.7 - (-59.4) = -3{,}289.3$ ft-kips

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

$= V_{mu} - V_d = 365.3 - 135.7 = 229.6$ kips

f_{pe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at the extreme fiber of the section where tensile stress is caused by externally applied loads.

The beam at this section is under net negative moment. Thus, f_{pe} should be evaluated at the top of the deck slab. Pretension has no effect on the deck slab, therefore, $f_{pe} = 0$

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \left(\frac{M_b + M_{ws}}{S_{tc}}\right) = \frac{(-162.0 - 283.6)(12)}{64{,}269} = 0.083 \text{ ksi}$$

f_r = the modulus of rupture of concrete

$$= 6\sqrt{f'_c} = 6\sqrt{4{,}000}\left(\frac{1}{1{,}000}\right) = 0.379 \text{ ksi}$$

M_{cr} = moment causing flexural cracking of section due to externally applied loads (after dead load)

$$= \left(6\sqrt{f'_c} + f_{pe} - f_d\right)S_{tc} \qquad \text{[STD Eq 9-28]}$$

$$= (0.379 + 0 - 0.083)\,\frac{64{,}269}{12} = 1{,}585.3 \text{ ft-kips}$$

d = distance from extreme compression fiber to centroid of pretensioned reinforcement. But d need not be taken less than $0.8h_c$ = 64.00 in. [STD Art. 9.20.2.2]. Negative moment reinforcement is assumed to be located at the mid-height of the deck, so d = 76.25 in.

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$ [STD Eq. 9-27]

$$= \frac{0.6\sqrt{7{,}000}(6)(76.25)}{1{,}000} + 135.7 + \frac{181.3(1{,}585.3)}{3{,}289.3} = 246.0 \text{ kips}$$

This value should not be less than:

Minimum $V_{ci} = 1.7\sqrt{f'_c}\, b'd$ [STD Art. 9.20.2.2]

$$= \frac{1.7\sqrt{7{,}000}(6.00)(76.25)}{1{,}000} = 65.1 \text{ kips} < V_{ci} = 246.0 \text{ kips} \quad \text{O.K.}$$

Compute V_{cw}: [STD Art 9.20.2.3]

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b'd + V_p$$ [STD Eq. 9-29]

where

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross-section resisting externally applied loads. This stress is due to both pretensioning and moments resisted by precast member acting alone.

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se}e(y_{bc} - y_b)}{I} + \frac{(M_g + M_s)(y_{bc} - y_b)}{I}$$

Compute eccentricity of strands, e, at h/2:

h/2 from centerline of support: 4.33 + 0.5 = 4.83 ft from end of beam

Center of gravity of the 10 harped strands at 4.83 ft from end of beam:

$$6 \text{ in.} + \frac{(72 \text{ in.} - 12 \text{ in.} - 6 \text{ in.})}{35.5 \text{ ft}}(4.83 \text{ ft}) = 13.35 \text{ in.}$$

Center of gravity of strand group at 4.83 ft from end of beam:

$$\frac{34(4) + 10(72 - 13.35)}{44} = 16.42 \text{ in.}$$

e = 36.60 – 16.42 = 20.18 in.

$$f_{pc} = \frac{1{,}026.6}{767} - \frac{1{,}026.6(20.18)(56.35 - 36.60)}{545{,}894} + \frac{386.2(12)(56.35 - 36.60)}{545{,}894}$$

= 1.338 – 0.750 + 0.168 = 0.756 ksi

V_p = vertical component of prestress force for harped strands

= P_{se} sin ψ (see Section 9.5.7.5 for calculations of ψ)

= 233.3sin(7.22°) = 29.3 kips

where

P_{se} = the effective prestress force for the harped strands

= 10(0.153)(152.5) = 233.3 kips

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b'd + V_p$$

$$= \left[3.5\sqrt{7,000} + 0.3(0.756)\right]\frac{6.00(76.25)}{1,000} + 29.3 = 267.0 \text{ kips}$$

The allowable nominal shear provided by concrete should be the lesser of V_{ci} (246.0 kips) and V_{cw} (267.0 kips), so V_{ci} governs.

$V_c = 246.0$ kips

$V_u < \phi(V_c + V_s)$ [STD Eq. 9-26]

where

V_s = nominal shear strength provided by shear reinforcement

ϕ = strength reduction factor for shear = 0.90 [STD Art. 9.14]

$$V_s = \frac{V_u}{\phi} - V_c = \frac{365.3}{0.90} - 246.0 = 159.9 \text{ kips}$$

The maximum shear force that may be carried by reinforcement:

Maximum $V_s = 8\sqrt{f'_c}\,b'd$ [STD Art. 9.20.3.1]

$$= 8\sqrt{7,000}\,\frac{6.00(76.25)}{1,000} = 306.2 \text{ kips} > \text{required } V_s = 159.9 \text{ kips} \quad \text{O.K.}$$

Compute area of shear steel required: [STD Art. 9.20.3.1]

$$V_s = \frac{A_v f_y d}{s}$$ [STD Eq. 9-30]

$$A_v = \frac{V_s\, s}{f_y\, d}$$

where

A_v = area of web reinforcement, in.^{2}

s = longitudinal spacing of the web reinforcement, in.

Set s = 12 in. to have units of in.^{2}/ft for A_v:

$$A_v = \frac{159.9(12)}{(60)(76.25)} = 0.42 \text{ in.}^2\text{/ft.}$$

Minimum shear reinforcement: [STD Art. 9.20.3.3]

$A_{v\text{-min}}$ = minimum area of web reinforcement

$$A_{v\text{-min}} = \frac{50\,b's}{f_y} = \frac{50(6)(12)}{60,000} = 0.06 \text{ in.}^2\text{/ft}$$ [STD Eq. 9-31]

The required shear reinforcement is the maximum of A_v (0.42 in.^{2}/ft) and $A_{v\text{-min}}$ (0.06 in.^{2}/ft).

Since $V_s = 159.9 \text{ kips} > 4\sqrt{f'_c}\,b'd = \dfrac{4\sqrt{7,000}(6.00)(76.25)}{1,000} = 153.1 \text{ kips}$,

then the maximum spacing of stirrups must be reduced by one-half. [STD Art. 9.20.3.2]

Maximum spacing = 0.75h = 0.75(72 + 7.5 + 0.5) = 60 in. or

= 24.00 in. (controls)

Therefore, maximum $s = \dfrac{24}{2} = 12$ in.

Use #4, two-legged bar stirrups with 10 in. spacing ($A_v = 0.48$ in.^{2}/ft)

The calculations above must be repeated along the entire length of the beam at regular intervals to determine the area and spacing of shear reinforcement.

9.5.12 HORIZONTAL SHEAR DESIGN

[STD Art. 9.20.4]

The *Standard Specifications* do not identify the location of the critical section. For convenience, it will be assumed here to be at the same location as the critical section for vertical shear.

$V_u = 365.3$ kips

$V_u \leq V_{nh}$ [STD Eq. 9-31a]

V_{nh} is nominal horizontal shear strength

$$V_{nh} \geq \frac{V_u}{\phi} = \frac{365.3}{0.9} = 405.9 \text{ kips}$$

Case (a&b): Contact surface is roughened or when minimum ties are used.

[STD Art. 9.20.4.3]

Allowable shear force:

$V_{nh} = 80b_v d$

where

b_v = width of cross-section at the contact surface being investigated for horizontal shear = 42.0 in.

d = distance from extreme compression fiber to centroid of the tension reinforcement = 76.25 in.

$V_{nh} = 80(42)(76.25)/1{,}000 = 256.2$ kips < 405.9 kips N.G.

Case (c): Minimum ties provided and contact surface roughened. [STD Art. 9.20.4.3]

Allowable shear force:

$V_{nh} = 350b_v d = 350(42)(76.25)/1{,}000 = 1{,}120.9$ kips > 405.9 kips O.K.

Determine required stirrups for horizontal shear: [STD Art. 9.20.4.5]

$$\text{Minimum } A_{vh} = 50\frac{b_v s}{f_y} = 50\frac{42(12)}{60{,}000} = 0.42 \text{ in.}^2\text{/ft}$$

The required minimum horizontal shear reinforcement, $A_{vh} = 0.42$ in.^{2}/ft, is less than the provided vertical shear reinforcement, $A_v = 0.48$ in.^{2}/ft. O.K.

Maximum spacing equals $4b' = 4(6) = 24.00$ in [STD Art. 9.20.4.5.a]

or = 24.00 in.

Therefore, s = 24.00 in.

9.5.13 PRETENSIONED ANCHORAGE ZONE

[STD Art. 9.22]

9.5.13.1 Minimum Vertical Reinforcement

In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi used to resist at least 4% of the total prestressing force, are placed within the distance of d/4 of the end of the beam. [STD Art. 9.22.1]

Minimum stirrups at the each end of the beam:

P_s = prestress force before initial losses equals

= 44(0.153)(202.5) = 1,363.2 kips

4% P_s = 0.04(1,363.2) = 54.5 kips

$$\text{Required } A_v = \frac{54.5}{20} = 2.73 \text{ in.}^2$$

Placed at a distance = 76.25/4 = 19 in. from the end of beam.

The shear reinforcement was determined in Section 9.5.11 to be #4 @ 10 in. However, the minimum vertical reinforcement criteria controls. Therefore, for a distance of 20 in. from the end of the member, use 5 #5 @ 4 in. The reinforcement provided is 5(2)(0.31) = 3.10 in.^{2}

Use 5 pairs of #5 bars at 4 in. spacing (Av = 3.10 in.2)

9.5.13.2 Confinement Reinforcement

[STD Art. 9.22.2]

For at least a distance d from the end of the beam, nominal reinforcement shall be provided to enclose the prestressing steel in the bottom flange.

9.5.14 DEFLECTION AND CAMBER

9.5.14.1 Deflection Due to Pretensioning Force at Transfer

For the harped strands:

$$\Delta_p = \frac{P_{si}}{E_{ci}I}\left(\frac{e_c L^2}{8} - \frac{e' a^2}{6}\right)$$

where

Δ_p = camber due to prestress force at transfer, in.

P_{si} = total prestress force after allowing initial losses = 1,026.6 kips

I = moment of inertia of non-composite section = 545,894 in.^{4}

L = overall beam length = 119.0 ft

E_{ci} = modulus of elasticity of the beam concrete at release = 4,496 ksi

e_c = eccentricity of pretensioning force at the midspan = 30.78 in.

e′ = difference between eccentricity of pretensioning steel at midspan and endspan = $e_c - e_e$ = 30.78 − (36.60 − 18.09) = 12.27 in.

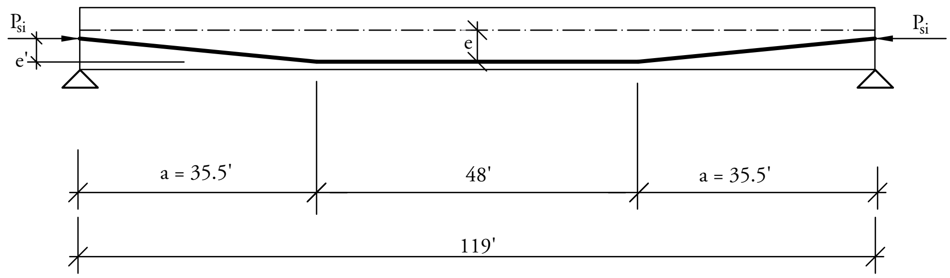
a = distance from the end of beam to harp point = 35.50 ft

(Note: see **Figure 9.5.14.1-1** for strand eccentricity)

$$\Delta_p = \frac{1{,}026.6}{(4{,}496)(545{,}894)}\left(\frac{(30.78)[(121)(12)]^2}{8} - \frac{(12.27)[(35.5)(12)]^2}{6}\right) = 3.24 \text{ in.} \uparrow$$

BULB-TEE (BT-72), THREE SPANS, COMPOSITE DECK STANDARD SPECIFICATIONS

Figure 9.5.14.1-1
Strand Eccentricity



9.5.14.2 Deflection Due to Beam Self-Weight

$$\Delta_{beam} = \frac{5\,w_g L^4}{384 E_{ci} I}$$

where w_g = beam weight = 0.799 kips/ft

Deflection due to beam self-weight at transfer (L = 119 ft):

$$= \frac{5(0.799/12)[(119)(12)]^4}{(384)(4,496)(545,894)} = 1.47 \text{ in.}\downarrow$$

Deflection due to beam self-weight, used to compute deflection at erection (L = 118 ft):

$$= \frac{5(0.799/12)[(118)(12)]^4}{(384)(4,496)(545,894)} = 1.42 \text{ in.}\downarrow$$

9.5.14.3 Total Initial Deflection

Total deflection at transfer is 3.24 – 1.47 = 1.77 in. ↑

Total deflection at erection, using PCI multipliers (see PCI Design Handbook):

1.80(3.24) – 1.85(1.42) = 3.21 in. ↑

PCI multipliers used for long-term deflection for commercial construction have proven not to be accurate for bridge construction. Therefore, it is recommend that the designer contact the owner agency of the bridge for further guidelines. Frequently, no additional multipliers are used.

9.5.14.4 Deflection Due to Deck and Haunch Weights

$$\Delta_{slab} = \frac{5\,w_s L^4}{384\,E_c I}$$

where

w_s = slab weight + haunch weight = 0.922 kips/ft

E_c = modulus of elasticity of the beam concrete at service = 5,072 ksi

$$\Delta_{slab} = \frac{5(0.922/12)[(118)(12)]^4}{(384)(5{,}072)(545{,}894)} = 1.45 \text{ in.}\downarrow$$

9.5.14.5 Deflection Due to Barrier and Wearing Surface Weights

From continuous beam analysis:

$\Delta_{SDL} = 0.04$ in. $\downarrow$

9.5.14.6 Deflection Due to Live Load and Impact

Live load deflection is not a required check according to the provisions of the *Standard Specifications*. It is usually not a concern for pretensioned concrete I-beam shapes especially when constructed to act as a continuous structure under super-imposed loads.

Live load deflection limit (optional): Span/800

$\Delta_{max} = 120(12)/800 = 1.80$ in.

Some state DOTs consider that all beams act together in resisting deflection due to live load and impact. Using that assumption, the distribution factor for deflection is:

$$\frac{\text{number of lanes}}{\text{number of beams}} = \frac{4}{6} = 0.667 \text{ lanes}$$

However, it is more conservative to use the distribution factor for moment.

The live load deflection may be conservatively estimated using the following formula:

$$\Delta = \frac{5L^2}{48EI_c}[M_s - 0.1(M_a + M_b)] \qquad \text{(Eq. 9.5.14.6-1)}$$

where

M_s = the maximum positive moment

M_a & M_b = the corresponding negative moments at the ends of the span being considered

In this example, a conservative approximation may be made by using the maximum positive moment in the center span and by ignoring the effect of M_a and M_b.

$$\Delta_L = \frac{5[(120)(12)]^2}{48(5{,}072)(1{,}148{,}942)}[(1{,}618.0)(12)] = 0.72 \text{ in.}\downarrow < 1.80 \text{ in.} \quad \text{O.K.}$$

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

NOTATION
DESIGN EXAMPLES

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pb} = compressive stress at bottom fiber of the beam due to prestress force

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = the modulus of rupture of concrete [STD]

f_r = modulus of rupture of concrete [LRFD]

f_s = allowable stress in steel

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress

f_{si} = effective initial pretension stress

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

f_t = concrete stress at top fiber of the beam for the non-composite section

f_{tc} = concrete stress at top fiber of the slab for the composite section

f_{tg} = concrete stress at top fiber of the beam for the composite section

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity, percent [LRFD]

H = height of wall [LRFD]

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_c = total height of composite section
h_f = compression flange depth [LRFD]
I = moment of inertia about the centroid of the non-composite precast beam [STD]
I = moment of inertia about the centroid of the non-composite precast beam [LRFD]
I = impact fraction (maximum 30%) [STD]
I_c = moment of inertia of composite section
IM = dynamic load allowance [LRFD]
J = St. Venant torsional constant
K = longitudinal stiffness parameter [STD]
K_g = longitudinal stiffness parameter [LRFD]
k = factor used in calculation of distribution factor for multi-beam bridges [LRFD]
k = factor used in calculation of average stress in pretensioning steel for Strength Limit State
L = live load [STD]
L = length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment [STD]
L = overall beam length or design span
L = span length measured parallel to longitudinal beams [STD]
L = span length [LRFD]
L_c = critical length of yield line failure pattern [LRFD]
LL = vehicular live load [LRFD]
ℓ_d = development length [LRFD]
ℓ_x = length required to fully develop the strand measured from the end of the strand
M_a = negative moment at the end of the span being considered
M_b = negative moment at the end of the span being considered
M_b = unfactored bending moment due to barrier weight
M_c = flexural resistance of cantilevered wall [LRFD]
M_{CIP} = unfactored bending moment due to cast-in-place topping slab
M_{const} = unfactored bending moment due to construction load
M_{col} = bending moment due to horizontal collision force
M_{cr} = moment causing flexural cracking at section due to externally applied loads (after dead load) [STD]
M_{cr} = cracking moment [LRFD]
M^*_{cr} = cracking moment [STD]
M_D = unfactored bending moment due to diaphragm weight
M_d = bending moment at section due to unfactored dead load
$M_{d/nc}$ = moment due to non-composite dead loads [STD]
M_f = unfactored bending moment due to fatigue truck per beam
M_g = unfactored bending moment due to beam self-weight
M_{LL} = unfactored bending moment due to lane load per beam
M_{LL+I} = unfactored bending moment due to live load + impact
M_{LL+I} = unfactored bending moment due to design vehicular load
M_{LT} = unfactored bending moment due to truck load with dynamic allowance per beam

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	$= \dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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BULB-TEE (BT-72), THREE SPANS, COMPOSITE DECK, LRFD SPECIFICATIONS

Bulb-Tee (BT-72), Three Spans, Composite Deck, LRFD Specifications

9.6.1 INTRODUCTION

This design example demonstrates the design of a three-span (110-120-110 ft) AASHTO-PCI bulb-tee beam bridge with no skew, as shown in **Figure 9.6.1-1**. This example illustrates in detail the design of a typical interior beam in the center span at the critical sections in positive flexure, negative flexure, shear, and deflection due to prestress, dead loads and live load. The superstructure consists of four beams spaced at 12'-0" centers as shown in **Figure 9.6.1-2**. Beams are designed to act compositely with the 8-in.-thick cast-in-place concrete deck slab to resist all superimposed dead loads, live loads and impact. A 1/2 in. wearing surface is considered to be an integral part of the 8-in. deck. Design live load is AASHTO LRFD HL-93. The design will be carried out in accordance with the *AASHTO LRFD Bridge Design Specifications,* 2nd Edition, 1998, and including through the 2003 Interim Revisions.

Figure 9.6.1-1
Longitudinal Section

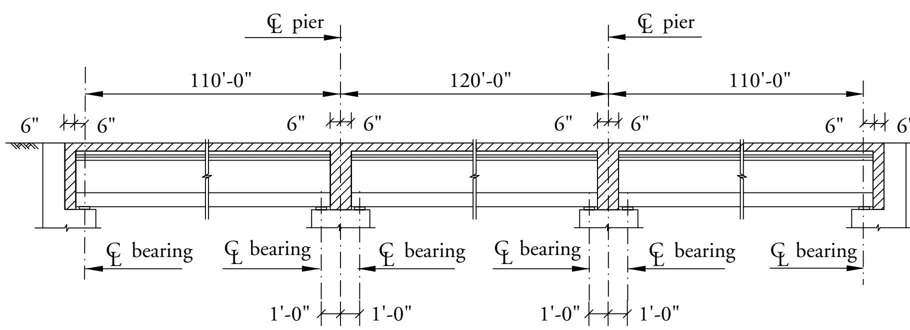
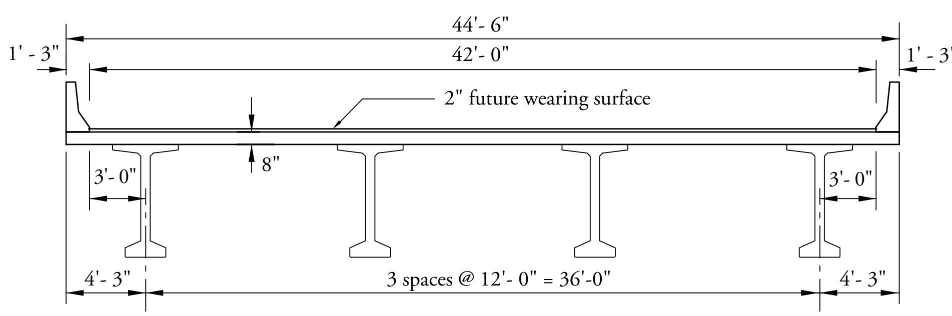


Figure 9.6.1-2
Cross-Section



9.6.2 MATERIALS

Cast-in-place slab: Actual thickness, t_s = 8.0 in.

Structural thickness = 7.5 in.

Note that a 1/2 in. wearing surface is considered to be an integral part of the 8-in. deck.

Concrete strength at 28 days, f'_c = 4.0 ksi

Concrete unit weight, w_c = 0.150 kcf

Precast beams: AASHTO-PCI, BT-72 bulb-tee beam shown in **Figure 9.6.2-1**.

Concrete strength at transfer, f'_{ci} = 5.5 ksi

Concrete strength at 28 days, f'_c = 7.0 ksi

Concrete unit weight, w_c = 0.150 kcf

Overall beam length (**Figure 9.6.1-1**) = 110.0 ft (end spans) and 119.0 ft (center span)

Design spans (**Figure 9.6.1-1**):

For non-composite beam: 109.0 ft (end spans) and 118.0 ft (center span)

For composite beam: 110.0 ft (end spans) and 120.0 ft (center span)

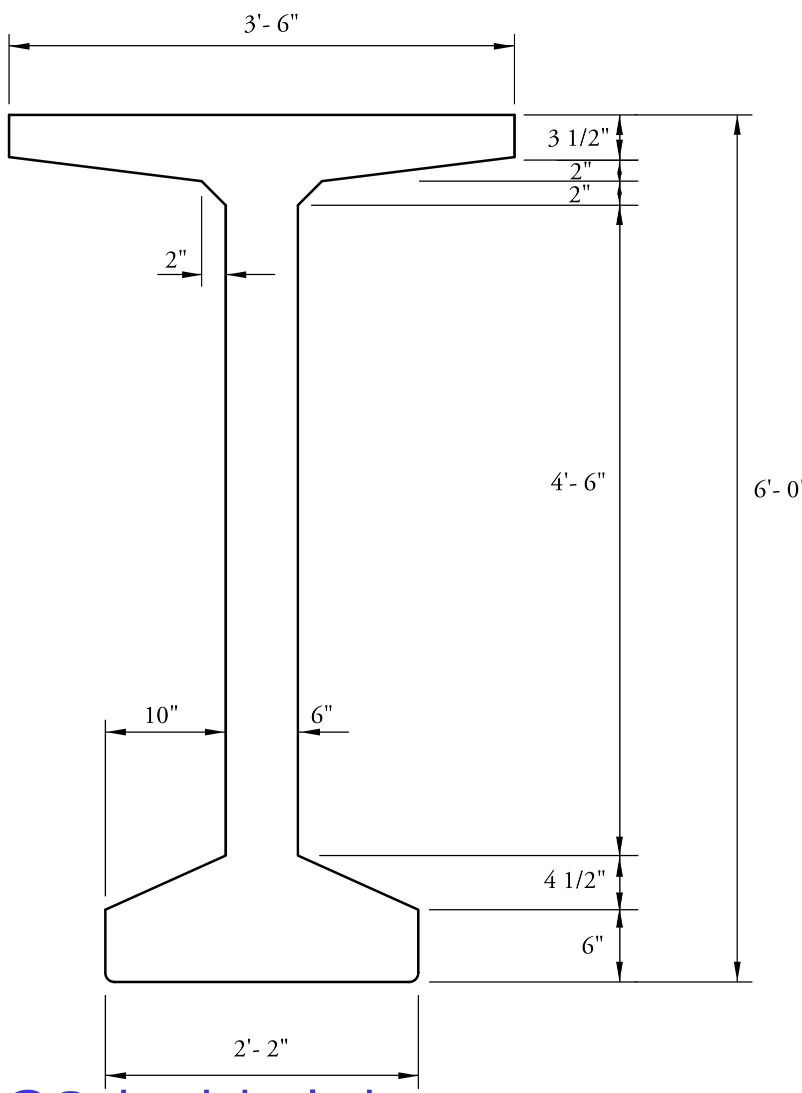


Figure 9.6.2-1
AASHTO-PCI Bulb-Tee, BT-72

Prestressing strands: 1/2 in. diameter, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

before transfer, $f_{pi} \leq 0.75f_{pu}$ = 202.5 ksi

at service limit state (after all losses)

$f_{pe} \leq 0.80f_{py}$ = 194.4 ksi

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Reinforcing bars:

Yield strength, f_y = 60 ksi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Future wearing surface: additional 2 in. with unit weight equal to 0.150 kcf

New Jersey-type barrier: Unit weight = 0.300 kip/ft/side

9.6.3 CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

9.6.3.1 Non-Composite Section

A = area of cross-section of beam = 767 in.^{2}

h = overall depth of beam = 72 in.

I = moment of inertia about the centroid of the non-composite precast beam = 545,894 in.^{4}

y_b = distance from centroid to extreme bottom fiber of the non-composite precast beam = 36.60 in.

y_t = distance from centroid to extreme top fiber of the non-composite precast beam = 35.40 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = I/y_b = 14,915 in.^{3}

S_t = section modulus for the extreme top fiber of the non-composite precast beam = I/y_t = 15,421 in.^{3}

Wt = 0.799 kip/ft

$E_c = 33{,}000(W_c)^{1.5}\sqrt{f'_c}$ [LRFD Eq. 5.4.2.4-1]

where

E_c = modulus of elasticity of concrete, ksi

w_c = unit weight of concrete = 0.150 kcf

The *LRFD Specifications*, commentary C5.4.2.4, indicates that the unit weight of normal weight concrete is 0.145 kcf. However, precast concrete mixes typically have a relatively low water/cementitious materials ratio and high density. Therefore, a unit weight of 0.150 kcf is used in this example. For high strength concrete, this value may need to be increased further based on test results.

f'_c = specified strength of concrete, ksi

Therefore, the modulus of elasticity for the cast-in-place concrete deck is:

$E_c = 33{,}000(0.150)^{1.5}\sqrt{4.0}$ = 3,834 ksi for the precast beam at transfer, $E_{ci} = 33{,}000(0.150)^{1.5}\sqrt{5.5} = 4{,}496$ ksi

for the precast beam at service loads, $E_c = 33{,}000(0.150)^{1.5}\sqrt{7.0} = 5{,}072$ ksi

9.6.3.2 Composite Section

9.6.3.2.1 Effective Flange Width

[LRFD Art. 4.6.2.6.1]

The effective flange width is the lesser of:

(1/4) span length: (120 x 12/4) = 360 in.

$12t_s$ plus greater of web thickness or 1/2 beam top flange width
= (12 x 7.5 + 0.5 x 42) = 111 in.; or,

average spacing between beams = (12 x 12) = 144 in.

Therefore, the effective flange width is = 111 in.

9.6.3.2.2 Modular Ratio Between Slab And Beam Materials

Modular ratio between slab and beam concrete, $n = \dfrac{E_c(\text{slab})}{E_c(\text{beam})} = \dfrac{3{,}834}{5{,}072} = 0.7559$

9.6.3.2.3 Transformed Section Properties

Transformed flange width = n (effective flange width) = (0.7559)(111) = 83.91 in.

Transformed flange area = n (effective flange width)(t_s) = (0.7559)(111)(7.5) = 629.29 in.^{2}

Note that only the structural thickness of the deck, 7.5 in., is considered.

Due to camber of the precast, prestressed beam, a minimum haunch thickness of 1/2 in., at midspan, is considered in the structural properties of the composite section. Also, the width of haunch must be transformed.

Transformed haunch width = (0.7559)(42) = 31.75 in.

Transformed area of haunch = (0.7559)(42)(0.5) = 15.87 in.^{2}

Figure 9.6.3.2.3-1 shows the dimensions of the composite section.

Figure 9.6.3.2.3-1
Composite Section

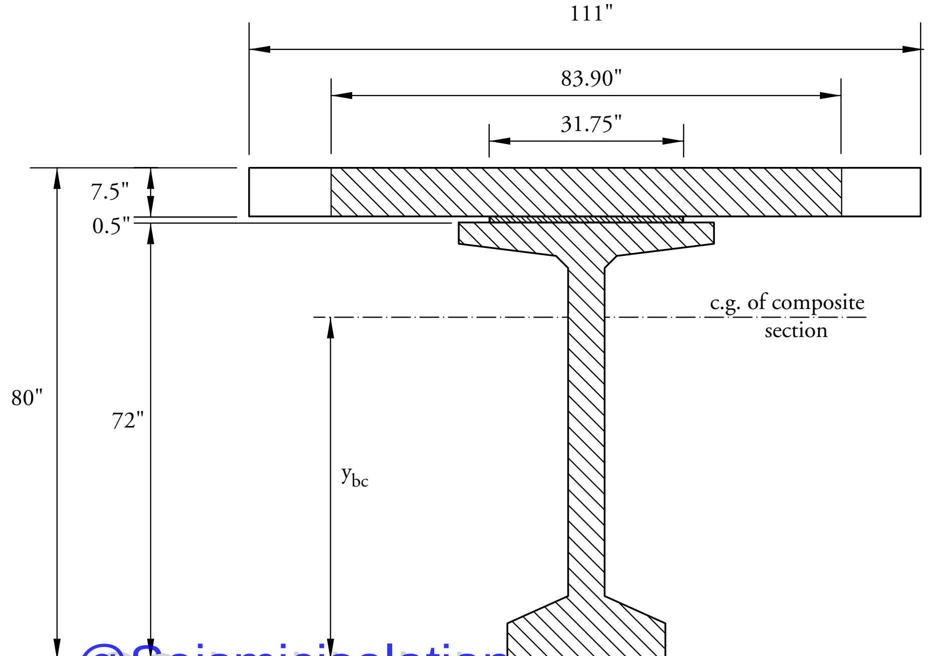


Table 9.6.3.2.3-1
Properties of Composite Section

	Transformed Area, in.^{2}	y_b in.	Ay_b in.^{3}	$A(y_{bc} - y_b)^2$ in.^{4}	I in.^{4}	$I + A(y_{bc} - y_b)^2$ in.^{4}
Beam	767.00	36.60	28,072.20	250,444.60	545,894.00	796,338
Haunch	15.87	72.25	1,146.61	4,904.73	0.33	4,905
Deck	629.29	76.25	47,983.36	293,058.09	2,949.61	296,007
$\sum$	1,412.16		77,202.17			1,097,251

Note that the haunch should only be considered to contribute to section properties if it is required to be provided in the completed structure. Some designers neglect its contribution to the section properties.

A_c = total area of composite section = 1,412 in.^{2}

h_c = overall depth of the composite section = 80 in.

I_c = moment of inertia of the composite section = 1,097,252 in.^{4}

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = $\dfrac{77,202}{1,412}$ = 54.67 in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 72 − 54.67 = 17.33 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the slab = 80 − 54.67 = 25.33 in.

S_{bc} = composite section modulus for the extreme bottom fiber of the precast beam

$$= (I_c/y_{bc}) = \left(\frac{1,097,252}{54.67}\right) = 20,070 \text{ in.}^3$$

S_{tg} = composite section modulus for the top fiber of the precast beam

$$= (I_c/y_{tg}) = \left(\frac{1,097,252}{17.33}\right) = 63,315 \text{ in.}^3$$

S_{tc} = composite section modulus for extreme top fiber of the deck slab

$$= \left(\frac{1}{n}\right)(I_c/y_{tc}) = \left(\frac{1}{0.7559}\right)\left(\frac{1,097,252}{25.33}\right) = 57,307 \text{ in.}^3$$

9.6.4 SHEAR FORCES AND BENDING MOMENTS

The self-weight of the beam and the weight of the slab and haunch act on the non-composite, simple-span structure, while the weight of barriers, future wearing surface, and live loads with impact act on the composite, continuous structure. Refer to **Table 9.6.4-1** which follows for a summary of unfactored values, calculated below:

9.6.4.1 Shear Forces and Bending Moments Due to Dead Loads

[LRFD Art. 3.3.2]

9.6.4.1.1 Dead Loads

DC = Dead load of structural components and non-structural attachments

Dead loads acting on the simple-span structure, non-composite section:

Beam self-weight = 0.799 kip/ft

8-in. deck weight = (8/12 ft)(12 ft)(0.150 kcf) = 1.200 kip/ft

1/2 in. haunch weight = (0.5)(42/144)(0.150) = 0.022 kip/ft

Notes:

1. Actual slab thickness (8 in.) is used for computing dead load.
2. A 1/2 in. minimum haunch thickness is assumed in the computations of dead load. If a deeper haunch will be used because of final beam camber, the weight of the actual haunch should be used.
3. The weight of cross-diaphragms is ignored since most agencies are moving away from cast-in-place concrete diaphragms to lightweight steel diaphragms.

Dead loads placed on the continuous structure, composite section:

LRFD Article 4.6.2.2.1 states that permanent loads (curbs and future wearing surface) may be distributed uniformly among all beams if the following conditions are met:

- Width of the deck is constant O.K.
- Number of beams, N_b, is not less than four ($N_b = 4$) O.K.
- Roadway part of the overhang, $d_e \leq 3.0$ ft

$$d_e = 3.0 - 1.25 - 0.5\left(\frac{6}{12}\right) = 1.5 \text{ ft} \qquad \text{O.K.}$$

- Curvature in plan is less than 4° (curvature = 0.0) O.K.
- Cross-section of the bridge is consistent with one of the cross-sections given in LRFD Table 4.6.2.2.1-1 O.K.

Since these criteria are satisfied, the barrier and wearing surface loads are equally distributed among the 4 beams.

Barrier weight = (2 barriers)(0.300 kip/ft)/(4 beams) = 0.150 kip/ft

DW = Dead load of future wearing surface = (2/12)(0.15) = 0.250 ksf = (0.025 ksf)(42.0 ft)/(4 beams) = 0.263 kip/ft

9.6.4.1.2 Unfactored Shear Forces and Bending Moments

For a simply supported beam with a span (L) loaded with a uniformly distributed load (w), the shear force (V_x) and bending moment (M_x) at any distance (x) from the support are given by:

$$V_x = w(0.5L - x) \qquad \text{(Eq. 9.6.4.1.2-1)}$$

$$M_x = 0.5wx(L - x) \qquad \text{(Eq. 9.6.4.1.2-2)}$$

Using the above equations, values of shear forces and bending moments for a typical interior beam, under self-weight of beam and weight of slab and haunch are computed and given in **Table 9.6.4-1** that is found at the end of Section 9.6.4. The span length for each span to be considered depends on the construction stage:

- overall length immediately after prestress release
- centerline-to-centerline distance between beam bearings at the time of deck placement
- centerline-to-centerline distance between supports after beams are made continuous

Shear forces and bending moments due to barrier weight and future wearing surface are calculated based on the continuous span lengths, 110, 120 and 110 ft. The three-span structure was analyzed using a continuous beam program. The shear forces and bending moments are given in **Table 9.6.4-1**.

9.6.4.2 Shear Forces and Bending Moments Due to Live Loads

9.6.4.2.1 Live Loads

Design live load is HL-93, which consists of a combination of: [LRFD Art. 3.6.1.2.1]

1. Design truck or design tandem with dynamic allowance. [LRFD Art. 3.6.1.2.1] The design truck is the same as the HS20 design truck specified by the *Standard Specifications* [STD Art. 3.6.1.2.2]. The design tandem consists of a pair of 25.0-kip axles spaced 4.0 ft apart. [LRFD Art. 3.6.1.2.3]. Spans in the range used in this example are much larger than those controlled by the tandem loading. For this reason, tandem loading effects are not included.
2. Design lane load of 0.64 kips/ft without dynamic allowance [LRFD Art. 3.6.1.2.4]

Art. 3.6.1.3.1 in the *LRFD Specifications* requires that for negative moment between points of dead load contraflexure, and, for reactions at interior piers only, 90% of the effect of two design trucks spaced at a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90% of the effect of the design lane load be considered. The distance between the 32 kip axles of each truck should be taken as 14 ft.

This three-span structure was analyzed using a continuous beam program that has the ability to generate live load shear force, and bending moment envelopes in accordance with the *LRFD Specifications* on a per-lane basis. The span lengths used are the continuous span lengths, 110, 120 and 110 ft.

9.6.4.2.2 Distribution Factor for a Typical Interior Beam

The live load bending moments and shear forces are determined by using the simplified distribution factor formulas [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions must be met. [LRFD Art. 4.6.2.2.1]

Width of deck is constant O.K.

Number of beams, $N_b \geq 4$ ($N_b = 4$) O.K.

Beams are parallel and approximately of the same stiffness O.K.

Roadway part of overhang, $d_e \leq 3.0$ ft $d_e = 3.0 - 1.25 - 0.5\left(\frac{6}{12}\right) = 1.50$ ft O.K.

Curvature is less than 4° [LRFD Table. 4.6.1.2.1-1] (Curvature = 0.0°) O.K.

For precast concrete I- or bulb-tee beams with cast-in-place concrete deck slab, the bridge type is (k) [LRFD Table 4.6.2.2.1-1]

The number of design lanes:

Number of design lanes = The integer part of the ratio of (w/12), where (w) is the clear roadway width, in ft, between the curbs. [Art. 3.6.1.1.1]

From **Figure 9.6.1-2**, w = 42 ft

Number of design lanes = integer part of (42/12) = 3 lanes

9.6.4.2.2.1 Distribution Factor for Bending Moment

- For all limit states except for fatigue limit state

For two or more lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \qquad \text{[LRFD Table 4.6.2.2.2b-1]}$$

Provided that: $3.5 \le S \le 16$ $\quad S = 12.0$ ft $\quad$ O.K.

$4.5 \le t_s \le 12$ $\quad t_s = 7.5$ in. $\quad$ O.K.

$20 \le L \le 240$ $\quad L = 120$ ft $\quad$ O.K.

$N_b \ge 4$ $\quad N_b = 4$ $\quad$ O.K.

$10{,}000 \le K_g \le 7{,}000{,}000$ $\quad$ O.K. (see below)

where

DFM = distribution factor for moment for interior beam

S = beams spacing, ft

L = beam span, ft

t_s = depth of concrete slab, in.

K_g = longitudinal stiffness parameter, in.^{4}, $= n(I + Ae_g^2)$ [LRFD Eq. 4.6.2.2.1-1]

where

n = modular ratio between beam and deck materials

$$= \frac{E_c(\text{beam})}{E_c(\text{slab})} = \frac{5{,}072}{3{,}834} = 1.3229$$

A = cross-section area of the beam (non-composite section), in.^{2}

I = moment of inertia of the beam (non-composite section), in.^{4}

e_g = distance between the centers of gravity of the beam and deck, in.
= (7.5/2 + 0.5 + 35.4) = 39.65 in.

Therefore,

$K_g = (1.3229)[545{,}894 + 767(39.65)^2] = 2{,}317{,}339.75$ in.^{4}

At center span:

$$DFM = 0.075 + \left(\frac{12}{9.5}\right)^{0.6}\left(\frac{12}{120}\right)^{0.2}\left(\frac{2{,}317{,}339.75}{(12.0)(120)(7.5)^3}\right)^{0.1}$$

$= 0.075 + (1.150)(0.631)(1.143) = 0.905$ lanes/beam

For one design lane loaded:

$$DFM = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \qquad \text{[LRFD Table 4.6.2.2.2b-1]}$$

$$= 0.06 + \left(\frac{12}{14}\right)^{0.4}\left(\frac{12}{120}\right)^{0.3}\left(\frac{2{,}217{,}339.75}{12.0(120)(7.5)^3}\right)^{0.1}$$

$= 0.06 + (0.940)(0.501)(1.138) = 0.596$ lanes/beam

Thus, the case of the two design lanes loaded controls, DFM = 0.905 lanes/beam.

- For Fatigue Limit State:

  LRFD Article 3.4.1 states that for fatigue limit state, the single design truck should be used. However, live load distribution factors given in LRFD Article 4.6.2.2 take into consideration the multiple presence factor, m. LRFD Article 3.6.1.1.2 states that the multiple presence factor, m, for one design lane loaded is 1.2. Therefore, the distribution factor for one design lane loaded, with the multiple presence factor removed, should be used. The distribution factor for fatigue limit state is: 0.596/1.2 = 0.497 lanes/beam.

  Fatigue limit state is not checked in this example. However, the live load moment that is used to compute the fatigue stress range is a moment due to a truck load with a constant spacing of 30 ft between the 32.0 kip axles.

9.6.4.2.2.2 Distribution Factor for Shear Force

For two or more lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 \qquad \text{[LRFD Table 4.6.2.2.3a-1]}$$

Provided that:	3.5	$\leq S$	≤ 16	$S = 12.0$ ft	O.K.
	20	$\leq L$	≤ 240	$L = 120$ ft	O.K.
	4.5	$\leq t_s$	≤ 12	$t_s = 7.5$ in.	O.K.
	N_b	≥ 4		$N_b = 4$	O.K.

where

DFV = Distribution factor for shear for interior beam

S = Beam spacing, ft

Therefore, the distribution factor for shear force for both end spans and center span is:

$$DFV = 0.2 + \left(\frac{12}{12}\right) - \left(\frac{12}{35}\right)^2 = 1.082 \text{ lanes/beam}$$

For one design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25.0}\right) = 0.36 + \left(\frac{12}{25.0}\right) = 0.840 \text{ lanes/beam}$$

Thus, the case of two or more lanes loaded controls, DFV = 1.082 lanes/beam.

9.6.4.2.3 Dynamic Allowance

IM = 33% [LRFD Table 3.6.2.1-1]

where IM = dynamic load allowance, applied only to truck load

9.6.4.2.4 Unfactored Shear Forces and Bending Moments

For all limit states except for fatigue limit state:

Unfactored shear forces and bending moments due to HL-93, per beam, are:

V_{LT} = (shear force per lane)(DFV)(1 + IM) = (shear force per lane)(1.082)(1 + 0.33) = (shear force per lane)(1.439) kips

M_{LT} = (bending moment per lane)(DFM)(1 + IM) = (bending moment per lane) (0.905)(1+0.33) = (bending moment per lane)(1.204) ft-kips

Values of V_{LT} and M_{LT} at different points are given in **Table 9.6.4-1**.

9.6.4.3 Load Combinations

[LRFD Art. 3.4]

Total factored load shall be taken as:

$Q = \eta\Sigma\gamma_i q_i$ [LRFD Eq. 3.4.1-1]

where

η = a factor relating to ductility, redundancy and operational importance. Here, η is considered to be 1.0 [LRFD Art. 1.3.2]

γ_i = load factors

q_i = specified loads [LRFD Table 3.4.1-1]

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

Service I: check compressive stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 1.00(LL + IM) [LRFD Table 3.4.1-1]

This load combination is the general combination for service limit state stress checks and applies to all conditions other than Service III.

Service III: Check tensile stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 0.80(LL + IM) [LRFD Table 3.4.1-1]

This load combination is a special combination for the service limit state stress check that applies only to tension in prestressed concrete structures to control cracks.

Strength I: Check ultimate strength: [LRFD Table 3.4.1-1 and 2]

Maximum Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)

Minimum Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)

This load combination is the general load combination for strength limit state design.

The minimum load factors for dead load (DC) and future wearing surface (DW) are used when dead load and future wearing surface stresses are of an opposite sign to that of the live load.

Fatigue: Check stress range in strands: [LRFD Table 3.4.1-1]

Q = 0.75(LL + IM)

This is a special load combination to check the tensile stress range in the strands due to live load and dynamic allowance.

Table 9.6.4-1
Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Location		Beam weight [simple span]		Deck plus Haunch weight [simple span]		Barrier weight [continuous span]		Future wearing surface [continuous span]		HL-93 live load envelope [continuous span]	
Distance x ft	Section x/L	Shear kips	Moment M_g ft-kips	Shear kips	Moment M_s ft-kips	Shear kips	Moment M_b ft-kips	Shear kips	Moment M_{ws} ft-kips	Shear V_{LL+I} kips	Moment M_{LL+I} ft-kips
0.0[1]	0.000	43.5	0.0	66.6	0.0	6.0	0.0	11.0	0.0	128.4	0.0
11.0	0.100	34.8	430.7	53.2	658.7	5.0	62.0	8.0	109.0	111.6	1,026.3
22.0	0.200	26.0	764.6	39.7	1,169.5	3.0	106.0	6.0	186.0	91.9	1,749.4
33.0	0.300	17.2	1,001.9	26.3	1,532.4	2.0	132.0	3.0	231.0	72.0	2,186.5
44.0	0.400	8.4	1,142.6	12.8	1,747.5	0.0	139.0	0.0	244.0	57.1	2,382.9
55.0	0.500	0.4	1,186.5	0.6	1,814.7	2.0	128.0	3.0	225.0	68.9	2,348.5
66.0	0.600	9.2	1,133.8	14.1	1,734.0	3.0	100.0	6.0	175.0	85.2	2,108.7
77.0	0.700	18.0	984.4	27.5	1,505.5	5.0	53.0	9.0	93.0	101.4	1,654.3
88.0	0.800	26.8	738.3	40.9	1,129.1	7.0	-12.0	12.0	-21.0	114.7	1,021.7
99.0	0.900	35.6	395.5	54.4	604.9	8.0	-95.0	15.0	-167.0	120.3	-1,316.8
109.0[1]	0.991	43.5	0.0	66.6	0.0	9.8*	-187.7*	17.7*	-328.8*	144.6*	-2,235.8*
110.0	1.000	0.0	0.0	0.0	0.0	10.0	-197.0	18.0	-345.0	147.0	-2,327.7
0.0[2]	0.000	0.0	0.0	0.0	0.0	9.0	-197.0	16.0	-345.0	144.6	-2,327.7
1.0[1]	0.008	47.1	0.0	72.1	0.0	8.8*	-188.9*	15.8*	-336.6*	143.6*	-2,276.9*
7.1[3]	0.059	42.3	272.7	64.6	417.1	7.8*	-139.6*	14.2	-244.4*	137.3*	-1,717.8*
12.0	0.100	38.4	470.2	58.7	719.1	7.0	-100.0	13.0	-175.0	132.3	-1,296.9
24.0	0.200	28.8	872.9	44.0	1,335.0	5.0	-24.0	10.0	-42.0	113.8	1,044.4
36.0	0.300	19.2	1,160.5	29.3	1,775.0	4.0	30.0	6.0	53.0	92.6	1,641.7
48.0	0.400	9.6	1,333.1	14.7	2,038.9	2.0	62.0	3.0	109.0	76.7	2,006.4
60.0	0.500	0.0	1,390.7	0.0	2,126.9	0.0	73.0	0.0	128.0	58.2	2,115.0

[1] Centerline of bearing
[2] Centerline of pier
[3] Critical section in shear
* Determined using linear interpolation
Note: Shear values shown are absolute values.

Note: The live load used in the above equation results only from a single design truck with a 30-ft constant spacing between the 32.0 kip axles with the special dynamic allowance, (IM) for fatigue.

9.6.5 ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber for the load combination at Service III at the section of maximum moment or at the harp points. For estimating the number of strands, only the stresses at midspan are considered.

9.6.5.1 Service Load Stresses at Midspan

Bottom tensile stress due to applied dead and live loads using load combination Service III, is:

$$f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + (0.8)(M_{LL+I})}{S_{bc}}$$

where

f_b = bottom tensile stresses, ksi

M_g = unfactored bending moment due to beam self-weight, ft-kips

M_s = unfactored bending moment due to slab and haunch weights, ft-kips

M_b = unfactored bending moment due to barrier weight, ft-kips

M_{ws} = unfactored bending moment due to weight of future wearing surface, ft-kips

M_{LL+I} = unfactored bending moment due to design vehicular live load including impact, ft-kips

Using values of bending moments from **Table 9.6.4-1**, the bottom tensile stress at midspan of the center span (point 0.5, centerspan), is:

$$f_b = \frac{(1{,}390.7 + 2{,}126.9)(12)}{14{,}915} + \frac{(73 + 128 + 0.8 \text{ x } 2{,}115)(12)}{20{,}070}$$

$$= 2.830 + 1.132 = 3.962 \text{ ksi}$$

9.6.5.2 Stress Limits for Concrete

The tensile stress limit at service loads = $0.19\sqrt{f'_c}$ [LRFD Art. 5.9.4.2.2b]

where f'_c = specified 28-day concrete strength, ksi

Therefore, the tensile stress limit in concrete = $0.19\sqrt{7.0} = -0.503$ ksi

9.6.5.3 Required Number of Strands

The required precompressive stress at the bottom fiber of the beam is the difference between bottom tensile stress due to the applied loads and the concrete tensile stress limit:

$f_{pb} = (3.962 - 0.503) = 3.459$ ksi.

The location of the strand center of gravity at midspan, ranges from 5 to 15% of the beam depth, measured from the bottom of the beam. A value of 5% is appropriate for newer efficient sections like the bulb-tee beams and 15% for less efficient AASHTO standard shapes.

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Assume the distance from the center of gravity of strands to the bottom fiber of the beam, y_{bs}, is equal to 7% of the beam depth.

$y_{bs} = 0.07h = 0.07(72) = 5.04$ in.

Then, the strand eccentricity at midspan, e_c, is $= (y_b - y_{bs}) = (36.60 - 5.04) = 31.56$ in.

If P_{pe} is the total prestressing force, the stress at the bottom fiber due to prestress is:

$f_{pb} = \dfrac{P_{pe}}{A} + \dfrac{P_{pe}\,e_c}{S_b}$, or, setting the required precompression (3.459 ksi) equal to the bottom fiber stress due to prestress, solve for the minimum required P_{pe}:

$$3.459 = \frac{P_{pe}}{767} + \frac{P_{pe}\,(31.56)}{14{,}915}$$

Solving for P_{pe}, the required $P_{pe} = 1{,}011.5$ kips

Final Prestress force per strand = (area of strand)(f_{pi})(1 – losses, %)

where f_{pi} = initial prestressing stress before transfer, ksi.

Assuming final loss of 25% of f_{pi}, the prestress force per strand after all losses = (0.153)(202.5)(1 – 0.25) = 23.2 kips

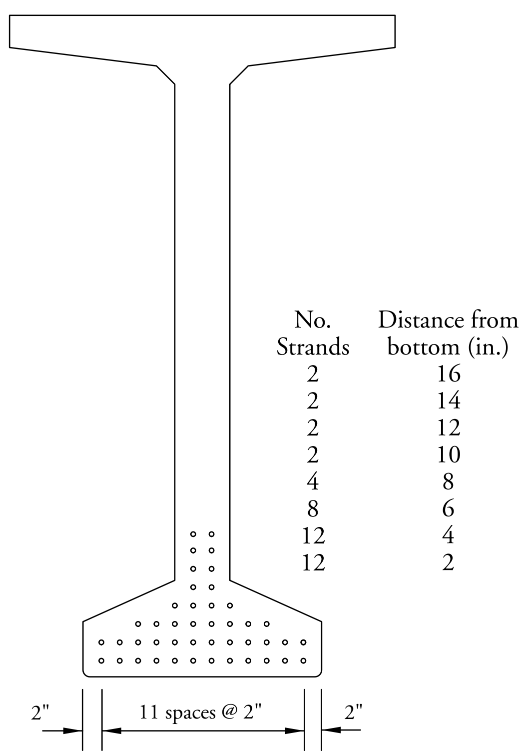
Number of strands required = (1,011.5/23.2) = 43.6 strands

Try (44) 1/2 in. diameter, 270 ksi, low-relaxation strands

9.6.5.4 Strand Pattern

The assumed strand pattern for the 44 strands at midspan is shown in **Figure 9.6.5.4-1**. Each available position was filled beginning with the bottom row.

Figure 9.6.5.4-1 Assumed Strand Pattern at Midspan



The distance between the center of gravity of strands and the bottom concrete fiber of the beam, y_{bs}, is:

y_{bs} = [(12(2) + 12(4) + 8(6) + 4(8) + 2(10) + 2(12) + 2(14) + 2(16)]/44 = 5.82 in.

Strand eccentricity at midspan, $e_c = y_b - y_{bs}$ = 36.60 − 5.82 = 30.78 in.

9.6.6 PRESTRESS LOSSES

[LRFD Art. 5.9.5]

Total prestress losses: [LRFD Eq. 5.9.5.1-1]

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

where

Δf_{pES} = loss due to elastic shortening, ksi

Δf_{pSR} = loss due to shrinkage, ksi

Δf_{pCR} = loss due to creep, ksi

Δf_{pR2} = loss due to relaxation of steel after transfer, ksi

9.6.6.1 Elastic Shortening

[LRFD Art. 5.9.5.2.3a]

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where

E_p = modulus of elasticity of prestressing reinforcement = 28,500 ksi

E_{ci} = modulus of elasticity of beam at release = 4,496 ksi

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member at sections of maximum moment

$$= \frac{P_i}{A} + \frac{P_i e_c^2}{I} - \frac{M_g e_c}{I}$$

The *LRFD Specifications*, Art. 5.9.5.2.3a, states that f_{cgp} can be calculated on the basis of a prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, common practice assumes the initial losses as a percentage of initial prestressing stress before release, f_{pi}. In both procedures, assumed initial losses should be checked and if different from the assumed value, a second iteration should be carried out. In this example, a 9% f_{pi} initial loss is used.

Force per strand at transfer = (area of strand) (prestress stress at release)

= 0.153(202.5)(1 − 0.09) = 28.2 kips

P_i = total prestressing force at release = (44 strands)(28.2) = 1,240.8 kips

M_g should be calculated based on the overall beam length of 119 ft. However, since the elastic shortening losses will be a part of the total losses, f_{cgp} will be conservatively computed based on M_g using the design span length of 118 ft.

$$f_{cgp} = \frac{1,240.8}{767} + \frac{(1,240.8)(30.78)^2}{545,894} - \frac{(1,390.7)(12)(30.78)}{545,894}$$

$$= 1.618 + 2.153 - 0.941 = 2.830 \text{ ksi}$$

Therefore, the loss due to elastic shortening is:

$$\Delta f_{pES} = \frac{28,500}{4,496}(2.830) = 17.9 \text{ ksi}$$

The percent of actual losses due to elastic shortening = (17.9/202.5)100 = 8.8%. Since calculated loss of 8.8% is approximately equal to the initial assumption of 9% a second iteration is not necessary. Note that this loss is equivalent to a stress after initial losses of 0.68 f_{pu}. This stress is lower than the estimate of 0.70 f_{pu}, provided in Article 5.9.5.2.3a. If the elastic shortening loss was calculated using a stress of 0.70 f_{pu}, a second iteration would be required to arrive at a steel stress of 0.68 f_{pu}.

9.6.6.2 Shrinkage

[LRFD Art. 5.9.5.4.2]

$\Delta f_{pSR} = (17 - 0.15H)$ [LRFDEq. 5.9.5.4.2-1]

where H = relative humidity (assume 70%)

Relative humidity varies significantly throughout the country. See LRFD Figure 5.4.2.3.3-1.

$\Delta f_{pSR} = 17 - 0.15(70) = 6.5$ ksi

9.6.6.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

where

Δf_{cdp} = Change of stresses at center of gravity of prestressing due to permanent loads except the loads acting at time of applying prestressing force, calculated at the same section as f_{cgp}

$$= \frac{M_s e_c}{I} + \frac{(M_{ws} + M_b)(y_{bc} - y_{bs})}{I_c}$$

$$= \frac{(2,126.9)(12)(30.78)}{545,894} + \frac{(73 + 128)(12)(54.67 - 5.82)}{1,097,252}$$

$$= 1.439 + 0.107 = 1.546 \text{ ksi}$$

Therefore, the loss due to creep is:

$\Delta f_{pCR} = 12(2.830) - 7(1.546) = 23.1$ ksi

9.6.6.4 Relaxation of Prestressing Strands

[LRFD Art. 5.9.5.4.4]

9.6.6.4.1 Relaxation before Transfer

[LRFD Art. 5.9.5.4.4b]

Initial loss due to relaxation of prestressing steel is accounted for in the beam fabrication process. Therefore, loss due to relaxation of the prestressing steel prior to transfer will not be computed, i.e. $\Delta f_{pR1} = 0$. Recognizing this for pretensioned members, LRFD Article 5.9.5.1 excludes the portion of the relaxation loss that occurs prior to transfer from the final loss.

9.6.6.4.2 Relaxation after Transfer

[LRFD Art. 5.9.5.4.4c]

For low-relaxation strands, loss due to strand relaxation after transfer is:

$\Delta f_{pR2} = 30\%[20.0 - 0.4(\Delta f_{pES}) - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$ [LRFD Art. 5.9.5.4.4c]

$= 0.30[20.0 - 0.4(17.9) - 0.2(6.5 + 23.1)] = 2.1$ ksi

9.6.6.5 Total Losses at Transfer

$\Delta f_{pi} = \Delta f_{pES} = 17.9$ ksi

Stress in tendons after transfer, $f_{pt} = f_{pi} - \Delta f_{pi} = (202.5 - 17.9) = 184.6$ ksi

Force per strand = (f_{pt})(strand area) = 184.6(0.153) = 28.2 kips

Therefore, the total prestressing force after transfer, P_i = 28.2 x 44 = 1,240.8 kips

Initial loss, % = (total losses at transfer)/(f_{pi}) = 17.9/(202.5) = 8.9%

The first estimation of loss at transfer, 9%, is very close to the actual computed initial loss of 8.9%. Thus, there is no need for a second iteration to refine the initial losses.

9.6.6.6 Total Losses at Service Loads

Total loss of prestress at service loads is:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} = 17.9 + 6.5 + 23.1 + 2.1 = 49.6$ ksi

Stress in tendon after all losses, $f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 49.6 = 152.9$ ksi

Check prestressing stress limit at service limit state:

$f_{pe} \leq 0.8 f_{py} = 0.8(243) = 194.4 \text{ ksi} > 152.9$ ksi O.K.

Force per strand = (f_{pe})(strand area) = (152.8)(0.153) = 23.4 kips

Therefore, the total prestressing force after all losses, P_{pe} = 23.4(44) = 1,029.6 kips

Final losses, % = $(\Delta f_{pT})/(f_{pi}) = \dfrac{49.6}{202.5}(100) = 24.5\%$

9.6.7 STRESSES AT TRANSFER

Force per strand after initial losses = 28.2 kips

Therefore, the total prestressing force after transfer is, P_i = 1,240.8 kips

9.6.7.1 Stress Limits For Concrete

[LRFD Art. 5.9.4]

Compression: $0.6f'_{ci} = 0.6(5.5) = +3.300$ ksi

Tension:

- without bonded auxiliary reinforcement:

$$-0.0948\sqrt{f'_{ci}} \leq -0.200 \text{ ksi}; \; -0.0948\sqrt{5.5} = -0.222 \text{ ksi} < -0.200 \text{ ksi}$$

Therefore, −0.200 ksi (Controls)

- with bonded auxiliary reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete:

$$-0.22\sqrt{f'_{ci}} = -0.22\sqrt{5.5} = -0.516 \text{ ksi}$$

9.6.7.2 Stresses at Transfer Length Section

Stresses at this location need only be checked at release since this stage almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length = 60(strand diameter) = 60(0.5) = 30 in. = 2.5 ft [LRFD Art. 5.8.2.3]

Due to the camber of the beam at release, the beam self-weight acts on the overall beam length (119 ft). Therefore, values of bending moment given in **Table 9.6.4-1** cannot be used since they are based on the span between centerlines of bearings (118 ft). Using Equation Eq. 9.6.4.1.2-2 given previously, the bending moment at a distance 2.5 ft from the end of the beam is calculated due to beam self-weight:

$M_g = (0.5)(0.799)(2.5)(119 - 2.5) = 116.4$ ft-kips

Compute top stress at the top fiber of the beam:

$$f_t = \frac{P_i}{A} - \frac{P_i e}{S_b} + \frac{M_g}{S_b} = \frac{1,240.8}{767} - \frac{(1,240.8)(30.78)}{14,915} + \frac{(116.4)(12)}{14,915}$$

$= 1.618 - 2.477 + 0.091 = -0.768$ ksi

Tensile stress limit for concrete with bonded reinforcement: -0.516 ksi N.G.

Compute bottom stress at the bottom fiber of the beam:

$$f_b = \frac{P_i}{A} + \frac{P_i e}{S_b} - \frac{M_g}{S_b} = \frac{1,240.8}{767} + \frac{(1,240.8)(30.78)}{14,915} - \frac{(116.4)(12)}{14,915}$$

$= 1.618 + 2.561 - 0.094 = +4.085$ ksi

Compressive stress limit for concrete: $+3.300$ ksi N.G.

Since the top and bottom concrete stresses exceed the stress limits, harp strands to make stresses fall within the specified limits. Harp 12 strands at the 0.3L points, as shown in **Figures 9.6.7.2-1** and **9.6.7.2-2**. This harp location is more appropriate for the end spans of multi-span continuous bridges because the maximum positive moment is closer to the abutment than in the interior spans. For simple spans, it is more common to use a harp point at least 0.4L from the ends.

Figure 9.6.7.2-1
Strand Pattern

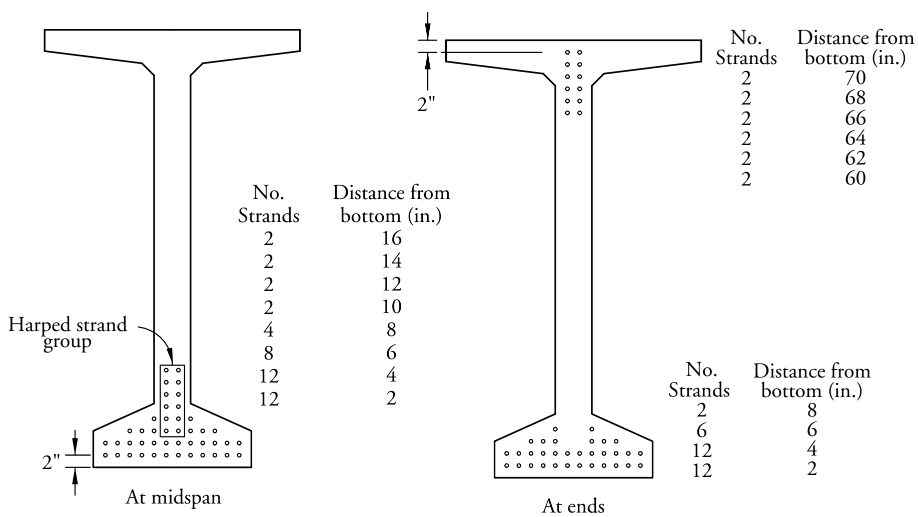
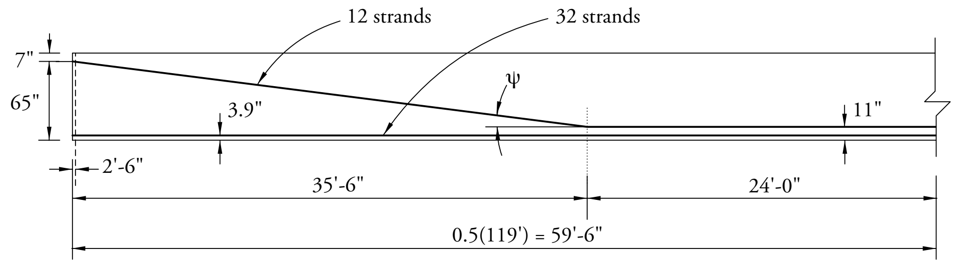


Figure 9.6.7.2-2
Longitudinal Strand Profile



Compute the center of gravity of the prestressing strands at the transfer length using the harped pattern.

The distance between the center of gravity of the 12 harped strands at the end of the beam and the top fiber of the precast beam is:

$$\frac{2(2) + 2(4) + 2(6) + 2(8) + 2(10) + 2(12)}{12} = 7.00 \text{ in.}$$

The distance between the center of gravity of the 12 harped stands at the harp point and the bottom fiber of the beam is:

$$\frac{2(6) + 2(8) + 2(10) + 2(12) + 2(14) + 2(16)}{12} = 11.0 \text{ in.}$$

The distance between the center of gravity of the 12 harped strands and the top fiber of the beam at the transfer length section is:

$$7 \text{ in.} + \frac{(72 - 11 - 7) \text{ in.}}{35.5 \text{ ft}}(2.5) \text{ ft} = 10.80 \text{ in.}$$

The distance between the center of gravity of the 32 straight bottom strands and the extreme bottom fiber of the beam is:

$$\frac{12(2) + 12(4) + 6(6) + 2(8)}{32} = 3.88 \text{ in.}$$

Therefore, the distance between the center of gravity of the total number of the strands and the bottom fiber of the precast beam at transfer length is:

$$\frac{12(72 - 10.80) + 32(3.88)}{44} = 19.51 \text{ in.}$$

Eccentricity of the strand group at transfer length, e, is: 36.60 – 19.51 = 17.09 in.

The distance between the center of gravity of the total number of the strands and the bottom fiber of the precast beam at the end of the beam is:

$$\frac{12(72 - 7) + 32(3.88)}{44} = 20.55 \text{ in.}$$

and the eccentricity at the end of the beam, e_e, is: 36.60 – 20.55 = 16.05 in.

Recompute top and bottom stresses at the transfer length section using the harped pattern:

Concrete stress at top fiber of the beam:

$$f_t = \frac{1,240.8}{767} - \frac{(1,240.8)(17.09)}{15,421} + \frac{(116.4)(12)}{15,421} = 1.618 - 1.375 + 0.091 = +0.334 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

Concrete stress at bottom fiber of the beam,

$$f_b = \frac{1,240.8}{767} + \frac{(1,240.8)(17.09)}{14,915} - \frac{(116.4)(12)}{14,915} = 1.618 + 1.422 - 0.094 = +2.946 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

9.6.7.3
Stresses at the Harp Points

The strand eccentricity at the harp points is the same as at midspan, e_c = 30.78 in.

The bending moment due to beam self-weight at a distance 35.5 ft (0.3L) from the end of the beam is:

M_g = (0.5)(0.799)(35.5)(119 – 35.5) = 1,184.2 ft-kips

Concrete stress at top fiber of the beam,

$$f_t = \frac{1,240.8}{767} - \frac{(1,240.8)(30.78)}{15,421} + \frac{(1,184.2)(12)}{15,421} = 1.618 - 2.477 + 0.921 = +0.062 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

Concrete stress at bottom fiber of the beam:

$$f_b = \frac{1,240.8}{767} + \frac{(1,240.8)(30.78)}{14,915} - \frac{(1,184.2)(12)}{14,915} = 1.618 + 2.561 - 0.953 = +3.226 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

9.6.7.4
Stresses at Midspan

The bending moment due to beam self-weight at a distance 59.5 feet from the end of the beam is:

M_g = (0.5)(0.799)(59.5)(119 – 59.5) = 1,414.3 ft-kips

Concrete stress at top fiber of the beam:

$$f_t = \frac{1,240.8}{767} - \frac{(1,240.8)(30.78)}{15,421} + \frac{(1,414.3)(12)}{15,421} = 1.618 - 2.477 + 1.101 = +0.242 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

Concrete stress at bottom fiber of the beam,

$$f_b = \frac{1,240.8}{767} + \frac{(1,240.8)(30.78)}{14,915} - \frac{(1,414.3)(12)}{14,915} = 1.618 + 2.561 - 1.138$$
$$= +3.041 \text{ ksi}$$

Compressive stress limit for concrete: +3.300 ksi O.K.

9.6.7.5 Hold-Down Forces

Assume that the stress in the strand at the time of prestressing, before any losses, is:

$0.80f_{pu} = 0.80(270) = 216$ ksi

Then, the prestress force per strand before any losses is: 0.153(216) = 33.0 kips

From **Figure 9.6.7.2-2**, harp angle,

$$\psi = \tan^{-1}\left(\frac{72-7-11}{35.5(12)}\right) = 7.2°$$

Therefore, hold-down force/strand = 1.05(force per strand)(sin ψ)

= 1.05(33.0) sin 7.2° = 4.3 kips/strand

Note that the factor, 1.05, is applied to account for friction.

Total hold-down force = 12 strands(4.3) = 51.6 kips

The hold-down force and the harp angle should be checked against maximum limits for local practices. Refer to Chapter 3, Fabrication and Construction, Section 3.3.2.2, and Chapter 8, Design Theory and Procedures, for additional details.

9.6.7.6 Summary Of Stresses at Transfer

	Top stresses f_t (ksi)	Bottom stresses f_b (ksi)
At transfer length section	+0.334	+2.946
At harp points	+0.062	+3.226
At midspan	+0.242	+3.041

Note that the bottom stresses at the harp points are more critical than the ones at midspan.

9.6.8 STRESSES AT SERVICE LOADS

The total prestressing force after all losses, P_{pe} = 1,029.6 kips

9.6.8.1 Stress Limits For Concrete

[LRFD Art. 5.9.4.2]

Compression:

Due to permanent loads, (i.e., beam self-weight, weight of slab and haunch, weight of future wearing surface, and weight of barriers), for load combination Service I:

for the precast beam: $0.45f'_c = 0.45(7.0) = +3.150$ ksi

for the deck: $0.45f'_c = 0.45(4.0) = +1.800$ ksi

Due to permanent and transient loads (i.e., all dead loads and live loads), for load combination Service I:

for precast beam: $0.60f'_c = 0.60(7.0) = +4.200$ ksi

for deck: $0.60f'_c = 0.60(4.0) = +2.400$ ksi

Tension:

For components with bonded prestressing tendons:

for load combination Service III: $-0.19\sqrt{f'_c}$

for precast beam: $-0.19\sqrt{7.0} = -0.503$ ksi

9.6.8.2 Stresses at Midspan

• Concrete stresses at the top fiber of the beam:

To check top compressive stress, two cases are checked:

1. Under permanent loads, Service I:

Using bending moment values given in **Table 9.6.4-1**, concrete stress at top fiber of the beam is:

$$f_{tg} = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{(M_g + M_s)}{S_t} + \frac{(M_{ws} + M_b)}{S_{tg}}$$

$$= \frac{1{,}029.6}{767} - \frac{(1{,}029.6)(30.78)}{15{,}421} + \frac{(1{,}390.7 + 2{,}126.9)(12)}{15{,}421} + \frac{(128 + 73)(12)}{63{,}315}$$

$$= 1.342 - 2.055 + 2.737 + 0.038 = +2.062 \text{ ksi}$$

Compressive stress limit for concrete: +3.150 ksi O.K.

2. Under permanent and transient loads, Service I:

$$f_{tg} = +2.062 + \frac{(M_{LL+I})}{S_{tg}} = +2.062 + \frac{(2{,}115)(12)}{63{,}315} = +2.062 + 0.401 = +2.463 \text{ ksi}$$

Compressive stress limit for concrete: +4.200 ksi O.K.

• Concrete stress at the top fiber of the deck, Service I:

Note: Compression stress in the deck slab at service loads never controls the design for typical applications. The calculations shown below are for illustration and may not be necessary in most practical applications.

1. Under permanent loads:

$$f_{tc} = \frac{(M_{ws} + M_b)}{S_{tc}} = \frac{(128 + 73)(12)}{57{,}307} = +0.042 \text{ ksi}$$

Compressive stress limit for concrete: +1.800 ksi O.K.

2. Under permanent and transient loads:

$$f_{tc} = \frac{(M_{ws} + M_b + M_{LL+I})}{S_{tc}} = \frac{(128 + 73 + 2{,}115)(12)}{57{,}307} = +0.485 \text{ ksi}$$

Compressive stress limit for concrete: +2.400 ksi O.K.

• Tension stress at the bottom fiber of the beam, Service III:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{(M_g + M_s)}{S_b} - \frac{(M_{ws} + M_b) + 0.8(M_{LL+I})}{S_{bc}}$$

$$= \frac{1{,}029.6}{767} + \frac{(1{,}029.6)(30.78)}{14{,}915} - \frac{(1{,}390.7 + 2{,}126.9)(12)}{14{,}915}$$

$$- \frac{(128 + 73)(12) + (0.8)(2{,}115)(12)}{20{,}070}$$

$$= 1.342 + 2.125 - 2.830 - 1.132 = -0.495 \text{ ksi}$$

Tensile stress limit for concrete: −0.503 ksi O.K.

9.6.8.3 Fatigue Stress Limit

9.6.8.3.1 Positive Moment Section

Fatigue limit state is not checked in this example. For an example of this calculation, refer to Examples 9.2 and 9.4, Sections 9.2.9.3 and 9.4.8.3 respectively.

9.6.8.3.2 Negative Moment Section

In order to perform the fatigue check, the reinforcement of the section should be determined. Therefore, the fatigue check for the negative moment section is addressed in Section 9.6.9.2.1.

9.6.8.4 Summary of Stresses at Service Loads

	Top of Deck (ksi) Service I		Top of Beam (ksi) Service I		Bottom of Beam (ksi)
	Permanent Loads	Total Loads	Permanent Loads	Total Loads	Service III
At midspan	+0.042	+0.485	+2.062	+2.463	-0.495

9.6.9 STRENGTH LIMIT STATE

9.6.9.1 Positive Moment Section

Total ultimate bending moment for Strength I is:

$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$ [LRFD Tables 3.4.1-1&2]

At midspan of center span, (center span point 0.5):

$$M_u = 1.25(1{,}390.7 + 2{,}126.9 + 73) + 1.5(128) + 1.75(2{,}115)$$
$$= 4{,}488.3 + 192.0 + 3{,}701.3 = 8{,}381.6 \text{ ft-kips}$$

Average stress in prestressing steel when $f_{pe} \geq 0.5\, f_{pu}$:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ [LRFD Eq. 5.7.3.1.1-1]

where

f_{ps} = average stress in prestressing steel

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$ [LRFD Eq. 5.7.3.1.1-2]

= 0.28 for low-relaxation strands [LRFD Table C5.7.3.1.1-1]

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons = $h - y_{bs}$ = 80.00 − 5.82 = 74.18 in.

c = distance between the neutral axis and the compressive face, in.
To compute c, assume rectangular section behavior, and check if the depth of the neutral axis, c is equal to or less than t_s:

[LRFD 5.7.3.2.2]

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y'}{0.85 f_c' \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.7.3.1.1.-4]

where

A_{ps} = area of prestressing steel = 44 x 0.153 = 6.732 in.^{2}

f_{pu} = specified tensile strength of prestressing steel = 270 ksi

A_s = area of mild steel tension reinforcement = 0.0 in.^{2}

f_y = yield strength of tension reinforcement = 60.0 ksi

A'_s = area of compression reinforcement = 0.0 in.^{2}

f'_y = yield strength of compression reinforcement = 60.0 ksi

f'_c = compressive strength of deck concrete = 4.0 ksi

β_1 = stress factor of compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f'_c \le 4.0$ ksi

= $0.85 - 0.05(f'_c - 4.0) \ge 0.65$ for $f'_c > 4.0$ ksi

= 0.85

b = effective width of compression flange = 111 in.

$$c = \frac{(6.732)(270) + 0.0 - 0.0}{(0.85)(4.0)(0.85)(111) + 0.28(6.732)\dfrac{270}{74.18}} = 5.55 \text{ in.} < t_s = 7.5 \text{ in.} \quad \text{O.K.}$$

a = depth of the equivalent stress block = $\beta_1 c$ [LRFD Eq. 9.6.9.1-1]

= 0.85(5.55) = 4.72 in.

Therefore, the assumption of rectangular section behavior is valid and the average stress in prestressing steel is:

$$f_{ps} = 270\left(1 - 0.28\frac{5.55}{74.18}\right) = 264.3 \text{ ksi}$$

Nominal flexural resistance: [LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right)$$ [LRFD Eq. 5.7.3.2.2-1]

This equation is a simplified form of LRFD Equation 5.7.3.2.2-1 because no compression reinforcement or mild tension reinforcement is considered and the section behaves as a rectangular section.

$$M_n = (6.732)(264.3)\left(74.18 - \frac{4.72}{2}\right)/12 = 10{,}648.9 \text{ ft-kips}$$

Factored flexural resistance:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where ϕ = resistance factor [LRFD Art. 5.5.4.2.1]

= 1.00, for flexure and tension of prestressed concrete

M_r = 10,648.9 ft-kips > M_u = 8,381.6 ft-kips O.K.

9.6.9.2 Negative Moment Section

9.6.9.2.1 Design of the Section

Total ultimate bending moment for Strength I is:

M_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM) [LRFD Tables 3.4.1-1&2]

At the pier section:

M_u = 1.25(–197) + 1.5(–345) + 1.75(–2,327.7) = –4,837.2 ft-kips

Notes:

1. At the negative moment section, the compression face is the bottom flange of the beam and is 26 in. wide.
2. This section is a nonprestressed reinforced concrete section, thus $\phi = 0.9$ for flexure.

Assume the deck reinforcement is at mid-height of the deck. The effective depth:

$d = 72 + 0.5 + 0.5(7.5) = 76.25$ in.

$$R_u = \frac{M_u}{\phi bd^2} = \frac{4{,}837.2\,(12)}{(0.9)(26)(76.25)^2} = 0.427 \text{ ksi}$$

$$m = \frac{f_y}{0.85f'_c} = \frac{60}{(0.85)(7.0)} = 10.084$$

$$\rho = \frac{1}{m}\left[1-\sqrt{1-\frac{2R_u m}{f_y}}\right] = \frac{1}{10.084}\left[1-\sqrt{1-\frac{2(0.427)(10.084)}{60}}\right] = 0.00739$$

$A_s = (\rho bd) = (0.00739)(26)(76.25) = 14.65$ in.^{2}

This is the amount of mild steel reinforcement required in the slab to resist the negative moment. Assume that the typical deck reinforcement consists of a bottom mat of #5 bars @ 12 in. and a top mat of #4 @ 12 in. for a total $A_s = 0.20 + 0.31 = 0.51$ in.^{2}/ft.

Since the *LRFD Specifications* do not provide guidance on the width over which this reinforcement is to be distributed, it is assumed here to be the same as the effective compression flange width which was determined earlier to be 111 in.

The typical reinforcement provided over this width is equal to (111 x 0.51/12) = 4.72 in.^{2}. Therefore, the required additional reinforcement at the negative moment section = 14.65 − 4.72 = 9.93 in.^{2}.

Provide 18 #7 bars additional reinforcement at 4 in. spacing (2 #7 bars in each space between #4 bars).

$A_s = 18(0.60) = 10.80$ in.^{2}

Therefore, the total A_s provided = 10.80 + 4.72 = 15.52 in.^{2} > 14.65 in.^{2} O.K.

Compute the capacity of the section in flexure at the pier:

Compute the depth of the compression block:

$$a = \frac{A_s f_y}{0.85\, b\, f'_c} = \frac{15.52(60)}{(0.85)(26)(7.0)} = 6.02 \text{ in.}$$

Note that this value is slightly larger than the flange thickness of 6.0 in. However, the adjustment in the moment capacity, ϕM_n, when using a more accurate non-rectangular section analysis, is extremely small.

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right) = 0.9(15.52)(60.0)\left(76.25 - \frac{6.02}{2}\right)/12$$

$$= 5{,}115.1 \text{ ft-kips} > 4{,}837.2 \text{ ft-kips} \quad \text{O.K.}$$

With time, creep of concrete members heavily pretensioned, may cause camber growth. Because this bridge is designed to have rigid connections between beams at the piers, camber growth is restrained. As a result, time-dependent positive moments will develop. Therefore, it is recommended that a nominal amount of positive moment continuity reinforcement be used over the piers to control potential cracking in this region. A common way to provide this reinforcement is to extend approximately 25 percent of the strands from the bottom flange and bend them up into the diaphragm. Another common detail is the addition of a quantity of mild steel reinforcement required to resist a moment equal to 1.2 M_{cr}. This reinforcement is also extended from the ends of the beam and bent up into the diaphragm.

9.6.9.2.2
Fatigue Stress Limit and Crack Control

The fatigue limit state and crack control for the negative moment zone over the piers are important design criteria that must be checked. This zone is expected to be cracked due to service loads and the steel stress range is expected to be significantly high.

For moment calculations, the fatigue truck loading must be introduced to the three-span continuous structure. The resulting moments are then used to determine whether or not the stress range in the longitudinal reinforcement steel is within the acceptable limits.

In order to control flexural cracking, the tensile stress in the mild steel reinforcement at service limit state, should not exceed the value given by LRFD Eq. 5.7.3.4.1.

9.6.10 LIMITS OF REINFORCEMENT

9.6.10.1 Positive Moment Section

This section is a prestressed section.

9.6.10.1.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The maximum amount of prestressed and nonprestressed reinforcement should be such that:

$$\frac{c}{d_e} \le 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

$$\text{where } d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y} \qquad \text{[LRFD Eq. 5.7.3.3.1-2]}$$

Since $A_s = 0$, then $d_e = d_p = 74.18$ in.

$$\frac{c}{d_e} = \frac{5.55}{74.18} = 0.075 \le 0.42 \quad \text{O.K.}$$

9.6.10.1.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

At any section, the amount of prestressed and nonprestressed tensile reinforcement should be adequate to developed a factored flexural resistance, M_r, equal to the lesser of:

- 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and,
- 1.33 times the factored moment required by the applicable strength load combination.

Check at midspan:

The *LRFD Specifications* do not give a procedure for computing the cracking moment. Therefore, the following equation adapted from the *Standard Specifications*, Art. 9.18.2.1 is used:

$$M_{cr} = (f_r + f_{pb})S_{bc} - M_{d/nc}(S_{bc}/S_b - 1)$$

where

f_r = modulus of rupture = $0.24\sqrt{f'_c} = 0.24\sqrt{7.0} = 0.635$ ksi [LRFD Art. 5.4.2.6]

f_{pb} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} = \frac{1{,}029.6}{767} + \frac{(1{,}029.6)(30.78)}{14{,}915} = 1.342 + 2.125 = 3.467 \text{ ksi}$$

$M_{d/nc}$ = moment due to non-composite dead loads, ft-kips

= $M_g + M_s$ = 1,390.7 + 2,126.9 = 3,517.6 ft-kips

S_{bc} = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 20,070 in.^{4}

S_b = non-composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 14,915 in.^{4}

$$M_{cr} = (0.635 + 3.467)(20{,}070/12) - (3{,}517.6)\left(\frac{20{,}070}{14{,}915} - 1\right) = 5{,}644.8 \text{ ft-kips}$$

$1.2M_{cr}$ = 1.2(5,644.8) = 6,773.8 ft-kips

At midspan, the factored moment required by the Strength I load combination is:

M_u = 8,381.6 ft-kips (as calculated in Section 9.6.9)

Therefore, $1.33M_u$ = 1.33 x 8,381.6 = 11,147.5 ft-kips

Since $1.2M_{cr} < 1.33M_u$, $1.2M_{cr}$ (Controls)

M_r = 10,648.9 ft-kips > $1.2M_{cr}$ O.K.

Note: Contrary to the *Standard Specifications*, the *LRFD Specifications* state that this requirement be met at every section.

9.6.10.2 Negative Moment Section

9.6.10.2.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The maximum amount of prestressed and nonprestressed reinforcement shall be such that:

$$\frac{c}{d_e} \le 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

where

$$d_e = \frac{A_{ps}f_{ps}d_p + A_sf_yd_s}{A_{ps}f_{ps} + A_sf_y} \qquad \text{[LRFD Eq. 5.7.3.3.1-2]}$$

Since A_{ps} = 0, then $d_e = d_s$ = 76.25 in.

β_1 = stress factor of compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f'_c \le 4.0$ ksi

= $0.85 - 0.05(f'_c - 4.0) \le 0.65$ for $f'_c \ge 4.0$

= 0.85 − 0.05(7.0 − 4.0) = 0.70

$$c = \frac{a}{\beta_1} = \frac{6.02}{0.70} = 8.60 \text{ in.}$$

$$\frac{c}{d_s} = \frac{8.60}{76.25} = 0.113 \leq 0.42 \quad \text{O.K.}$$

Note that the value of "a" used here is not exact because the geometry of the bottom flange must be accommodated. But since "a" is slightly larger than 6 in., the uniform width portion of the bottom flange, and since (c/d_s) is much lower than the maximum limit, further refinement is not warranted.

9.6.10.2.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

For nonprestressed sections, the minimum reinforcement provision may be considered satisfied if:

$$\rho \geq 0.03\frac{f'_c}{f_y} \quad \text{[LRFD Eq. 5.7.3.3.2-1]}$$

where ρ = ratio of tension steel to gross area = $A_s/(bd) = \dfrac{(15.52)}{(26)(76.25)} = 0.008$

$$0.008 \geq 0.03\frac{7.0}{60} = 0.0035 \quad \text{O.K.}$$

At the negative moment section, the bottom flange of the precast beam acts as the compression block of the composite section. Therefore, the 28-day strength of the beam concrete, 7.0 ksi, is used.

9.6.11 SHEAR DESIGN

The area and spacing of shear reinforcement must be determined at regular intervals along the entire length of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement must be provided when:

$V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

where

V_u = total factored shear force, kips

V_c = shear strength provided by concrete, kips

V_p = component of the effective prestressing force in the direction of the applied shear, kips

ϕ = resistance factor [LRFD Art. 5.5.4.2.1]

9.6.11.1 Critical Section

Critical section near the supports is the greater of: [LRFD Art. 5.8.3.2]

$0.5d_v\cot\theta$, or, d_v

where

d_v = effective shear depth

= distance between resultants of tensile and compressive forces, ($d_e - a/2$) but not less than $0.9d_e$ or $0.72h$ [LRFD Art. 5.8.2.7]

where

d_e = the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement = 76.25 in.

(Note: d_e is calculated considering the nonprestressed reinforcement in the slab as the main reinforcement and neglecting the prestress reinforcement. This is because this section lies in the negative moment zone.)

a = equivalent depth of the compression block = 6.02 in. (from negative moment flexural design)

h = total height of the section = 80.0 in.

θ = angle of inclination of diagonal compressive stresses, assumed = 32° (slope of compression field)

9.6.11.1.1 Angle of Diagonal Compressive Stresses

The design for shear depends on the angle of diagonal compressive stresses at the section under consideration. The shear design is an iterative process that begins with assuming a value for θ. For this example, only the results of the final cycle of calculations are shown.

9.6.11.1.2 Effective Shear Depth

$d_v = 76.25 - 0.5(6.02) = 73.24$ in.

$\geq 0.9d_e = 0.9(76.25) = 68.63$ in.

$\geq 0.72h = 0.72(80.0) = 57.60$ in.

Therefore, $d_v = 73.24$ in.

9.6.11.1.3 Calculation of Critical Section

The critical section near the support is the greater of:

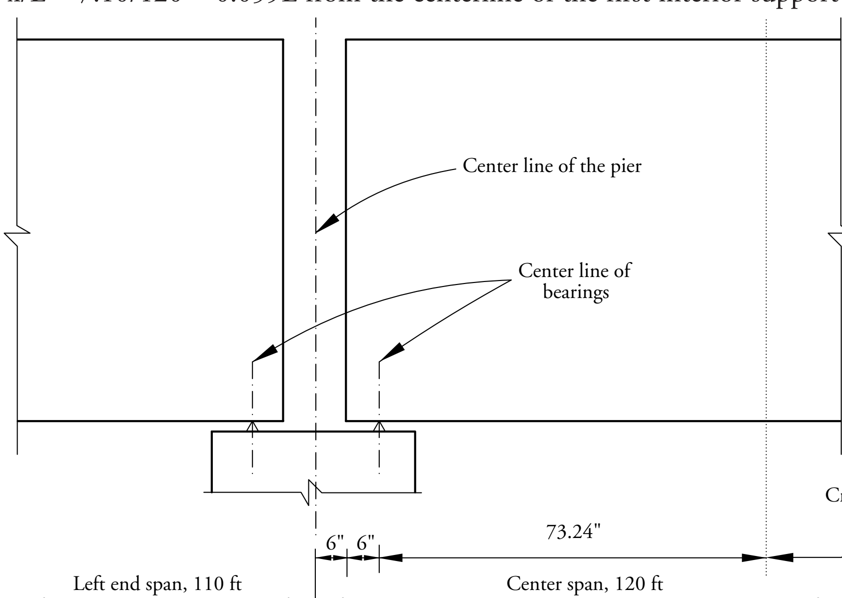
$d_v = 73.24$ in.   (Controls)

$0.5d_v\cot\theta = 0.5(73.24)\cot 32° = 58.60$ in.

Because the width of the bearing is not yet determined, the width of bearing was conservatively assumed to be equal to zero for the computation of the critical section of shear, as shown in **Figure 9.6.11-1**. Therefore the critical section in shear is at a distance of 0.5 + 0.5 + 73.19/12 = 7.10 ft from the centerline of the first interior support (pier).

x/L = 7.10/120 = 0.059L from the centerline of the first interior support (pier)

Figure 9.6.11-1 Critical Section in Shear of the Center Span



Using values from **Table 9.6.4-1**, compute the factored shear force and bending moment at the critical section for shear (center span point 0.059), according to Strength I load combinations.

$V_u = 1.25(42.3 + 64.6 + 7.8) + 1.50(14.2) + 1.75(137.3) = 405.0$ kips

$M_u = 1.25(272.7 + 417.1 - 139.6) + 1.50(-244.4) + 1.75(-1{,}717.8) = -2{,}685.0$ ft-kips

or,

$V_u = 0.9(42.3 + 64.6 + 7.8) + 1.50(14.2) + 1.75(137.3) = 364.8$ kips

$M_u = 0.9(272.7 + 417.1 - 139.6) + 1.50(-244.4) + 1.75(-1{,}717.8) = -2{,}877.6$ ft-kips

When determining M_u at a particular section, it is conservative to take M_u as the highest factored moment that will occur at that section, rather than the moment corresponding to maximum V_u, (LRFD Art. C5.8.3.4.2). Therefore,

$V_u = 405.0$ kips

$M_u = -2{,}877.6$ ft-kips

9.6.11.2 Contribution of Concrete to Nominal Shear Resistance

The contribution of the concrete to the nominal shear resistance is:

$$V_c = 0.0316\beta\sqrt{f_c'}\ b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

Several quantities must be determined before this expression can be evaluated.

9.6.11.2.1 Strain in Flexural Tension Reinforcement

Calculate strain in the reinforcement, ε_x:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \le 0.001 \qquad \text{[LRFD Eq. 5.8.3.4.2-2]}$$

where

N_u = applied factored normal force at the specified section = 0

V_p = component of the effective prestressing force in the direction of the applied shear = (force per strand)(number of draped strands)(sin ψ)

Force per strand = 23.4 kips

From Section 9.6.7.5, $\psi = 7.2°$

$V_p = (23.4)(12)\sin 7.2° = 35.2$ kips

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi). For pretensioned members, LRFD Article C5.8.3.4.2 indicates that f_{po} can be taken as the stress in the strands when the concrete is cast around them, which is the jacking stress, f_{pj}, or $0.75f_{pu}$.

= 0.75(270.0) = 202.5 ksi

A_{ps} = area of prestressing steel on the flexural tension side of the member. The flexural tension side of the member should be taken as the half-depth containing the flexural tension zone as illustrated in LRFD Figure 5.8.3.4.2-3.

A_{ps} = 12(0.153) = 1.836 in.^{2}

A_s = area of nonprestressing steel on the flexural tension side of the member = 15.52 in.^{2}

$$\varepsilon_x = \frac{\dfrac{2{,}877.6\,(12)}{73.24} + 0 + 0.5(405.0 - 35.2)(\cot 32°) - 1.836(202.5)}{2[29{,}000(15.52) + 28{,}500(1.836)]} = +0.394 \times 10^{-3}$$

9.6.11.2.1.1
Shear Stress

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} \qquad \text{[LRFD Eq. 5.8.3.4.2-1]}$$

where

v_u = shear stress in concrete

b_v = effective web width of the beam = 6 in.

V_p = component of the effective prestressing force in the direction of the applied shear (calculated in Sect. 9.6.11.2.1)

$$v_u = \frac{405.0 - 0.9(35.2)}{(0.9)(6)(73.24)} = 0.944 \text{ ksi}$$

$(v_u/f'_c) = (0.944/7.0) = 0.135$

9.6.11.2.2
Values of β and θ

Having computed ε_x and v_u/f'_c, find a better estimate of θ from LRFD Table 5.8.3.4.2-1. Since the computed value of v_u/f'_c is likely to fall between two rows in the table, a linear interpolation may be performed. However, for hand calculations, interpolation is not recommended (LRFD Art. C5.8.3.4.2). The values of θ in the lower row that bounds the computed value may be used. Similarly, the values of β in the first column to the right of the computed value may be used. For this example, the applicable row and column are the ones labeled "≤ 0.150" and "≤ 0.50", respectively. The values of θ and β contained in the cell of intersection of that row and column are:

θ = 32.1° which is close to assumed θ of 32.0°.

Thus, no further iteration is needed. However, if the designer desires to go through further iteration, it should be kept in mind that the position of the critical section of shear and consequently the values of V_u and M_u will need to be based on the new value of θ, 32.1°.

β = 2.36

where β = a factor indicating the ability of diagonally cracked concrete to transmit tension; a value indicating concrete contribution.

9.6.11.2.3
Concrete Contribution

The nominal shear resisted by the concrete is:

$$V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v d_v \qquad \text{[LRFD Eq. 5.8.3.3-3]}$$

$$= 0.0316(2.36)\sqrt{7.0}\,(6)(73.24) = 86.7 \text{ kips}$$

9.6.11.3
Contribution of Reinforcement to Nominal Shear Resistance

9.6.11.3.1
Requirement for Reinforcement

Check if $V_u > 0.5\phi(V_c + V_p)$ [LRFD Eq. 5.8.2.4-1]

$V_u = 405.0 \text{ kips} > 0.5\phi(V_c + V_p) = 0.5(0.9)(86.7 + 35.2) = 54.9 \text{ kips}$

Therefore, transverse shear reinforcement should be provided.

9.6.11.3.2 Required Area of Reinforcement

$V_u/\phi \le V_n = V_c + V_s + V_p$ [LRFD Eq. 5.8.3.3-1]

where

V_s = shear force carried by transverse reinforcement

$= (V_u/\phi) - V_c - V_p = (405.0/0.9) - 86.7 - 35.2 = 328.1$ kips

$$V_s = \frac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s}$$ [LRFD Eq. 5.8.3.3-4]

where

A_v = area of shear reinforcement within a distance s, in.^{2}

s = spacing of stirrups, in.

f_y = yield strength of shear reinforcement, ksi

α = angle of inclination of transverse reinforcement to longitudinal axis = 90°

Therefore, area of shear reinforcement (in.^{2}) within a spacing, s, is:

req'd $A_v = (sV_s)/(f_y d_v \cot\theta)$

$= s(328.1)/(60)(73.24\cot 32°) = 0.047(s)$ in.^{2}

if s = 12 in., the required $A_v = 0.56$ in.^{2}

Try #5 double legs @ 12 in. spacing.

$$A_v \text{ provided} = (2)(0.31)\left(\frac{12}{12}\right) = 0.62 \text{ in.}^2/\text{ft} > A_v \text{ required} = 0.56 \text{ in.}^2/\text{ft}$$ O.K.

$$V_s \text{ provided} = \frac{(0.62)(60)(73.24)\cot 32°}{12} = 363.2 \text{ kips}$$

9.6.11.3.3 Spacing of Reinforcement

Check maximum spacing of transverse reinforcement: [LRFD Art. 5.8.2.7]

Check if $v_u < 0.125f'_c$ [LRFD Eq. 5.8.2.7-1]

or if $v_u \ge 0.125f'_c$ [LRFD Eq. 5.8.2.7-2]

$0.125f'_c = (0.125)(7.0) = 0.875$ ksi

$v_u = 0.944$ ksi

Since $v_u > 0.125f'_c$

Then s ≤ 12 in. ≤ $0.4d_v = 0.4(73.24) = 29.3$ in.

Therefore, s ≤ 12 in.

Actual spacing, s = 12 in. O.K.

9.6.11.3.4 Minimum Reinforcement Requirement

The area of transverse reinforcement should not be less than:

$$0.0316\sqrt{f'_c}\frac{b_v s}{f_y}$$ [LRFD Eq. 5.8.2.5-1]

$$0.0316\sqrt{7.0}\left(\frac{6(12)}{60}\right) = 0.100 \text{ in.}^2 < A_v \text{ provided}$$ O.K.

9.6.11.4 Maximum Nominal Shear Resistance

In order to ensure that the concrete in the web of the beam will not crush prior to yielding of the transverse reinforcement, the *LRFD Specifications* give an upper limit of V_n.

$V_n = 0.25f'_c b_v d_v + V_p$ [LRFD Eq. 5.8.3.3-2]

Comparing this equation with LRFD Eq. 5.8.3.3-2, it can be concluded that,

$V_c + V_s \leq 0.25f'_c b_v d_v$

$86.7 + 328.1 = 414.8 \text{ kips} \leq 0.25(7.0)(6)(73.24) = 769.0 \text{ kips}$ O.K.

Using the foregoing procedures, the transverse reinforcement can be determined at increments along the entire length of the beam.

9.6.12 INTERFACE SHEAR TRANSFER

[LRFD Art. 5.8.4]

9.6.12.1 Factored Horizontal Shear

At the strength limit state, the horizontal shear at a section on a per unit basis can be taken as:

$$V_h = \frac{V_u}{d_v}$$ (LRFD Eq. C5.8.4.1-1)

where

V_h = horizontal factored shear force per unit length of the beam, kips/in.

V_u = factored shear force due to superimposed loads, kips

d_v = distance between resultants of tensile and compressive forces, $(d_e - a/2)$

The *LRFD Specifications* does not identify the location of the critical section. For convenience, it will be assumed here to be the same location as the critical section for vertical shear, at point 0.059 of the center span.

Using load combination Strength I:

$V_u = 1.25(7.8) + 1.50(14.2) + 1.75(137.3) = 271.3 \text{ kips}$

$d_v = 73.24 \text{ in.}$

$$\text{Required } V_h = \frac{271.3}{73.24} = 3.70 \text{ kips/in.}$$

9.6.12.2 Required Nominal Resistance

$\text{Required } V_n = V_h/0.9 = 3.70/0.9 = 4.11 \text{ kips/in.}$

9.6.12.3 Required Interface Shear Reinforcement

The nominal shear resistance of the interface plane is:

$V_n = c\,A_{cv} + \mu[A_{vf}f_y + P_c]$ [LRFD Eq. 5.8.4.1-1]

where

c = cohesion factor, ksi [LRFD Art. 5.8.4.2]

μ = friction factor [LRFD Art. 5.8.4.2]

A_{cv} = area of concrete engaged in shear transfer, in.^{2}

A_{vf} = area of shear reinforcement crossing the shear plane, in.^{2}

P_c = permanent net compressive force normal to the shear plane, kips

f_y = shear reinforcement yield strength, ksi

For concrete placed against clean, hardened concrete with the surface intentionally roughened: [LRFD Art. 5.8.4.2]

c = 0.1 ksi

μ = 1.0λ

where λ = 1.0 for normal weight concrete

The actual contact width, b_v, between the deck and the beam is 42 in. Therefore,

A_{cv} = (42 in.)(1 in.) = 42 in.^{2}/in.

LRFD Eq. 5.8.4.1-1 can be solved for A_{vf} as follows:

$4.12 = 0.1(42) + 1.0[A_{vf}(60) + 0]$

$A_{vf} < 0$

Since the resistance provided by cohesion is higher than the applied force, provide the minimum required interface reinforcement.

9.6.12.3.1 Minimum Interface Shear Reinforcement

Minimum shear reinforcement, $A_{vf} \geq (0.05b_v)/f_y$ [LRFD Eq. 5.8.4.1-4]

where b_v = width of the interface

From the design of vertical shear reinforcement, a #5 double-leg bar at 12-in. spacing is provided from the beam extending into the deck. Therefore, A_{vf} = 0.62 in.^{2}/ft.

A_{vf} = (0.62 in.^{2}/ft) > $(0.05b_v)/f_y$ = 0.05(42)/60 = 0.035 in.^{2}/in. = 0.42 in.^{2}/ft O.K.

Consider further that LRFD Article 5.8.4.1 states that the minimum reinforcement requirement may be waived if V_n/A_{cv} < 0.100 ksi.

4.11 kips/in./42.0 in. = 0.098 ksi < 0.100 ksi

Therefore, the minimum reinforcement requirement could be waived had it governed.

9.6.12.4 Maximum Nominal Shear Resistance

$$\text{Provided } V_n = (0.1)(42) + 1.0\left[\frac{0.62}{12}(60) + 0\right] = 7.3 \text{ kips/in.}$$

$0.2f'_cA_{cv} = 0.2(4.0)(42) = 33.6$ kips/in.

$0.8A_{cv} = 0.8(42) = 33.6$ kips/in.

V_n provided $\leq 0.2f'_c A_{cv}$ O.K. [LRFD Eq. 5.8.4.1-2]

$\leq 0.8A_{cv}$ O.K. [LRFD Eq. 5.8.4.1-3]

9.6.13 MINIMUM LONGITUDINAL REINFORCEMENT REQUIREMENT

[LRFD Art. 5.8.3.5]

The LRFD Specifications state that if the reaction force or the load at the maximum moment location introduces direct compression into the flexural compression face of the member, the area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone.

This reason that the longitudinal reinforcement requirement is relaxed for this condition, is based on the following explanation. At maximum moment locations, the shear force changes sign and, hence, the inclination of the diagonal compressive stresses also changes. At direct supports and point loads, this change of inclination is associated with a fan-shaped pattern of compressive stresses radiating from the point load or the direct support. This fanning of the diagonal stresses reduces the tension in the longitudinal reinforcement caused by the shear, i.e., angle θ becomes steeper.

The conditions mentioned above exist at the interior supports. Directly over the support, the angle θ becomes 90° and the contribution of shear to the longitudinal reinforcement requirement is zero. Therefore, at this location, the longitudinal reinforcement is sized only for the moment applied to the section and there is no need to check the minimum longitudinal reinforcement requirement.

However, for sections within a distance of $(d_v \cot\theta)/2$ from the interior supports, the shear will again affect the required longitudinal reinforcement and the requirement must be checked. It should be noted that at locations near the interior supports of continuous members, the minimum longitudinal reinforcement requirement is used to check the quantity of reinforcement in the deck. The longitudinal reinforcement requirement must also be checked for the prestressing strands at the simply-supported ends of continuous span units. Refer to Design Example 9.4, Section 9.4.13.

9.6.14 PRETENSIONED ANCHORAGE ZONE

[LRFD Art. 5.10.10]

9.6.14.1 Anchorage Zone Reinforcement

[LRFD Art. 5.10.10.1]

Design of the anchorage zone reinforcement is computed using the force in the strands just before transfer.

Force in the strands before transfer = F_{pi} = 44(0.153)(202.5) = 1,363.2 kips

The bursting resistance, P_r, should not be less than 4.0% of F_{pi}.

[LRFD Arts. 5.10.10.1 and C3.4.3]

$P_r = f_s A_s \geq 0.04 f_{pi} = 0.04(1{,}363.2) = 54.5$ kips

where

A_s = total area of transverse reinforcement located within the distance h/4 from the end of the beam, in.^{2}

f_s = stress in steel, but not taken greater than 20 ksi

Solving for the required area of steel, A_s = 54.5/(20) = 2.73 in.^{2}

At least 2.73 in.^{2} of vertical transverse reinforcement should be provided at the end of the beam for a distance equal to one-fourth of the depth of the beam, h/4 = 72/4 = 18.0 in.

The shear reinforcement was determined in Section 9.6.11 to be #5 (double legs)@ 10 in. However, the minimum vertical reinforcement criteria controls. Therefore, for a distance of 18.0 in. from the end of the member, use 5 #5 @ 4 in. The reinforcement provided is 5(2)(0.31) = 3.10 in.^{2} > 2.73 in.^{2} O.K.

9.6.14.2 Confinement Reinforcement

[LRFD Art. 5.10.10.2]

For a distance of 1.5d = 1.5(72) = 108 in., from the end of the beam, reinforcement is placed to confine the prestressing steel in the bottom flange. The reinforcement should not be less than #3 deformed bars, with spacing not exceeding 6.0 in., and shaped to enclose the strands.

9.6.15 DEFLECTION AND CAMBER

[LRFD Art. 5.7.3.6.2]

Deflections are calculated using the modulus of elasticity of concrete calculated in Section 9.6.2 and the moment of inertia of the non-composite precast beam.

9.6.15.1 Deflection Due to Prestressing Force at Transfer

Force per strand at transfer = 28.2 kips

$$\Delta_p = \frac{P_i}{E_{ci}I}\left(\frac{e_c L^2}{8} - \frac{e' a^2}{6}\right)$$

where

P_i = total prestressing force at transfer = 44(28.2) = 1,240.8 kips

e_c = eccentricity of prestressing force at midspan = 30.78 in.

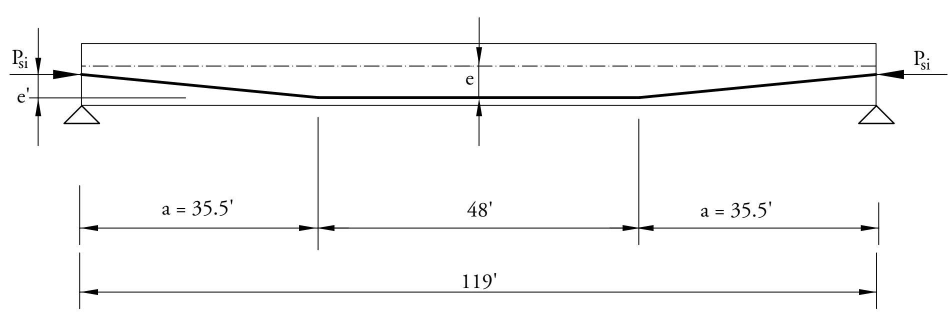
e' = difference between eccentricity of prestressing force at midspan and end, as shown in **Figure 9.6.15.1-1**

= $e_c - e_e$ = 30.78 − 19.42 = 11.36 in.

a = distance from end of beam to harp point = 35.5 ft

L = beam length = 119.0 ft

Figure 9.6.15.1-1 Strand Eccentricity



$$\Delta_p = \frac{1,240.8}{4,496(545,894)}\left(\frac{30.78(119\text{x}12)^2}{8} - \frac{11.36\,(35.5\text{x}12)^2}{6}\right) = 3.79 \text{ in.} \uparrow$$

9.6.15.2 Deflection Due to Beam Self-Weight

$$\Delta_g = \frac{5wL^4}{384E_{ci}I}$$

where w = beam self-weight, kip/ft

Deflection due to beam self-weight at transfer:

L = overall beam length = 119 ft

$$\Delta_g = \frac{5\left(\frac{0.799}{12}\right)(119\text{x}12)^4}{(384)(4,496)(545,894)} = 1.47 \text{ in.} \downarrow$$

Deflection due to beam self-weight at erection:

L = span length between centerlines of bearings = 118 ft

$$\Delta_g = \frac{5\left(\frac{0.799}{12}\right)(118 \text{ x } 12)^4}{(384)(4,496)(545,894)} = 1.42 \text{ in.} \downarrow$$

9.6.15.3 Deflection Due to Haunch and Deck

$$\Delta_s = \frac{5wL^4}{384E_cI}$$

where

w = deck slab plus haunch weights, kip/ft

L = span length between centerlines of bearings, ft

E_c = modulus of elasticity of precast beam at 28 days

$$\Delta_s = \frac{5\left(\frac{1.222}{12}\right)(118 \text{ x } 12)^4}{(384)(5,072)(545,894)} = 1.93 \text{ in.} \downarrow$$

9.6.15.4 Deflection Due to Barrier and Future Wearing Surface

Δ_{b+ws} = 0.048 in. ↓

(This value was calculated using a continuous beam program.)

9.6.15.5 Deflection and Camber Summary

For midspan:

At transfer, $(\Delta_p + \Delta_g) = 3.79 - 1.47 = 2.32$ in. ↑

Total deflection at erection, using PCI multipliers (see the *PCI Design Handbook*)

$= 1.8(3.79) - 1.85(1.42) = 4.20$ in. ↑

Long-Term Deflection

LRFD Article 5.7.3.6.2 states that the long-term deflection may be taken as the instantaneous deflection multiplied by a factor of 4.0, if the instantaneous deflection is based on the gross moment of inertia. However, a factor of 4.0 is not appropriate for this type of precast construction. Therefore, it is recommended that the designer follow the guidelines of the owner agency for whom the bridge is being designed, or undertake a more rigorous time-dependent analysis.

9.6.15.6 Deflection Due to Live Load and Impact

Live load deflection is not a required check, according to the provisions of the *LRFD Specifications*. Further, live load deflections are usually not a problem for prestressed concrete I- and bulb-tee shapes especially when they are constructed to act as a continuous structure under superimposed loads. If the designer chooses to check deflection, the following recommendations are from the *LRFD Specifications*.

Live load deflection limit: Span/800 = (120)(12)/800 = 1.80 in. [LRFD Art. 2.5.2.6.2]

If the owner invokes the optional live load deflection criteria specified in LRFD Article 2.5.2.6.2, the deflection is the greater of: [LRFD Art. 3.6.1.3.2]

- that resulting from the design truck alone, or,
- that resulting from 25% of the design truck taken together with the design lane load

The *LRFD Specifications* state that all beams may be assumed to be deflecting equally under the applied live load and impact. [LRFD Art. 2.5.2.6.2]

Therefore, the distribution factor for deflection is calculated as follows:

(number of lanes/number of beams) = 3/4 = 0.75 [LRFD Art. C2.5.2.6.2]

However, it is more conservative to use the distribution factor for moment.

The live load deflection may be conservatively estimated using the following formula:

$$D = \frac{5L^2}{48EI_c}\left[M_s - 0.1(M_a + M_b)\right] \qquad \text{(Eq. 9.6.15.6-1)}$$

where

M_s = the maximum positive moment

M_a and M_b = the corresponding negative moments at the ends of the span being considered.

The live load combination specified in LRFD Article 3.6.1.3.2 calls for the greater of design truck alone or 0.25 design truck plus lane load.

In this example, a conservative approximation may be made by using the positive moment for Service III load combination, 0.8 truck plus lane load, and by ignoring the effect of M_a and M_b.

$$\Delta_L = \frac{5(120 \text{ x } 12)^2}{48(5{,}072)(1{,}097{,}252)}\left[0.8 \text{ x } 2{,}115.0 \text{ x } 12\right] = 0.79 \text{ in.} \downarrow < 1.80 \text{ in.} \quad \text{O.K.}$$

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci} = specified compressive strength of concrete at time of initial loading or pretensioning [LRFD]

f_{cgp} = concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment [LRFD]

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pb} = compressive stress at bottom fiber of the beam due to prestress force

f_{pc} = compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads [STD]

f_{pc} = compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone [LRFD]

f_{pe} = compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads [STD]

f_{pe} = effective stress in the pretensioning steel after losses [LRFD]

f_{pi} = initial stress immediately before transfer

f_{po} = stress in the pretensioning steel when the stress in the surrounding concrete is zero [LRFD]

f_{ps} = average stress in pretensioning steel at the time for which the nominal resistance of member is required [LRFD]

f_{pt} = stress in pretensioning steel immediately after transfer [LRFD]

f_{pu} = specified tensile strength of pretensioning steel [LRFD]

f_{py} = yield strength of pretensioning steel [LRFD]

f_r = the modulus of rupture of concrete [STD]

f_r = modulus of rupture of concrete [LRFD]

f_s = allowable stress in steel

f'_s = ultimate stress of pretensioning reinforcement [STD]

f_{se} = effective final pretension stress

f_{si} = effective initial pretension stress

f^*_{su} = average stress in pretensioning steel at ultimate load [STD]

f_t = concrete stress at top fiber of the beam for the non-composite section

f_{tc} = concrete stress at top fiber of the slab for the composite section

f_{tg} = concrete stress at top fiber of the beam for the composite section

f_y = yield strength of reinforcing bars [STD]

f_y = specified minimum yield strength of reinforcing bars [LRFD]

f_y = yield stress of pretensioning reinforcement [STD]

f'_y = specified minimum yield strength of compression reinforcement [LRFD]

f_{yh} = specified yield strength of transverse reinforcement [LRFD]

H = average annual ambient mean relative humidity, percent [LRFD]

H = height of wall [LRFD]

h = overall depth of precast beam [STD]

h = overall depth of a member [LRFD]

h_c	= total height of composite section	
h_f	= compression flange depth	[LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD]
I	= moment of inertia about the centroid of the non-composite precast beam	[LRFD]
I	= impact fraction (maximum 30%)	[STD]
I_c	= moment of inertia of composite section	
IM	= dynamic load allowance	[LRFD]
J	= St. Venant torsional constant	
K	= longitudinal stiffness parameter	[STD]
K_g	= longitudinal stiffness parameter	[LRFD]
k	= factor used in calculation of distribution factor for multi-beam bridges	[LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
L_c	= critical length of yield line failure pattern	[LRFD]
LL	= vehicular live load	[LRFD]
ℓ_d	= development length	[LRFD]
ℓ_x	= length required to fully develop the strand measured from the end of the strand	
M_a	= negative moment at the end of the span being considered	
M_b	= negative moment at the end of the span being considered	
M_b	= unfactored bending moment due to barrier weight	
M_c	= flexural resistance of cantilevered wall	[LRFD]
M_{CIP}	= unfactored bending moment due to cast-in-place topping slab	
M_{const}	= unfactored bending moment due to construction load	
M_{col}	= bending moment due to horizontal collision force	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads (after dead load)	[STD]
M_{cr}	= cracking moment	[LRFD]
M^*_{cr}	= cracking moment	[STD]
M_D	= unfactored bending moment due to diaphragm weight	
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_f	= unfactored bending moment due to fatigue truck per beam	
M_g	= unfactored bending moment due to beam self-weight	
M_{LL}	= unfactored bending moment due to lane load per beam	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_{LL+I}	= unfactored bending moment due to design vehicular load	
M_{LT}	= unfactored bending moment due to truck load with dynamic allowance per beam	

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S = span length of deck slab [STD]

S = effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web LRFD]

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam [STD]

S_{bc} = composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)

SH = loss of pretension due to concrete shrinkage [STD]

SR = fatigue stress range

S_t = section modulus for the extreme top fiber of the non-composite precast beam

S_{tc} = composite section modulus for top fiber of the deck slab

S_{tg} = composite section modulus for top fiber of the precast beam

s = longitudinal spacing of the web reinforcement [STD]

s = length of a side element [LRFD]

s = spacing of rows of ties [LRFD]

T = collision force at deck slab level

t = thickness of web

t = thickness of an element of the beam

t_f = thickness of flange

t_s = cast-in-place deck thickness

t_s = depth of concrete deck [LRFD]

V_c = nominal shear strength provided by concrete [STD]

V_c = nominal shear resistance provided by tensile stresses in the concrete [LRFD]

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment [STD]

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web [STD]

V_d = shear force at section due to unfactored dead load [STD]

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} [STD]

V_{LL} = unfactored shear force due to lane load per beam

V_{LL+I} = unfactored shear force due to live load plus impact

V_{LL+I} = unfactored shear force due design vehicular live load

V_{LT} = unfactored shear force due to truck load with dynamic allowance per beam

V_{mu} = ultimate shear force occurring simultaneously with M_u

V_n = nominal shear resistance of the section considered [LRFD]

V_{nh} = nominal horizontal shear strength [STD]

V_p = vertical component of effective pretension force at section [STD]

V_p = component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear [LRFD]

V_s = nominal shear strength provided by web reinforcement [STD]

V_s = shear resistance provided by shear reinforcement [LRFD]

V_u = factored shear force at the section [STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	= $\dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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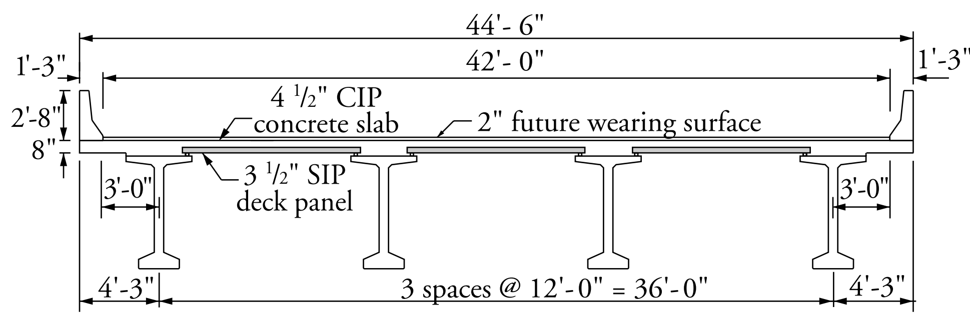
STAY-IN-PLACE PANEL DECK, STANDARD SPECIFICATIONS

Precast Concrete Stay-In-Place Deck Panel System, Standard Specifications

9.7.1 INTRODUCTION

This design example demonstrates the design of a 3-1/2-in.-thick precast, pretensioned stay-in-place (SIP) deck panel with a 4-1/2-in.-thick cast-in-place (CIP) concrete topping. A 1/2-in. wearing surface is considered to be an integral part of the 4-1/2-in. topping slab. The bridge has 3-lanes and a total width of 44'-6". The deck slab is supported over four AASHTO-PCI bulb-tee beams spaced at 12 ft on center and has overhangs of 4'-3", as shown in **Figure 9.7.1-1**. The CIP concrete requires a concrete strength of 4,000 psi at the time of opening the bridge for traffic. The SIP panel needs a strength of 6,500 psi at the time of transfer of the pretensioning force to the panel and a design strength of 8,000 psi at the time of casting the slab. A New Jersey-type barrier is included. The design is conducted in compliance with the *Standard Specifications,* 17th edition, 2002.

Figure 9.7.1-1
Bridge Cross-Section



9.7.2 MATERIALS

Cast-in-place concrete:

Actual thickness = 4-1/2 in.

Structural thickness = 4 in. (a 1/2-in. wearing surface is considered to be an integral part of the deck)

Specified concrete strength at 28 days, f'_c = 4,000 psi

Concrete unit weight, w_c = 150 pcf

Superstructure beams: AASHTO-PCI bulb-tee beam as shown in **Figure 9.7.2-1**

Beam spacing = 12.0 ft

Deck overhang = 4.25 ft

Top flange width = 42 in.

Precast pretensioned SIP deck panels:

Concrete strength at transfer, f'_{ci} = 6,500 psi

28-Day strength, f'_c = 8,000 psi

Concrete unit weight, w_c = 150 pcf

Prestressing strands:

1/2-in. diameter, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate stress, $f_s' = 270{,}000$ psi

Yield strength, $f_y^* = 0.9f_s' = 243{,}000$ psi

Initial pretensioning, $f_{si} = 0.75f_s' = 202{,}500$ psi [STD Art. 9.15]

Modulus of elasticity, $E_s = 28{,}000$ ksi [STD Art. 9.16.2.1.2]

Panel dimensions: 8-ft wide x 9 ft-6 in. long x 3-1/2 in. deep

Reinforcement is welded wire reinforcement:

Yield strength, $f_y = 60{,}000$ psi

Modulus of elasticity, $E_s = 29{,}000$ ksi [STD Art. 8.7.2]

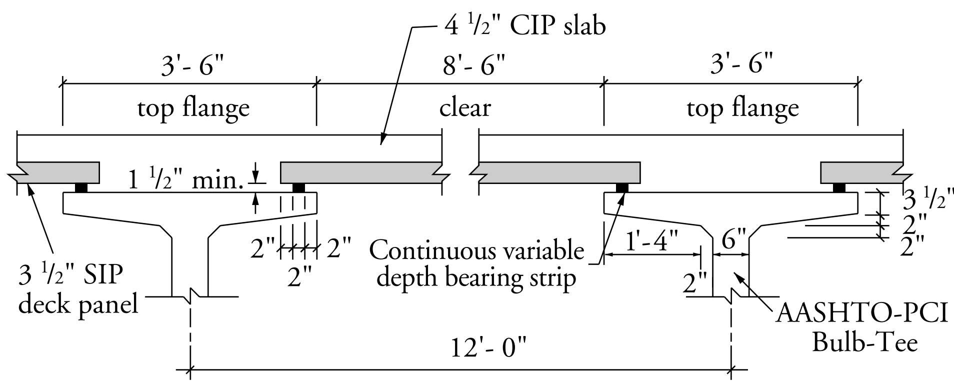
Top reinforcement clear cover = 2.5 in. [STD Art. 8.22.1]

Bottom reinforcement clear cover = 1.0 in.[STD Art. 8.22.1]

Future wearing surface: additional 2 in., unit weight = 150 pcf

New Jersey-type barrier: unit weight = 300 lb/ft/side

Figure 9.7.2-1
Details of the SIP Deck Panel on Supports



9.7.3
LOADS

[STD Art. 9.12]

Art. 9.12.1 states that precast SIP panels should be analyzed assuming that they support their own weight, any construction loads and the weight of the CIP concrete. Under subsequent superimposed dead and live loads, the precast panels should be analyzed assuming they act compositely with the CIP concrete.

9.7.3.1
Effective Span Length

In the design of the SIP panel, the effective span length is taken conservatively as the panel length, 9'-6", as shown in **Figure 9.7.2-1.**

For the composite section, STD Art. 3.24.1.2 states that slabs continuous over more than two supports and supported on the rigid top flange of pretensioned beams, with top flange width-to-minimum-thickness ratio greater than 4.0, S is the distance between edges of the top flange plus one-half of beam top flange width.

Therefore, $S = 8.5 + 0.5\left(\dfrac{42}{12}\right) = 10.25$ ft.

9.7.3.2
Dead Loads

Weight of 3.5-in.-thick SIP panel $= \left(\dfrac{3.5}{12}\right)(0.150) = 0.044$ kip/ft^2

Weight of 4.5-in.-thick CIP slab = $\left(\frac{4.5}{12}\right)(0.150) = 0.056$ kip/ft²

Weight of 2 in. wearing surface = $\left(\frac{2.0}{12}\right)(0.150) = 0.025$ kip/ft²

Weight of New Jersey barrier = 0.300 kip/ft/side

Construction load (applied to the SIP precast panel only) = 0.050 kip/ft²

9.7.3.3 Live Load

[STD Art. 9.12.2.1]

Use HS20 loading with impact. For HS20, STD Article 3.24.3.1 states that for deck slabs with main reinforcement perpendicular to traffic, with spans from 2 to 24 ft,the live load bending moment without impact is = $\left(\frac{S+2}{32}\right)P_{2.0}$, ft-lb/ft [STD Eq. 3-15]

where:

S = effective span length = 10.25 ft

P_{20} = load on one rear wheel of the HS20 truck = 16,000 lb

Impact, $I = \dfrac{50}{(L+125)} \leq 0.3$ [STD Eq. 3.1]

where:

L = length in ft of the portion of the span that is loaded = 10.25 ft

$I = \dfrac{50}{(10.25+125)} = 0.37 > 0.3$, therefore, $I = 0.3$ (Controls)

In slabs continuous over three or more supports, a continuity factor of 0.8 is applied to both positive and negative moment. [STD Art. 3.24.3.1]

Therefore, live load bending moment with impact:

$$0.8\left(\frac{10.25+2}{32}\right)(16{,}000)(1+0.3)/1{,}000 = 6.37 \text{ ft-kips/ft}$$

9.7.3.4 Load Combination

[STD Art. 3.22]

For Group (N) = $\gamma[\beta_D(D) + \beta_L(L+I)]$ [STD Eq. 3-10]

For Service I = 1.0[(1.0)D + (1.0)(L+I)] [STD Table 3.22.1A]

For Load Factor Design I = 1.3[(1.0)D + (1.67)(L+I)] [STD Table 3.22.1A]

Fatigue: [STD Art. 8.16.8.3]

Fatigue need not be checked for concrete slabs with primary reinforcement perpendicular to traffic and designed in accordance with the specifications.

9.7.4 SECTION PROPERTIES

Section properties are calculated for a 12-in.-wide strip.

9.7.4.1 Non-Composite Section

A = area of cross-section of the precast panel = 3.5(12) = 42 in.²/ft

S_b = section modulus for the extreme bottom fiber of the non-composite precast panel

= $(12)(3.5)^2/6$ = 24.5 in.³/ft

S_t = section modulus for the extreme top fiber of the non-composite precast panel

= $(12)(3.5)^2/6$ = 24.5 in.³/ft

$E_c = (w_c)^{1.5}(33)\sqrt{f_c'}$ [STD Art. 8.7.1]

where

E_c = modulus of elasticity, psi

w_c = unit weight of concrete, pcf = 150 pcf

STD Article 8.7.1, indicates that the unit weight of normal weight concrete is 145 pcf. However, precast concrete mixes exhibit a relatively low water/cementitious material ratio and high density. Therefore, a unit weight of 150 pcf is used in this example. For high strength concrete, even this value may need to be increased based on test results.

f_c' = specified strength of concrete, (psi)

Therefore, the modulus of elasticity:

at transfer, $E_{ci} = 33(150)^{1.5}\sqrt{6{,}500}/1{,}000 = 4{,}888$ ksi

at service loads, $E_c = 33(150)^{1.5}\sqrt{8{,}000}/1{,}000 = 5{,}423$ ksi

9.7.4.2 Composite Section

For simplicity, the pretensioning reinforcement is ignored in the calculations of the composite section properties.

E_c (for the SIP) = 5,423 ksi

E_c (for the CIP slab) = $33(150)^{1.5}\sqrt{4{,}000}/1{,}000 = 3{,}834$ ksi

n = modular ratio between CIP slab and SIP panel = 3,834/5,423 = 0.707

The transformed width of the CIP slab = (0.707)(12) = 8.48 in.

Figure 9.7.4.2-1 shows the dimensions of the transformed composite section. Since a 1/2-in. wearing surface is considered to be an integral part of the 4-1/2 in. CIP slab, only the structural depth of the CIP slab, 4 in., is considered.

A_c = total area of the composite section = 12(3.5) + 8.48(4) = 42 + 33.92 = 75.92 in.^{2}/ft

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast panel = [42(3.5/2) + 33.92(3.5 + 2)]/(75.92) = 3.43 in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast panel = 3.50 − 3.43 = 0.07 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the CIP slab = 3.5 + 4.0 − 3.43 = 4.07 in.

I_c = moment of inertia of the composite section

$= (42)(3.5)^2/12 + 42(3.43 - 3.5/2)^2 + (33.92)(4)^2/12 + (33.92)(3.5 + 2 - 3.43)^2$

= 352 in.^{4}/ft

S_{bc} = composite section modulus for the extreme bottom fiber of the precast panel

$= (I_c/y_{bc}) = 352/3.43 = 102.6$ in.^{3}/ft

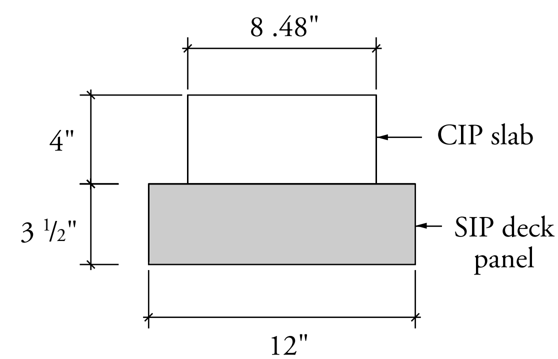
S_{tg} = composite section modulus for the top fiber of the precast panel

$= (I_c/y_{tg}) = 352/0.07 = 5{,}028.6$ in.^{3}/ft

S_{tc} = composite section modulus for extreme top fiber used to compute stresses at top fiber of the slab

$$= \left(\frac{1}{n}\right)(I_c/y_{tc}) = (1/0.707)(352/4.07) = (1/0.707)(86.5) = 122.4 \text{ in.}^3\text{/ft}$$

Figure 9.7.4.2-1
Transformed Composite Section



9.7.5 REQUIRED PRESTRESS

9.7.5.1 Service Load Stresses, Bottom Fiber, Composite Section

The number of strands required is usually governed by concrete tensile stress at service loads. Tensile stress at the bottom fiber of the composite section due to the self-weight of the SIP panel, the CIP concrete slab, wearing surface, barrier load and live loads is:

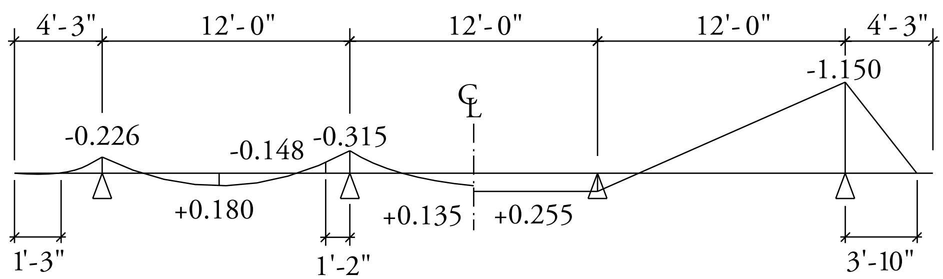
$$f_b = \frac{M_{SIP} + M_{CIP}}{S_b} + \frac{M_{ws} + M_b + M_{LL+I}}{S_{bc}}$$

For bending moment due to SIP panel and CIP slab, the span length is taken conservatively as the panel length, 9'-6", while for the wearing surface and live load, the span length is 10'-3".

$M_{SIP} = (0.044)(9.5)^2/8 = 0.496$ ft-kips/ft
$M_{CIP} = (0.056)(9.5)^2/8 = 0.632$ ft-kips/ft

The *Standard Specifications* does not give any guidance about how force actions of the composite deck can be determined. Therefore, the guidelines provided by the *LRFD Specifications* in Article 4.6.2.1.6 are used in this example, where the composite deck is considered as a continuous beam supported by infinitely rigid supports. **Figure 9.7.5.1-1** gives the bending moment due to the wearing surface and barrier load.

Figure 9.7.5.1-1
Bending Moments in ft-kips/ft



Moment due to Wearing Surface   Moment due to Barrier

To obtain maximum effects, consider the interior span,
where:

M_{ws} = 0.135 ft-kips/ft
M_b = 0.255 ft-kips/ft
M_{LL+I} = 6.370 ft-kips/ft (as calculated in Section 9.7.3.3)

$$f_b = \frac{(0.496+0.632)12}{24.5} + \frac{(0.135+0.255+6.370)12}{102.6} = 0.552 + 0.791$$
$$= 1.343 \text{ ksi}$$

9.7.5.2
Allowable Tensile Stress for Concrete

Allowable concrete tensile stress at service loads [STD Art. 9.15.2.2]

$$6\sqrt{f'_c} = 6\sqrt{8{,}000}/1{,}000 = 0.537 \text{ ksi}$$

9.7.5.3
Required Number of Strands

The required precompression stress at the bottom fiber = 1.343 – 0.537 = 0.806 ksi.

P_{se} is the total effective prestress force after all losses. Since the center of gravity of strands is concentric with the center of gravity of the SIP panel:

$$0.806 = \frac{P_{se}}{A} = \frac{P_{se}}{42}$$

P_{se} = 33.9 kips/ft = (33.9)(8.0) = 271.2 kips/panel

Using 1/2-in. diameter, 270 ksi, low-relaxation strand and assuming 15% final losses, the final prestress force per strand is = f_{si}(area of strand)(1-final losses)

$= (202.5)(0.153)(1 - 0.15) = 26.3$ kips

Required number of strands = 271.2/26.3 = 10.3 strands/panel

Try (10) 1/2-in. diameter, 270 ksi, low-relaxation strands per panel. The strands are placed in one layer at mid-height of the SIP panel.

9.7.6 PRESTRESS LOSSES

[STD Art. 9.16]

Total losses = SH + ES + CR_c + CR_s [STD Eq. 9-3]

where

SH = loss of prestress due to concrete shrinkage

ES = loss of prestress due to elastic shortening

CR_c = loss of prestress due to creep of concrete

CR_s = loss of prestress due to relaxation of pretensioning steel

9.7.6.1 Shrinkage

[STD Art. 9.16.2.1.1]

Relative humidity varies significantly within the country, see the U.S. map [STD Figure 9.16.2.1.1].

Assume relative humidity, RH = 70%

SH = 17,000 − 150(RH) [STD Eq. 9-4]

= [17,000 − 150(70)]/1,000 = 6.500 ksi

9.7.6.2 Elastic Shortening

[STD Art. 9.16.2.1.2]

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$ [STD Eq. 9-6]

where

E_s = 28,000 ksi

E_{ci} = 4,888 ksi

f_{cir} = concrete stress at the center of gravity of pretensioning steel due to pretensioning force and the self-weight of the panel, immediately after transfer

Assume an initial prestress loss of 5%.

Therefore, the total prestress force after transfer = (0.153)(202.5)(1 − 0.05)(10 strands) = 294.3 kips

f_{cir} = 294.3/[(3.5)(8)(12)] = 0.876 ksi

$$ES = \frac{28{,}000}{4{,}888} 0.876 = 5.018 \text{ ksi}$$

9.7.6.3 Creep of Concrete

[STD Art. 9.16.2.1.3]

$CR_c = 12f_{cir} - 7f_{cds}$ [STD Eq. 9-9]

where

f_{cir} = 0.876 ksi

f_{cds} = concrete stress at the center of gravity of pretensioning steel due to all dead loads except the dead load present at the time of applying the pretensioning force; i.e. due to the weight of the CIP slab, wearing surface and barriers.

Since the weight of the CIP slab acts on the non-composite section, i.e. the SIP panel, it provides zero stress at the center of gravity of pretensioning reinforcement. Therefore, only the stress due to the wearing surface and barriers is considered.

M_{ws} = 0.135 ft-kips/ft (as calculated in Section 9.7.5.1)

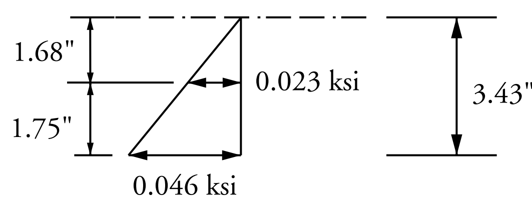
M_b = 0.255 ft-kips/ft

Bottom fiber stress = –(0.135 + 0.255)(12)/102.6 = –0.046 ksi

Therefore, as shown in **Figure 9.7.6.3-1**,
f_{cds} = (–0.046)(1.68)/3.43 = –0.023 ksi

CR_c = 12(0.876) – 7(0.023) = 10.351 ksi

Figure 9.7.6.3-1
Concrete Stresses due to Wearing Surface and Barriers



9.7.6.4 Relaxation of Pretensioning Steel

[STD Art. 9.16.2.1.4]

CR_s = 5000 – 0.1(ES) – 0.05(SH+CR_c) [STD Eq. 9-10A]
= 5.0 – 0.1(5.018) – 0.05(6.500 + 10.351) = 3.656 ksi

9.7.6.5 Total Loss at Transfer

Total initial losses, ES = 5.018 ksi

Initial prestress loss ratio = 5.018/202.5 = 2.5%

The 2.5% initial loss is close enough to the first estimation (5%) that there is no need to perform a second iteration to refine initial loss.

Effective prestress after transfer, f_{si} = (202.5 – 5.018) = 197.5 ksi

Total prestress force at transfer, P_{si} = (0.153)(197.5)(10 strands)= 302.2 kips/panel
= 302.2/8 = 37.8 kips/ft

9.7.6.6 Total Loss at Service Loads

Final prestress losses = 6.500 + 5.018 + 10.351 + 3.656 = 25.525 ksi

Final prestress loss ratio = 25.525/202.5 = 12.6%

Effective prestress, f_{se} = 202.5 – 25.525 = 176.975 ksi

Total prestress force at service loads, P_{se} = (0.153)(176.975)(10 strands)
= 270.8 kips/panel = 270.8/8.0 = 33.9 kips/ft

9.7.7 STRESSES IN SIP PANEL AT TRANSFER

9.7.7.1 Allowable Stress Limits

[STD Art. 9.15]

Compression: $0.6f_{ci}' = 0.6(6{,}500)/1{,}000 = +3.900$ ksi

Tension: the maximum tensile stress should not exceed:

- $3\sqrt{f_{ci}'} \leq 200$ ksi (with no bonded reinforcement)

  $-3\sqrt{6{,}500}/1{,}000 = -0.242 \text{ ksi} > -0.200 \text{ ksi}$

  Therefore, -0.200 ksi (Controls)

- $-7.5\sqrt{f_{ci}'} = -7.5\sqrt{6{,}500}/1{,}000 = -0.605$ ksi (with bonded reinforcement)

Because the strand group is concentric with the precast concrete panel, the midspan section is the critical section that should be checked.

9.7.7.2 Stresses at Midspan

Effective pretensioning stress at the end of pretensioning process = 37.8 kips/ft

Bending moment due to self-weight of the panel, $M_{SIP} = 0.496$ ft-kips/ft

Top concrete stress of the SIP panel, $f_t = \dfrac{P_i}{A} + \dfrac{M_{SIP}}{S_t}$

$$f_t = \frac{37.8}{42} + \frac{0.496(12)}{24.5} = +0.900 + 0.243 = +1.143 \text{ ksi}$$

Allowable compressive concrete stress: +3.900 ksi OK

Bottom concrete stress in the SIP panel, $f_b = \dfrac{P_i}{A} - \dfrac{M_{SIP}}{S_b}$

$$f_b = \frac{37.9}{42} - \frac{0.496(12)}{24.5} = +0.900 - 0.243 = +0.657 \text{ ksi}$$

Allowable compressive concrete stress: +3.900ksi OK

9.7.8 STRESSES IN SIP PANEL AT TIME OF CASTING TOPPING SLAB

9.7.8.1 Allowable Stress Limits

[STD Art. 9.15.2.2]

Assume that at the time of casting the CIP concrete slab, the precast SIP concrete panel reaches its 28-day concrete strength, 8,000 ksi.

Compression:

Case I: due to all load combinations:

$0.6f_c' = 0.6(8{,}000)/1{,}000 = +4.800$ ksi

Case II: due to effective prestress and permanent (dead) loads:

$0.4f_c' = 0.4(8{,}000)/1{,}000 = +3.200$ ksi

Case III: due to live loads plus 1/2 the sum of the prestressing force and permanent (dead) loads:

$0.4f_c' = 0.4(8{,}000)/1{,}000 = +3.200$ ksi

Tension:

For members with bonded reinforcement:

$$6\sqrt{f_c'} = -6\sqrt{8,000}/1,000 = -0.537 \text{ ksi}$$

9.7.8.2
Stresses at Midspan after Non-Composite Loads

Effective prestress after all losses = 33.9 kips/ft

Bending moment due to the self-weight of the SIP panel, the CIP concrete and construction load are:

$$M_{SIP} = 0.496 \text{ ft-kips/ft}$$
$$M_{CIP} = 0.632 \text{ ft-kips/ft}$$
$$M_{const} = (0.050)(9.5)^2/8 = 0.564 \text{ ft-kips/ft}$$

Since the construction load is a live load, Case I is the critical case.

Top compressive concrete stress in the SIP panel:

$$f_t = \frac{P_{se}}{A} + \frac{M_{SIP} + M_{CIP} + M_{const}}{S_t} = \frac{33.9}{42} + \frac{(0.496 + 0.632 + 0.564)(12)}{24.5}$$
$$= +0.807 + 0.829 = +1.636 \text{ ksi}$$

Allowable compressive concrete stress: +4.800 ksi OK

Bottom concrete stress of the SIP panel:

$$f_b = \frac{P_{se}}{A} - \frac{M_{SIP} + M_{CIP} + M_{const}}{S_b} = \frac{33.9}{42} - \frac{(0.496 + 0.632 + 0.564)(12)}{24.5}$$
$$= +0.807 - 0.829 = -0.022 \text{ ksi}$$

Allowable tensile concrete stress: –0.537 ksi OK

9.7.9 STRESSES IN SIP PANEL AT SERVICE LOADS

9.7.9.1
Allowable Stress Limits

[STD Art. 9.15.2.2]

Compression:

Case I: due to all load combinations:

for the SIP panel: $0.6f_c' = 0.6(8,000)/1,000 = +4.800$ ksi

for the CIP topping: $0.4f_c' = 0.4(4,000)/1,000 = +1.600$ ksi

[STD Art. 8.15.2.1]

Case II: due to effective prestress and permanent (dead) loads:

for the SIP panel: $0.4f_c' = 0.4(8,000)/1,000 = +3.200$ ksi

Case III: due to live loads plus 1/2 of the prestressing force and permanent (dead) loads:

for the SIP panel: $0.4f_c' = 0.4(8,000)/1,000 = +3.200$ ksi

Tension:

for members with bonded reinforcement:

for the SIP panel: $6\sqrt{f_c'} = -6\sqrt{8,000}/1,000 = -0.537$ ksi

9.7.9.2
Service Load Stresses at Midspan

Effective prestress after all losses = 33.9 kips/ft

The weights of SIP panel and CIP slab act on the non-composite section, while the wearing surface, barrier load and live load act on the composite section.

M_{SIP} = 0.496 ft-kips/ft

M_{CIP} = 0.632 ft-kips/ft

M_{ws} = 0.135 ft-kips/ft

M_b = 0.255 ft-kips/ft

M_{LL+I} = 6.370 ft-kips/ft

Concrete stress at top fiber of the CIP slab:

(Since the prestress force is acting only on the SIP, Case I controls)

$$f_{tc} = \frac{M_{ws} + M_b + M_{LL+I}}{S_{tc}} = \frac{(0.135 + 0.255 + 6.370)12}{122.4} = +0.663 \text{ ksi}$$

Allowable compressive concrete stress: +1.600 ksi OK

Concrete stress at top fiber of the SIP panel:

Case I:

$$f_{tg} = \frac{P_{se}}{A} + \frac{M_{SIP} + M_{CIP}}{S_t} + \frac{M_{ws} + M_b}{S_{tg}} + \frac{M_{LL+I}}{S_{tg}}$$

$$= +\frac{(33.9)}{42} + \frac{(0.496 + 0.632)12}{24.5} + \frac{(0.135 + 0.255)12}{5{,}028} + \frac{(6.370)12}{5{,}028}$$

= +0.807 + 0.553 + 0.001 + 0.015 = +1.376 ksi

Allowable compressive concrete stress: +4.800 ksi OK

Case II:

f_{tg} = +0.807 + 0.553 + 0.001 = +1.361 ksi

Allowable compressive concrete stress: +3.200 ksi OK

Case III:

f_{tg} = 0.5(+0.807 + 0.553 + 0.001) + 0.015 = +0.696 ksi

Allowable compressive concrete stress: +3.200 ksi OK

Concrete stress at bottom fiber of the SIP panel:

$$f_b = \frac{P_{se}}{A} - \frac{M_{SIP} + M_{CIP}}{S_b} - \frac{M_{ws} + M_b + M_{LL+I}}{S_{bc}}$$

$$= +\frac{(33.9)}{42} - \frac{(0.496 + 0.632)12}{24.5} - \frac{(0.135 + 0.255 + 6.370)12}{102.6}$$

= +0.807 − 0.553 − 0.791 = −0.537 ksi

Allowable tensile concrete stress: −0.537ksi OK

9.7.10 FLEXURAL STRENGTH OF POSITIVE MOMENT SECTION

[STD Art. 9.17]

M_{SIP} = 0.496 ft-kips/ft
M_{CIP} = 0.632 ft-kips/ft
M_{ws} = 0.135 ft-kips/ft
M_b = 0.255 ft-kips/ft
M_{LL+I} = 6.370 ft-kips/ft
M_u = $1.3[M_D + 1.67M_{LL+I}]$
= 1.3[(0.496 + 0.632 + 0.135 + 0.255) + 1.67(6.370)] = 15.803 ft-kips/ft

The design flexural strength, ϕM_n, for rectangular sections, is given by:

$$\phi M_n = \phi\left[A_s^* f_{su}^* d\left(1 - 0.6\frac{p^* f_{su}^*}{f_c'}\right)\right] \quad \text{[STD Eq. 9-13]}$$

where

ϕ = strength reduction factor = 1.0
A_s^* = area of pretensioning steel = (10)(0.153)/(8.0) = 0.191 in.²/ft
d = distance from extreme compressive fiber to the centroid of the prestressing reinforcement = 0.5(3.5) + 4 = 5.75 in.

p^* = ratio of pretensioning steel = $\dfrac{A_s^*}{bd} = \dfrac{0.191}{12(5.75)} = 0.00277$

f_c' = specified compressive strength at 28 days = 4.0 ksi
f_{su}^* = average stress in pretensioning steel at ultimate load
= $f_s'[1 - (\gamma^*/\beta_1)(p^* f_s'/f_c')]$ [STD Eq. 9.17]

where

γ^* = factor for type of pretensioning reinforcement = 0.28 (for low-relaxation strand)
β_1 = stress factor of compression block:
0.85 for $f_c' \le 4.0$ ksi
$0.85 - 0.05(f_c' - 4.0) \ge 0.65$ for $f_c' > 4.0$ ksi
= 0.85
f_s' = ultimate strength of pretensioning reinforcement = 270 ksi

$$f_{su}^* = 270\left[1 - \left(\frac{0.28}{0.85}\right)\left(\frac{0.00277(270)}{4.0}\right)\right] = 253.4 \text{ ksi}$$

Check limit on f_{su}^*: [STD 9.17.4.2]

$$f_{su}^* \le \frac{\ell_x}{D} + \frac{2}{3}f_{se} \quad \text{[STD Eq. 9-19]}$$

where

ℓ_x = distance from end of the prestressing strand to center of panel = 9.5/2 = 4.75 ft
D = nominal diameter of strand = 0.5 in.
f_{se} = effective stress in prestressing strand after losses = 176.975 ksi

Thus, $\frac{\ell_x}{D} + \frac{2}{3}f_{se} = \frac{4.75(12)}{0.5} + \frac{2}{3}(176.975) = 231.983$ ksi

Therefore, $f^*_{su} = 231.983$ ksi (Controls)

Depth of the rectangular stress block:

$$a = \frac{A^*_s f^*_{su}}{0.85 f'_c b} = \frac{(0.191)(231.983)}{(0.85)(4.0)(12)}$$

$= 1.09$ in. < thickness of the CIP topping = 4.00 in. OK

$$\phi M_n = 1.0\left[(0.191)(231.983)(5.75)\left(1 - 0.6\frac{(0.00277)(231.983)}{4.0}\right)\right]/12$$

$= 19.185$ ft-kips/ft > $M_u = 15.803$ ft-kips/ft OK

Notes:

1. The effect of the compression reinforcement in the CIP topping is neglected.
2. The calculations here are conducted for flexure at the midspan section. It is possible that intermediate sections between midspan and the supports will exhibit critical stresses due to partial development of the pretensioning strands. Because the *Standard Specifications* does not provide guidance relative to live load bending moment at intermediate sections, it is difficult to perform this check without making certain assumptions. One way to estimate the live load moment is to assume that it varies parabolically from $0.8M_{LL+I}$ at midspan to $0.2M_{LL+I}$ over the beam centerline.

9.7.11 DUCTILITY LIMITS OF THE SIP PANEL

[STD Art. 9.18]

9.7.11.1 Maximum Reinforcement

[STD Art. 9.18.1]

Pretensioned concrete members are designed so that the steel yields as ultimate capacity is approached.

The reinforcement index for rectangular sections = $\rho^* \frac{f^*_{su}}{f'_c} < 0.36\beta_1$ [STD Eq. 9.20]

$$p^* \frac{f^*_{su}}{f'_c} = 0.00277\left(\frac{231.983}{4.0}\right) = 0.16 < 0.36\beta_1 = 0.36(0.85) = 0.306 \quad \text{OK}$$

Note that the compression strength of the topping should be used to determine β_1.

9.7.11.2 Minimum Reinforcement

[STD Art. 9.18.2]

STD Article 9.18.2.1 states that the total amount of pretensioned reinforcement should be adequate to develop an ultimate moment at the critical section of at least 1.2 times the cracking moment, M^*_{cr}; i.e.:

$$\phi M_n \geq 1.2M^*_{cr}$$

Compute cracking moment:

$M^*_{cr} = (f_r + f_{pe})S_{bc} - (M_{d/nc})(S_{bc}/S_b - 1)$ [STD Art. 9.18.2.1]

where

f_r = modulus of rupture [STD Art. 9.15.2.3]

$= 7.5\sqrt{f'_c} = 7.5\sqrt{8{,}000}/1{,}000 = 0.671$ ksi

f_{pe} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$= \dfrac{P_{se}}{A} = \dfrac{33.9}{42} = 0.807$ ksi

$M_{d/nc}$ = non-composite dead load moment at the section

$= M_{SIP} + M_{CIP} = 0.496 + 0.632 = 1.128$ ft-kips/ft

$M^*_{cr} = (0.671 + 0.807)(102.6/12) - (1.128)[(102.6/24.5) - 1] = 9.041$ ft-kips/ft

$1.2M^*_{cr} = 10.849$ ft-kips/ft $< \phi M_n = 19.185$ ft-kips/ft OK

9.7.12 LONGITUDINAL REINFORCEMENT IN THE SIP PANEL

[STD Art. 9.24.2]

Minimum amount of distribution reinforcement = 0.11 in.^{2}/ft.

Use #3 reinforcing bars @ 12 in. centers. Area of steel provided = 0.11 in.^{2}/ft

or use 4x4 W4xW4. Reinforcement area provided = 3(0.04) = 0.12 in.^{2}/ft.

9.7.13 NEGATIVE MOMENT SECTION OVER INTERIOR BEAMS

9.7.13.1 Critical Section

STD Article 3.24.1.2 specifies the span length that can be used for calculating bending moments for slabs continuous over more than two supports. However, the *Standard Specifications* does not specify the location of the critical section. Therefore, as a conservative approach, the critical section is considered to be at the center of the beam.

9.7.13.2 Bending Moment

Negative moment occurs only from wearing surface and live load while barrier load results in positive moment.

Due to wearing surface, $M_{ws} = -0.315$ ft-kips/ft

Due to barrier load, $M_b = +0.255$ ft-kips/ft

Due to live load, $M_{LL+I} = -6.370$ ft-kips/ft

Total service bending moment = $-0.315 + 0.255 - 6.370 = -6.430$ ft-kips/ft

Total factored bending moment, $M_u = 1.3[-0.315 + 0.255 - 1.67(6.370)]$
$= -13.907$ ft-kips/ft

9.7.13.3 Design of Section

Assume welded wire reinforcement will be used with D26 wire. With 2.5-in. clear cover, the effective depth is:

d $= 3.5 + 4.0 - 0.5(0.575) - 2.5 = 4.712$ in.

b $= 12.0$ in.

$R_n = (M_u/\phi bd^2) = \dfrac{13.907(12)(1{,}000)}{0.9(12)(4.712)^2} = 696.0$ psi/ft, $(\phi = 0.9)$

$$m = (f_y/0.85f'_c) = \frac{60,000}{0.85(4,000)} = 17.65$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2(17.65)(696.0)}{60,000}}\right) = 0.0131$$

$A_s = \rho(bd) = 0.0131(12)(4.712) = 0.74$ in.²/ft

Use D26 @ 4-in. spacing, $A_s = 0.78$ in.²/ft

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.78(60.0)}{(0.85)(4.0)(12)} = 1.15 \text{ in.}$$

$$\phi M_n = 0.9(A_s f_y)(d - 0.5a)/12 = 0.9(0.78)(60.0)[4.712 - 0.5(1.15)]/12$$
$$= 14.521 \text{ ft-kips/ft} > M_u = 13.907 \text{ ft-kips/ft} \quad \text{OK}$$

9.7.13.4 Maximum Reinforcement

[STD Art. 8.16.3.1]

$$\rho_{actual} = \frac{0.78}{12(4.712)} = 0.0138$$

$$\rho_b = \frac{0.85\,\beta_1\,f'_c}{f_y}\left(\frac{87,000}{87,000 + f_y}\right) \quad \text{[STD Eq. 8.18]}$$

$$= \frac{0.85\,(0.85)(4.0)}{60}\left(\frac{87,000}{87,000 + 60,000}\right) = 0.0285$$

$$\rho_{max} = 0.75\rho_b = 0.0214 > \rho_{actual} \quad \text{OK}$$

9.7.13.5 Minimum Reinforcement

[STD Art. 8.17.1]

The total amount of non-pretensioned reinforcement should be adequate to develop ultimate moment at the critical section of at least 1.2 times the cracking moment, i.e.:

$\phi M_n \geq 1.2M_{cr}$

Compute cracking moment: $M_{cr} = Sf_r$

where

f_r = modulus of rupture [STD Art. 8.15.2.1.1]

$$= 7.5\sqrt{f'_c} = 7.5\sqrt{4,000}/1,000 = 0.474 \text{ ksi}$$

S = section modulus = $12(7.5)^2/6 = 112.5$ in.³

$M_{cr} = 112.5(0.474)/12 = 4.444$ ft-kips/ft

$1.2M_{cr} = 1.2(4.444) = 5.333$ ft-kips/ft $< \phi M_n = 19.185$ ft-kips/ft OK

9.7.13.6 Crack Control

[STD Art. 8.16.8.4]

The calculated stress in the reinforcement at service loads, f_s, should not exceed the value computed by:

$$f_s = \frac{z}{(d_c A)^{1/3}} \le 0.6f_y \qquad \text{[STD Eq. 8-61]}$$

where

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar, in. For calculation purposes, the thickness of the concrete clear cover used to compute d_c should be taken not greater than 2.0 in.

= 2.0 + 0.5(0.575) = 2.288 in.

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement divided by the number of bars, in.^{2} = 2(2.288)(4) = 18.30 in.^{2}

z = 130 kip/in. (severe exposure conditions)

$$f_s = \frac{130}{[2.288(18.30)]^{1/3}} = 37.4 \text{ ksi} > 0.6f_y = 36 \text{ ksi}$$

Therefore, f_s = 36.0 ksi.

Actual stress in reinforcement:

E_c = 3,834 ksi

n = E_s/E_c = 29,000/3,834 = 7.564

$$\rho = \frac{0.78}{12(4.712)} = 0.01379$$

ρn = (0.01379)(7.564) = 0.1043

$$k = \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.36$$

$$j = (1 - k/3) = 0.88$$

$$f_s = \frac{M_{service}}{(jdA_s)} = \frac{(6.430)(12)}{(0.88)(4.712)(0.78)} = 23.9 \text{ ksi} < 36.0 \text{ ksi} \quad \text{OK}$$

9.7.14 NEGATIVE MOMENT SECTION OVER EXTERIOR BEAMS

9.7.14.1 Critical Section

As noted before, STD Article 3.24.1.2 specifies the span length that can be used in calculating bending moments for slabs continuous over more than two supports. However, the *Standard Specifications* does not specify the position for the critical section.

In this example, the critical section is considered to be at a distance of one-fourth of the top flange width measured from the centerline of the beam. This assumption is made because the supporting beam has a top flange width-to-minimum thickness ratio greater than 4.0. Critical section = 42/4 = 10.5 in.

9.7.14.2 Bending Moment

Two loading cases should be considered for the overhang design:

Case 1: under live load, HS20, combined with dead loads. STD Article 3.24.2.1 states that the wheel load is applied at 1.0 ft from face of barrier, as shown in Case 1, **Figure 9.7.14.2-1.**

$$M_{CIP} = \left(\frac{8}{12}\right)(0.150)\left(\frac{40.5}{12}\right)^2/2 = 0.570 \text{ ft-kips/ft}$$

$$M_{ws} = (0.025)\left(\frac{12+13.5}{12}\right)^2/2 = 0.056 \text{ ft-kips/ft}$$

$$M_b = (0.300)\left(\frac{35.5}{12}\right) = 0.888 \text{ ft-kips/ft}$$

Live load moment: [STD Art. 3.24.5.1.1]

Live load moment with impact, $M_{LL+I} = (P/E)X(1 + I)$ (Eq. 9.7.14.2-1)

where

$P = P_{20}$ = load of the design truck wheel of the rear axle = 16 kips

X = the distance from load to point of support = 13.5 in.

E = width of slab over which a wheel load is distributed [STD Eq. 3-17]

$$= 0.8X + 3.75 = 0.8\left(\frac{13.5}{12}\right) + 3.75 = 4.65 \text{ ft}$$

I = impact fraction = 0.3

$M_{LL+I} = (16/4.65)(13.5/12)(1+0.3) = 5.032$ ft-kips/ft

Total service bending moment:

$M_{service} = (0.570 + 0.056 + 0.888 + 5.032) = 6.546$ ft-kips/ft

Total factored bending moment, $M_u = 1.3[(0.570 + 0.056 + 0.888) + 1.67(5.032)]$
$= 12.893$ ft-kips/ft

Case 2: under collision force with the barrier combined with dead loads. STD Article 2.7.1.3 states that a 10-kip horizontal force should be applied at the top of the barrier, as shown in Case 2, **Figure 9.7.14.2-1.**

$M_{CIP} = 0.570$ ft-kips/ft

$M_{ws} = 0.056$ ft-kips/ft

$M_b = 0.888$ ft-kips/ft

Live load moment: [STD Art. 3.24.5.1.1]

Bending moment due to collision force, M_{col}, is distributed over a distance E,

where

$E = 0.8X + 5.0$ [STD Art. 3.24.5.2]

where

X = distance from center of the post to the point under investigation
= 15/2 + 25.5 = 33.0 in.

Therefore, $E = 0.8\left(\frac{33.0}{12}\right) + 5 = 7.20$ ft

$M_{col} = (10/7.20)(2.67) = 3.708$ ft-kips/ft

Total service bending moment, $M_{service} = (0.570 + 0.056 + 0.888 + 3.708) = 5.222$ ft-kips/ft

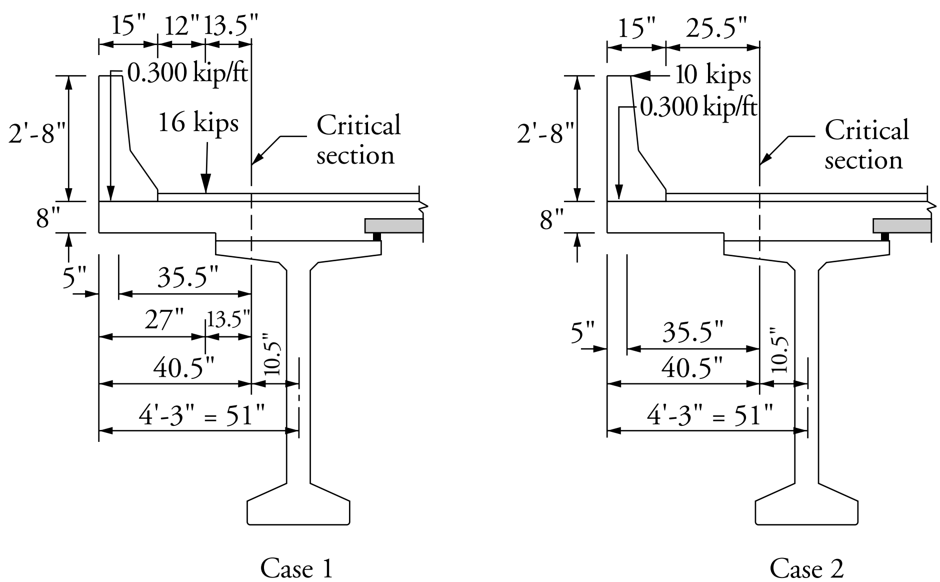
Since the *Standard Specifications* does not provide guidelines for the load factor that should be used with the collision force, a conservative approach is considered by using the load factor used with live loads; i.e., $\gamma = 1.3$ and $\beta = 5/3 = 1.67$.

Total factored bending moment, $M_u = 1.3[(0.570 + 0.056 + 0.888) + (1.67)(3.708)]$
$= 10.018$ ft-kips/ft

Service axial tension force at critical section = 10/7.2 = 1.389 kips/ft

Factored axial tension force = 1.3[1.67(1.389)] = 3.015 kips/ft

Figure 9.7.14.2-1 Loading Cases for the Overhang



9.7.14.3 Design of Section

Case 1: $M_u = 12.893$ ft-kips/ft

Assume welded wire reinforcement will be used with D26 wire. With 2.5-in.-clear cover, the effective depth is:

$d = 7.5 - 0.5(0.575) - 2.5 = 4.712$ in.

$b = 12.0$ in.

$$R_n = (M_u/\phi bd^2) = \frac{12.893(12 \times 1{,}000)}{0.9(12)(4.712)^2} = 645.2 \text{ lb/in.}^2, \ (\phi = 0.9)$$

$$m = (f_y/0.85 f'_c) = \frac{60{,}000}{0.85(4{,}000)} = 17.65$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{17.65}\left(1 - \sqrt{1 - \frac{2(17.65)(645.2)}{60{,}000}}\right) = 0.0120$$

$A_s = \rho(bd) = 0.0120(12)(4.712) = 0.68$ in.²/ft

Use D26 @ 4 in. spacing, $A_s = 0.78$ in.²

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.78)(60.0)}{(0.85)(4.0)(12)} = 1.15 \text{ in.}$$

$\phi M_n = 0.9(A_s f_y)(d - 0.5a)/12 = 0.9(0.78)(60.0)[4.712 - 0.5(1.15)]/12$
$= 14.521$ ft-kips/ft $> M_u = 12.893$ ft-kips/ft OK

Case 2: $M_u = 10.018$ ft-kips/ft, $T_u = 3.015$ kips/ft
Check D26 @ 4 in. spacing, $A_s = 0.78$ in.²
$d = 4.712$ in.; $b = 12$ in.
$A_s f_y = 0.78(60) = 46.800$ kips
$C = A_s f_y - T_u = 46.800 - 3.015 = 43.785$ kips
$a = C/(0.85bf'_c) = (43.785)/(0.85)(12)(4.0) = 1.07$ in.
$\phi M_n = \phi[A_s f_y(d - a/2) - T_u(0.5d - 0.5a)]$
$= 0.9[(0.78)(60)(4.712 - 1.07/2) - (3.015)(4.712/2 - 1.07/2)]/12$
$= 14.250$ ft-kips/ft $> M_u = 10.018$ ft-kips/ft OK

9.7.14.4 Maximum Reinforcement

[STD Art. 8.16.3.1]

$$\rho_{actual} = \frac{0.78}{12(4.712)} = 0.0138$$

$$\rho_b = \frac{0.85\,\beta_1\, f'_c}{f_y}\left(\frac{87{,}000}{87{,}000 + f_y}\right) \quad \text{[STD Eq. 8.18]}$$

$$= \frac{0.85\,(0.85)(4.0)}{60}\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right) = 0.0285$$

$\rho_{max} = 0.75\rho_b = 0.0214 > \rho_{actual}$ OK

9.7.14.5 Minimum Reinforcement

[STD Art. 8.17.1]

The total amount of non-pretensioned reinforcement should be adequate to develop ultimate moment at the critical section of at least 1.2 times the cracking moment; i.e.:

$\phi M_n \geq 1.2 M_{cr}$

Compute cracking moment: $M_{cr} = Sf_r$

where

f_r = modulus of rupture [STD Art. 8.15.2.1.1]

$= 7.5\sqrt{f'_c} = 7.5\sqrt{4{,}000}/1{,}000 = 0.474$ ksi

S = section modulus = $12(7.5)^2/6 = 112.5$ in.^{3}

$M_{cr} = 112.5(0.474)/12 = 4.444$ ft-kips

$1.2M_{cr} = 1.2(4.444) = 5.333$ ft-kips/ft $< \phi M_n = 14.250$ ft-kips/ft OK

9.7.14.6 Crack Control

[STD Art. 8.16.8.4]

The calculated stress in the reinforcement at service loads, f_s, should not exceed the value computed by:

$$f_s = \frac{z}{(d_cA)^{1/3}} \le 0.6f_y$$ [STD Eq. 8-61]

where

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar, in. For calculation purpose, the thickness of the concrete clear cover used to compute d_c should be taken not greater than 2.0 in.

= 2.0 + 0.5(0.575) = 2.288 in.

A = effective tension area of concrete surrounding the flexure tension reinforcement and having the same centroid as that reinforcement divided by the number of bars, in.^{2} = 2(2.288)(4) = 18.30 in.^{2}

z = 130 kip/in. (severe exposure conditions)

$$f_s = \frac{130}{\left[(2.288)(18.30)\right]^{1/3}} = 37.4 \text{ ksi} > 0.6f_y = 36 \text{ ksi}$$

Therefore, $f_s = 36.0$ ksi

Actual stress in reinforcement:

E_c = 3,834 ksi

n = E_s/E_c = 29,000/3,834 = 7.564

$$\rho = \frac{0.78}{12(4.712)} = 0.01379$$

$\rho n = 0.01379(7.564) = 0.1043$

$$k = \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.36$$

j = (1 – k/3) = 0.88

$$f_s = \frac{M_{service}}{(jdA_s)} = \frac{(6.546)(12)}{(0.88)(4.712)(0.78)} = 24.3 \text{ ksi} < 36.0 \text{ ksi} \quad \text{OK}$$

9.7.14.7 Development Length of Steel Reinforcement

[STD Art. 8.30.1.2]

$$\ell_d = 0.03d_b(f_y - 20{,}000)/\sqrt{f_c'} \geq 0.20\left[\frac{A_w}{s_w}\left(\frac{f_y}{\sqrt{f_c'}}\right)\right]$$

where

A_w = area of individual wire to be developed, in.^{2}

s_w = spacing of wire to be developed

$\ell_d = 0.03(0.575)(60{,}000 - 20{,}000)/\sqrt{4{,}000} = 10.9 \geq 0.20\left[\frac{0.26}{4}\left(\frac{60{,}000}{\sqrt{4{,}000}}\right)\right]$

= 12.3 in.

Thus, ℓ_d = 12.3 in. (Controls)

Since this length is less than the width of the barrier, 15 in., development of the reinforcement at the face of the barrier should be satisfactory.

9.7.15 LONGITUDINAL REINFORCEMENT IN THE CIP SLAB

[STD Art. 9.18.2.2]

The minimum amount of non-prestressed longitudinal reinforcement provided in the CIP portion of the slab, should be 0.25 in.^{2}/ft

Use D14@6 in. spacing; A_s = 2(0.14) = 0.28 in.^{2}/ft OK

TABLE OF CONTENTS
DESIGN EXAMPLES

Note: Each design example contains a thorough table of contents.

A	= cross-sectional area of the precast beam or section	[STD], [LRFD]
A	= effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement divided by the number of bars	[STD], [LRFD]
A_b	= area of an individual bar	[LRFD]
A_c	= total area of the composite section	
A_c	= area of concrete on the flexural tension side of the member	[LRFD]
A_{cv}	= area of concrete section resisting shear transfer	[LRFD]
A_o	= area enclosed by centerlines of the elements of the beam	[LRFD]
A_{ps}	= area of pretensioning steel	[LRFD]
A_{PT}	= transverse post-tensioning reinforcement	
A_s	= area of non-pretensioning tension reinforcement	[STD]
A_s	= area of non-pretensioning tension reinforcement	[LRFD]
A_s	= total area of vertical reinforcement located within the distance (h/5) from the end of the beam	[LRFD]
A_{sf}	= steel area required to develop the ultimate compressive strength of the overhanging portions of the flange	[STD]
A_{sr}	= steel area required to develop the compressive strength of the web of a flanged section	[STD]
A^*_s	= area of pretensioning steel	[STD]
A'_s	= area of compression reinforcement	[LRFD]
A_v	= area of web reinforcement	[STD]
A_v	= area of transverse reinforcement within a distance 's'	[LRFD]
A_{vf}	= area of shear-friction reinforcement	[LRFD]
A_{vh}	= area of web reinforcement required for horizontal shear	
$A_{v\text{-}min}$	= minimum area of web reinforcement	
a	= depth of the compression block	[STD]
a	= distance from the end of beam to drape point	
a	= depth of the equivalent rectangular stress block	[LRFD]
b	= effective flange width	
b	= width of beam	[STD]
b	= width of bottom flange of the beam	
b	= width of the compression face of a member	[LRFD]
b'	= width of web of a flanged member	[STD]
b_e	= effective web width of the precast beam	
b_v	= width of cross section at the contact surface being investigated for horizontal shear	[STD]
b_v	= effective web width	[LRFD]
b_v	= width of interface	[LRFD]
b_w	= web width	[LRFD]
CR_c	= loss of pretension due to creep of concrete	[STD]
CR_s	= loss of pretension due to relaxation of pretensioning steel	[STD]
c	= distance from the extreme compression fiber to the neutral axis	[LRFD]
c	= cohesion factor	[LRFD]
D	= dead load	[STD]
D	= strand diameter	[STD]

DC	= dead load of structural components and non structural attachments	[LRFD]
DFD	= distribution factor for deflection	
DFM	= distribution factor for bending moment	
DF_m	= live load distribution factor for moment	
DFV	= distribution factor for shear force	
DW	= load of wearing surfaces and utilities	[LRFD]
d	= distance from extreme compressive fiber to centroid of the pretensioning force	[STD]
d_b	= nominal strand diameter	[LRFD]
d_c	= thickness of concrete cover measured from extreme tension fiber to center of the closest bar	[STD], [LRFD]
d_e	= distance from exterior web of exterior beam and the interior side of curb or traffic barrier	[LRFD]
d_e	= effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	[LRFD]
d_p	= distance from extreme compression fiber to the centroid of the pretensioning tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
E	= width of slab over which a wheel load is distributed	[STD]
E_c	= modulus of elasticity of concrete	[STD]
E_c	= modulus of elasticity of concrete	[LRFD]
E_{ci}	= modulus of elasticity of the beam concrete at transfer	
E_p	= modulus of elasticity of pretensioning tendons	[LRFD]
ES	= loss of pretension due to elastic shortening	[STD]
E_s	= modulus of elasticity of pretensioning reinforcement	[STD]
E_s	= modulus of elasticity of reinforcing bars	[LRFD]
e	= eccentricity of the strands at h/2	
e	= eccentricity of strands at transfer length	
e'	= difference between eccentricity of pretensioning steel at midspan and end span	
e_c	= eccentricity of the strand at the midspan	
e_e	= eccentricity of pretensioning force at end of beam	
e_g	= distance between the centers of gravity of the beam and the slab	[LRFD]
F_b	= allowable tensile stress in the precompressed tensile zone at service loads	
F_{pi}	= total force in strands before release	
F_ε	= reduction factor	[LRFD]
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified concrete strength at 28 days	[STD]
f'_c	= specified compressive strength at 28 days	[LRFD]
f_{cdp}	= change of stresses at center of gravity of prestress due to permanent loads, except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp}	[LRFD]
f_{cds}	= concrete stress at the center of gravity of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied	[STD]
f_{cir}	= average concrete stress at the center of gravity of the pretensioning steel due to pretensioning force and dead load of beam immediately after transfer	[STD]
f'_{ci}	= concrete strength at release	[STD]

f'_{ci}	= specified compressive strength of concrete at time of initial loading or pretensioning	[LRFD]
f_{cgp}	= concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	[LRFD]
f_d	= stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pb}	= compressive stress at bottom fiber of the beam due to prestress force	
f_{pc}	= compressive stress in concrete (after allowance for all pretension losses) at centroid of cross section resisting externally applied loads	[STD]
f_{pc}	= compressive stress in concrete after all prestress losses have occurred either at the centroid of the cross section resisting live load or at the junction of the web and flange when the centroid lies in the flange. In a composite section, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies with in the flange, due to both prestress and to the bending moments resisted by the precast member acting alone	[LRFD]
f_{pe}	= compressive stress in concrete due to effective pretension forces only (after allowance for all pretension losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pe}	= effective stress in the pretensioning steel after losses	[LRFD]
f_{pi}	= initial stress immediately before transfer	
f_{po}	= stress in the pretensioning steel when the stress in the surrounding concrete is zero	[LRFD]
f_{ps}	= average stress in pretensioning steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pt}	= stress in pretensioning steel immediately after transfer	[LRFD]
f_{pu}	= specified tensile strength of pretensioning steel	[LRFD]
f_{py}	= yield strength of pretensioning steel	[LRFD]
f_r	= the modulus of rupture of concrete	[STD]
f_r	= modulus of rupture of concrete	[LRFD]
f_s	= allowable stress in steel	
f'_s	= ultimate stress of pretensioning reinforcement	[STD]
f_{se}	= effective final pretension stress	
f_{si}	= effective initial pretension stress	
f^*_{su}	= average stress in pretensioning steel at ultimate load	[STD]
f_t	= concrete stress at top fiber of the beam for the non-composite section	
f_{tc}	= concrete stress at top fiber of the slab for the composite section	
f_{tg}	= concrete stress at top fiber of the beam for the composite section	
f_y	= yield strength of reinforcing bars	[STD]
f_y	= specified minimum yield strength of reinforcing bars	[LRFD]
f_y	= yield stress of pretensioning reinforcement	[STD]
f'_y	= specified minimum yield strength of compression reinforcement	[LRFD]
f_{yh}	= specified yield strength of transverse reinforcement	[LRFD]
H	= average annual ambient mean relative humidity, percent	[LRFD]
H	= height of wall	[LRFD]
h	= overall depth of precast beam	[STD]
h	= overall depth of a member	[LRFD]

h_c	= total height of composite section	
h_f	= compression flange depth	[LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD]
I	= moment of inertia about the centroid of the non-composite precast beam	[LRFD]
I	= impact fraction (maximum 30%)	[STD]
I_c	= moment of inertia of composite section	
IM	= dynamic load allowance	[LRFD]
J	= St. Venant torsional constant	
K	= longitudinal stiffness parameter	[STD]
K_g	= longitudinal stiffness parameter	[LRFD]
k	= factor used in calculation of distribution factor for multi-beam bridges	[LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
L	= live load	[STD]
L	= length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	[STD]
L	= overall beam length or design span	
L	= span length measured parallel to longitudinal beams	[STD]
L	= span length	[LRFD]
L_c	= critical length of yield line failure pattern	[LRFD]
LL	= vehicular live load	[LRFD]
ℓ_d	= development length	[LRFD]
ℓ_x	= length required to fully develop the strand measured from the end of the strand	
M_a	= negative moment at the end of the span being considered	
M_b	= negative moment at the end of the span being considered	
M_b	= unfactored bending moment due to barrier weight	
M_c	= flexural resistance of cantilevered wall	[LRFD]
M_{CIP}	= unfactored bending moment due to cast-in-place topping slab	
M_{const}	= unfactored bending moment due to construction load	
M_{col}	= bending moment due to horizontal collision force	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads (after dead load)	[STD]
M_{cr}	= cracking moment	[LRFD]
M^*_{cr}	= cracking moment	[STD]
M_D	= unfactored bending moment due to diaphragm weight	
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= moment due to non-composite dead loads	[STD]
M_f	= unfactored bending moment due to fatigue truck per beam	
M_g	= unfactored bending moment due to beam self-weight	
M_{LL}	= unfactored bending moment due to lane load per beam	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_{LL+I}	= unfactored bending moment due to design vehicular load	
M_{LT}	= unfactored bending moment due to truck load with dynamic allowance per beam	

M_{max} = maximum factored moment at section due to externally applied loads [STD]
M_n = nominal moment strength of a section [STD]
M_n = nominal flexural resistance [LRFD]
$M_{n/dc}$ = non-composite dead load moment at the section
M_r = factored flexural resistance of a section in bending [LRFD]
M_s = maximum positive moment
M_s = unfactored bending moment due to slab and haunch weights
M_{SDL} = unfactored bending moment due to super-imposed dead loads
$M_{service}$ = total bending moment for service load combination
M_{SIP} = unfactored bending moment due to stay-in-place panel
M_u = factored bending moment at section [STD]
M_u = factored moment at a section [LRFD]
M_{ws} = unfactored bending moment due to wearing surface
M_x = bending moment at a distance (x) from the support
m = material parameter
m = stress ratio = $(f_y/0.85f'_c)$
N_b = number of beams [LRFD]
N_L = number of traffic lanes [STD]
N_u = applied factored axial force taken as positive if tensile [LRFD]
n = modular ratio between deck slab and beam materials
P = diaphragm weight concentrated at quarter points
P = load on one rear wheel of design truck (P_{15} or P_{20}) [STD]
P_c = permanent net compression force [LRFD]
P_{eff} = effective post-tensioning force
P_i = total pretensioning force immediately after transfer
P_{pe} = total pretensioning force after all losses
P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement
P_s = prestress force before initial losses
P_{se} = effective pretension force after allowing for all losses
P_{si} = effective pretension force after allowing for the initial losses
P_{20} = load on one rear wheel of the H20 truck [STD]
Q = total factored load [LRFD]
Q_i = specified loads [LRFD]
q = generalized load [LRFD]
RH = relative humidity [STD]
R_n = coefficient of resistance
R_u = flexural resistance factor
R_w = total transverse resistance of the railing or barrier [LRFD]
S = width of precast beam [STD]
S = average spacing between beams in feet [STD]
S = spacing of beams [LRFD]

S	= span length of deck slab	[STD]
S	= effective span length of the deck slab; clear span plus distance from extreme flange tip to face of web	LRFD]
S_b	= section modulus for the extreme bottom fiber of the non-composite precast beam	[STD]
S_{bc}	= composite section modulus for extreme bottom fiber of the precast beam (equivalent to S_c in the *Standard Specifications*)	
SH	= loss of pretension due to concrete shrinkage	[STD]
SR	= fatigue stress range	
S_t	= section modulus for the extreme top fiber of the non-composite precast beam	
S_{tc}	= composite section modulus for top fiber of the deck slab	
S_{tg}	= composite section modulus for top fiber of the precast beam	
s	= longitudinal spacing of the web reinforcement	[STD]
s	= length of a side element	[LRFD]
s	= spacing of rows of ties	[LRFD]
T	= collision force at deck slab level	
t	= thickness of web	
t	= thickness of an element of the beam	
t_f	= thickness of flange	
t_s	= cast-in-place deck thickness	
t_s	= depth of concrete deck	[LRFD]
V_c	= nominal shear strength provided by concrete	[STD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[LRFD]
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	[STD]
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	[STD]
V_d	= shear force at section due to unfactored dead load	[STD]
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	[STD]
V_{LL}	= unfactored shear force due to lane load per beam	
V_{LL+I}	= unfactored shear force due to live load plus impact	
V_{LL+I}	= unfactored shear force due design vehicular live load	
V_{LT}	= unfactored shear force due to truck load with dynamic allowance per beam	
V_{mu}	= ultimate shear force occurring simultaneously with M_u	
V_n	= nominal shear resistance of the section considered	[LRFD]
V_{nh}	= nominal horizontal shear strength	[STD]
V_p	= vertical component of effective pretension force at section	[STD]
V_p	= component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear	[LRFD]
V_s	= nominal shear strength provided by web reinforcement	[STD]
V_s	= shear resistance provided by shear reinforcement	[LRFD]
V_u	= factored shear force at the section	[STD]

V_u = factored shear force at section [LRFD]
V_{uh} = factored horizontal shear force per unit length of the beam [LRFD]
V_x = shear force at a distance (x) from the support
v = factored shear stress [LRFD]
W = overall width of bridge measured perpendicular to the longitudinal beams [STD]
w = a uniformly distributed load [LRFD]
w = width of clear roadway [LRFD]
w_b = weight of barriers
w_c = unit weight of concrete [STD]
w_c = unit weight of concrete [LRFD]
w_g = beam self-weight
w_s = slab and haunch weights
w_{ws} = weight of future wearing surface
X = distance from load to point of support [STD]
x = the distance from the support to the section under question
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam
y_t = distance from centroid to the extreme top fiber of the non-composite precast beam
y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab
y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam
Z (or z)= factor reflecting exposure conditions [LRFD], [STD]
α = angle of inclination of transverse reinforcement to longitudinal axis
β = factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution) [LRFD]
β_D = load combination coefficient for dead loads [STD]
β_L = load combination coefficient for live loads [STD]
β_1 = factor for concrete strength [STD]
β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone [LRFD]
Δ_{beam} = deflection due to beam self-weight
Δ_{b+ws} = deflection due to barrier and wearing surface weights
Δf_{cdp} = change in concrete stress at center of gravity of pretensioning steel due to dead loads except the dead load acting at the time of the pretensioning force is applied [LRFD]
Δf_{pCR} = loss in pretensioning steel stress due to creep [LRFD]
Δf_{pES} = loss in pretensioning steel stress due to elastic shortening [LRFD]
Δf_{pi} = total loss in pretensioning steel stress immediately after transfer
Δf_{pR} = loss in pretensioning steel stress due to relaxation of steel [LRFD]
Δf_{pR1} = loss in pretensioning steel stress due to relaxation of steel at transfer [LRFD]
Δf_{pR2} = loss in pretensioning steel stress due to relaxation of steel after transfer [LRFD]
Δf_{pSR} = loss in pretensioning steel stress due to shrinkage [LRFD]

Δf_{pT}	= total loss in pretensioning steel stress	[LRFD]
Δ_D	= deflection due to diaphragm weight	
Δ_L	= deflection due to specified live load	
Δ_{LL+I}	= deflection due to live load and impact	
Δ_{LL}	= deflection due to lane load	
Δ_{LT}	= deflection due to design truck load and impact	
Δ_{max}	= maximum allowable live load deflection	
Δ_p	= camber due pretension force at transfer	
Δ_{SDL}	= deflection due to barrier and wearing surface weights	
Δ_{slab}	= deflection due to the weights of slab and haunch	
ε_x	= longitudinal strain in the web reinforcement on the flexural tension side of the member	[LRFD]
γ	= load factor	[STD]
γ^*	= factor for type of pretensioning reinforcement, 0.28 for low relaxation strand	[STD]
γ_i	= load factor	[LRFD]
η	= load modifier (a factor relating to ductility, redundancy, and operational importance)	[LRFD]
ϕ	= strength reduction factor for moment = 1.0	[STD]
ϕ	= strength reduction factor for shear = 0.90	[STD]
ϕ	= resistance factor	[LRFD]
λ	= parameter used to determine friction coefficient μ	[LRFD]
μ	= Poisson's ratio for beams	[STD]
μ	= coefficient of friction	[LRFD]
θ	= angle of inclination of diagonal compressive stresses	[LRFD]
ρ_{actual}	= actual ratio of non-pretensioned reinforcement	
ρ_b	= reinforcement ratio producing balanced strain condition	[STD]
ρ^*	$= \dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	[STD]
ψ	= angle of harped pretensioned reinforcement	

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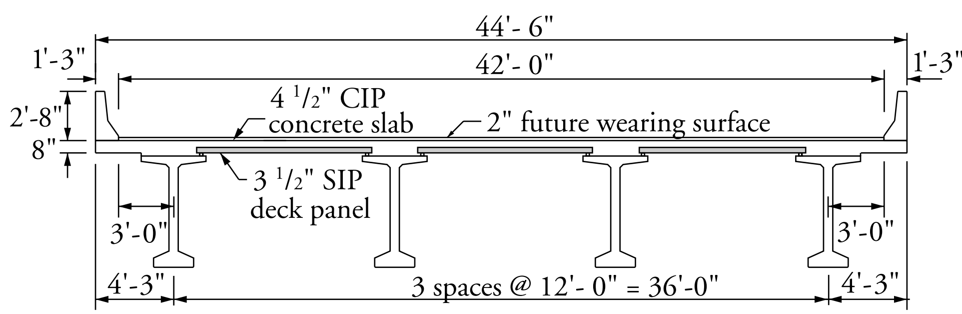
STAY-IN-PLACE PANEL DECK, LRFD SPECIFICATIONS

Precast Concrete Stay-In-Place Deck Panel System, LRFD Specifications

9.8.1 INTRODUCTION

This example demonstrates the design of a 3-1/2-in.-thick precast pretensioned stay-in-place (SIP) deck panel with a 4-1/2-in.-thick cast-in-place (CIP) concrete topping. A 1/2-in. wearing surface is considered to be an integral part of the 4-1/2-in. topping slab. The example bridge has 3-lanes with a total width of 44'-6". The deck slab is supported over four AASHTO-PCI bulb-tee beams spaced at 12 ft on center and includes overhangs of 4'-3", as shown in **Figure 9.8.1-1**. The CIP concrete requires a design strength of 4,000 psi. The SIP panel requires a strength of 6,500 psi at the time of transfer of the prestress force to the panel and a design strength of 8,000 psi at the time of casting the CIP slab. A New Jersey-type barrier is included. The design is conducted in accordance with the *LRFD Specifications*, 2nd Edition, 1998, and including through the 2003 Interim Revisions. The strip design method is used.

Figure 9.8.1-1
Bridge Cross-Section



9.8.2 MATERIALS

Cast-in-place composite slab: Actual thickness = 4-1/2 in.

Structural thickness = 4 in. (a 1/2-in. wearing surface is considered to be an integral part of the deck)

Specified concrete strength at 28 days, f'_c = 4,000 psi

Concrete unit weight, w_c = 0.150 kcf

Superstructure beams: AASHTO-PCI bulb-tee beam shown in **Figure 9.8.2-1**

Beam spacing = 12.0 ft

Top flange width = 42 in.

Deck overhang = 4.25 ft

Precast pretensioned SIP deck panels: Concrete strength at transfer, f'_{ci} = 6,500 psi

28-day concrete strength, f'_c = 8,000 psi

Concrete unit weight, w_c = 0.150 kcf

Prestressing strands: 1/2-in. diameter, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Stress limits for prestressing strands: [LRFD Table 5.9.3-1]

Before transfer, $f_{pi} \leq 0.75f_{pu}$ = 202.5 ksi

At service limit state (after all losses)

$f_{pe} \leq 0.80f_{py}$ = 194.4 ksi

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Panel dimensions: 8-ft wide x 9 ft-6 in. long x 3-1/2 in. deep

Mild reinforcement: reinforcing bars

Yield strength, f_y = 60,000 psi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

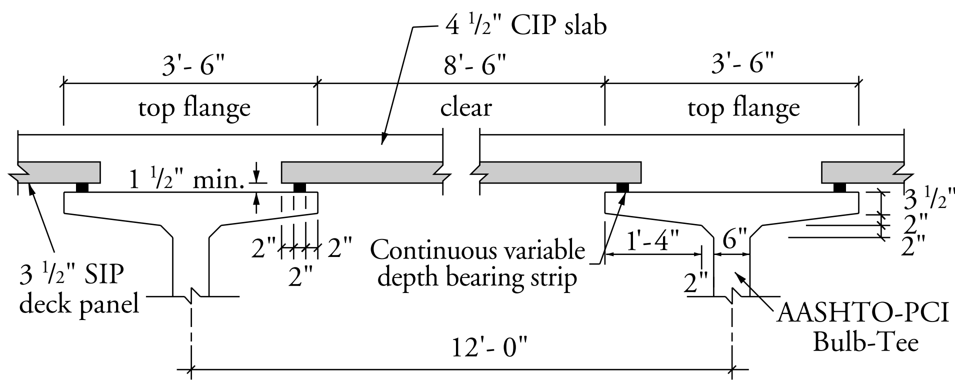
Top reinforcement clear cover = 2.5 in. [LRFD Table 5.12.3-1]

Bottom reinforcement clear cover = 1.0 in. [LRFD Table 5.12.3-1]

Future wearing surface: additional 2 in., unit weight = 0.150 kcf

New Jersey-type barrier: unit weight = 0.300 kip/ft/side

Figure 9.8.2-1
Details of the SIP Deck Panel on Supports



9.8.3 MINIMUM SLAB THICKNESS

[LRFD Art. 9.7.1.1]

For interior spans: 8 in. − 0.5 in. sacrificial layer = 7.5 in.

For overhangs: 8 in. [LRFD Art. 13.7.3.1.2]

Depth of the SIP panel: [LRFD Art. 9.7.4.3.1]

SIP thickness should be ≤ 55%(total depth) = 0.55(8.0) = 4.4 in. > 3.5 in. OK

Select the 3.5-in.-thick precast SIP panel with 4.5-in.-thick CIP slab. A 1/2-in. wearing surface is considered to be an integral part of the 4.5-in. topping slab.

9.8.4
LOADS

The precast SIP panels support their own weight, any construction loads, and the weight of the CIP slab. For superimposed dead and live loads, the precast panels are analyzed assuming that they act compositely with the CIP concrete.

9.8.4.1
Dead Loads

Weight of 3.5-in.-thick SIP panel = (3.5/12)(0.150) = 0.044 kip/ft^2

Weight of 4.5-in.-thick CIP slab = (4.5/12)(0.150) = 0.056 kip/ft^2

Weight of New Jersey barrier = 0.300 kip/ft/side

9.8.4.2
Wearing Surface and Construction Loads

Weight of 2-in. wearing surface = (2/12)(0.150) = 0.025 kip/ft^2

Construction load (applied to the SIP precast panel only) = 0.050 kip/ft^2

[LRFD Art. 9.7.4.1]

Note that LRFD Article 3.4.2 requires a load factor of at least 1.5 be used with construction loads.

9.8.4.3
Live Loads

LRFD Article 3.6.1.3.3 states that for decks where the primary strips are transverse and their span does not exceed 15 ft, the transverse strips are designed for the wheels of the 32.0-kip axle of the design truck.

Multiple Presence Factor: [LRFD Art. 3.6.1.1.2]

Single truck = 1.2

Two trucks = 1.0

Dynamic Load Allowance = 33% [LRFD Art. 3.6.2.1]

LRFD Table A4.1-1 gives the values of maximum positive and negative bending moments for different spans. This table is valid for decks supported on at least three girders and having a width measured between the centerlines of the exterior girders of not less than 14 ft. Multiple presence factors and the dynamic load allowance are included in the tabulated values. Values of negative bending moments provided by this table do not apply to the deck overhang.

For the deck under consideration, where S = 12.0 ft, the maximum positive bending moment, with dynamic allowance, M_{LL+I} = 8.010 ft-kips/ft.

For the overhang, a minimum distance of 12 in. from center of wheel of the design truck to the inside face of parapet should be considered [LRFD Art. 3.6.1.3]. However, LRFD Article 3.6.1.3.4 states that for overhangs less than 6.0 ft with continuous barrier, the outside row of wheels may be replaced with a uniformly distributed, 1.0-kip/ft line load, located 1.0 ft from railing face. In this example, the case of concentrated wheel loads is considered.

9.8.4.4
Load Combination

[LRFD Art. 3.4]

Total factored load shall be taken as:

$Q = \eta \Sigma \gamma_i Q_i$ [LRFD Eq. 3.4.1-1]

where

η = a factor relating to ductility, redundancy, and operational importance. For this example, η is considered to be 1.0. [LRFD Art. 1.3.2]

γ_i = load factors [LRFD Table 3.4.1-1]

Q_i = specified loads

Evaluating the various limit states given in LRFD Article 3.4.1, the following are applicable:

• Service I: to investigate compressive stresses in prestressed concrete components

Q = 1.00(DC + DW) + 1.00(LL + IM) [LRFD Table 3.4.1-1]

This load combination is the general combination for service limit state stress checks and applies to all conditions other than Service III.

• Service III: to investigate tensile stresses in prestressed concrete components

Q = 1.00(DC + DW) + 0.80(LL + IM) [LRFD Table 3.4.1-1]

This load combination is a special combination for service limit state stress checks that applies only to tension in prestressed concrete structures to control cracks. Note that the 0.8 factor provided for the live load with dynamic allowance is intended for application to longitudinal prestressed concrete beams only. Therefore, it is replaced with a factor of 1.0 for use in this example.

• Strength I: to check ultimate strength [LRFD Tables 3.4.1-1 and 2]

Maximum Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)

Minimum Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)

This load combination is the general load combination for strength limit state design.

Note: For simple-span bridges, the maximum load factors produce maximum effects. Use minimum load factors for dead load (DC) and wearing surface (DW) when dead load and wearing surface stresses are opposite to those of the live load.

• Fatigue: [LRFD Art. 9.5.3 and 5.5.3.1]

Fatigue need not be investigated for concrete slabs in multi-beam bridges.

9.8.5 SECTION PROPERTIES

9.8.5.1 Non-Composite Section

A = area of cross-section of the precast panel = (3.5)(12) = 42 in.^{2}/ft

S_b = section modulus for the extreme bottom fiber of the non-composite precast panel

= $(12)(3.5)^2/6$ = 24.5 in.^{3}/ft

S_t = section modulus for the extreme top fiber of the non-composite precast panel

= $(12)(3.5)^2/6$ = 24.5 in.^{3}/ft

$E_c = 33{,}000\,(w_c)^{1.5}\sqrt{f'_c}$ [LRFD Eq. 5.4.2.4-1]

where

E_c = modulus of elasticity, (ksi)

w_c = unit weight of concrete, (kcf) = 0.150 kcf

f'_c = specified strength of concrete, (ksi)

LRFD Article C5.4.2.4, indicates that the unit weight of normal weight concrete is 0.145 kcf. However, precast concrete mixes typically have a relatively low water/cementitious material ratio and high density. Therefore, a unit weight of 0.150 kcf is used in this example. For high strength concrete, even this value may need to be increased based on test results.

Therefore, the modulus of elasticity is:

At release, $E_{ci} = 33{,}000(0.150)^{1.5}\sqrt{6.5} = 4{,}888$ ksi

At service loads, $E_c = 33{,}000(0.150)^{1.5}\sqrt{8.0} = 5{,}423$ ksi

9.8.5.2 Composite Section

The pretensioning reinforcement is ignored in the calculations of the composite section properties for simplicity.

E_c (for the SIP panel) = 5,423 ksi

E_c (for the CIP slab) = $33{,}000(0.150)^{1.5}\sqrt{4.0} = 3{,}834$ ksi

n = modular ratio between CIP Slab and SIP panel = 3,834/5,423 = 0.707

Transformed width of the CIP slab = (0.707)(12) = 8.48 in./ft

Figure 9.8.5.2-1 shows the dimensions of the composite section. Since a 1/2 in. wearing surface is considered to be an integral part of the 4-1/2 in. CIP slab, only the structural depth of the CIP slab, 4 in., is considered.

A_c = total area of the composite section

$= 12(3.5) + 8.48(4) = 42 + 33.92 = 75.92$ in.^{2}/ft

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast panel = [42(3.5/2) + 33.92(3.5 + 2)]/(75.92) = 3.43 in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast panel = 3.50 − 3.43 = 0.07 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the CIP slab = 3.5 + 4.0 − 3.43 = 4.07 in.

I_c = moment of inertia of the composite section

$= (42)(3.5)^2/12 + 42(3.43 - 3.5/2)^2 + (33.92)(4)^2/12 + (33.92)(3.5 + 2 - 3.43)^2$

$= 352$ in.^{4}/ft

S_{bc} = composite section modulus for the extreme bottom fiber of the precast panel

$= (I_c/y_{bc}) = 352/3.43 = 102.6$ in.^{3}/ft

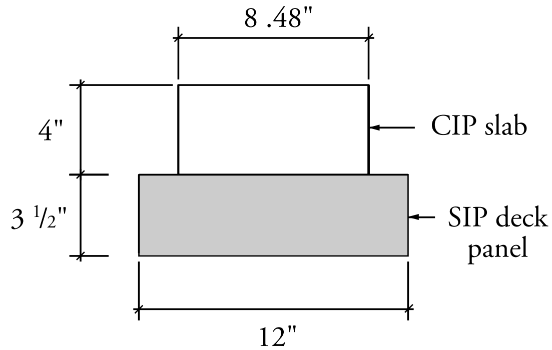
S_{tg} = composite section modulus for the top fiber of the precast panel

$= (I_c/y_{tg}) = 352/0.07 = 5{,}028.6$ in.^{3}/ft

S_{tc} = composite section modulus for extreme top fiber used to compute stresses at top fiber of the slab

$$= \left(\frac{1}{n}\right)(I_c/y_{tc}) = (1/0.707)(352/4.07) = (1/0.707)(86.5) = 122.4 \text{ in.}^3\text{/ft}$$

Figure 9.8.5.2-1
Transformed Composite Section



9.8.6 REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stress at service loads. Bottom tensile stress due to applied dead and live loads, using the modified Service III load combination (see Section 9.8.4.4), is:

$$f_b = \frac{M_{SIP} + M_{CIP}}{S_b} + \frac{M_{ws} + M_b + M_{LL+I}}{S_{bc}}$$

where

f_b = bottom tensile stresses, ksi
M_{SIP} = unfactored bending moment due to self-weight of SIP panel
M_{CIP} = unfactored bending moment due to weight of CIP slab
M_{ws} = unfactored bending moment due to future wearing surface
M_b = unfactored bending moment due to weight of barriers
M_{LL+I} = unfactored bending moment due to live load with impact
= 8.010 ft-kips/ft (as calculated in Section 9.8.4.3)

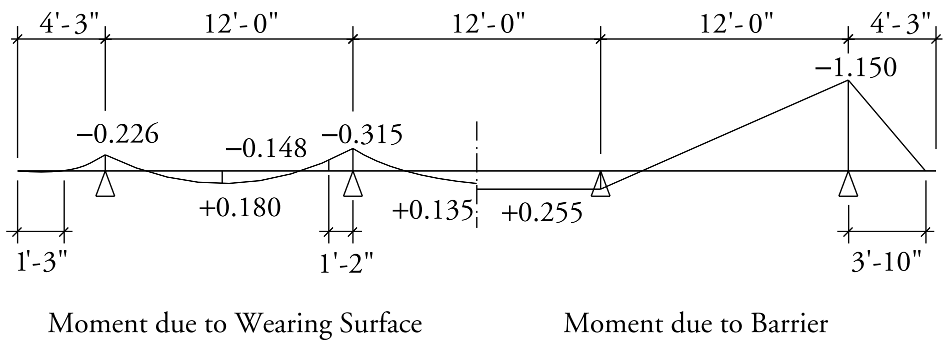
For bending moments due to the weight of the SIP panel and CIP slab, which are acting on the non-composite section, the span length is taken conservatively as the panel length, 9'-6".

$M_{SIP} = (0.044)(9.5)^2/8 = 0.496$ ft-kips/ft
$M_{CIP} = (0.056)(9.5)^2/8 = 0.632$ ft-kips/ft

For the superimposed dead and live loads, LRFD Article 4.6.2.1.6 states that force effects should be calculated based on analyzing the strip as a continuous beam supported by infinitely rigid supports. The maximum value of positive moment applies to all positive moment sections [LRFD Art. 4.6.2.1.1]. Also, LRFD Article 4.6.2.1.6 states that the effective span is the center-to-center distance between the supporting beams, which is 12.0 ft. Using software for continuous beam analysis, bending moments due to wearing surface and barrier weight are as shown in **Figure 9.8.6-1**.

To arrive at maximum effects, consider the interior span, where:
$M_{ws} = 0.135$ ft-kips/ft
$M_b = 0.255$ ft-kips/ft

Figure 9.8.6-1
Bending Moments in ft-kips/ft



$$f_b = \frac{(0.496 + 0.632)12}{24.5} + \frac{(0.135 + 0.255 + 8.010)12}{102.6} = 0.553 + 0.983 = 1.536 \text{ ksi}$$

Tensile stress limit at service loads = $0.19\sqrt{f_c'}$ [LRFD Art. 5.9.4.2.2b]

$0.19\sqrt{8.0} = 0.537$ ksi

Required precompression stress at bottom fiber = 1.536 − 0.537 = 0.999 ksi

If P_{se} is the total effective prestress force after all losses, and the center of gravity of strands is concentric with the center of gravity of the SIP panel:

$$0.999 = \frac{P_{se}}{A} = \frac{P_{se}}{42}$$

P_{se} = 42.0 kips/ft = (42)(8.0) = 336.0 kips/panel

Using 1/2-in. diameter, 270 ksi, low-relaxation strand and assuming 15% final losses, the final prestress force per strand = f_{pi}(area of strand)(1−final losses)

= (202.5)(0.153)(1 − 0.15) = 26.3 kips

The required number of strands = 336/26.3 = 12.8 strands/panel

Try (13) 1/2-in.-diameter, 270 ksi, low-relaxation strands per panel.

9.8.7 PRESTRESS LOSSES

[LRFD Art. 5.9.5]

Total prestress losses, $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ [LRFD Eq. 5.9.5.1-1]

where

Δf_{pES} = loss due to elastic shortening

Δf_{pSR} = loss due to shrinkage

Δf_{pCR} = loss due to creep

Δf_{pR2} = loss due to relaxation of steel after transfer

9.8.7.1 Elastic Shortening

[LRFD Art. 5.9.5.2.3a]

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp}$$ [LRFD Eq. 5.9.5.2.3a-1]

where

E_p = modulus of elasticity of prestressing reinforcement = 28,500 ksi

E_{ci} = modulus of elasticity of the SIP panel at transfer = 4,888 ksi

f_{cgp} = sum of concrete stresses at the center of gravity of prestressing tendons due to prestressing force at transfer and the self-weight of the member at sections of maximum positive moment

Total prestress force at transfer, P_i = (area of strand)(prestress stress at transfer)(number of strands)

The LRFD Article 5.9.5.2.3a states that f_{cgp} can be calculated on the basis of a prestressing steel stress assumed to be $0.7f_{pu}$ for low-relaxation strands. However, common practice assumes the initial losses as a percentage of initial prestressing stress before transfer, f_{pi}. In both procedures, assumed initial losses should be checked, and if different from the assumed value, a second iteration should be conducted. In this example, a 5% f_{pi} initial loss is used. The total prestress force after transfer is:

P_i = (0.153)(202.5)(1–0.05)(13 strands) = 382.6 kips/panel = 382.6/8 = 47.8 kips/ft

$$f_{cgp} = \frac{P_i}{A} = \frac{47.8}{42} = 1.138 \text{ ksi}$$

$$\Delta f_{pES} = \frac{28{,}500}{4{,}888}(1.138) = 6.635 \text{ ksi}$$

9.8.7.2 Shrinkage

[LRFD Art. 5.9.5.4.2]

$\Delta f_{pSR} = (17 - 0.15H)$ [LRFD Eq. 5.9.5.4.2-1]

where H = relative humidity (assume 70%). Relative humidity varies significantly throughout the country. See LRFD Figure 5.4.2.3.3-1.

$\Delta f_{pSR} = 17 - 0.15(70) = 6.500$ ksi

9.8.7.3 Creep of Concrete

[LRFD Art. 5.9.5.4.3]

$\Delta f_{pCR} = 12f_{cgp} - 7\Delta f_{cdp}$ [LRFD Eq. 5.9.5.4.3-1]

where

f_{cdp} = change of stresses at center of gravity of prestress due to permanent loads (weights of CIP slab, wearing surface and barrier), except dead load acting at the time the prestress force is applied (at transfer), calculated at the same section as f_{cgp},

M_{CIP} = 0.632 ft-kips/ft (acts on the non-composite section)

M_{ws} = 0.135 ft-kips/ft (acts on the composite section)

M_b = 0.255 ft-kips/ft (acts on the composite section)

However, the weight of the CIP slab provides zero stress at the center of gravity of pretensioning force. So, stresses due only to wearing surface and barriers are considered as shown in **Figure 9.8.7.3-1**.

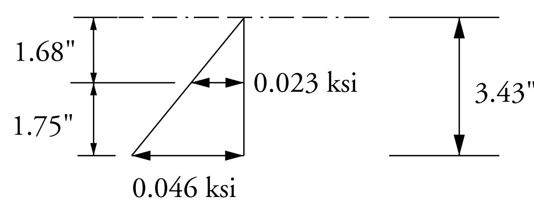
Stress at bottom fiber of the SIP panel:

$$\frac{M_{ws} + M_b}{S_{bc}} = \frac{(0.135 + 0.255)(12)}{102.6} = 0.046 \text{ ksi}$$

Therefore, $\Delta f_{cdp} = 0.046(1.68)/3.43 = 0.023$ ksi

$\Delta f_{pCR} = 12(1.138) - 7(0.023) = 13.495$ ksi

Figure 9.8.7.3-1
Concrete Stresses Due to Wearing Surface and Barrier



9.8.7.4 Relaxation of Pretensioning Strands

[LRFD Art. 5.9.5.4.4]

9.8.7.4.1 Relaxation before Transfer

The relaxation loss between tensioning and transfer, Δf_{pR1}, which will be used to compute the initial prestress loss, Δf_{pi}, is calculated by Eq. 8.6.5.3-1 [similar to LRFD Eq.5.9.5.4.4b-2 (see discussion in Section 8.6.5.3)] with $K_r = 45$ and $t = 0.75$ days (18 hrs):

$$\Delta f_{pR1} = \frac{\log(24(0.75))}{45}\left(\frac{202.5}{243.0} - 0.55\right)202.5 = 1.600 \text{ ksi} \qquad \text{(Eq. 8.6.5.3-1)}$$

This initial loss due to relaxation is assumed to be included in the total relaxation loss, Δf_{pR2}. Therefore, Δf_{pR1} is not used to compute the total prestress loss.

9.8.7.4.2 Relaxation after Transfer

For low-relaxation strand, the total loss due to relaxation is:

$$\Delta f_{pR2} = 6.0 - 0.12\Delta f_{pES} - 0.06(\Delta f_{pSR} + \Delta f_{pCR}) \qquad \text{(Eq. 8.6.10.3-1)}$$

$$= 6.0 - 0.12(6.635) - 0.06(6.500 + 13.495) = 4.004 \text{ ksi}$$

9.8.7.5 Total Losses at Transfer

The total loss of prestress at transfer is equal to the sum of the elastic shortening loss and the relaxation of the strand prior to transfer:

$\Delta f_{pi} = \Delta f_{pES} + \Delta f_{pR1} = 6.635 + 1.600 = 8.235$ ksi

Initial loss percentage: 8.235/202.5 x 100 = 4.1%

The first estimation of losses that occur prior to transfer, 5%, is relatively close to the computed losses of 4.1%. Therefore, there is no need to perform a second iteration to refine these losses.

Stress in tendons after transfer:

$f_{pt} = f_{pi} - \Delta f_{pi} = (202.5 - 8.235) = 194.265$ ksi

Force per strand = (f_{pt})(area of strand) = (194.265)(0.153) = 29.72 kips

Therefore total prestress force after transfer:

$P_i = (29.72)(13) = 386.4$ kips/panel = 386.4/8 = 48.3 kips/ft

9.8.7.6
Total Losses at Service Loads

Total loss of prestress at service loads is:

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2} = 6.635 + 6.500 + 13.495 + 4.004 = 30.634$ ksi

Final loss, % = (Total losses at service loads)/(f_{pi}) = 30.634/(202.5) = 15.1 %

Stress in tendon after all losses, $f_{pe} = f_{pi} - \Delta f_{pT} = 202.5 - 30.634 = 171.866$ ksi

Force per strand = (f_{pe})(area of strand) = 171.866(0.153) = 26.3 kips

Therefore, the total prestress force after all losses:

$P_{pe} = 26.3(13) = 341.9$ kips/panel = 341.9/8 = 42.7 kips/ft

9.8.8 STRESSES IN THE SIP PANEL AT TRANSFER

9.8.8.1
Stress Limits for Concrete

[LRFD Art. 5.9.4]

Compression: $0.6f'_{ci} = 0.6(6.5) = 3.900$ ksi

where f'_{ci} = concrete strength at release

Tension: without bonded reinforcement: $-0.0948\sqrt{f'_{ci}} \le -0.2$ ksi

$-0.0948\sqrt{6.5} = -0.242 \text{ ksi} > -0.2$ ksi

therefore, use –0.200 ksi (Controls)

with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete:

$-0.22\sqrt{f'_{ci}} = -0.22\sqrt{6.5} = -0.561$ ksi

Because the strand group is concentric with the precast concrete panel, the midspan section is the critical section that should be checked.

9.8.8.2
Stresses at Midspan

Effective prestress after transfer = 48.3 kips/ft

Bending moment due to self-weight of the panel, $M_{SIP} = 0.496$ ft-kips/ft

Top concrete stress in the SIP panel:

$$f_t = \frac{P_i}{A} + \frac{M_{SIP}}{S_t} = \frac{48.3}{42} + \frac{0.496\,(12)}{24.5} = +1.150 + 0.243 = +1.393 \text{ ksi}$$

Compressive stress limit: +3.900 ksi > +1.393 ksi OK

Bottom concrete stress of the SIP panel:

$$f_b = \frac{P_i}{A} - \frac{M_{SIP}}{S_b} = \frac{48.3}{42} - \frac{0.496\,(12)}{24.5} = +1.150 - 0.243 = +0.907 \text{ ksi}$$

Compressive stress limit: +3.900 ksi > +0.907 ksi OK

9.8.9 STRESSES IN SIP PANEL AT TIME OF CASTING TOPPING SLAB

The total prestress force after all losses, P_{pe} = 42.7 kips/ft

9.8.9.1 Stress Limits for Concrete

[LRFD Art. 9.7.4.1]

LRFD Article 9.7.4.1 states that flexural stresses in the SIP formwork due to unfactored construction loads should not exceed 65% of the 28-day compressive strength for concrete in compression, or the modulus of rupture in tension for prestressed concrete form panels.

Note that the definition of construction loads according to the *LRFD Specifications* includes the weight of the SIP panel, CIP topping, and an additional 0.050 ksf.

Therefore, the stress limit for concrete in compression, for load combination Service I:
$0.65f'_c = 0.65(8.0) = 5.200$ ksi.

Stress limit for concrete in tension, for load combination Service I:

Modulus of rupture, $f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{8.0} = 0.679$ ksi [LRFD Art. 5.4.2.6]

9.8.9.2 Stresses at Midspan after all Non-Composite Loads

Bending moment due to the self-weight of the SIP panel, the CIP topping and construction load:

M_{SIP} = 0.496 ft-kips/ft
M_{CIP} = 0.632 ft-kips/ft
$M_{const} = (0.050)(9.5)^2/8 = 0.564$ ft-kips/ft

Concrete stress at top fiber of the SIP panel:

$$f_t = \frac{P_{pe}}{A} + \frac{M_{SIP} + M_{CIP} + M_{const}}{S_t}$$

$$= \frac{42.7}{42} + \frac{(0.496 + 0.632 + 0.564)(12)}{24.5} = +1.017 + 0.829 = +1.846 \text{ ksi}$$

Compressive stress limit: +5.200 ksi > +1.846 ksi OK

Concrete stress at bottom fiber of the SIP panel:

$$f_b = \frac{P_{pe}}{A} - \frac{M_{SIP} + M_{CIP} + M_{const}}{S_b} = \frac{42.7}{42} - \frac{(0.496 + 0.632 + 0.564)(12)}{24.5}$$

$$= +1.017 - 0.829 = +0.188 \text{ ksi}$$

Compressive stress limit: +5.200 ksi > +0.188 ksi OK

9.8.9.3
Elastic Deformation

[Art. 9.7.4.1]

LRFD Article 9.7.4.1, states that for SIP panels spanning less than 10 ft, the elastic deformation due to dead load of the panel plus the CIP topping should not exceed either the panel span divided by 180 or 0.25 in.

$$\text{Elastic deformation} = \frac{5}{48}\frac{(M_{SIP} + M_{CIP})L^2}{E_c I}$$

$$= \left(\frac{5}{48}\right)\frac{(0.496 + 0.632)(12)\left[(9.5)(12)\right]^2}{(5,423)(42.875)}$$

$$= 0.08 \text{ in.} < 0.25 \text{ in.} \quad \text{OK}$$

9.8.10
STRESSES IN SIP PANEL AT SERVICE LOADS

The total prestress force after all losses, P_{pe} = 42.7 kips

9.8.10.1
Stress Limits for Concrete

[LRFD Art. 5.9.4.2]

Compression: for load combination Service I:

- Due to permanent loads, (i.e. self-weight of SIP panel, CIP slab, wearing surface and barriers) = $0.45f'_c$
  for the SIP panel = 0.45(8.0) = 3.600 ksi
  for the CIP slab = 0.45(4.0) = 1.800 ksi

- Due to permanent and transient loads, (i.e. all dead and live loads) = $0.60f'_c$
  for the SIP panel = 0.60(8.0) = 4.800 ksi
  for the CIP slab = 0.60(4.0) = 2.400 ksi

- Due to live load plus one-half sum of effective prestress and permanent loads = $0.40f'_c$
  for the SIP panel = 0.40(8.0) = 3.200 ksi
  for the CIP slab = 0.40(4.0) = 1.600 ksi

Tension: for load combination Service III: $0.19\sqrt{f'_c}$
for the SIP panel = $-0.19\sqrt{8.0} = -0.537$ ksi

9.8.10.2
Service Load Stresses at Midspan

Effective prestress after all losses, P_{se} = 42.7 kips

The weights of the SIP panel and the CIP concrete act on the non-composite section:

M_{SIP} = 0.496 ft-kips/ft
M_{CIP} = 0.632 ft-kips/ft

At the time of opening the bridge to traffic, the wearing surface, barriers, and live loads act on the composite section.

M_{ws} = 0.135 ft-kips/ft
M_b = 0.255 ft-kips/ft
M_{LL+I} = 8.010 ft-kips/ft

- Concrete stress at top surface of the CIP slab:

  Due to permanent loads, Service I:

$$f_{tc} = \frac{M_{ws} + M_b}{S_{tc}} = +\frac{(0.135 + 0.255)(12)}{122.4} = +0.038 \text{ ksi}$$

  Compressive stress limit: +1.800 ksi > +0.038 ksi OK

  Due to permanent and transient loads, Service I

$$f_{tc} = \frac{M_{ws} + M_b + M_{LL+I}}{S_{tc}} = +\frac{(0.135 + 0.255 + 8.010)(12)}{122.4} = +0.824 \text{ ksi}$$

  Compressive stress limit: +2.400 ksi > +0.824 ksi OK

- Concrete stress at top fiber of the SIP panel:

  Due to permanent loads, Service I:

$$f_{tg} = \frac{P_{pe}}{A} + \frac{M_{SIP} + M_{CIP}}{S_t} + \frac{M_{ws} + M_b}{S_{tg}}$$

$$= +\frac{(42.7)}{42} + \frac{(0.496 + 0.632)(12)}{24.5} + \frac{(0.135 + 0.255)(12)}{5,028}$$

$$= +1.017 + 0.553 + 0.001 = +1.571 \text{ ksi}$$

  Compressive stress limit: +3.600 ksi > +1.571 ksi OK

  Due to permanent and transient loads, Service I:

$$f_{tg} = \frac{P_{pe}}{A} + \frac{M_{SIP} + M_{CIP}}{S_t} + \frac{M_{ws} + M_b + M_{LL+I}}{S_{tg}}$$

$$= +\frac{(42.7)}{42} + \frac{(0.496 + 0.632)(12)}{24.5} + \frac{(0.135 + 0.255 + 8.010)(12)}{5,028}$$

$$= +1.017 + 0.553 + 0.020 = +1.590 \text{ ksi}$$

  Compressive stress limit: +4.800 ksi > +1.590 ksi OK

  Due to live load plus one-half sum of effective prestress and permanent loads:

$$f_{tg} = \frac{0.5P_{pe}}{A} + \frac{0.5(M_{SIP} + M_{CIP})}{S_t} + \frac{0.5(M_{ws} + M_b) + M_{LL+I}}{S_{tg}}$$

$$= +0.5\frac{42.7}{42} + 0.5\frac{(0.496 + 0.632)(12)}{24.5} + \frac{[0.5(0.135 + 0.255) + (8.010)](12)}{5,028}$$

$= +0.508 + 0.276 + 0.019 = +0.803$ ksi

Compressive stress limit: +3.200 ksi > +0.803 ksi OK

• Concrete stress at bottom fiber of the SIP panel:

$$f_b = \frac{P_{pe}}{A} - \frac{M_{SIP} + M_{CIP}}{S_b} - \frac{M_{ws} + M_b + M_{LL+I}}{S_{bc}}$$

$$= +\frac{(42.7)}{42} - \frac{(0.496 + 0.632)(12)}{24.5} - \frac{(0.135 + 0.255 + 8.010)(12)}{102.6}$$

$= +1.017 - 0.553 - 0.982 = -0.518$ ksi

Tensile stress limit: –0.537 ksi > –0.518 ksi OK

9.8.11 FLEXURAL STRENGTH OF POSITIVE MOMENT SECTION

Total ultimate bending moment for Strength I:

$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$

$M_{SIP} = 0.496$ ft-kips/ft

$M_{CIP} = 0.632$ ft-kips/ft

$M_{ws} = 0.135$ ft-kips/ft

$M_b = 0.255$ ft-kips/ft

$M_{LL+I} = 8.010$ ft-kips/ft

$M_u = 1.25(0.496 + 0.632 + 0.255) + 1.5(0.135) + 1.75(8.010) = 15.949$ ft-kips/ft

Factored flexural resistance:

$M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where

ϕ = Resistance factor [LRFD Art. 5.5.4.2.1]

= 1.00 for flexure and tension of prestressed concrete

M_n = Nominal flexural resistance [LRFD Art. 5.7.3.2.3]

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$ [LRFD Eq. 5.7.3.2.2-1]

where

A_{ps} = area of prestressing steel = 13(0.153) = 1.989 in.^{2}

d_p = distance from extreme compression fiber of the composite section to the centroid of the prestressing tendons = 7.5 – 0.5(3.5) = 5.75 in.

a = depth of equivalent rectangular stress block = $\beta_1 c$

where

c = distance from neutral axis to extreme compression fiber

β_1 = stress factor of compression block [LRFD Art. 5.7.2.2]

= 0.85 for $f'_c \leq 4.0$ ksi

$0.85 - 0.05(f'_c - 4.0) \geq 0.65$ for $f'_c > 4.0$ ksi

= 0.85

To compute c, use rectangular section behavior [LRFD Art. C5.7.3.2.2]

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A_s' f_y'}{0.85f_c'\beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} \qquad \text{[LRFD Eq. 5.7.3.1.1.-4]}$$

where

A_s = area of mild steel tension reinforcement = 0
A_s' = area of compression reinforcement = 0
f_c' = compressive strength of slab concrete = 4.0 ksi
f_y = yield strength of mild steel tension reinforcement, (ksi)
f_y' = yield strength of mild steel compression reinforcement, (ksi)
f_{pu} = specified tensile strength of prestressing steel = 270 ksi
b = effective width of compression flange = 8.0(12) = 96.0 in.

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad \text{[LRFD Eq. 5.7.3.1.1-2]}$$

for low-relaxation strands, $k = 0.28$ [LRFD Table C5.7.3.1.1-1]

$$c = \frac{1.989(270) + 0 - 0}{0.85(4.0)(0.85)(96) + 0.28(1.989)\left(\dfrac{270}{5.75}\right)} = 1.77 \text{ in.}$$

$a = \beta_1 c = 0.85(1.77) = 1.51$ in.
f_{ps} = average stress in prestressing steel

When $f_{pe} \geq 0.5f_{pu}$, $f_{ps} = f_{pu}\left(1 - k\dfrac{c}{d_p}\right)$ [LRFD Eq. 5.7.3.1.1-1]

$$f_{ps} = 270\left(1 - 0.28\frac{1.77}{5.75}\right) = 246.7 \text{ ksi}$$

Check stress in prestressing steel according to available development length, ℓ_d:

$\ell_d = \left(f_{ps} - \dfrac{2}{3}f_{pe}\right)d_b$, or, rearranging, [LRFD Eq. 5.11.4.1-1]

$$f_{ps} = \frac{\ell_d}{d_b} + \frac{2}{3}f_{pe} \qquad \text{(Eq. 9.8.11-1)}$$

where

d_b = nominal strand diameter
f_{pe} = effective stress in prestressing steel after losses
= 171.9 ksi

Available development length at midspan of the SIP panel: 0.5(9.5) = 4.75 ft

$$f_{ps} = \frac{4.75\,(12)}{0.5} + \frac{2}{3}(171.9) = 228.6 \text{ ksi} \quad \text{(Controls)}$$

Therefore, the design flexural strength is:

$$\phi M_n = \frac{1.0\,(1.989)\,(228.6)\left(5.75 - \dfrac{1.51}{2}\right)}{12}$$

$= 189.2$ ft-kips/panel $= 189.2/8 = 23.7$ ft-kips/ft $> M_u = 15.949$ ft-kips/ft OK

The calculations here were conducted for flexure in the midspan section. It is possible that intermediate sections between midspan and the supports will have critical stresses due to only partial development of the strands.

9.8.12 LIMITS OF REINFORCEMENT

9.8.12.1 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

The amount of prestressed and non-prestressed reinforcement should be such that:

$$\frac{c}{d_e} \leq 0.42 \qquad \text{[LRFD Eq. 5.7.3.3.1-1]}$$

where $d_e = \dfrac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$ [LRFD Eq. 5.7.3.3.1-2]

Since $A_s = 0$, $d_e = d_p = 5.75$ in.

$$\frac{c}{d_e} = \frac{1.77}{5.75} = 0.308 \leq 0.42 \quad \text{OK}$$

9.8.12.2 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

At any section, the amount of prestressed and non-prestressed tensile reinforcement should be adequate to developed a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and
- 1.33 times the factored moment required by the applicable strength load combination.

Check at midspan:

The *LRFD Specifications* do not give a procedure for computing the cracking moment. Therefore, the following equation adapted from STD Article 9.18.2.1 is used.

$$M_{cr} = (f_r + f_{pb})S_{bc} - (M_{d/nc})(S_{bc}/S_b - 1)$$

where

f_r = modulus of rupture [LRFD Art. 5.4.2.6]

$= 0.24\sqrt{f'_c} = 0.24\sqrt{8.0} = 0.679$ ksi

f_{pb} = compressive strength in concrete due to effective prestress force only, (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$= \frac{P_{pe}}{A} = \frac{42.7}{42} = 1.017 \text{ ksi}$$

$M_{d/nc}$ = non-composite dead load moment at the section

$= M_{CIP} + M_{SIP} = 0.632 + 0.496 = 1.128$ ft-kips/ft

S_{bc} = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 102.6 in.^{3}/ft

S_b = non-composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads = 24.5 in.^{3}/ft

$$M_{cr} = (0.679 + 1.017)\left(\frac{102.6}{12}\right) - (1.128)\left(\frac{102.6}{24.5} - 1\right) = 10.905 \text{ ft-kips/ft}$$

$1.2M_{cr} = 1.2(10.905) = 13.086$ ft-kips/ft

At midspan, the factored moment required by Strength I load combination,
$M_u = 15.949$ ft-kips/ft. Therefore, $1.33M_u = 1.33(15.949) = 21.212$ ft-kips/ft.
Since $1.2M_{cr} < 1.33M_u$, $1.2M_{cr}$ controls.
$\phi M_n = 23.7$ ft-kips/ft $> 1.2M_{cr} = 13.086$ ft-kips/ft OK

Note: Contrary to the *Standard Specifications*, the *LRFD Specifications* requires that this criterion be met at every section.

9.8.13 NEGATIVE MOMENT SECTION OVER INTERIOR BEAMS

9.8.13.1 Critical Section

The design section for negative moments and shear forces, for precast I-shaped concrete beams, is at a distance of 1/3 of the flange width from the centerline of the support but not exceeding 15 in. [LRFD Art. 4.6.2.1.6]. Since 1/3 the beam flange width

$$= \frac{1}{3}(42) = 14 \text{ in.} < 15 \text{ in.},$$

the design section for negative moment is at a distance of 14 in. (1.17 ft) from the centerline of the beam.

9.8.13.2 Bending Moment

[LRFD Art. 4.6.2.1.6]

LRFD Article 4.6.2.1.6 states that force effects be calculated based on analyzing the strip as a continuous beam supported by infinitely rigid supports. The maximum value of moment shall apply at all sections. Using appropriate software for beam analysis, the bending moment is:

DC: Because the weight of the barrier produces positive moment at the interior girders, as shown in **Figure 9.8.6-1**, bending moment due to this load is conservatively ignored.

DW: Due to wearing surface, M_{ws} = 0.148 ft-kips/ft

LL+IM: From LRFD Table A4.1-1, for S = 12.0 ft, maximum negative bending moment at 14 in. from beam center line, with impact and multiple presence factor, M_{LL+I} = 6.347 ft-kips/ft

Therefore, the negative service bending moment is:

$M_{service} = 0.148 + 6.347 = 6.495$ ft-kips/ft

Negative factored bending moment:

$M_u = 1.5(0.148) + 1.75(6.347) = 11.329$ ft-kips/ft

9.8.13.3
Design of Section

Assume # 5 reinforcing bars, and 2.5-in. clear cover.

$d = 7.5 - 0.5(0.625) - 2.5 = 4.688$ in.

$R_n = (M_u/\phi bd^2) = (11.329)(12)/[(0.9)(12)(4.688)^2] = 0.573$ kip/in.^{2}

$m = (f_y/0.85f'_c) = (60)/[(0.85)(4.0)] = 17.65$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{f_y}}\right) = \frac{1}{17.65}\left(1-\sqrt{1-\frac{2(17.65)(0.573)}{60}}\right) = 0.01053$$

$A_s = \rho(bd) = (0.01053)(12)(4.688) = 0.59$ in.^{2}/ft

Use #5 bars @ 6-in. centers, $A_s = 0.31(12/6) = 0.62$ in.^{2}/ft

Check:

$a = (A_s f_y)/(0.85bf'_c) = (0.62)(60)/[(0.85)(12)(4.0)] = 0.91$ in.

$\phi M_n = 0.9(A_s f_y)(d - a/2) = 0.9(0.62)(60)[4.688 - 0.5(0.91)]/12$

$= 11.810$ ft-kips/ft $> M_u = 11.329$ ft-kips/ft OK

9.8.13.4
Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

$d_e = 4.688$ in.

$c = (a/\beta_1) = (0.91/0.85) = 1.07$

$c/d_e = (1.07/4.688) = 0.23 < 0.42$ OK

9.8.13.5
Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

$$\rho_{actual} = \frac{0.62}{12(4.688)} = 0.011$$

$\rho_{min} = 0.03(f'_c/f_y) = 0.03(4.0)/(60) = 0.002 < \rho_{actual}$ OK

9.8.13.6 Crack Control

[LRFD Art. 5.7.3.4]

Calculated stress in the reinforcement at service loads, f_s, should not exceed the value computed by: $f_s = \dfrac{Z}{(d_c A)^{1/3}} \leq 0.6f_y$ [LRFD Eq. 5.7.3.4-1]

where

d_c = thickness of concrete cover measured from extreme tension fiber to center of the closest bar, (in.). For calculation purposes, the thickness used for clear cover should be taken not greater than 2.0 in.

= 2.0 + 0.5(0.625) = 2.313 in.

A = effective tension area of concrete surrounding the flexure tension reinforcement and having the same centroid as that reinforcement divided by the number of bars, (in.^{2})

= 2(2.313)(6) = 27.756 in.^{2}

Z = 130 kip/in. (severe exposure condition)

$$f_s = \frac{130}{\left[2.313(27.756)\right]^{1/3}} = 32.5 \text{ ksi} < 0.6f_y = 36 \text{ ksi}$$

Actual stress in reinforcement:

E_c = 3,834 ksi

n = E_s/E_c = 29,000/3,834 = 7.56

ρ_{actual} = 0.011

k $= \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.33$

j = (1 – k/3) = 0.89

f_s = $M_{service}/(jdA_s)$ = (6.495)(12)/[(0.89)(4.688)(0.62)] = 30.1 ksi < 32.5 ksi OK

9.8.14 NEGATIVE MOMENT SECTION OVER EXTERIOR BEAMS

9.8.14.1 Critical Section

The critical section for negative moment over the beams is at a distance of 14 in., 1.17 ft, from the centerline of the beam.

Therefore, cantilever span is = 4.25 – 1.17 = 3.08 ft

9.8.14.2 Design of Section

Article A13.4.1 states that three design cases need to be checked when designing the overhang regions. These cases are:

- **Case 1**: check overhang for horizontal vehicular collision load: [LRFD Art. A13.4.1]

The overhang is designed to resist an axial tension force from vehicular collision at the extreme event limit state acting simultaneously with the moment (collision + dead loads).

From design of the barrier under Performance Level PL-3: [LRFD Art. A13.3]

Flexural resistance of the barrier at its base, M_c = 17.200 ft-kips/ft

Total transverse resistance of the barrier, i.e. collision horizontal force at top of barrier, R_w = 166.0 kips

The force, R_w, is distributed over a width of L_c at the top fiber of the barrier = 13.36 ft
Height of the barrier, H = 32 in.

Assume that this force is distributed at an angle of 45° from the top fiber of the barrier to its base, thus:

Collision force at deck slab level:

$$T = \frac{R_w}{L_c + 2H} \quad \text{[LRFD Eq. A13.4.2-1]}$$

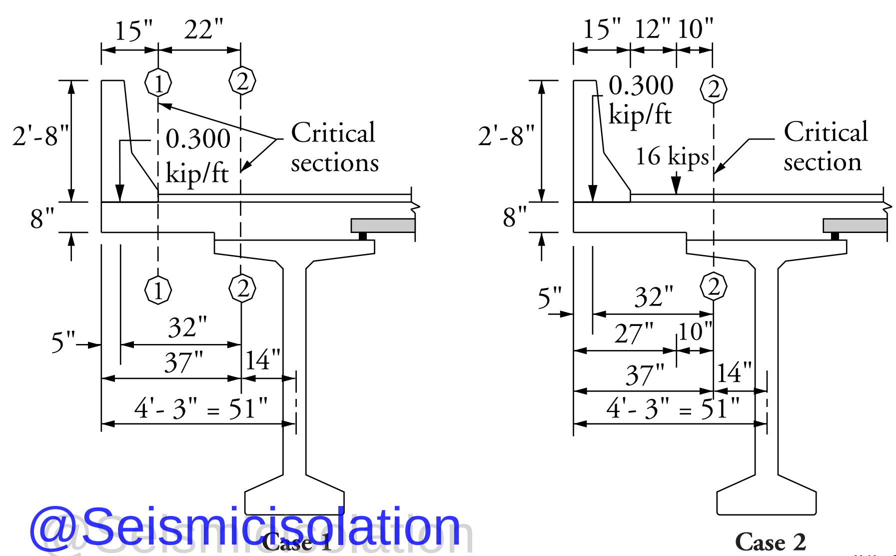
$$= \frac{166.0}{160.32 + 2(32)} = 0.74 \text{ kip/in.} = 8.88 \text{ kips/ft}$$

Note that the slab thickness is ignored.

Design the section at the inner face of barrier; i.e. Section 1-1 in Case 1 in **Figure 9.8.14.2-1**:

Factored bending moment, M_u, at face of barrier due to collision force and dead loads:
$17.200 + 1.25\{(0.5)[8(0.150)/12](15/12)^2 + 0.300(15 - 5)/12\} = 17.610$ ft-kips/ft

Figure 9.8.14.2-1
Loading Cases for the Overhang



Try #6 bars @ 5 in. centers, $A_s = 0.44(12)/5 = 1.056$ in.²/ft

For # 6 bars and 2.5 in. clear cover, the effective depth,

$d = 7.5 - 0.5(0.75) - 2.5 = 4.625$ in.

$b = 12$ in.

Check development length of steel reinforcement [LRFD Art. 5.11.2]

$$\ell_d = \frac{1.25A_b f_y}{\sqrt{f'_c}} = \frac{1.25(0.44)(60)}{\sqrt{4.0}} = 16.5 \text{ in.}$$

Since the width of the barrier, 15 in., is less than the length required to fully develop the #6 bars, available stress of the #6 bar should be reduced as follows:

$f_s = 60(15/16.5) = 54.55$ ksi.

$A_s f_s = 1.056(54.55) = 57.61$ kips

$T = 8.88$ kips/ft

$C = A_s f_s - T = 57.61 - 8.88 = 48.73$ kips

$a = C/(0.85bf'_c) = 48.73/[0.85(12)(4.0)] = 1.19$ in.

$\phi = 1.0$ (for extreme event, LRFD Art. 1.3.2.1)

$\phi M_n = \phi[A_s f_s(d - a/2) - T(0.5d - 0.5a)]$

$= 1.0[57.61(4.625 - 1.19/2) - 8.88(4.625/2 - 1.19/2)]/12$

$= 18.079$ ft-kips/ft $> M_u = 17.610$ ft-kips/ft OK

For the critical section over the exterior beam, Section 2-2 in Case 1 in **Figure 9.8.14.2-1**:

At the inner face of the barrier, the flexural resistance of the barrier at its base, 17.200 ft-kips/ft, is distributed over a length L_c, while the collision axial force T is distributed over a length of (L_c+2H) where H is the height of the barrier. Assume that the moment at the face of the barrier and the axial force are distributed at an angle of 30° from the inner face of the barrier to the design section. The collision bending moment at the critical section is:

$$\frac{M_c L_c}{L_c + 2(22)\tan 30} = \frac{17.200(160.32)}{160.32 + 2(22)\tan 30} = 14.847 \text{ ft-kips/ft}$$

The factored bending moment at Section 2-2 due to collision force and dead loads is:

$M_u = 14.847 + 1.25[(8x0.150/12)(37/12)^2/2 + 0.300(32/12)]$

$+ 1.5[(2x0.150/12)(22/12)^2/2] = 16.504$ ft-kips/ft

Collision axial force at Section 2-2 is:

$$T = \frac{R_w}{L_c + 2H + 2(22)\tan 30}$$

$$= \frac{166.0}{160.32 + 2(32) + 2(22)\tan 30} = 0.665 \text{ kip/in.} = 7.98 \text{ kips/ft}$$

Check #6 bars @ 5-in. centers:

A_s = 0.44(12)/5 = 1.056 in.², d = 7.5 – 0.5(0.75) – 2.5 = 4.625 in.

T = 7.98 kips/ft

C = $A_s f_y$ – T = 63.36 – 7.98 = 55.38 kips/ft

a = $C/(0.85bf_c')$ = 55.38/[0.85(12)(4.0)] = 1.36 in.

ϕ = 1.0 (for extreme event, LRFD Art. 1.3.2.1)

$\phi M_n = \phi[A_s f_y(d - a/2) - T(0.5d - 0.5a)]$

$= 1.0[63.36(4.625 - 1.36/2) - 7.98(4.625/2 - 1.36/2)]/12$

$= 19.744$ ft-kips/ft > M_u = 16.504 ft-kips/ft OK

- **Case 2**: check overhang for vertical collision force [LRFD Art. A13.4.1]

  For concrete parapets, the case of vertical collision never controls.

- **Case 3**: check overhang for dead and live loads: [LRFD Art. A13.4.1]

  DC:

  due to weight of slab, $M_{CIP} = (8x0.150/12)(37/12)^2/2 = 0.475$ ft-kips/ft

  due to barrier load, $M_b = 0.300(32/12) = 0.800$ ft-kips/ft

  DW:

  due to wearing surface, $M_{ws} = (2x0.150/12)(22/12)^2/2 = 0.042$ ft-kips/ft

  LL+IM:

  For maximum negative moment, the truck wheel should be at 12 in. from the face of the barrier, LRFD Art. 3.6.1.3.1, as shown in Case 2 in **Figure 9.8.14.2-1.**

  The wheel load is distributed over a width of (45.0 + 10.0 X), inches. [LRFD Art. 4.6.2.1.3]

  where X = distance from load to point of support (ft) = 10/12 = 0.833 ft

  Therefore, the width is 45 + 10(0.833) = 53.33 in. = 4.44 ft

  Dynamic allowance = 33% [LRFD Art. 3.6.2.1]

  Multiple presence factor for single truck = 1.2 [LRFD Art. 3.6.1.1.2]

  Maximum negative bending moment at Section 2-2, with impact and multiple presence factor is:

  $M_{LL+I} = (16/4.44)(0.833)(1+0.33)(1.2) = 4.791$ ft-kips/ft

Therefore, the negative service bending moment at Section 2-2:

$M_{service} = 0.475 + 0.800 + 0.042 + 4.791 = 6.108$ ft-kips/ft

Negative factored bending moment at Section 2-2:

$M_u = 1.25(0.475+0.800) + 1.5(0.042) + 1.75(4.791) = 10.041$ ft-kips/ft

Check # 6 bars @ 5 in. centers:

$A_s = 1.056$ in.², $d = 7.5 - 0.5(0.75) - 2.5 = 4.625$ in.

$a = (A_s f_y)/(0.85bf_c') = (1.056)(60)/[(0.85)(12)(4.0)] = 1.55$ in.

$\phi M_n = 0.9(A_s f_y)(d - a/2)$

$= (0.9)(1.056)(60)(4.625 - 1.55/2)/12 = 18.295$ ft-kips/ft $> M_u$ OK

9.8.14.3 Maximum Reinforcement

[LRFD Art. 5.7.3.3.1]

$d_e = 4.625$ in.

$c = (a/\beta_1) = 1.55/0.85 = 1.83$

$c/d_e = 1.83/4.625 = 0.40 < 0.42$ OK

9.8.14.4 Minimum Reinforcement

[LRFD Art. 5.7.3.3.2]

$\rho_{actual} = 1.056/(12)(4.625) = 0.019$

$\rho_{min.} = 0.03(f_c'/f_y) = 0.03(4.0/60) = 0.002 < \rho_{actual}$ OK

9.8.14.5 Crack Control

[LRFD Art. 5.7.3.4]

$d_c = [2.0 + 0.5(0.75)] = 2.375$ in.

$A = 2(2.375)(5) = 23.75$ in.²

$Z = 130$ kips/in.

$$f_s = \frac{Z}{(d_c A)^{1/3}} = \frac{130}{(23.75 \text{ x } 2.375)^{1/3}} = 33.9 \text{ ksi} < 0.6f_y = 36 \text{ ksi}$$

Actual stress in reinforcement:

$E_c = 3{,}834$ ksi

$n = E_s/E_c = 29{,}000/3{,}834 = 7.56$

$$\rho_{actual} = \frac{1.056}{12(4.625)} = 0.019$$

$$k = \sqrt{(\rho n)^2 + (2\rho n)} - \rho n = 0.41$$

$j = (1 - k/3) = 0.86$

$f_s = M_{service}/(jdA_s) = (6.108)(12)/[(0.86)(4.625)(1.056)] = 17.450$ ksi < 33.9 ksi OK

9.8.15 DISTRIBUTION REINFORCEMENT

[LRFD Art. 9.7.3.2]

The *LRFD Specifications* do not provide guidelines for the distribution reinforcement required for a SIP panel system. However, LRFD Article 9.7.3.2 gives guidance for deck slabs, which have four layers of reinforcement, as follows:

$$\text{(Distribution reinforcement/primary reinforcement), \%} = \frac{220}{\sqrt{S}} \le 67\%$$

where S = clear span + distance from extreme flange tip to the face of the web

$= (12.0 - 42/12) + 2(18/12) = 11.5$ ft [LRFD Art. 9.7.2.3]

Therefore, the percentage of distribution reinforcement $= 220/\sqrt{11.50}$

$= 64.9 < 67\%$

Based on the area of the strands, which are the main positive reinforcement in the SIP panel, the distribution reinforcement = (0.649)(13)(0.153)/8.0 = 0.16 in.^{2}/ft.

If the strand area is converted to equivalent mild reinforcement area, the required distribution reinforcement = 0.16(243/60) = 0.65 in.^{2}/ft. Note that the yield strength of each material is used as the basis for equivalence.

This amount of reinforcement is 45% higher than that required by the Empirical Design Method, LRFD Article 9.7.2.5, where a total amount of (0.27 + 0.18) = 0.45 in.^{2}/ft is provided in two layers.

Therefore, the designer may opt to use #5 bars @ 5.5-in. centers to satisfy LRFD Article 9.7.3.2 or #5 bars @ 8.0-in. centers to satisfy LRFD Article 9.7.2.5.

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NOTATION

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a	= depth of equivalent rectangular stress block	[STD], [LRFD]
A	= area of beam cross-section	[STD]
A_c	= total area of the composite section	
A_{ps}	= area of prestressing steel	[LRFD]
A_s^*	= area of prestressing steel	[STD]
A_s	= area of nonprestressed tension reinforcement	[STD], [LRFD]
A'_s	= area of compression reinforcement	[STD], [LRFD]
A_v	= area of a transverse reinforcement within distance, s	[STD], [LRFD]
b	= width of compression face of member	[STD], [LRFD]
b_e	= effective web width of the precast beam	
b_v	= effective web width of the precast beam	[STD]
b_w	= width of web of a flanged member	[STD], [LRFD]
c	= distance from extreme compression fiber to neutral axis	[STD], [LRFD]
d_e	= effective depth from extreme compression fiber to the centroid of tensile force in the tensile reinforcement	[LRFD]
d_1	= friction loss over a given length	
d_p	= distance from extreme compression fiber to the centroid of the prestressing tendons	[LRFD]
d_v	= effective shear depth	[LRFD]
DC	= dead load of structural components and nonstructural attachments	[LRFD]
DFM	= distribution factor for bending moment	
DFV	= distribution factor for shear force	
DW	= dead load of wearing surfaces and utilities	
E_p	= modulus of elasticity of pretensioning and post-tensioning reinforcement	[STD], [LRFD]
E_s	= modulus of elasticity of non-pretensioned reinforcement	[STD], [LRFD]
e	= base of natural logarithm	
e	= eccentricity of strands at transfer length	
e_c	= eccentricity of strands at the midspan	
f_b	= concrete stress at the bottom fiber of the beam	
f'_c	= specified compressive strength of concrete at 28 days, unless another age is specified	[STD], [LRFD]
f'_{ci}	= compressive strength of concrete at time of initial prestress	[STD], [LRFD]
f_{pb}	= compressive stress at bottom fiber of the beam due to prestress force	
f_{pe}	= compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads	[STD]
f_{pj}	= initial stress immediately before transfer	
f_{pj}	= stress in the prestressing steel at jacking	[LRFD]

f_{ps}	= average stress in prestressing steel at the time for which the nominal resistance of member is required	[LRFD]
f_{pt}	= stress in prestressing steel immediately after transfer	[LRFD]
f_{pu}	= ultimate strength of prestressing steel	[LRFD]
f'_s	= ultimate strength of prestressing steel	[STD]
f_{py}	= yield point stress of prestressing steel	[STD], [LRFD]
f_r	= the modulus of rupture of concrete	[STD], [LRFD]
f_s	= allowable stress in steel, taken not greater than 20 ksi	
f_t	= concrete stress at top fiber of the beam for the non-composite section	
f_{tc}	= concrete stress at top fiber of the beam for the composite section	
f_{tg}	= concrete stress at top fiber of the beam for the composite section	
f_{sy}	= specified yield strength of nonprestressed conventional reinforcement	[STD]
f_y	= specified yield strength of nonprestressed conventional reinforcement	[STD], [LRFD]
f'_y	= specified yield strength of nonprestressed conventional reinforcement	[STD]
h	= overall thickness or depth of a member	[STD], [LRFD]
h_c	= total height of composite section	
h_f	= compression flange thickness	[STD], [LRFD]
I	= moment of inertia about the centroid of the non-composite precast beam	[STD], [LRFD]
I_c	= moment of inertia for the composite section	
IM	= vehicular dynamic load allowance	[STD], [LRFD]
k	= factor used in calculation of average stress in pretensioning steel for Strength Limit State	
K	= wobble friction coefficient	[STD], [LRFD]
L	= overall beam length or design span	
L	= live load	[STD]
LL	= live load	[LRFD]
M_b	= unfactored bending moment due to barrier weight	
M_{cr}	= moment causing flexural cracking at section due to externally applied loads	[STD], [LRFD]
M_d	= bending moment at section due to unfactored dead load	
$M_{d/nc}$	= non-composite dead load moment	[STD]
M_g	= unfactored bending moment due to beam self-weight	
M_{LL+I}	= unfactored bending moment due to live load + impact	
M_n	= nominal flexural resistance	[STD], [LRFD]
M_r	= factored flexural resistance of a section in bending	[LRFD]
M_S	= unfactored bending moment due to deck slab and haunch weights	

$M_{secondary}$	= secondary bending moment due to post-tensioning	
M_{Total}	= total unfactored bending moment due to post-tensioning	
M_u	= factored moment at section	[STD], [LRFD]
M_{ws}	= unfactored bending moment due to wearing surface	
n	= modular ratio of elasticity	[STD], [LRFD]
P	= prestress force	
P_i	= total pretensioning force immediately after transfer	
P_{pe}	= total pretensioning force after all losses	
P_{PT}	= total post-tensioning force after all losses	
s	= spacing of shear reinforcement in direction parallel to the longitudinal reinforcement, in.	[STD], [LRFD]
S_b	= non-composite section modulus for the extreme bottom fiber of section where the tensile stress is caused by externally applied loads	[STD]
S_{bc}	= composite section modulus for the extreme bottom fiber of the precast beam	
S_t	= section modulus for extreme top fiber of the non-composite precast beam	
S_{tc}	= composite section modulus for top fiber of the slab	
S_{tg}	= composite section modulus for the top fiber of the precast beam	
t_s	= depth of concrete slab	[LRFD]
V_c	= nominal shear resistance provided by tensile stresses in the concrete	[STD], [LRFD]
V_d	= shear force at section due to unfactored dead load	[LRFD]
V_{LL+I}	= unfactored shear force due to live load plus impact	
V_s	= shear resistance provided by shear reinforcement	[STD], [LRFD]
$V_{secondary}$	= secondary shear force due to post-tensioning	
V_u	= factored shear force at section	[STD], [LRFD]
w_c	= unit weight of concrete	[STD], [LRFD]
W_{eq}	= equivalent load for post-tensioning	
x	= distance from the support to the section under question	
x	= length influenced by anchor set	
X	= distance from load to point of support	[LRFD]
y_b	= distance of the centroid to the extreme bottom fiber of the non-composite precast beam	
y_{bc}	= distance of the centroid of the composite section to the extreme bottom fiber of the precast beam	
y_{bs}	= distance from the center of gravity of strands to the bottom fiber of the beam	
y_t	= distance from centroid to the extreme top fiber of the non-composite precast beam	
y_{tc}	= distance of the centroid of the composite section to the top fiber of the slab	
y_{tg}	= distance of the centroid of the composite section to the top fiber of the precast beam	

α	= sum of the absolute values of angular change of prestressing steel path from jacking end	
β	= factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension	[STD]
β_1	= ratio of depth of equivalent compression zone to depth of actual compression zone	[STD], [LRFD]
Δf_{pA}	= loss in prestressing steel stress due to anchor set	[STD]
Δf_{pa}	= prestress loss at point, a	
Δf_{pES}	= loss in prestressing steel stress due to elastic shortening	[STD], [LRFD]
Δf_{pF}	= loss in prestressing steel due to friction	[STD]
Δf_{pT}	= total loss in prestressing steel stress	[STD]
ΔL	= anchor set	
θ	= angle of inclination of diagonal compressive stresses	[STD]
μ	= coefficient of friction	[STD], [LRFD]
ϕ	= resistance factor	[STD], [LRFD]

EXTENDING SPANS

11.1 INTRODUCTION

Precast, prestressed concrete beams have been used widely for highway bridges throughout the United States and the world. The simplest and most economical application for precast concrete beam bridges is where full-span beams are used in the bridge. The full-span beams have most often been used as simple spans, although continuity has also been established between spans using a continuity diaphragm at interior piers and various methods to counter negative moments.

For simple span precast, prestressed concrete bridges using conventional materials, the maximum spans for each standard section type are shown in Appendix B. However, the excellent durability and structural performance, low maintenance, and low cost of bridges using precast, prestressed concrete beams have encouraged designers to find ways to use them for even longer spans.

A number of methods have been identified for extending the typical span ranges of prestressed concrete beams. These include the use of:

- high strength concrete
- increased strand size or strength
- modified section dimensions
  - widening the web
  - thickening or widening the top flange
  - thickening the bottom flange
  - increasing the section depth (haunch) at interior piers
  - casting the deck with the girder (deck bulb tee)
- lightweight concrete
- post-tensioning
- continuity
- use of pier tables

Of these methods, the use of high strength concrete, lightweight aggregate concrete (both of which are considered to be high performance concrete) and continuity are discussed in this chapter.

As designers attempt to use longer full-span beams, limitations on handling and transportation are encountered. Some of the limitations are imposed by the states regarding the size and weight of vehicles allowed on highways. Some states limit the maximum transportable length of a beam to 120 ft and the weight to 70 tons. Other states, including Pennsylvania, Washington, Nebraska and Florida, for example, have allowed precast beams with lengths up to 185 ft and weights of 100 tons to be shipped by truck. In other cases, the size of the erection equipment may be limited, either by availability to the contractor or by access to the site. There are sites where access will not allow long beams to reach the bridge.

When any of these limitations preclude the use of full-span beams, shorter beam segments can be produced and shipped. These beam segments are then spliced together at or near the jobsite or in their final location. The splices are located in the spans, away from the piers. The beam segments are typically post-tensioned for the full length of the bridge unit, which can be either a simple span or a multiple span continuous unit.

While the introduction of splices and post-tensioning increases the complexity of the construction and adds cost, precast bridges of this type have been found to be very cost competitive with other systems and materials. The longest span in a modern spliced beam bridge in the United States is currently the 320 ft long channel span in a three-span bridge near Moore Haven, Fla. This bridge was originally designed using a steel plate girder, but was redesigned at the request of the contractor to reduce project costs, which clearly demonstrates the comparative economy of the spliced concrete beam system.

Since splicing is an important tool for extending span ranges, and since it also incorporates some additional design issues not discussed elsewhere in this Manual, a significant portion of this chapter is devoted to providing designers with information on this type of bridge. Design theory, post-tensioning analysis and details, segment-to-segment joint details and examples of recently constructed spliced-beam bridges are given. The chapter includes examples intended to help designers understand the various design criteria and to develop preliminary superstructure designs.

A significant additional resource for the design of precast prestressed concrete beams for extended spans is the research project performed as part of the National Cooperative Highway Research Program (NCHRP) titled "Extending Span Ranges of Precast Prestressed Concrete Girders" by Castrodale and White (2004). The final report contains considerable information on methods for extending span ranges, as well as an extended discussion of issues related to the design of spliced beam bridges, including three design examples. The report also identifies nearly 250 spliced beam bridges constructed in the United States and Canada.

11.2 HIGH PERFORMANCE CONCRETE

High performance concrete (HPC) has been defined in a number of different ways, but in general, it includes modifications to concrete that improve the efficiency, durability or structural capability of members over that achieved using conventional concrete. A number of HPC tools can be used to extend the spans of precast, prestressed concrete beams. In this chapter, the discussion will be limited to the use of high strength and lightweight concrete.

11.2.1 High Strength Concrete

11.2.1.1 Benefits

High strength concrete (HSC) has several advantages over conventional strength concrete. These include increased:

- compressive strength
- modulus of elasticity
- tensile strength

In addition, high strength concrete is nearly always enhanced by these other benefits:

- a smaller creep coefficient
- less shrinkage strain
- lower permeability
- improved durability

Specifically, beams made with high strength concrete exhibit the following structural benefits:

1. Permit the use of high levels of prestress and therefore a greater capacity to carry gravity loads. This, in turn, allows the use of:
   - fewer beam lines for the same width of bridge
   - longer spans for the same beam depth and spacing
   - shallower beams for a given span
2. For the same level of initial prestress, reduced axial shortening and short-term and long-term deflections.
3. For the same level of initial prestress, reduced creep and shrinkage result in lower prestress losses, which can be beneficial for reducing the required number of strands.
4. Higher tensile strength results in a slight reduction in the required prestressing force if the tensile stress limit controls the design.
5. Strand transfer and development lengths are reduced.

11.2.1.2 Costs

The benefits of high strength concrete are not attained without cost implications. For example, when high concrete compressive strength is used to increase member capacity, a higher prestress force is required. This in turn offsets the effect of a lower creep coefficient and results in larger losses and deflections. Furthermore, very long and shallow members require an investigation of live load deflections, as well as constructability and stability during design.

High strength concrete is more expensive per cubic yard than conventional concrete. In some areas, increasing concrete strength from 7,000 psi to 14,000 psi could double the cost from $70 to $140 per cubic yard. However, a modest increase to 10,000 psi might add only $10 to $20 per cubic yard. Concrete mixes with strengths higher than 10,000 psi may be difficult to attain with consistency and require large quantities of admixtures. It is difficult to generalize about costs and capabilities. The materials, experience and equipment may be more a regional issue for the industry. Generally, the technology to produce high strength precast concrete is advancing very rapidly.

Other consequential costs that should be taken into consideration include:

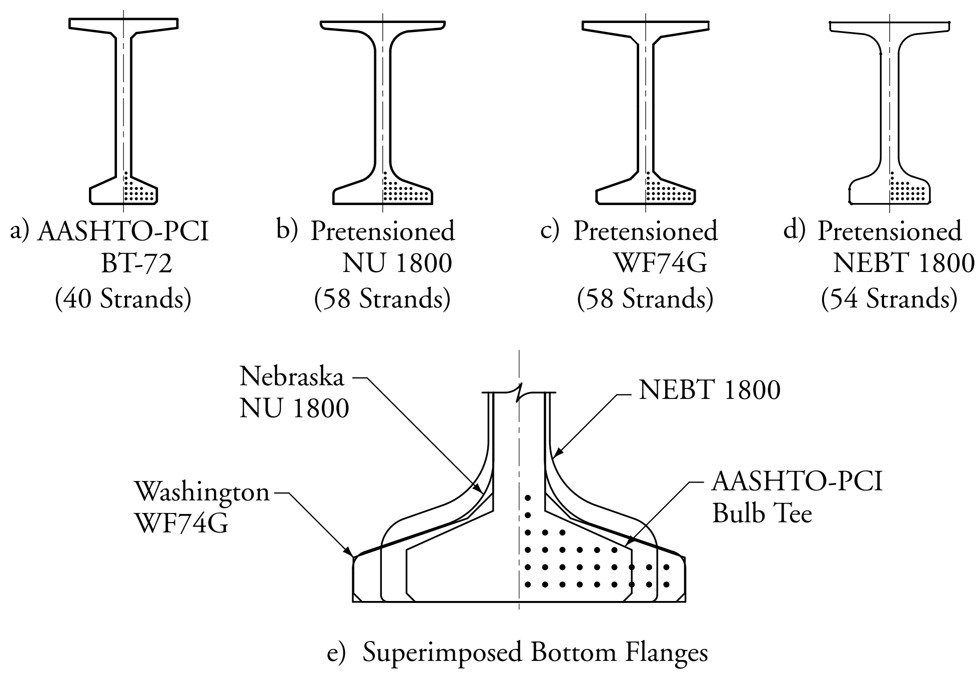
- Achieving high transfer strengths could extended the production cycle to more than one day
- High prestress forces may exceed available bed capacity for some plants
- Larger capacity equipment to handle, transport and erect longer and heavier beams may be required than is normally available

Costs associated with the production of high strength concrete should be weighed against the reduction in volume and the net result may well be both initial savings as well as long-term durability enhancements. Producers near the project (and their state and regional associations) should be consulted about these issues.

11.2.1.3
Effects of Section Geometry and Strand Size

High strength concrete increases the effectiveness of precast, prestressed concrete beams. High concrete strength at prestress transfer permits the application of a larger pretension force, which in turn increases the member's capacity to resist design loads. The number of strands that can be used is limited by the size of the bottom flange. The primary reason that the NU I-Girders, the Washington Super Girders and the New England Bulb-Tee beams have higher span capacities than the AASHTO-PCI Bulb Tee is that they all have significantly larger bottom flanges as shown in **Figure 11.2.1.3-1**.

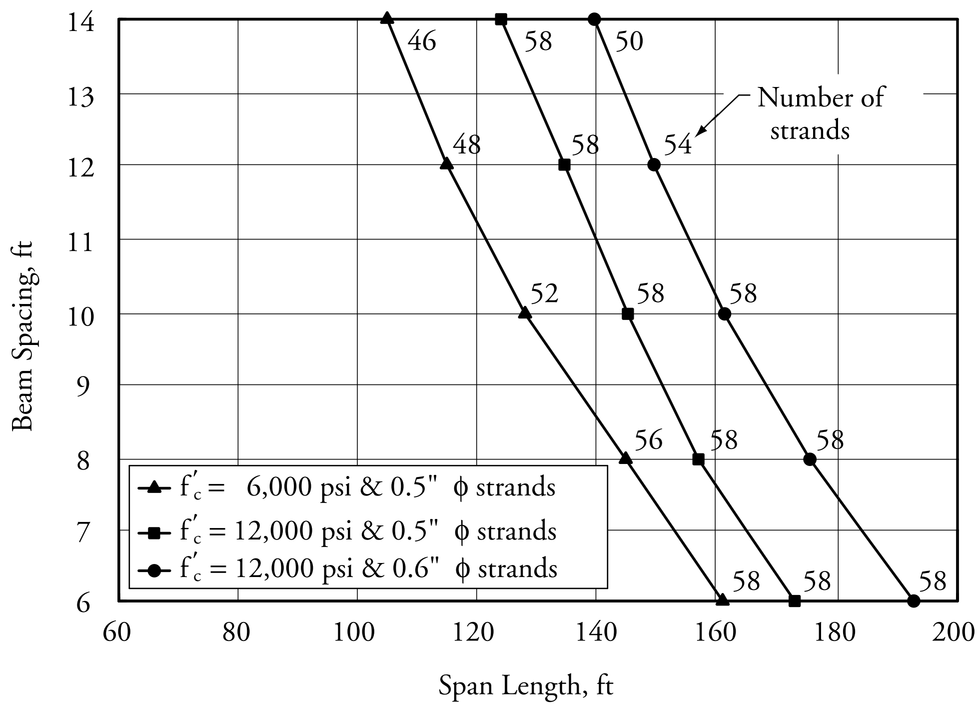
Figure 11.2.1.3-1
I-Beam Shapes with Large Bottom Flanges to Accommodate More Strand



Designers are rapidly implementing the use of 0.6-in.-dia strands. This will improve the efficiency of all beam shapes because each 0.6-in.-dia strand provides 40 percent more pretension force for only a 20 percent increase in diameter. The *Standard* and *LRFD Specifications* allow the same center-to-center spacing for 0.6-in.-dia strand as for ½-in.-dia strand.

Figure 11.2.1.3-2 shows the maximum span of a NU2000 (78.7 in.-deep) beam.

Figure 11.2.1.3-2 Maximum Span of NU2000 Beam



The maximum span varies with the beam spacing and number of strands. The number of strands must increase to allow for a greater span length. Likewise, as the beam spacing increases, the number of strands must also increase. An investigation conducted by the Washington State Department of Transportation and the Pacific Northwest PCI shows that the maximum span of the W21MG beam (now referred to as the W83G beam), with 7,500-psi transfer strength, using 0.6-in.-dia strands, is 180 ft.(Seguirant, 1998).

At a small beam spacing of about 6 to 8 feet, however, the potential for increased span length with high strength concrete may be limited by the number of strands that can be placed in the bottom flange. For the NU beam with 6,000 psi concrete and beam spacing of 6 ft, 58 strands are required to achieve the maximum span length of 161 ft. This is the maximum number of strands that can be placed in the bottom flange of the NU beam. If the concrete strength is increased to 12,000 psi, the maximum span will increase only 12 ft, about 7.5 percent greater than the original maximum span. However, when the beam spacing is increased to 14 ft, the number of strands can be increased from 46 for concrete with a design strength of 6,000 psi to 58 for 12,000-psi concrete with an increase in span from 105 ft to 124 ft.

If concrete strength and strand size are both increased, the span length can be extended further. The 12,000-psi concrete is still adequate to fully utilize the bottom flange by filling it with 58 strands. This confirms the work by Russell, et al (1997), who found that concrete with a compressive strength lower than 12,000 psi would be adequate when ½-in.-dia strands are used.

Based on these results, two conclusions can be made regarding effective utilization of beams with high strength concrete:

1. The effectiveness of HSC is largely dependent on the number of strands that the bottom flange can hold. The more strands contained in the bottom flange, the farther the beam can span and the greater the capacity to resist positive moment. It is recognized that designers do not always have a large number of choices of available beam sections. Nonetheless, a beam that provides for the greatest number of strands in the bottom flange is preferred when using HSC.
2. Allowable stresses are increased when using HSC. If these limiting stresses cannot be fully utilized with ½-in.-dia strands, then 0.6-in.-dia strands should be used. The tensile strength of 0.6-in.-dia strands is nearly 40 percent greater than the capacity of ½-in.-dia strands. Both the *Standard* and *LRFD Specifications* permit the use of 0.6-in. strands at the common 2-in. spacing. The use of 0.6-in.-dia strand is expected to increase in the future even with the use of conventional strength concrete due to economy in production.

11.2.1.4 Compressive Strength at Transfer

Higher concrete compressive strength at transfer allows a beam to contain more strands and increases the capability of the beam to resist design loads. To achieve the largest span for a given beam size, designers should use concrete with the compressive strength needed to resist the effect of the maximum number of strands that can be accommodated in the bottom flange. However, the availability of high compressive strength at transfer varies throughout the country. Strength at transfer should not be higher than required for the span being designed because strengths in excess of 5,500 to 6,500 psi may increase the required duration of the production cycle at the manufacturing plant. This would in turn increase the cost of the beams. Early compressive strength is influenced by local materials and sometimes production facilities and regional practices. Producers should be consulted about available concrete strengths before beginning design.

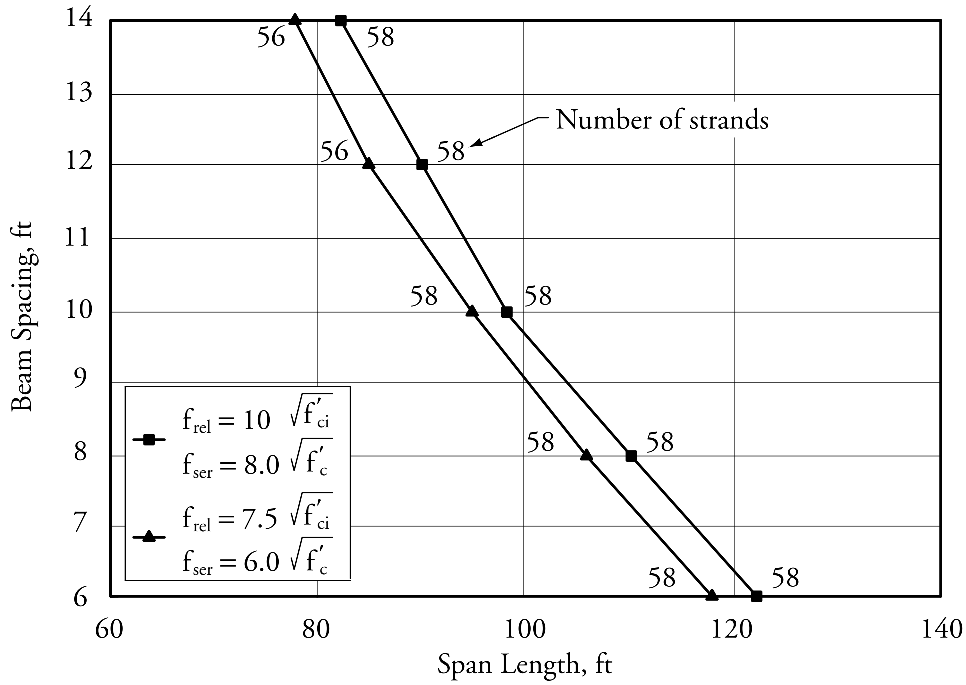
11.2.1.5 Reduction of Pretension Force by Post-Tensioning

When it is necessary to reduce compressive strength at transfer or when there are limitations on the capacity of the pretensioning bed, the total amount of prestress can be provided in two stages. The first is pretensioning during production followed by post-tensioning after production. Compared to using only pretensioning during production, combining pre- and post-tensioning generally increases the cost of the beam but has been used very effectively to solve strength and plant constraints.

11.2.1.6 Tensile Stress Limit at Service Limit State

Numerous test results on HSC have shown a modulus of rupture as high as $12\sqrt{f'_c}$ compared to $7.5\sqrt{f'_c}$ indicated for conventional concrete (ACI Committee 363, 1992). Since the limiting tensile stress is directly proportional to the modulus of rupture, some designers and researchers have suggested an increase in the tensile stress limit. As shown in **Figure 11.2.1.6-1**, the use of higher tensile stress limits has relatively small effect on the maximum achievable spans of prestressed concrete I-beams.

Figure 11.2.1.6-1
Variation of Maximum Span of NU1100 Beams with Spacing and Allowable Tensile Stress



11.2.1.7
Prestress Losses

Depending on specific aggregates, the general characteristics of HSC are reduced creep, reduced shrinkage strain, and increased modulus of elasticity. Consequently, prestress losses are lower for HSC compared to conventional concrete at a constant level of prestress. However, higher levels of prestress are generally used in HSC members. Therefore, the absolute value of loss may be comparable, or even higher compared to conventional strength concrete (Seguirant, 1998).

The *LRFD Specifications* provides two methods for estimating loss of prestress: the refined method and the lump-sum method. The refined method accounts for level of prestress but not for reduced creep and shrinkage characteristics. Thus, its application could significantly overestimate prestress losses in HSC. The lump-sum method does not account for either the prestress level or for variation in creep and shrinkage. Therefore, it contains two counteracting incorrect assumptions but could result in more reasonable values for losses than those of the refined method.

A study for the National Cooperative Highway Research Program (NCHRP) by Tadros, et al (2002), resulted in recommendations for the determination of certain concrete properties in HSC (modulus of elasticity, creep and shrinkage), as well as a proposal for both a new approximate and a new detailed method of prestress loss estimation. If these recommendations are adopted by AASHTO, they would result in more realistic, reduced values of prestress losses for HSC applications and comparable loss values resulting from the current provisions for conventional concrete strength. Until the *LRFD Specifications* is revised, it is recommended that the method described in Section 8.6 of this manual be used.

11.2.2
Lightweight Aggregate Concrete

Structural lightweight aggregate concrete has been used extensively to reduce the weight of precast members. The weight of a concrete beam accounts for about one-third of its total load, and increases in proportion as the span increases. Reducing member weight allows the beam to carry higher superimposed loads and to span farther. Structural lightweight aggregate (LWA) concrete bridges have been reported in the literature from the earliest days of the prestressed concrete industry and those applications continue. Useful publications on LWA concrete applications are provided by the Expanded Shale, Clay and Slate Institute (ESCSI) (website www.escsi.org). LWA concrete with a specified strength of 10,000 psi has reportedly been used in Norway and Canada (Meyer and Kahn, 2001). Research performed at Georgia Institute of Technology (Meyer and Kahn, 2002) includes a study of the advantages of lightweight, high strength concrete to 12,000 psi. The production and testing of full-size beams has verified the important design and long-term properties of the material. When lighter weight is combined with higher strength and improved durability, the benefits are compounded.

11.3 CONTINUITY

11.3.1 Introduction

Precast, prestressed concrete beams are most often placed on their supports as simple-span beams. In this configuration, the beams support self-weight and the weight of deck formwork. Generally, the weight of the deck slab is also supported by the simple span. If the details used allow for rotation of beam-ends, further loads applied to the bridge may also be applied to the simple span.

Simple span systems have sometimes not performed well. Whether the deck slab is placed continuously over abutting ends at a pier or a joint is placed in the deck at this location, rotation of beam ends can result in significant deck cracking. The use of simple span systems can lead to leakage through the deck and deterioration of beam-ends, bearings and the substructure. This is especially critical in cold weather regions where deicing chemicals are used.

However, when beams are made continuous, structural efficiency and long-term performance are significantly improved.

Two methods have been used to create continuity in precast, prestressed concrete beam bridges:

- Deck reinforcement
- Post-tensioning

Two additional methods have been introduced recently for establishing continuity. They are accomplished prior to placing the deck and have shown promising results:

- Coupling beams with high strength rods
- Coupling beams with prestressing strands

The use of post-tensioning and the latter two methods provide the structural benefit of making the beam continuous to resist the deck weight – a considerable portion of the total load. This significantly improves the structural performance of the bridge.

Discussion of the features of each of the four methods follows.

11.3.2 Method 1 – Conventional Deck Reinforcement

Continuity can be established by casting abutting beam-ends on the pier into cast-in-place concrete diaphragms. Reinforcement is placed in the cast-in-place deck to resist the negative design moments that develop. Section 3.2.3.2.2 provides more details of this method. Design considerations and calculations are shown in Design Examples 9.5 and 9.6.

The method has been used very successfully in a number of states beginning as early as the 1950s. It is the simplest of the existing methods because it does not require additional equipment or specialized labor to make the connections between beams to establish continuity. The beam acts as a simple span under its own weight and the weight of the deck slab but as a continuous beam for other dead loads and the live load. Since the deck is mildly reinforced and not pretensioned, tensile stresses in the deck are not usually checked.

11.3.3 Method 2 – Post-Tensioning

This method is somewhat more expensive than the previous method per unit volume of beam concrete. It generally requires full-length post-tensioning of the bridge beams. The beam web must be wider than 6 in. that is common in many pretensioned beams. It also requires enlargement of the webs at the ends of some beams (end blocks) to accommodate post-tensioning anchorage hardware, or special anchorage details in the back wall of the abutment. A specialized contractor may be required to perform the post-tensioning and grouting operations.

However, significant advantages of this method are the ability to:

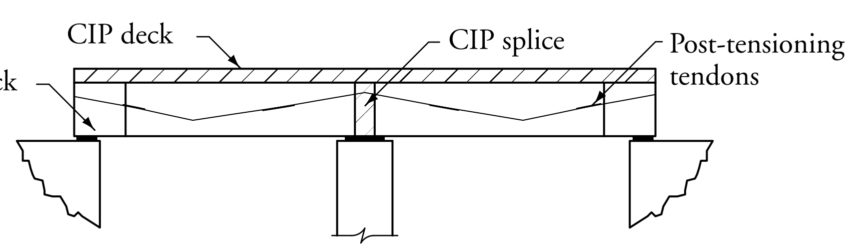
- splice segments into longer spans
- create efficient, multiple-span continuous bridges
- pre-compress the deck in the negative moment regions to virtually eliminate transverse surface cracking in the deck at piers
- improve structural efficiency by having a continuous beam for the deck weight and all subsequent loads
- have post-tensioning resist part of the self-weight of the beam
- use plant pretensioning only to counteract the weight of the beam and for handling stresses. This relatively small prestress results in small cambers and minimizes the need for high strength concrete at transfer.

For these reasons, much of the remainder of this chapter is devoted to the use, the analysis and design of post-tensioning for extending the spans of precast concrete beams.

In general, the construction of a post-tensioned beam bridge proceeds in the following way: The beams or beam segments are erected first, the post-tensioning ducts are spliced and then the beam splices or diaphragms are formed, cast and cured. Some or all of the post-tensioning tendons may then be installed and tensioned. The cast-in-place composite deck is cast. The remainder of the post-tensioning tendons are installed and tensioned.

Post-tensioning may be applied in one or more stages. If an appropriate level of prestressing is applied by first stage post-tensioning before the deck is cast, the beams will be continuous for the deck weight and construction loads as shown in **Figure 11.3.3-1**.

Figure 11.3.3-1
Full-Length Post-Tensioned Beam Bridge



First-stage post-tensioning must be large enough to control concrete stresses throughout the continuous member for the loads applied before the next post-tensioning stage. If a second post-tensioning stage is used, it is usually applied after the deck has cured and before superimposed dead loads are applied. Issues associated with applying post-tensioning after the deck has been placed are discussed in Section 11.4.9.

In cases where all post-tensioning is applied prior to placement of the deck, tensile stresses in the deck are not usually checked.

In cases where the deck is pretensioned by the application of post-tensioning, tensile stresses in the deck may be checked. If tensile stresses in the deck in negative moment regions exceed design requirements, one of the following could be considered:

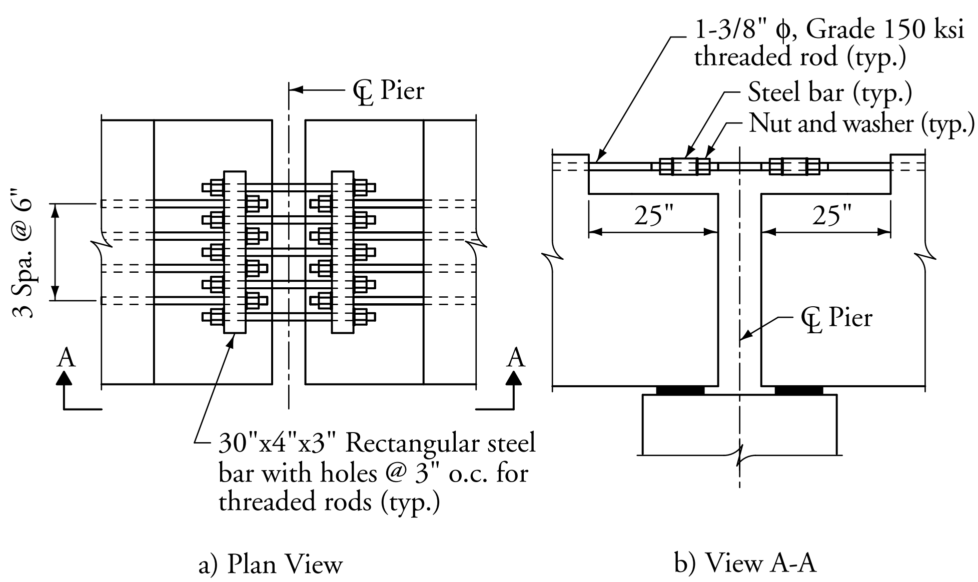
- consider the deck partially prestressed at this section. This condition would be superior to other continuous beam systems where the deck has no prestressing and is expected to crack under service load.
- increase post-tensioning to bring deck concrete stresses within limits
- increase the specified concrete strength of the deck

11.3.4 Method 3 – Coupled High-Strength Rods

In this method, nonprestressed, high strength threaded rods are extended from the top of the beam and coupled over the piers to provide resistance to negative moments from the weight of the deck slab. Conventional longitudinal reinforcement as described in Section 11.3.2, Method 1, is placed in the deck in the negative moment to resist the additional negative moments due to superimposed dead and live loads. Therefore, this method provides continuity conditions for deck weight, superimposed dead load and live load.

An earlier version of the connection shown in **Figure 11.3.4-1** has undergone full-scale testing (Ma, et al, 1998). It was shown to be structurally effective and simple to construct. The detail has been adopted by the Nebraska Department of Roads. A similar detail has been used on a four-span, Florida Department of Transportation double tee bridge on U.S. 41 over the Imperial River at Bonita Springs, Florida.

Figure 11.3.4-1
Threaded-Rod Connection in Top Flange of I-Beam



Another application of the method was a successful value-engineering change to a project in Nebraska in 2002. The contractor redesigned the Clarks Bridge from a haunched plate girder system that varied from 4- to 6-ft deep, to a modified, 50-in.-deep bulb tee. The project is shown nearing completion in **Figure 11.3.4-2**.

Figure 11.3.4-2 Clarks Bridge, over U.S. Highway 30 and the Union Pacific Railroad, Omaha, Nebraska

The bridge has four spans, 100 ft, 148 ft, 151 ft, and 128 ft. It has a composite deck thickness of 8 in. and a beam spacing of 10.75 ft to match the original steel beam design. For a precast I-beam system at this relatively wide spacing, the bridge has an impressive span-to-depth ratio of 31 = 151x12/(50+8). It also uses unique individual cast-in-place pier tables to support the beams. These tables become composite with cast-in-place extensions of the beams and later, with the bridge deck. **Figure 11.3.4-3a** shows a typical beam with high strength rods extended from the top flange. **Figure 11.3.4-3b** shows the beams on their pier tables with extended rods spliced between ends of the beams (Hennessey and Bexten, 2002).

Figure 11.3.4-3 Clarks Bridge, Omaha, Nebraska

a) Beam Showing High Strength Rods

b) Spliced Negative Moment Reinforcement

The coupled-rod splice combines the simplicity of adding reinforcement in the deck (Method 1) with some of the structural efficiency of post-tensioning (Method 2) where the beam may be made continuous for certain dead loads. A cost comparison of this method with Method 1 (Saleh, et al, 1995) indicates that savings in positive moment strands offsets the added cost of the threaded rods and hardware. Moreover, any need for positive moment reinforcement at the piers due to creep restraint is totally eliminated because the compression introduced into the bottom of the splice from the negative dead load moment is expected to counteract any possible positive moment generated from time-dependant effects. This method also increases the span capacity of a given beam size by about 10 percent.

There are no major disadvantages of this method compared to Method 1. There are two disadvantages of this method compared to Method 2:

- the deck in the negative moment region is not pre-compressed
- the beam-to-beam connection can only be made over the piers

11.3.5 Method 4 – Coupled Prestressing Strands

This method uses pretensioning strands, which are left extended at beam-ends. Strands are positioned so that after production they project from the ends of the beam near the top surface. After the pier diaphragm concrete is placed and hardened, but before the cast-in-place deck slab is placed, the strands are spliced and tensioned. This method has been utilized in the construction of a pedestrian/bicycle overpass in Lincoln, Nebraska, and is described in detail in Ficenec, et al (1993). The prestressing strand continuity method provides all the advantages of full-length post-tensioning but may cost less because it does not require large jacks, end blocks or the grouting associated with post-tensioning. This method is very efficient because it utilizes the existing pretensioning strands. However, the hardware and procedures for strand splicing and the procedures for the transfer of prestress in the plant are somewhat complex.

11.4 SPLICED-BEAM STRUCTURAL SYSTEMS

11.4.1 Introduction and Discussion

Spans greater than about 165 ft cannot usually be achieved economically with one-piece, precast, pretensioned concrete beams because of transportation and lifting restrictions. Owners tend to specify structural steel for these relatively large spans. However, many are becoming familiar with the efficiency and economy of spliced concrete beams. This system, which is described in detail in the remainder of this chapter, has been demonstrated in the past several decades to be cost-competitive with structural steel and has advantages with regard to durability and aesthetics (Abdel-Karim, 1991; Abdel-Karim and Tadros, 1992).

To provide simple spans, precast, pretensioned beam segments are sometimes post-tensioned together at or near the project site and lifted as one piece onto final supports. In most cases, however, the precast segments are erected on temporary towers to span the full distance between supports. When the segments are post-tensioned together, they lift off the temporary falsework and span between their permanent pier and abutment supports.

As discussed in Section 11.3.3, these spliced, continuous, post-tensioned beam bridges offer the advantage, over steel and pretensioned, precast concrete bridges, of having pre-compressed concrete in the deck at the negative moment regions. While competitive with steel, they require more design and construction steps, and are generally, but not always, more expensive than pretensioned-only concrete systems.

In situations that required these longer spans, precast concrete beams that are only pretensioned are usually not viable. Today, it is becoming more common for designers to think of segmental I-beam, post-tensioning solutions. Owner agencies should be encouraged to develop designs using this system as an alternative to steel plate beams for as many projects as possible. The experience in a number of states of offering a spliced concrete beam and a steel plate beam alternative has resulted in healthy competition and significant savings. Even when the steel alternate is the successful one, its bid price has been shown to be dramatically lower than before facing a concrete alternative. This has proven to more than justify the cost of preparing alternatives for contractor bidding. In instances when structural steel suppliers sacrifice profits or provide plate beams at a loss, continuing alternative designs have resulted in the concrete solution eventually being selected for construction and further rewarding the owner through lower long-term maintenance costs.

11.4.1.1 Combined Pretensioning and Post-Tensioning

The combination of plant pretensioning and subsequent post-tensioning offers an opportunity for structural optimization of simple spans made continuous, where the prestressing is introduced in stages corresponding to the introduction of design loads. The conventional system is to design a precast, pretensioned beam as simple span for self-weight and deck weight, and to make spans continuous through longitudinal deck reinforcement for superimposed dead loads and live loads. Alternatively, the same beam can be pretensioned to resist self-weight as a simple span and then spliced and post-tensioned to resist all other loads as a continuous beam. This optimization can result in the reduction of one or two beam lines or a reduction in structural depth while maintaining the same beam spacing. Several bridges have been built in Nebraska using the combination of two types of prestressing. In nearly all cases, combined prestressing was successfully bid against a structural steel alternate.

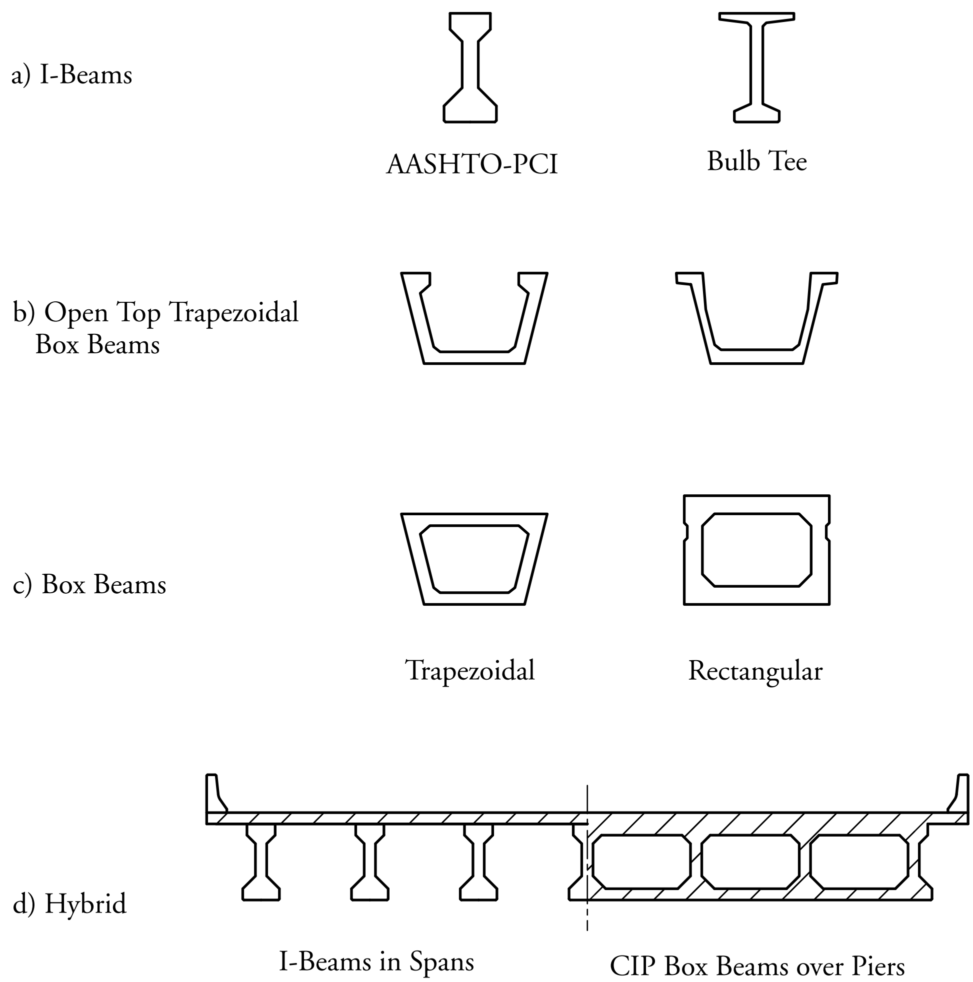
This type of system offers a practical introduction for agencies that have little or no experience with spliced-beam bridges or post-tensioning. Once the agency becomes familiar with the design and construction process, and the techniques are introduced in practice, applications can advance to longer span systems that require splices away from the permanent pier supports.

When pretensioning and post-tensioning are combined, additional losses will occur due to the interaction of different prestress forces.

11.4.2 Types of Beams

Shapes typically used in spliced-beam bridge applications are shown in **Figure 11.4.2-1**. Prestressed I-beams are the most popular, mainly due to their moderate self-weight, ease of fabrication and ready availability. For these reasons, much of the discussion that follows will focus on I-beams.

Figure 11.4.2-1
Shapes Used for Spliced-Beam Bridges



As the trend continues toward continuous superstructures, the need becomes evident for optimum I-beam sections. The I-beam geometry should perform well in both the positive and negative moment regions. This is clearly a different goal from shapes that were developed specifically for simple spans. Simple-span beams generally have inadequate sections for negative moment resistance and have webs too thin for post-tensioning ducts. A minimum web width to accommodate the post-tensioning tendon ducts and shear reinforcement is required, as discussed in Section 11.4.5.1.

Open-topped trapezoidal beams, or U-beams, are increasingly popular because of their aesthetic appeal. They are not suitable for pier segments where the non-composite beam is required to resist significant negative moments.

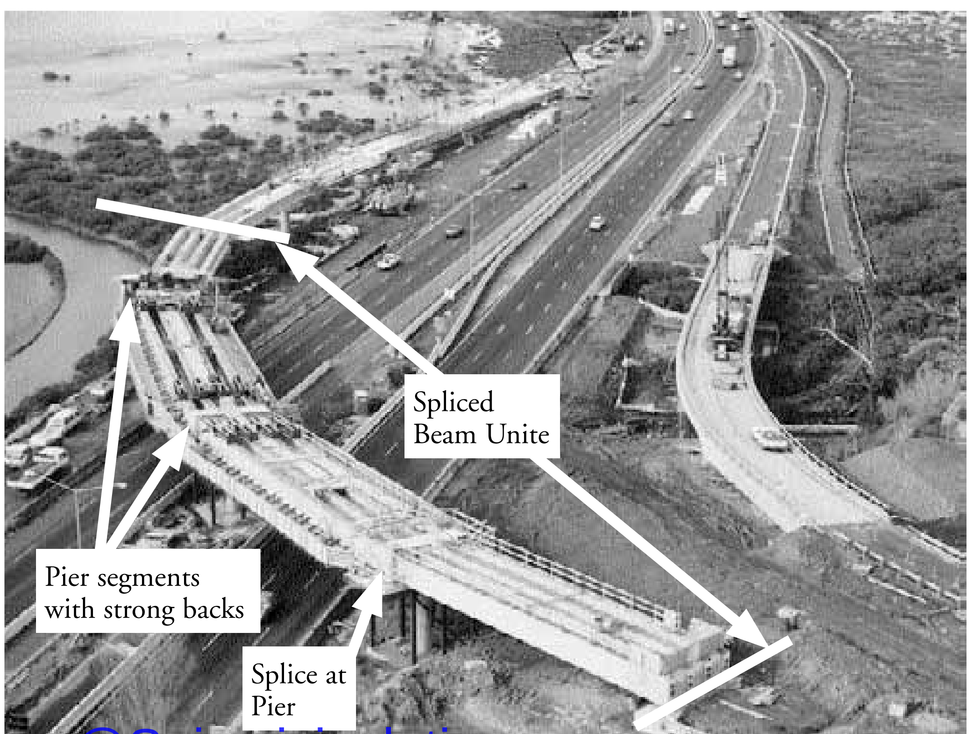
Figure 11.4.2-1d depicts a unique solution, which uses a hybrid combination of precast and cast-in-place concrete. Precast I-beams achieve a slender, light-looking mid-span element and are combined with cast-in-place concrete box beams at the piers where compressive forces caused by negative moments require a large bottom flange. While this solution has the benefit of improved section properties to resist negative moments at the interior piers, construction is more complex and lengthy than for more conventional precast construction. However, where structure depth is severly restricted, a section like this has proven to be an economical solution for several bridges.

11.4.3 Span Arrangements and Splice Location

By considering spliced beams, the designer has more flexibility to select the most advantageous span lengths, beam depths, number and locations of piers, segment lengths for handling, hauling and construction, and splice locations. As discussed in Section 11.3.3, a commonly used splicing technique is to post-tension a series of beams that are simply supported on piers or abutments. This achieves continuity for deck weight and superimposed loads. In addition to the enhanced structural efficiency of this system, post-tensioning can be used to assure that the deck is stressed below its cracking limit, which improves durability considerably.

Another feature of spliced beams is the ability to adapt to horizontally-curved alignments. By casting the beam segments in appropriately short lengths and providing the necessary transverse diaphragms, spliced beams may be chorded along a curved alignment. This is shown clearly in **Figure 11.4.3-1** that shows the Rosebank-Patiki Interchange in New Zealand with a 492-ft radius.

Figure 11.4.3-1
The Rosebank-Patiki Interchange, New Zealand

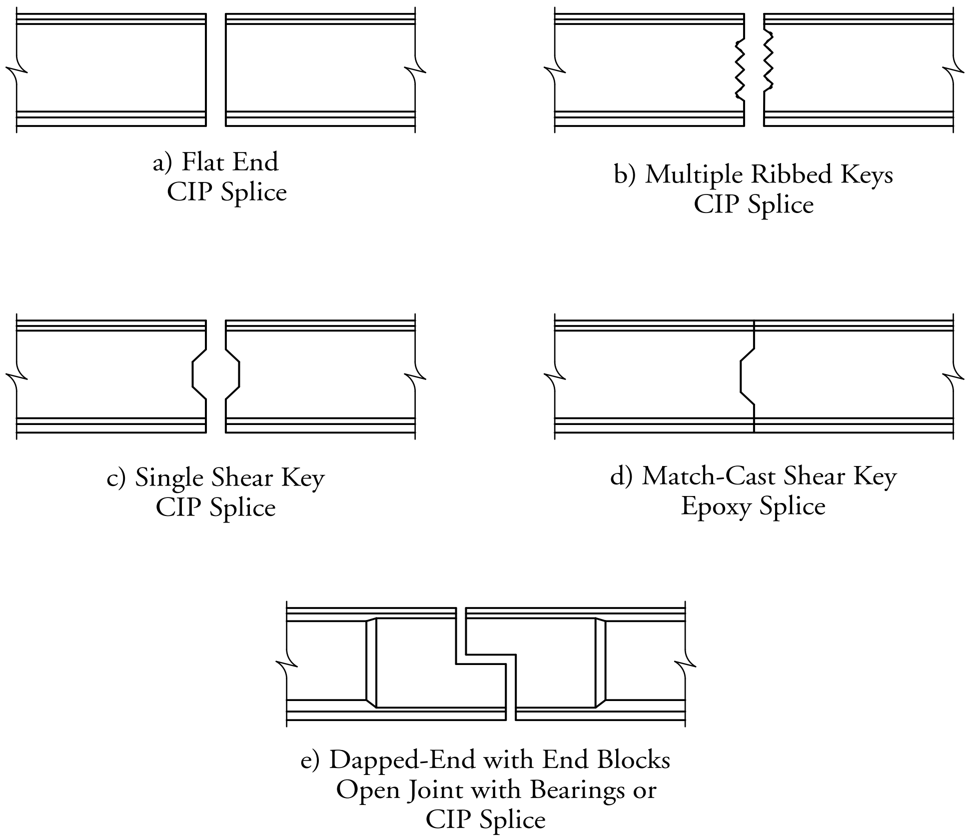


The chorded solution results in an efficient framing system while enhancing aesthetics. More details for chorded curved bridges are given in Chapter 12.

11.4.4 Details at Beam Splices

A wide variety of joint details have been used for splicing between beams. **Figure 11.4.4-1** shows some of the beam splice configurations used for I-beams.

Figure 11.4.4-1
I-Beam Splice Configurations



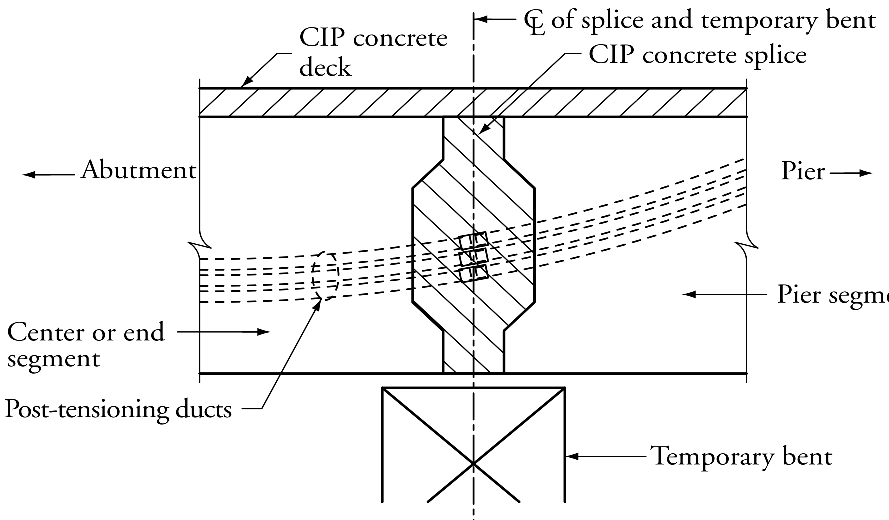
Most precast concrete beam splices are cast-in-place as shown in **Figure 11.4.4-1a,b** and **c**. Cast-in-place splices give the designer more construction tolerances. These details use a gap width of from 6 to 18 or even 24 in. The space is filled with high-early-strength concrete. Detail **a** is not recommended, even when the end of the beam is roughened by sandblasting or other means, because the high vertical interface shear generally requires a more positive shear key system. Detail **d** is an epoxy-coated, match-cast joint. This detail is discouraged because of the difficulty in adequately matching two pretensioned beam-ends, especially when the beams are of different lengths and with different pretensioning levels. Detail **e** is used with continuous post-tensioning but is sometimes used when the designer desires to have an expansion joint in the bridge. For an expansion joint, the post-tensioning tendons are terminated at the joint. While this detail has been used very successfully for a number of bridges, it has not been used for most recent structures.

With proper mix designs and proportions, the required strength and quality of jobsite concrete can be achieved. Three-day concrete strengths in the range of 5,000 psi can be achieved. It should be noted that more jobsite labor is needed for cast-in-place splices than for other splicing techniques, such as match-cast splicing.

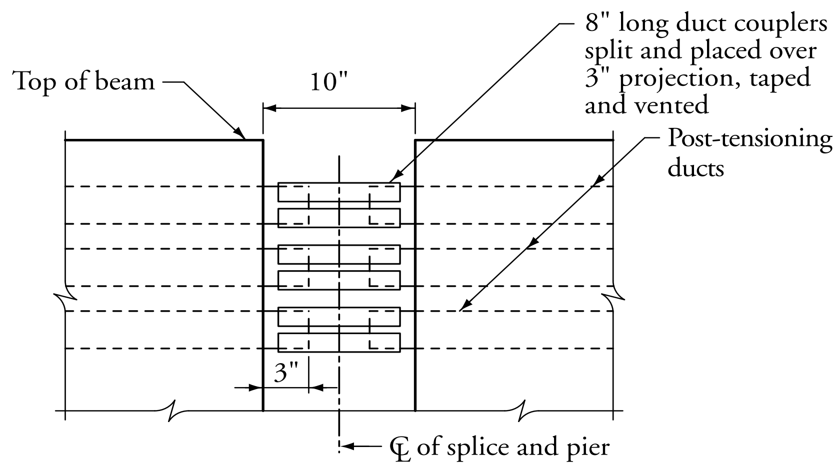
11.4.4.1 Cast-In-Place Post-Tensioned Splice

Cast-in-place, post-tensioned splices are most commonly used because of their simplicity and their ability to accommodate fabrication and construction tolerances. The segments are erected on falsework, the ducts are coupled and post-tensioning tendons installed. Concrete for the deck slab may be placed at the same time as the concrete for the splice, or the deck concrete may be placed after the splice and following the first stage of post-tensioning. **Figure 11.4.4.1-1** shows details of a cast-in-place, post-tensioned splice.

Figure 11.4.4.1-1 Cast-In-Place Post-Tensioned Splice



a) Post-Tensioned In-Span Splice with Shear Key



b) Duct Splice Detail at Pier

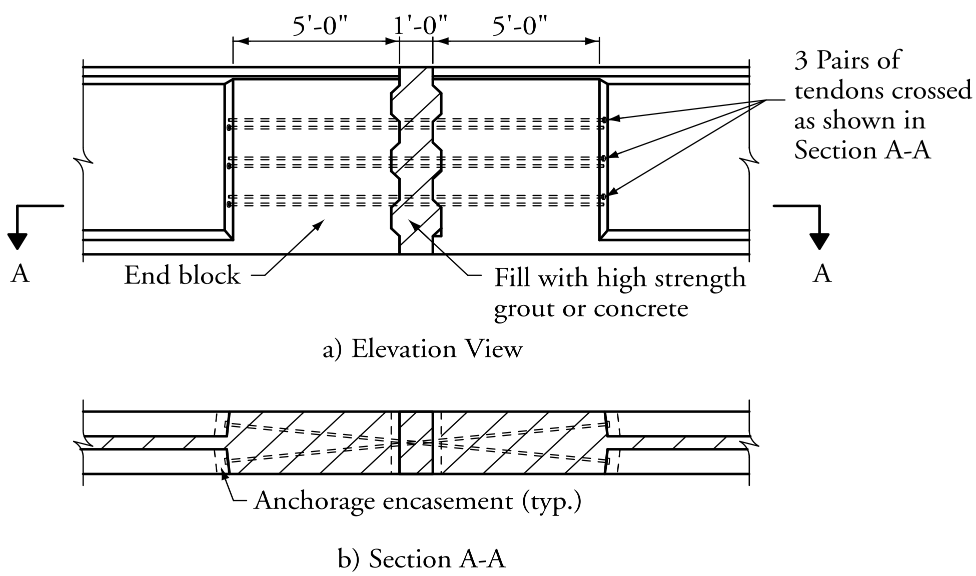
Figure 11.4.4.1-2 shows a typical splice during construction.

Figure 11.4.4.1-2 Cast-In-Place Post-Tensioned Splice

11.4.4.1.1 "Stitched" Splice

A schematic view of a "stitched" splice is shown in **Figure 11.4.4.1.1-1**.

Figure 11.4.4.1.1-1 "Stitched" Splice



A similar splice is shown during construction in **Figure 11.4.4.1.1-2**. The photo is of the Shelby Creek Bridge in Kentucky. The workman is tensioning a stitch tendon at the end of the widened end section. Grout ports are located near each tendon. This project used precast diaphragms that can be seen in the left foreground (See Caroland, et al, 1992).

Figure 11.4.4.1.1-2 Stitched Splice in the Shelby Creek Bridge, Kentucky

In this type of cast-in-place splice, the precast, pretensioned segments are post-tensioned across the splice using short tendons or threaded bars. It should be noted that precise alignment of the post-tensioning ducts is essential for the effectiveness of the post-tensioning. If proper alignment is not achieved, considerable frictional losses can result. Oversized ducts are often used to provide additional tolerance. In addition, because of the short length of the tendons, anchor seating losses could be unacceptably large. To reduce anchor seating losses, the use of a power wrench to tension threaded bars is recommended.

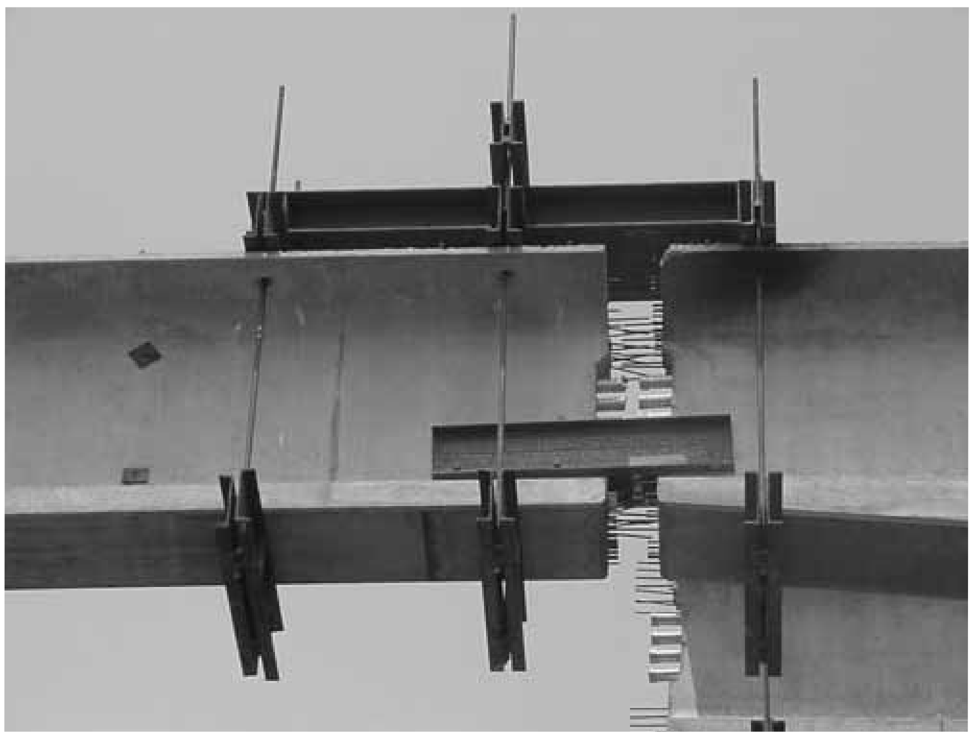
End blocks are required at the spliced ends of the beams in order to house the post-tensioning hardware and provide the “end zone” reinforcement to resist concentrated stresses due to the anchorages.

This type of splice may be suitable for long bridges where continuous tendon post-tensioning over the full length produces excessive friction losses.

11.4.4.1.2 Structural Steel Strong Back at Splice

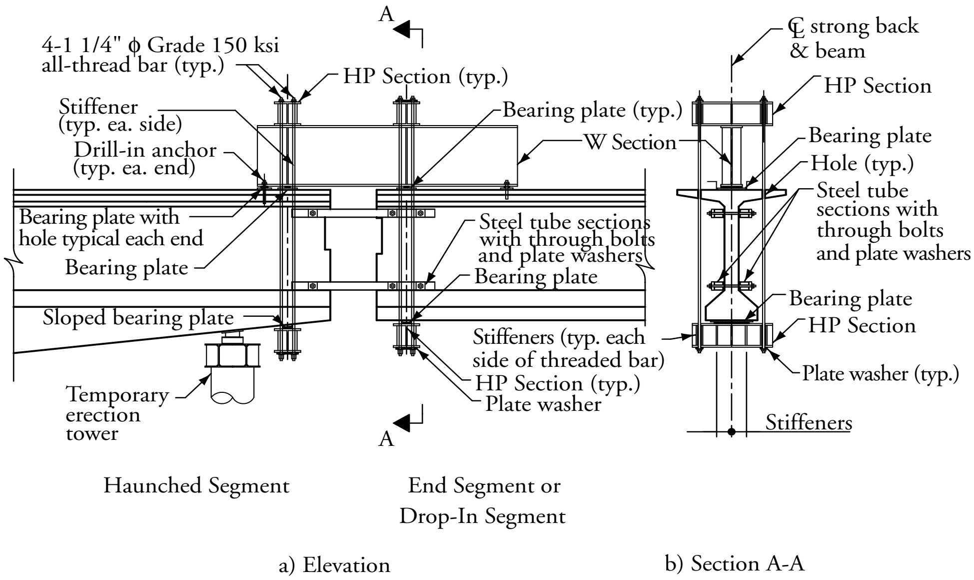
Some projects have used a removable structural steel “strong back” assembly in place of dapped ends or temporary support towers or falsework. **Figure 11.4.4.1.2-1** shows a strong back in use.

Figure. 11.4.4.1.2-1 Strong Back Used to Support a Drop-in Beam



Structural steel strong backs are rigidly connected to the top of the “drop-in” or end segments. They are used to hang these segments from the cantilevered pier segments until the splice joint is cast and the beams are post-tensioned. The strong back is attached to the drop-in beam with threaded-rod yokes. It bears on the top of the end of the cantilevered pier segment. Additional supports are used across the joint at the webs to maintain alignment and to prevent the tendency of the cantilevered beam to roll under the weight of the drop-in segment. As for all joint details, alignment of the ducts is important. The strong back is removed after the joint is cast and the segments are post-tensioned together. This device is especially recommended for situations where falsework is not economical. It requires detailed structural design and careful erection due to the large forces involved. A typical detail is shown in **Figure 11.4.4.1.2-2**.

Figure 11.4.4.1.2-2
Strong Back at Splice



11.4.4.1.3
Structural Steel Hanger at Splice

Another device used to avoid falsework towers is a unique adaptation of the "Cazaly Hanger" used for many years in the precast industry. It employs steel shapes that are embedded in both ends of the beams at a joint. The embedments in the pier segment support the hangers that have also been embedded in the drop-in segment. This solution requires even more control of fabrication and erection tolerances, alignment of ducts and care in construction. The details include "keepers" to assist with alignment and prevent dislodging the hangers from the seats. Additional alignment brackets are required on the webs to provide for stability as in the strong back details previously described. The use of this device is described by Caroland, et al (1992). **Figure 11.4.4.1.3-1a** shows the large rectangular steel bars extending from pier beams in storage. At the project site, a steel "shoe" will be fitted over and pinned to these bars as shown in **Figure 11.4.4.1.3-1b**. The drop-in beams will sit on the shoe and will in turn be pinned to it.

Figure 11.4.4.1.3-1
Hanger Supports, Shelby Creek Bridge, Kentucky

a) Beam Segment with Hanger Bars.

b) Beam Segments with Hanger Support Bars and Guide Shoes in Place.

11.4.4.2 Match-Cast Splice

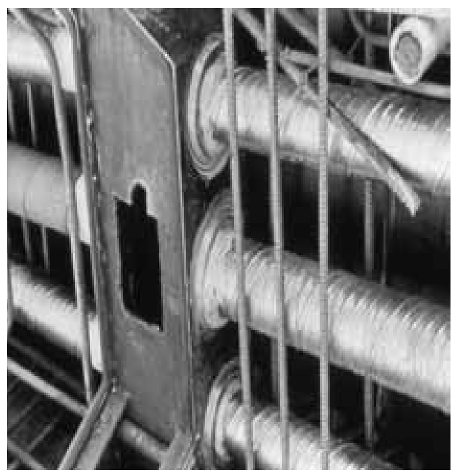
Match-cast segments were used in early applications of spliced beam bridges to eliminate the time and expense of cast-in-place joints. They are seldom used today. Match-castinging of I- or other beam sections has significant challenges. Beams that are pretensioned and cast on a long-line system, as most are, have continuous pretensioning strands that must be cut before these products are removed from the form. That operation is usually facilitated by the use of "headers" that form the ends of beams. The space between headers is used to cut the strands.

Emulated match-casting has been used where a machined steel header provides precisely formed concrete surfaces. The header is precision-made in a machine shop to exacting tolerances. Installed in the casting bed, it has stubs to accurately position the ends of the post-tensioning ducts and access ports to allow cutting the strands that have been threaded through it.

Figure 11.4.4.2-1 shows the header in the form. **Figure 11.4.4.2-2** shows the resulting match-cast joint on a temporary support tower being compressed through the use of external threaded rods. The mating surfaces of the beams have been coated with epoxy.

Figure. 11.4.4.2-1 Fabrication of a Match-Cast Joint

a) Machined "Match-Cast" Header in Form



b) Close-Up of Header

Figure. 11.4.4.2-2 Match-Cast Joint Being Compressed During Installation

Other necessary details to consider include:

- the coupling of post-tensioning ducts. This requires the forming of small recesses around the duct where it meets the header.
- sealing of the coupling zone against leakage of post-tensioning grout
- camber in the pretensioned beams that causes the ends to rotate. The rotation must be accounted for during fit-up of the beams at the joints as shown in **Figure 11.4.4.2-2**.

11.4.5
System Optimization

The main reason for segmenting and post-tensioning precast beams is to overcome the size and weight limitations for handling, shipping and erection. For example, in a bridge with two spans of 180 ft, the beams can be produced and shipped in 3, 120-ft segments. These pieces are one segment on the pier located between two end segments.

For very long spans, the critical location is generally at the pier due to large negative moments or large shear forces. The beams at the pier may need to be deepened to accommodate these forces. This will result in a considerably heavier pier segment and therefore, special planning and attention for production and transportation. Haunched pier beams are shown in storage in the manufacturing plant in **Figure 11.4.5-1**.

Figure 11.4.5-1
Haunched Pier Beams

Deepening the pier beam is but one choice available to the designer. This option should be carefully evaluated and compared to other options before a final decision is made on its use. Other options include:

- placement of a cast-in-place bottom slab
- gradual widening of a member toward the support
- using higher concrete strength
- adding compression reinforcement in the bottom flange
- the use of a hybrid system like that discussed in Section 11.4.2
- the use of a composite steel plate in the bottom of the bottom flange. See discussion in the design example, Section 11.8.11.2.

11.4.5.1
Minimum Web Width to Accommodate Post-Tensioning

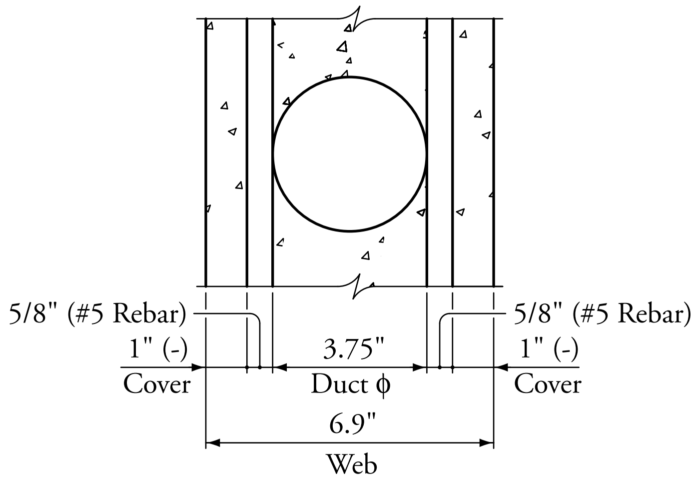
Web width should be as small as possible to optimize cross-section shape and minimize weight. Yet it should be large enough to accommodate a post-tensioning duct, auxiliary reinforcement and minimum cover for corrosion protection.

The requirements of the AASHTO Specifications changed with the introduction of the *LRFD Specifications*. LRFD Article 5.4.6.2 states that the duct cannot be larger than 40 percent of the web width. This requirement has been traditionally used to size webs for internal ducts in segmental bridge construction. Historically, this requirement has not existed and has not been observed for segmental I-beams.

When the NU I-Beam was developed in the early 1990s, a 6.9-in. (175-mm) web was selected to provide approximately 1-in. (25-mm) cover on each side plus two #5

(16-mm-dia) vertical bars plus a 3.75-in. (95-mm)-dia post-tensioning duct. The dimensions are shown in **Figure 11.4.5.1-1**.

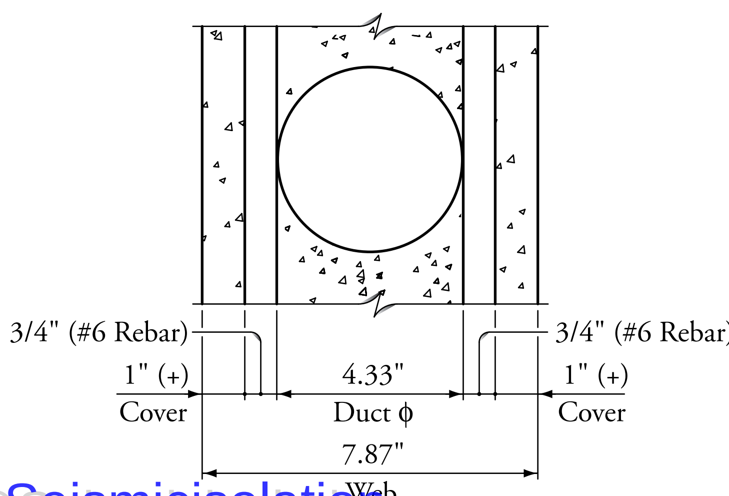
Figure 11.4.5.1-1
Web Configuration for NU I-Beam



Another requirement of LRFD Article 5.4.6.2, and a requirement of the Post-Tensioning Institute's *Specifications for Grouting* (2003) is that the inside duct area be at least 2.5 times the tendon cross-sectional area for the "pull through" method of tendon installation and 2.0 times the tendon cross-sectional area for the "push through" method of tendon installation. The NU I-Beam duct diameter satisfied the minimum requirement of 2.5 times the tendon cross-sectional area for the standard (15) 0.6-in.-dia tendons used in Nebraska. The corresponding minimum inside duct area is calculated as 2.5(15)(0.217) = 8.14 in.^{2}. This corresponds to a required inside diameter of 3.22 in. These values have been the standard practice in Nebraska, backed up by significant experimental research and actual bridge applications.

The Washington State Department of Transportation (WSDOT) chose a web width of 200 mm (7.87 in.) for their new series of beams (Seguirant, 1998). This was derived as shown in **Figure 11.4.5.1-2**. The 4.33-in. duct can accommodate commercially available post-tensioning systems of up to (19) 0.6-in.-dia strands per tendon, or (29) ½-in.-dia trands per tendon. The corresponding distance between the duct and the concrete surface of 1.77 in. is more than twice the maximum aggregate size of ¾-in. used in Washington. A number of other states and Canadian provinces have adopted similar practices with no reported problems.

Figure 11.4.5.1-2
Web Configuration, Washington State I-Beam



Eleven bridges were described in the PCI report on spliced girder bridges (1992). Of those bridges, two had web widths of 6 in., five had web widths of 7 in., two had web widths of 7.5 in., and two had web widths of 8 in. None of the bridges had a web width that was more than 8 in. Most of the segmental I-beam bridges built using post-tensioning over the past four decades have not met the limit of duct diameter and web width.

11.4.5.2 Haunched Pier Segments

In situations where it is not possible to avoid a splice joint in the span, and prismatic pier segments are not adequate, haunched pier segments can be used effectively. For these haunched segments to be most efficient, Girgis (2002) has shown that the haunch depth should be about 1.75 times the standard depth and the haunch length 20 percent of the span. Shallower depths or shorter lengths may have to be used, with less efficiency, to satisfy clearance criteria.

11.4.6 Design and Fabrication Details

To assure a satisfactory beam splice, proper design details must be used along with good workmanship and fabrication techniques. Wet-cast splice joints are the standard practice. The ends of the beams at splices should have formed shear keys, if required, similar to those shown in **Figure 11.4.4.1-1**. Ducts for post-tensioning should be made of semi-rigid galvanized metal, high density polyethylene (HDPE) or polypropylene (PP). They must be adequately supported within the beam during casting to maintain alignment and minimize friction losses.

11.4.7 Construction Methods and Techniques

11.4.7.1 Splicing and Shoring Considerations

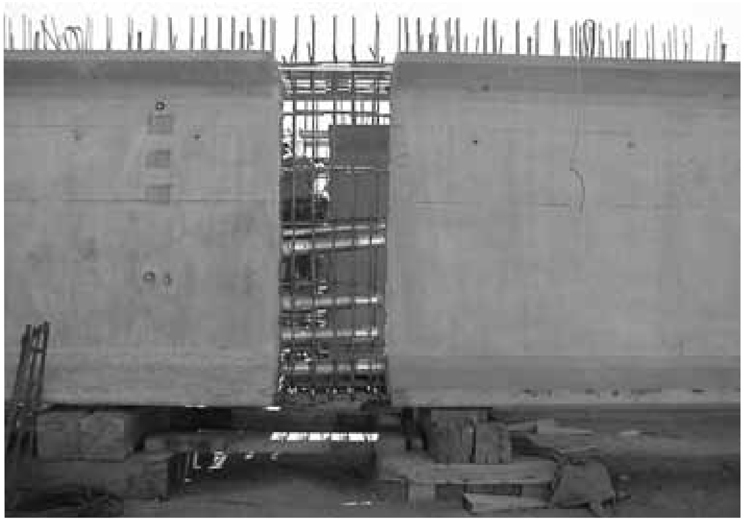
In a conventionally reinforced or post-tensioned splice away from the piers, it is usually necessary to support the ends of both beams on temporary supports. For bridges over inaccessible terrain or for water crossings, structural steel strong backs like those described in Section 11.4.4.1.2 are commonly used to support one beam from another instead of using towers. A common solution for a three-span channel crossing is to use towers for the side spans, where land is accessible during construction and strong backs in the center span over the water.

Important factors to consider when deciding whether to use falsework to support the segments in place or to splice the segments on the ground include:

- Space at the site is needed to position the segments, cast the joints and post-tension the beam.
- The assembled beam will be heavy and require larger cranes.
- Access for trucks and cranes.
- Towers may need to be excessively tall.

The principal advantage of splicing on the ground vs. in-place is the saving of the cost of falsework. On the ground, the splice is readily accessible by the workers and is close to material and equipment. The resulting improved labor productivity is an additional advantage. Splicing on the ground requires a large level area and temporary supports such as concrete pads. Segments need to be accurately aligned during splicing. **Figure 11.4.7.1-1** shows segments aligned, ducts spliced and reinforcement installed for splicing on the ground.

Figure 11.4.7.1-1 Segments Aligned for Splicing on the Ground



In-place splicing requires stiff falsework constructed with the capability to make adjustments for final elevations. **Figure 11.4.7.1-2** shows falsework supporting the ends of a pier beam and drop-in beam.

Figure 11.4.7.1-2 Segment Ends Supported on Falsework for Splicing

Precise vertical alignment of the beam segments is usually accomplished by the use of shims or screw jacks between the falsework and the segments. The major advantages of in-place splicing over splicing on the ground is that the beam segments are handled only once and require smaller lifting equipment. Additional assembly space at the site is not required.

Some or all of the falsework requirements can be eliminated through the use of strong backs or hangers that were described in Sections 11.4.4.1.2 and 11.4.4.1.3.

11.4.7.2 Construction Sequencing and Impact on Design

11.4.7.2.1 Single Spans

Single-span beams can be made-up of two or more segments. Using three segments as an example, as shown in **Figure 11.4.7.2.1-1**, the segments are installed on temporary towers and braced. Next, the splice joints are cast, tendons inserted in ducts and post-tensioning introduced, completing the assembly of the beam. Before the splice

Figure 11.4.7.2.1-1 Three Segments Supported on Falsework for Splicing

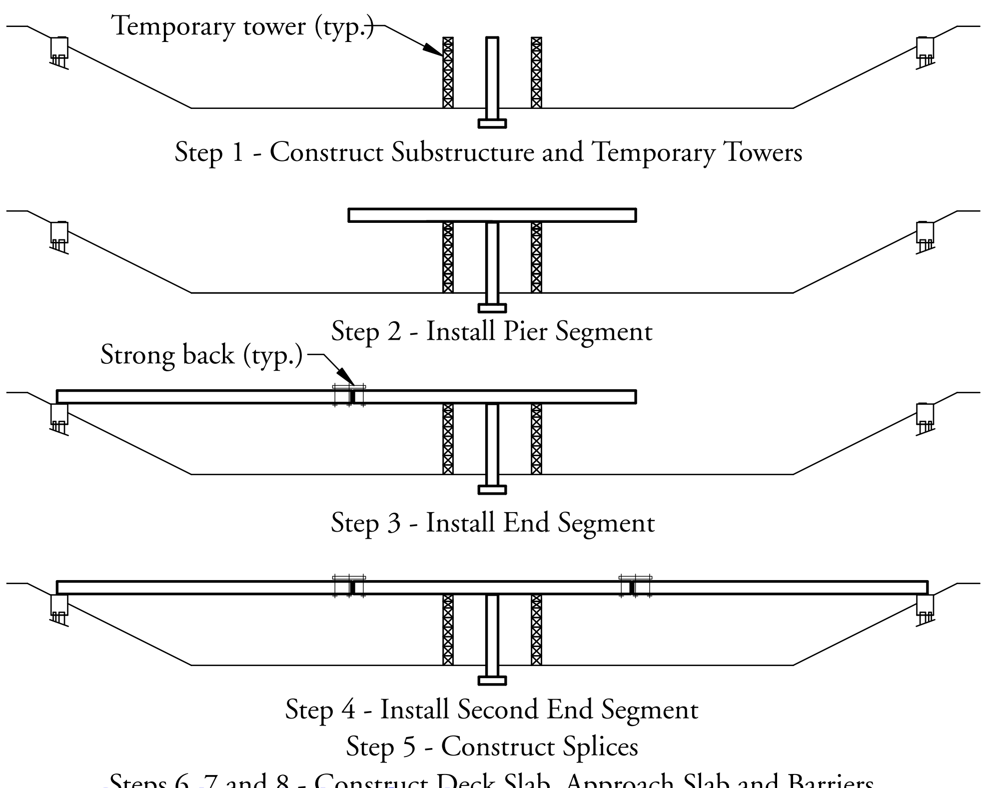
joints are cast, the end elevations of the segments need to be carefully positioned to allow for calculated long-term deflection. This also impacts the aesthetic appearance of the profile due to camber in the beam. These elevations also determine the amount of concrete needed for the haunches – the space between the top of the top flange and the bottom of the deck.

When the post-tensioning is applied, the full span, spliced beam cambers upwards and lifts up away from the temporary towers. The beam reactions that were being carried by the temporary towers are now carried by the spliced girder, so they must be considered in the analysis.

11.4.7.2.2 Multiple Spans

The same issues apply to multispan spliced beams erected on temporary towers. **Figure 11.4.7.2.2-1** shows the erection sequencing of a two-span overpass where traffic does not allow for temporary towers at the splice joint.

Figure 11.4.7.2.2-1 Two-Span Bridge Construction Sequence



The pier segment is installed on the pier and adjacent towers and a connection is made to the pier. Ideally, the pier connection should be one that allows for horizontal displacement of the beam at the time of post-tensioning. However, a fully integral joint can be utilized as long as the supports at the abutment allow for horizontal movement during tensioning of the post-tensioning tendons.

Placement of the first end segment, as shown in Step 3, **Figure 11.4.7.2.2-1**, creates moments in the pier segment and overturning effects on the tower and pier that must be evaluated. When an end segment is erected on the second span, the temporary overturning effect is eliminated. After the concrete in the splice has achieved the specified compressive strength and the post-tensioning tendons are stressed, the tower reactions must be applied as loads to the continuous two-span system as the beam lifts from the towers. The balance of construction sequencing is as described earlier.

11.4.8 Grouting of Post-Tensioning Ducts

Grouting of the ducts after tensioning is a critical step in the construction process. Good workmanship in grouting ensures proper performance of the structure and longevity. Inadequate attention to grouting can lead to problems that can compromise the integrity of the bridge.

Grouting of ducts should be performed as soon as possible after completion of the post-tensioning. Leaving the tendons ungrouted for an extended period of time could cause accumulation of moisture and chlorides in coastal areas, and the onset of corrosion. Moisture accumulation in the ducts may result in water lenses and ultimately in air pockets that could compromise the durability of the system.

Specific grouts and grouting techniques must be strictly observed in order to achieve high-quality construction. For example, the grout must be flowable and must be pumped at a pressure high enough to displace the air in the ducts yet low enough to avoid cracking or blow-outs of the concrete cover over the ducts. Air vent tubes must be placed at strategic locations to prevent air encapsulation.

The grout mix generally contains a shrinkage compensating or an expansive admixture. Current recommendations are that the grout be the commercially-packaged type manufactured for this purpose. The current edition of the *PTI Specifications for Grouting* (2003) should be followed.

Since proper grouting is such an important step in the construction process, it should be performed by experienced and well-qualified personnel. The American Segmental Bridge Institute (ASBI) has developed grouting training courses and a personnel certification program, which should be required. These will serve as important resources for good grouting practices.

11.4.9 Deck Removal Considerations

The removal of a bridge deck that has been in service has been a subject of concern among bridge owners who are interested in using spliced-beam and segmental box-beam bridges. In the snow belt areas of the United States, due to the large number of freeze-thaw cycles and the liberal use of deicing chemicals, it has been common to expect that a bridge deck will deteriorate to the point of needing replacement in 20 to 30 years.

When the deck is in place when the beams are post-tensioned, it becomes an integral part of the resistance system. Removal of the deck for replacement may temporarily overstress the bare beam. This would require an elaborate analysis and possibly a complicated temporary support scheme until the new deck is in place. However, if properly analyzed and the economics are verified, there is no reason this approach should not be considered. Computing power and available software make this a viable alternative.

Some states have avoided this issue by requiring designers to apply the post-tensioning in its entirety before the deck is placed (Nebraska, 2001). An additional benefit of this single-stage post-tensioning is simplified scheduling and coordination of construction. It eliminates multiple mobilizations for specialized subcontractors.

However, there are significant benefits to multistage post-tensioning in terms of structural efficiency, compared with single-stage post-tensioning. A convenient option is to divide the post-tensioning into thirds: two-thirds applied to the bare beam and one-third applied to the composite section. This is demonstrated in the example of Section 11.8. There are a number of benefits to this division. The deck is subject to compression that controls transverse cracking and extends its “first” life before it might need replacement. The ratio of initial post-tensioning on the composite system to total post-tensioning, 0.33, is partially offset by the gain in concrete strength of beam and time-dependent prestress loss which is approximately 20 percent. Therefore, the beams would not be appreciably more overstressed than when initially post-tensioned.

It may be desirable to apply all of the post-tensioning after the deck becomes part of the composite section. This case would be similar to the conditions of a segmental box beam system where the top flange is an integral part of the cross-section when the post-tensioning tendons are stressed. This solution in the United States and abroad has proven to provide a deck surface of excellent durability, perhaps not requiring any provisions for deck removal and replacement. The position of the American Segmental Bridge Institute (ASBI) is to provide a small additional thickness of sacrificial concrete in the original deck that can be removed and replaced with a wearing overlay if chloride diffusion measurements warrant such action. However, if the designer wishes to do so, the analysis of deck removal and replacement as part of the original design of the bridge is entirely possible.

Analysis for deck removal and replacement generally requires use of a continuous beam computer program (Tadros, et al, 1977). First, concrete stresses in the deck at time of anticipated deck removal are calculated with due consideration of time-dependent effects. Then, analysis is performed on the continuous precast member due to two sets of loads: the deck weight reversed, and the deck stress resultants reversed. The resulting stress increments in the beam are then added to the stresses just before deck removal and the net values checked against maximum stress limits.

Deck removal and replacement is a temporary loading case requiring temporary measures. If the concrete tensile stress exceeds the stress limit, then one should check if there is enough reinforcement to control cracking. If concrete compressive stress exceeds the $0.6f'_c$ specification limit, then a temporary support may be required. A more practical approach would be that the designer consider waiving that limit temporarily if the resistance strength moment is greater than the factored load, i.e., required strength moment.

11.4.10 Post-Tensioning Anchorages

Post-tensioning anchorages require the use of end blocks, which are thickened webs for a short length at the anchorages. End blocks can increase production costs of beams considerably due to the need for special forms and forming changes during production. I-beams with end blocks are also heavier to handle and transport, especially if the dimensions are selected according to the LRFD Article 5.10.9.1. It states that the end block length should be at least equal to the beam depth and its width at least equal to the smaller of the widths of the two flanges. End blocks are shown in **Figure 11.4.10-1**. The beam in the center shows the typical cross-section.

Figure 11.4.10-1
Beam End Block

It is possible to use the cast-in-place diaphragm at the abutment to house the anchorage located there. This practice is used in the Pacific Northwest because of the availability of contractors experienced with cast-in-place, post-tensioned concrete. For regions where post-tensioning is not prevalent, it is preferred to have the anchorage hardware placed by the precast concrete producer in order to control quality, reduce contractor risk and reduce construction time. Post-tensioning anchorage zones are discussed further in Section 11.7.

11.5 EXAMPLES OF SPLICED-BEAM BRIDGES

The PCI report on spliced-girder bridges (1992) contains information on some of the bridges that had been constructed during the preceding three decades. The following is a brief description of some additional notable bridges not contained in that report.

11.5.1 Eddyville-Cline Hill Section, Little Elk Creek Bridges 1 through 10, Corvallis-Newport Highway (US20), OR (2000)

In 2000, the Oregon Department of Transportation (ODOT) completed 6.4 miles of US20 Highway realignment between Corvallis and Newport. This two-lane section of highway is located 25 miles from the Pacific Ocean in the Coastal Mountain Range. The new alignment crosses the Little Elk Creek at 10 locations. The creek is environmentally sensitive and has a history of channel shifting during flood conditions; therefore, simple spans ranging from 99 ft to 184 ft were required to minimize stream impact and eliminate piers in the water.

ODOT selected a three-piece precast, post-tensioned, composite spliced-beam structure for four of the bridges that exceeded a span of 164 ft. **Figure 11.5.1-1** shows Bridge #7 upon completion.The roadway width is 46 ft and six lines of beams spaced at 7'-10" support an 8-in. deck. The precast beam is the ODOT Bulb-I 2440 (BI96). The top and bottom flanges are 24-in. wide and the web is 7.5-in. thick. End blocks were incorporated at the abutment ends to receive the multiple post-tensioning anchorages. Up to five tendons with 19, ½-in.-dia strands each were placed in the beam segments. The beams were supported at the abutments and on two temporary towers located at the third points. The post-tensioning tendons were spliced at the interior, 24-in.-wide closure pours. End diaphragms were cast followed by the deck placement. Post-tensioning was applied to complete the superstructure. Pretensioning was provided to control shipping stresses and to carry the non-composite loads. This method of construction allowed work to continue during critical in-water limitation periods and the project was completed one year ahead of schedule.

Figure 11.5.1-1 Bridge #7 over Little Elk Creek

11.5.2 Rock Cut Bridge, Stevens and Ferry Counties, WA (1997)

The Rock Cut Bridge is a single span of 190'-6" spliced using three segments. It is shown after completion in **Figure 11.5.2-1**.

Figure 11.5.2-1
Rock Cut Bridge

The bridge consists of four, 7.5-ft-deep special beams and is 24.5-ft wide. Transportation difficulties, elimination of a center pier and environmental restraints presented major design-construction challenges in a mountainous region of northeast Washington State. The restrictions imposed on constructing the new bridge were unusually severe. First, because it is located in an environmentally sensitive area, the surroundings were to be left as undisturbed as possible. Second, for environmental and structural reasons, no center pier (permanent or temporary) was allowed. Third, the route leading to the project site was along a highway with steep slopes and sharp bends. Therefore, even though a one-piece, 200-ft-long prestressed concrete beam was feasible, it was ruled out because such a long beam could not be transported along the winding highway.

The key to solving the problem was to divide the long beam into three, 63-ft-long beam segments with each segment weighing only 40 tons. The segments were fabricated and transported 150 miles to a staging area near the bridge site. There, the segments for each beam were precisely aligned, closure pours were made, post-tensioning tendons threaded and jacked. The fully-assembled beams were then carefully transported to the bridge site. At the site, the leading end of the beam was secured on a rolling trolley on a launching truss. Next, the transport vehicle backed the beam across the truss. When the leading end of the beam reached the opposite side of the river, the beam was picked up and set in place by cranes at both ends. All four beams were erected into final position using this method.

Using precast, prestressed concrete spliced beams for this bridge resulted in several benefits including a shortened construction time (3½ months), protection of the river environment, and cost savings due to constructing the bridge in one restricted construction season. This construction method resulted in a highly successful project. There was no pier in the water, no environmental issues were challenged by agencies, no construction delays occurred due to high water or weather, no stoppage occurred due to fishery constraints, and no special equipment or non-standard concrete strengths were needed. The total cost of the bridge was $660,471 ($141.50/sf). The cost of the precast concrete portion of the project, which included production, transportation, installation, and post-tensioning prior to launching, amounted to $229,482 ($49.17/sf). For more details, see Nicholls and Prussack, 1997.

11.5.3 US 27-Moore Haven Bridge, FL (1999)

The Moore Haven Bridge crosses the Caloosahatchee River. The bridge has record spans for precast, prestressed concrete at the time of its construction. The bridge consists of 11 total spans with a three-span continuous unit over the water. The three spans have a total length of 740 ft and a total width of 105 ft. The main span is 320 ft. The bridge is shown in **Figure 11.5.3-1**.

Figure 11.5.3-1
Moore Haven Bridge

Each three-span continuous unit consists of five segments: two haunched beams, one center drop-in beam and two end beams. The haunched beams are 138-ft long and vary in depth from 6.75 ft to 15 ft. The drop-in beam is 182-ft long and 8-ft deep. The end beams are 141-ft long and 6.75-ft deep. The beams were constructed in segments and made continuous using post-tensioning.

11.5.4 Bow River Bridge, Calgary, AB (2002)

The Bow River Bridge is 774-ft-long. It is shown during construction in **Figure 11.5.4-1**.

Figure 11.5.4-1
Bow River Bridge during Construction

The twin structures consist of four spans: two at 174-ft long and two at 213-ft. The project is described by Bexten, et al (2002). The precast concrete alternative provided a cost savings of about 10 percent over the steel plate beam option. This bridge marked the first time a single piece, 211-ft-long beam weighing 268,000 lbs spanned the entire distance between permanent pier supports without use of segmental I-beams, intermediate splice joints, or temporary falsework towers. Another source of economy was the relatively wide beam spacing of 11.65 ft. This spacing resulted in fewer beam lines despite the relatively long spans and the uncommonly heavy live loading mandated in Alberta due to the heavy hauling demands of its oil refinery industry. The maximum live load moment in Alberta bridge design practice is significantly larger than the moment resulting from the AASHTO Specifications.

An NU 2800 beam with a depth of 9.2 ft and a web width of 6.9 in. was used for the 213-ft-long span. The thin web is one of the important reasons for the minimized beam weight and increased span efficiency. The beam is shown in transit in **Figure 11.5.4-2**.

Figure 11.5.4-2
Transportation of the 211-Ft-Long Bow River Beam

The largest NU 2800 bridge beam used prior to this project was part of the spliced beam Oldman River Bridge, also in Alberta, which was completed a year earlier. It had a length of 188.6 ft and weighed 240,000 lbs. The Oldman River Bridge, however, utilized pier segments and jobsite-cast joints to span the 230 ft interior spans of the five-span bridge (180 ft, 230 ft, 230 ft, 230 ft, 180 ft).

The Bow River Bridge beams were pretensioned for lifting, shipping and erection. They were checked for top flange buckling during each of these stages. The stability analysis methods given in Section 8.10 were utilized. A steel stiffening truss was used in the center 100 ft section of the beams and a special lifting device that allowed shifting of the lifting point several feet above the top flange were some of the measures taken to assure safety during beam handling. At the site, the first beam erected in each span was braced to the top of the pier. After erection of the second beam, structural steel diagonal bracing diaphragms were placed between the first and second beams, which provided the necessary stability for both.

After erection, the beams were post-tensioned using four tendons, each with (12) 0.6-in.-dia strands, placed in 3-in.-dia ducts. One tendon was stressed prior to placing the deck making the beams continuous for deck weight. The remaining tendons were post-tensioned following placement of the deck.

11.6 POST-TENSIONING ANALYSIS

11.6.1 Introduction

Several issues related to the analysis and design of post-tensioned beams differ significantly from those for pretensioned beams. These include:

- losses in post-tensioning tendons. Additional sources of prestress losses must be considered such as friction and anchor losses.
- the interaction of losses between pretensioned strands and post-tensioned tendons
- time-dependent analysis. The method of analysis should take into account the effects of creep and shrinkage of concrete and the relaxation of steel. It should be applicable to statically indeterminate structures.
- the effect of post-tensioning on continuous beams. The method of analysis should properly account for post-tensioning, including secondary moments.
- the effect of post-tensioning ducts on shear capacity

These issues are discussed in this section. Chapter 8 provides a detailed discussion of prestress losses and deflections. In this chapter, only friction and anchorage set losses are discussed, which are unique to post-tensioning.

Other issues that are significant in the design and analysis of post-tensioned beams include:

- the methods used to show post-tensioning on plans. For example, should each tendon be shown or just the centroid of the group?
- the analysis and design of anchorage zones. The design must include consideration of potential conflicts between the anchorage hardware with its accompanying reinforcement and other reinforcement in the anchorage zone.
- estimation of deflection, camber and end rotation of beams with multiple construction stages.
- web thickness to accommodate ducts
- the difference between the center of gravity of the duct and the tendon
- flexural strength for post-tensioned tendons

Information on some of these subjects can be found elsewhere in this chapter, including in the design examples. Additional information can be found in texts that discuss design of post-tensioned structures. An additional resource regarding the use of post-tensioning in precast, prestressed concrete beams is TRB Report 517 of NCHRP Project 12-57, which is entitled "Extending Span Ranges of Precast Prestressed Concrete Girders" (Castrodale and White, 2004). This document includes an extensive discussion of issues and three design examples related to spliced beam construction.

11.6.2 Losses at Post-Tensioning

11.6.2.1 Friction Loss

In the design of post-tensioned structures, the designer ordinarily provides in the contract plans, the geometry of a tendon path and the required design forces at one or more locations along the path. This allows the contractor to select the post-tensioning system and procedures that lead to the best economy for the project, without neglecting safety.

The first step in analyzing a tendon is to plot a diagram of the stress or force along the tendon path. When a tendon is jacked from one or both ends, the stress along the tendon decreases away from the jack due to the effects of friction. The loss of stress may be expressed by the following equation:

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx + \mu\alpha)}\right) \qquad \text{[LRFD Eq. 5.9.5.2.2b-1]}$$

where

f_{pj} = stress in the prestressing tendon at jacking

e = base of natural logarithm

x = length of a prestressing tendon from the jacking end to any point under consideration, ft

K = wobble friction coefficient, typically about 0.0002/ft for rigid and semi-rigid galvanized metal ducts [LRFD Table 5.9.5.2.2b-1]

μ = coefficient of friction due to local deviations from tendon path, typically about 0.2/rad for rigid and semi-rigid galvanized metal sheathing and polyethylene ducts [LRFD Table 5.9.5.2.2b-1]

α = sum of the absolute values of angular change of post-tensioning tendon from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation, rad

11.6.2.2 Anchor Set Loss

Anchor set loss of prestress occurs in the vicinity of the jacking end of post-tensioned members as the post-tensioning force is transferred from the jack to the anchorage block. During this process, the wedges move inward as they seat and grip the strand. This results in a loss of elongation and therefore force in the tendon.

The value of the strand shortening, generally referred to as anchor set, ΔL, varies from about 0.125 in. to 0.375 in. It depends on the anchorage hardware and jacking equipment. An average value of 0.25 in. may be assumed in design with the stipulation on the plans that the post-tensioning contractor is to verify the accuracy of this assumed value and appropriate adjustment be made to the expected force and elongation.

The anchor set loss is highest at the anchorage. It diminishes gradually due to friction effects as the distance from the anchorage increases. Anchor set loss is more significant in shorter tendons. On very short tendons, the anchor set loss can be nearly as high as the initial tendon elongation. Therefore, the initial prestress could be ineffective.

11.6.2.3 Design Example

Calculation of friction and anchor set losses is best demonstrated by an example.

Figure 11.6.2.3-1 Anchor Set Loss

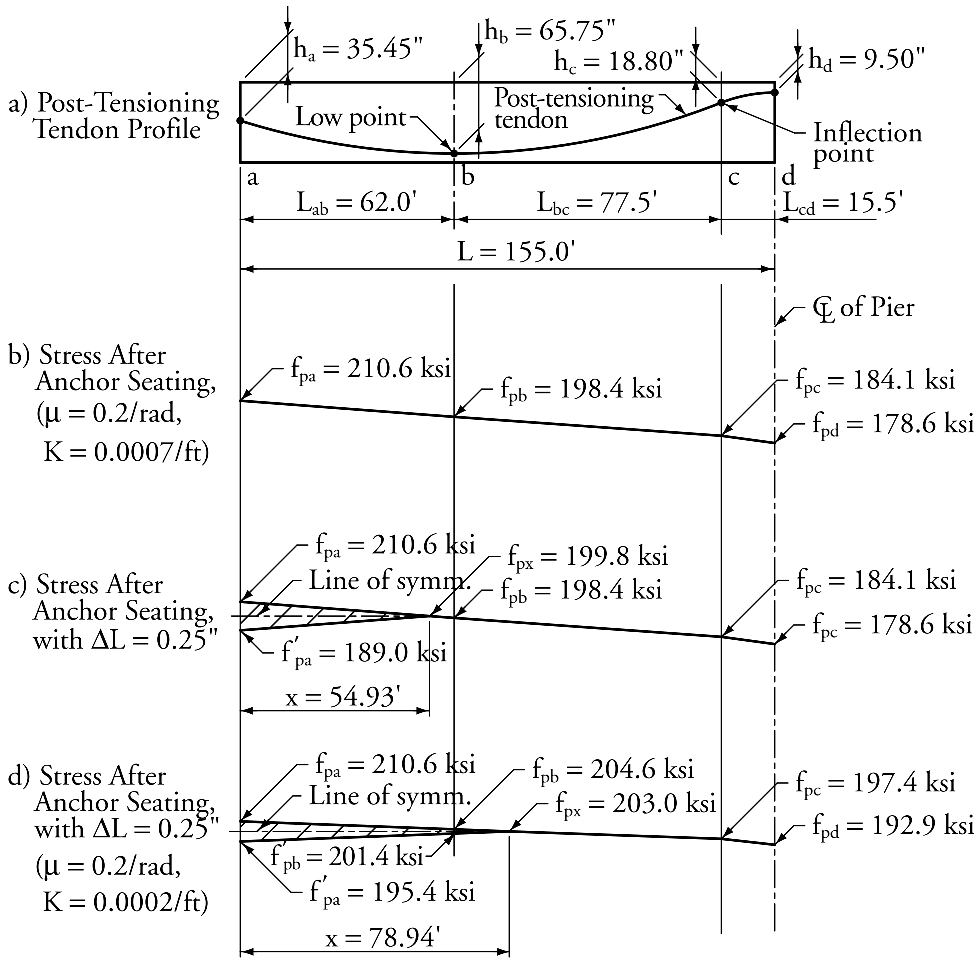


Figure 11.6.2.3-1a shows the elevation of the end span of a multispan beam. Its length is 155 ft. The tendon profile consists of three segments, L_{ab}, L_{bc} and L_{cd} with three different curvatures.

A jacking stress, $f_{pa} = 0.78f_{pu} = 210.6$ ksi is often used for design. A curvature coefficient $\mu = 0.20$/rad, and a wobble coefficient, $K = 0.0007$/ft are assumed. The value of K in this part of the example is significantly overestimated for simpler presentation of the anchor set loss. The typical value is 0.0002/ft as stated in Section 11.6.2.1.

11.6.2.3.1 Friction Loss

The stress values before seating can be calculated by applying LRFD Eq. 5.9.5.2.2b-1 for each of the three segments. The results are $f_{pb} = 198.4$ ksi, $f_{pc} = 184.1$ ksi, and $f_{pd} = 178.6$ ksi.

For this reason, the stress diagram before accounting for anchor set loss, shown in **Figure 11.6.2.3-1b**, consists of three linear segments. The slope of each segment is partly a function of the tendon curvature as discussed in Section 11.6.2.1.

11.6.2.3.2 Anchor Set Loss

The hatched area in **Figure 11.6.2.3-1c** and **d** represent the drop in tendon stress over the affected beam length, x, after the post-tensioning tendon is anchored. This total length may be shorter than L_{ab} or as large as the beam length between anchorages. After seating, the highest stress will be at the right end of the hatched area. The stresses before seating and after seating are symmetrical about a horizontal line passing through f_{px}, the tendon stress at distance, x. This symmetry results from the fact that friction effects are equal in both directions, i.e., as the tendon is being pulled out of the beam during stressing, or pulled back into the beam during seating of the anchorage. Since the distance, x, is not yet known, it is best calculated by numerical iteration until the following condition is satisfied:

$$\Delta L = \text{Hatched area}/E_p \qquad \text{(Eq. 11.6.2.3.2-1)}$$

11.6.2.3.2.1 Length Affected by Seating is Within L_{ab}

First, assume that x, measured from point a, is equal to L_{ab} = 62 ft. Dividing the hatched area in **Figure 11.6.2.3-1** by the steel modulus of elasticity, (210.6 – 198.4)(62)(12)/28,500 = 0.318 in. which is greater than the assumed ΔL = 0.25 in. Thus, the length affected by seating is within L_{ab}, and therefore the hatched area is bounded by straight lines.

In this case, a closed form solution is possible using the Eq. (11.6.2.3.2-1):

$$x = \sqrt{\frac{\Delta L(E_p)(L_{ab})}{(f_{pa} - f_{pb})}} \qquad \text{(Eq. 11.6.2.3.2.1-1)}$$

Substituting for ΔL, f_{pa}, f_{pb}, E_p, and L_{ab}, the values 0.25 in., 210.6 ksi, 198.4 ksi, 28,500 ksi, and 62 ft respectively, x is found to be = 54.93 ft, which is less than 62 ft as expected. The corresponding anchor set loss, Δf_{pA}, is:

$$\Delta f_{pA} = \frac{2(f_{pa} - f_{pb})(x)}{L_{ab}} \qquad \text{(Eq. 11.6.2.3.2.1-2)}$$

Substituting the value of x = 54.93 ft, Δf_{pA} = 21.6 ksi, see **Figure 11.6.2.3-1c**.

Therefore

$$f_{px} = f_{pa} - 0.5\Delta f_{pA} = 210.6 - 0.5(21.6) = 199.8 \text{ ksi} \le 0.74 f_{pu} = 199.8 \text{ ksi}$$

(LRFD Table 5.9.3-1)

and

$$f'_{pa} = f_{pa} - \Delta f_{pA} = 210.6 - (21.6) = 189.0 \text{ ksi} \le 0.70 f_{pu} = 189.0 \text{ ksi}$$

(LRFD Table 5.9.3-1)

11.6.2.3.2.2 Length Affected by Seating is Within L_{ac}

To illustrate the case where the length affected by seating is greater than the distance to the low point in the tendon profile L_{ab} (hatched area longer than 62 ft), the example will be reworked with μ = 0.20/rad and K = 0.0002/ft which are the typical values according to LRFD Table 5.9.5.2.2-1. With these values, f_{pb}, f_{pc}, and f_{pd} can be found to be equal to 204.6 ksi, 197.4 ksi, and 192.9 ksi.

In this case, two quantities are unknown: x and Δf_{pa}. An iterative procedure will be used to reach a solution.

The first condition is that the stress diagrams before and after seating are symmetrical about a horizontal line passing through f_{px}, therefore:

$$\Delta f_{pb} = \frac{(2)(f_{pb} - f_{pc})(x - L_{ab})}{L_{bc}} \quad \text{(Eq. 11.6.2.3.2.2-1)}$$

$$\Delta f_{pa} = 2(f_{pa} - f_{pb}) + \Delta f_{pb} \quad \text{(Eq. 11.6.2.3.2.2-2)}$$

The second condition is that the hatched area divided by the steel modulus of elasticity is equal to the anchor seating, ΔL:

$$\Delta L = \frac{1}{E_p}\left[\frac{1}{2}\left(\Delta f_{pa} + \Delta f_{pb}\right)\left(L_{ab}\right) + \frac{1}{2}\left(\Delta f_{pb}\right)\left(x - L_{ab}\right)\right] \quad \text{(Eq. 11.6.2.3.2.2-3)}$$

With x assumed equal to L_{ab}, the first estimate of Δf_{pa}, using Eq. (11.6.2.3.2.2-2), is 12 ksi. Substituting this value in Eq. (11.6.2.3.2.2-3) gives a tendon shortening of 0.16 in., which is less than the assumed value of 0.25 in.

The next iteration would be to try $x = L_{ab} + L_{bc}$. The corresponding tendon shortening using this value is 0.76 in., which is greater than 0.25 in. Since the two computed values bracket the assumed value, values of x between the above two limits are tried until a solution is found. The use of spreadsheet software simplifies this iteration. The results of the iteration are x = 947.25 in. (78.94 ft) and Δf_{pa} = 15.2 ksi. This corresponds to stress at point a = 210.6 – 15.2 = 195.4 ksi, stress at point b = 204.6 – (15.2 – 12.0) = 204.6 – 3.2 = 201.4 ksi, and stress at distance x = 204.6 – 3.2/2 = 203.0 ksi. **Figure 11.6.2.3-1d** shows the tendon stress diagram for this case.

As noted in the previous section, the *LRFD Specifications* limits tendon stresses, f'_{pa} and f'_{px} after seating. The stress f'_{pa} at point a exceeds the limit by 6.4 ksi, while the stress, f'_{px} at point x exceeds the limit by 3.2 ksi. Therefore, tendon stress at point a governs and the initial jacking stress must be reduced by approximately 6.4 ksi. The required maximum jacking stress is therefore 210.6 – 6.4 = 204.2 ksi. The tendon stress should be recomputed using this new jacking stress, resulting in the entire tendon stress diagram being lowered by approximately 6.4 ksi.

11.6.2.4 Elastic Shortening Loss

Post-tensioned beams are typically post-tensioned sequentially using one multi-strand tendon jack. It is not generally economical to tension more than one tendon at a time. When the first tendon is tensioned, it is anchored at the end of the beam. Tensioning of subsequent tendons in the same beam, and to some extent moving across the bridge width (if the deck has been cast before the tendons are stressed), causes the concrete along with previously tensioned tendons to shorten. This sequential elastic shortening loss is highest in the first tendon tensioned. There are formulas in the *LRFD Specifications* to estimate the average elastic shortening loss in this situation. A second round of tendon tensioning to restore the original tensile stress in the tendons may substantially eliminate the losses due to sequential shortening, but it is generally not required.

11.6.3 Time-Dependant Analysis

Some commercial computer programs that include time-dependent analysis are based on European creep and shrinkage prediction formulas. There has been no evidence that these formulas are more accurate than those on which the *LRFD Specifications* are based. These spliced-beam programs tend to follow programs developed for segmental box beam bridges in those regions. In most cases, it is unwarranted to spend a considerable amount of time and resources to model spliced I-beam bridges with sophisticated finite-element, time-dependent programs. Experience has shown no advantage or superior performance in calculating, in detail, the effects of differential creep and shrinkage and the effects of temperature gradients, compared to the practice used with conventional, non-post-tensioned I-beam systems.

11.6.4 Equivalent Loads for Effects of Post-Tensioning

In a pretensioned beam, when the prestress force is transferred from the strands to the concrete, it causes the member to camber and become supported at its ends. The beam acts as a simply-supported member. At any section, the effect of the prestress is an axial force equal to the effective prestress force and a bending moment equal to the product of the effective prestress force and its eccentricity. Because the member is statically determinate, the support reactions due to prestressing are zero. The end reactions are caused only by member weight. The same is true for a simple span, post-tensioned beam.

For continuous members, post-tensioning is usually introduced at the construction site.

Because the continuous member is statically indeterminate at the time of post-tensioning, its support reactions are affected by the deformations of the beam. The member cannot camber freely as the post-tensioning tendons are stressed.

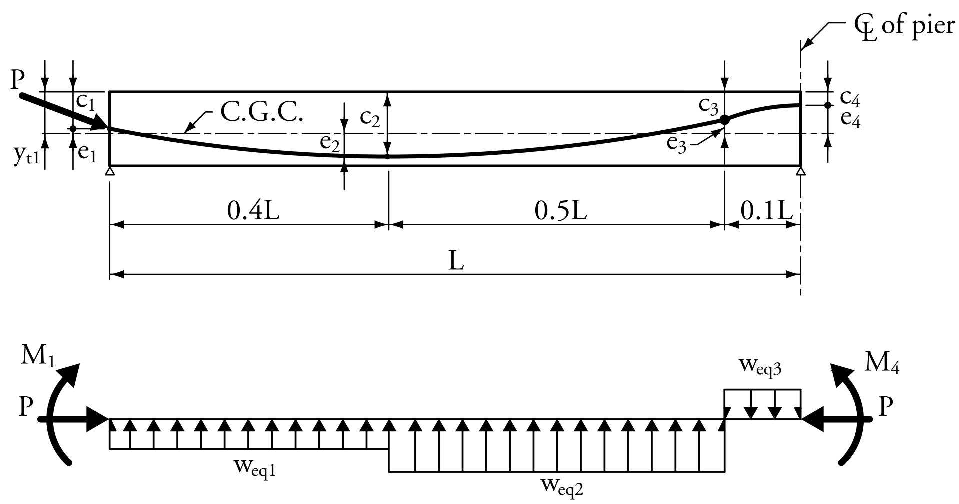
Support reactions caused by the restrained deformations due to post-tensioning result in additional moments called “secondary” moments. There are secondary shears as well, but usually not additional axial forces, unless the member is restrained by the supports against axial deformation. The term “secondary” is somewhat misleading. The effects are called “secondary” only because they are caused as the result of another effect – the post-tensioning of the beam. The effect of the secondary moments may not be minor as could be implied by the term, because it is conceivable that the secondary moment at the intermediate support of a two-span bridge could totally offset the primary moment caused by post-tensioning. This would result in a uniform stress at that location equal to P/A, where A is the cross-sectional area of the member.

11.6.4.1 Conventional Analysis Using Equivalent Uniformly Distributed Loads

A common approach to evaluate secondary moments due to post-tensioning, is to model the effect of the post-tensioning tendon as a series of equivalent uniformly distributed loads. **Figure 11.6.4.1-1** shows the required equations for calculation of the equivalent loads for a typical end span of a post-tensioned beam.

Figure 11.6.4.1-2 shows one span of a two-span continuous bridge beam.

Figure 11.6.4.1-1
Post-Tensioning Equivalent Loads for Two-Span Continuous Bridge



$e_1 = y_{t1} - c_1$

$e_2 = c_2 - y_{t1}$

$w_{eq1} = 2P(e_2 + e_1)/(0.4L)^2$

$M_1 = Pe_1$

$e_4 = y_{t1} - c_4$

$c_3 = (c_2 - 5c_4)/6$

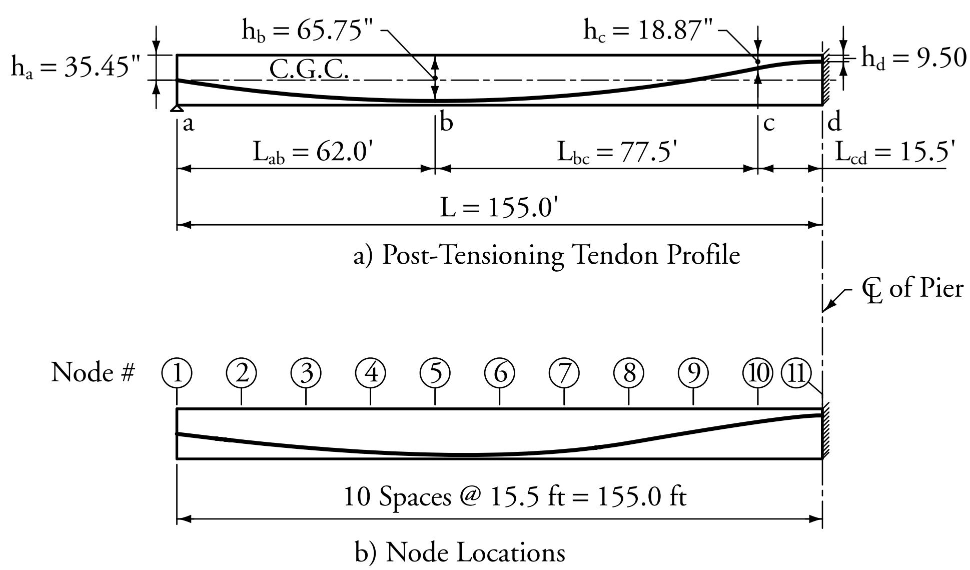
$e_4 = y_{t1} - c_3$

$w_{eq2} = 2P(e_2 + e_3)/(0.5L)^2$

$w_{eq3} = 2P(e_4 - e_3)/(0.1L)^2$

$M_4 = Pe_4$

Figure 11.6.4.1-2
One Span of a Two-Span Continuous Bridge



The two spans are equal, 155 ft. The beam depth is 72 in. and the centroidal distance from the top fiber, 35.45 in.

The span is divided into 10 segments with 11 nodes. The eccentricities at Nodes 5, 10 and 11, at 0.4L, 0.9L and 1.0L, are given, based on available concrete cover at the lowest and highest points, and on a common tangent of the curves connected at Node 10. Note that although the 0.9L node is commonly used as the inflection point for the tendon as it approaches the pier location (1.0L), it may not be the optimal location in terms of overall effects of post-tensioning. The designer may wish to investigate other locations. The geometric properties of the curves between Nodes 1 and 5, 5 and 10, and 10 and 11, are used to determine the tendon eccentricities at the remainder of the nodes. If the curve is a parabola, as is usually assumed, the relationship, $y = ax^2$, can be used. The distance, y, is the height above the lowest point or below the highest point, and x is the horizontal distance from that point. The eccentricities at all 11 nodes for the example have been calculated and are shown in **Table 11.6.4.1-1**.

Table 11.6.4.1-1
Post-Tensioning Effect – Approximate Method

Node #	1	2	3	4	5	6	7	8	9	10	11
Distance from left end, in.	0	186	372	558	744	930	1,116	1,302	1,488	1,674	1,860
Tendon eccentricity, in.	0.00	-13.26	-22.73	-28.41	-30.30	-28.43	-22.80	-13.43	-0.30	16.58	25.95
Post-tensioning stress, ksi	181.4	182.9	184.4	185.9	187.4	188.3	187.7	186.3	184.8	183.4	178.9
Equivalent loads:											
Vertical force, kips/ft	1.050	1.050	1.050	1.050	1.050	0.310	0.310	0.310	0.310	-5.210	-5.210
Moment, in.-kip	0	0	0	0	0	0	0	0	0	0	-20,798.1
Total moment, in.-kip	0	-9,456.1	-15,969.0	-19,538.0	-20,163.4	-17,859.0	-12,638.0	-4,501.1	6,551.5	20,520.2	28,659.6
Primary moment, in.-kip	0	-10,625.0	-18,214.0	-22,767.0	-24,284.5	-22,782.0	-18,274.0	-10,760.0	-240.4	13,284.8	20,798.1
Secondary moment, in.-kip	0	786.1	1,572.3	2,358.4	3,144.9	3,930.7	4,716.9	5,503.0	6,289.2	7,075.3	7,861.5

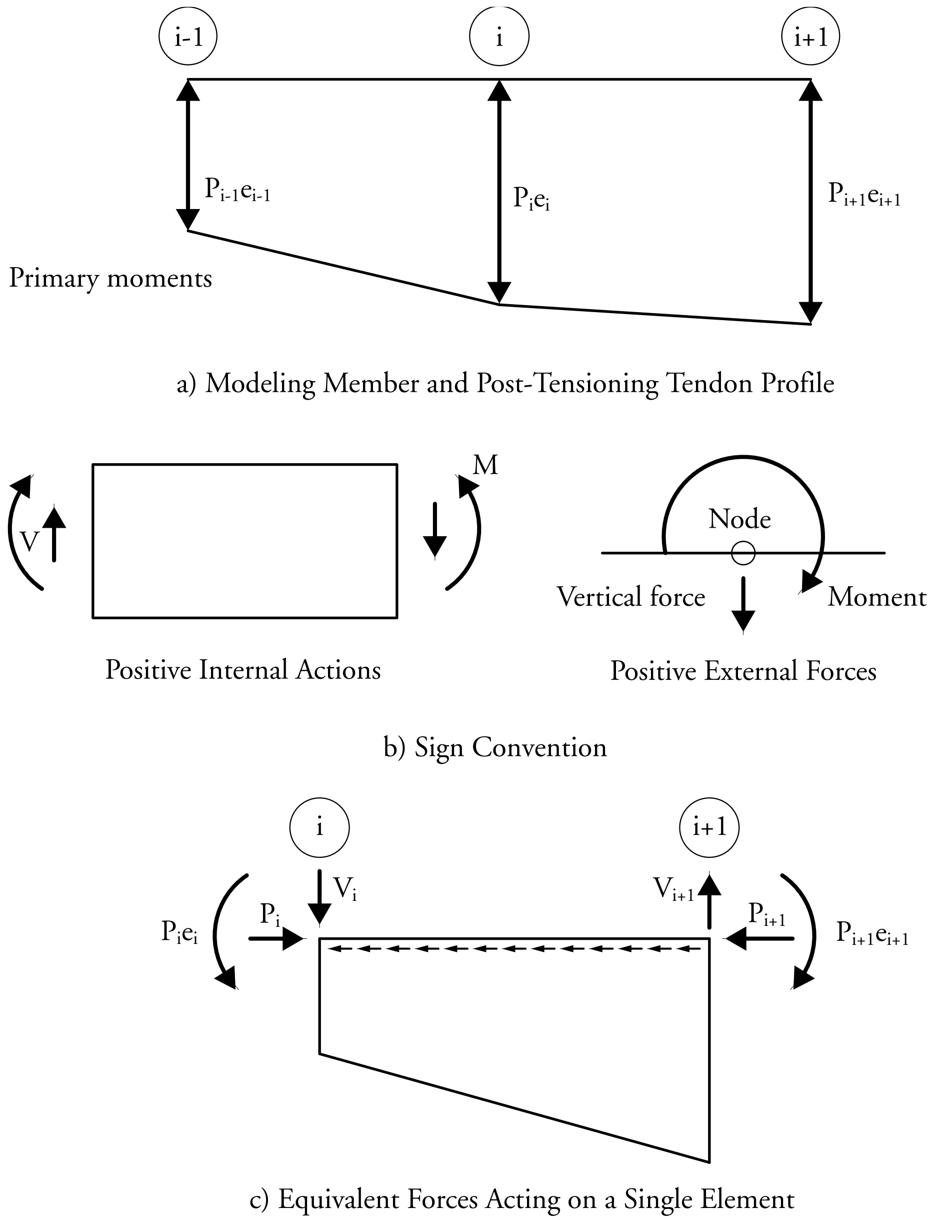
Table 11.6.4.1-1 shows the post-tensioning stresses at each node after accounting for friction and anchor set losses. The average post-tensioning tendon stress along the length of the span is 184.7 ksi. Assuming the area of post-tensioning tendons is 4.34 in.^{2}, which corresponds to a (20) 0.6 in.-dia strand tendon, the average post-tensioning force is equal to 801.5 kips. Using this average force, equivalent loads are calculated according to **Figure 11.6.4.1-1**. The loads are then input into a continuous beam analysis computer program to obtain the total moments due to post-tensioning. For the analysis of this particular example, only one span needs to be modeled due to symmetry. The support at point 'a' is assumed to be restrained against vertical movement only, while the support at point 'd' is fully restrained due to symmetry. The secondary moments are the difference between the primary and the total moments. The total, primary, and secondary moments using this method are shown in **Table 11.6.4.1-1**.

This approach is appropriate only if the effective prestress force is relatively constant along the entire beam length. However, friction and anchor set losses in large multi-strand tendons, which are generally used in bridge applications, may cause the variation in post-tensioning force over the member length to be as high as 30 percent. Thus, assuming constant P and uniform equivalent loads may be only appropriate in preliminary design.

11.6.4.2 Refined Modeling Using a Series of Nodal Forces

A convenient way to determine the effects of post-tensioning is to use a spreadsheet program. The post-tensioning effects at each of the nodes of an element are converted to equivalent nodal forces: a vertical force, a horizontal force and a couple at each node. In addition, a distributed horizontal force is applied to the element between each pair of adjacent nodes to account for the change between horizontal nodal forces due to friction and anchor set losses. The beam and post-tensioning modeling are shown in **Figure 11.6.4.2-1**.

Figure 11.6.4.2-1 Numerical Assumptions and Sign Convention in Proposed Method



The only approximation used in this method is to assume the post-tensioning profile as a series of straight lines between the nodes. A structural analysis program for continuous beams is then used to determine the total bending moment diagram. For clarity of presentation, an axial load diagram is not included. It can easily be obtained from the axial nodal and element forces.

The primary bending moment diagram can be obtained directly as Pe. It can also be obtained through the same structural analysis program with the same loads, but with enough supports removed to render the beam statically determinate. The example used in Section 11.6.4.1 will be used below to illustrate the calculation steps.

Figure 11.6.4.2-1 shows three nodes in sequence and the sign convention used. The equivalent load at each node is calculated using the post-tensioning force and its eccentricity at that point. The global (structure) sign convention for this analysis is that downward loads are positive, a couple acting clockwise is positive, tendon eccentricity below the concrete centerline is positive, and prestress force is always positive. The standard sign convention for internal forces, including axial force, shearing force and bending moment is used.

Consistent units of measurement must be maintained throughout the analysis. All supports except one are assumed to be free to move horizontally.

The vertical point load at a node, i, is computed as:

$$F_{yi} = \frac{P_{i-1}e_{i-1} - P_i e_i}{L_{i-1}} - \frac{P_i e_i - P_{i+1}e_{i+1}}{L_i} \qquad \text{(Eq. 11.6.4.2-1)}$$

where

e_i = tendon eccentricity from concrete section centroid to tendon centroid at node i

L_i = distance between nodes i and i + 1, or length of segment i

P_i = post-tensioning force at node i.

The couples shown at element-ends, see **Figure 11.6.4.2-1c**, cancel each other when the elements are combined into the full member. Two exceptions to this are the first node, Node 1, and the last node, Node 11, in this example. Thus, the external couples at Nodes 2, 3, 4, 5, 6, 7, 8, 9 and 10 = 0.0. The couples at Nodes 1 and 11 are computed using:

$$M_i = P_i e_i \qquad \text{(Eq. 11.6.4.2-2)}$$

$$M_1 = -P_1 e_1 \text{ and } M_{11} = P_{11}e_{11}$$

Using the sign convention for moments, M_1 will be negative and M_{11} will be positive.

When using Eq. (11.6.4.2-1), the first term should be taken equal to 0.0 when i = 1 and the second term equal to 0.0 when i = n.

11.6.4.2.1 Example

As an example, calculate the equivalent loads at Nodes 1 and 2 and on Segment 1. The post-tensioning forces are 787.28 and 793.79 kips. The eccentricities are 0.00 and 13.26 in. The loads at Nodes 1 and 2 are:

$$F_{y1} = 0 - \frac{787.3(0) - 793.8(13.26)}{186} = 56.59 \text{ kips}$$

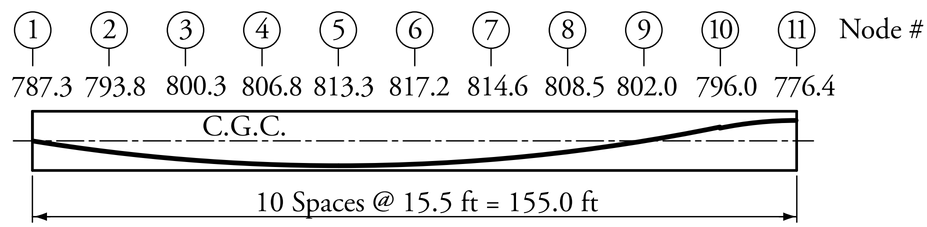
$$M_1 = (787.3)(0) = 0.00$$

$$F_{y2} = \frac{787.3(0) - 793.8(13.26)}{186} - \frac{793.8(13.26) - 800.3(22.73)}{186} = -15.38 \text{ kips}$$

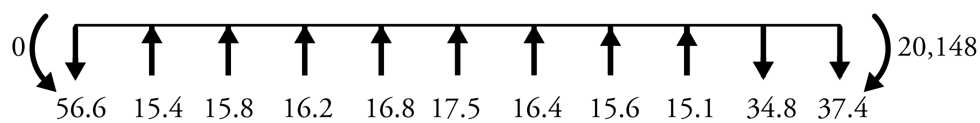
$$M_2 = 793.8\ (13.26) - 793.8\ (13.26) = 0.00$$

The post-tensioning force at each node is calculated as the product of the post-tensioning stress, after accounting for friction and anchor set losses, and the area of post-tensioning tendons. The forces at each node are given in **Table 11.6.4.2.1-1** and **Figure 11.6.4.2.1-1a**.

Figure 11.6.4.2.1-1
Post-Tensioning Profile and Equivalent Loads



a) Post-Tensioning Forces, P_i (kips)



b) Post-Tensioning Equivalent Vertical Loads and Moments (Force - kips, Moment - in.-k)

The equivalent vertical loads for this example are shown in **Figure 11.6.4.2.1-1b**. A relatively large number of nodes in a span would result in greater accuracy. For most applications, nodes at tenth or twentieth points provide sufficient accuracy.

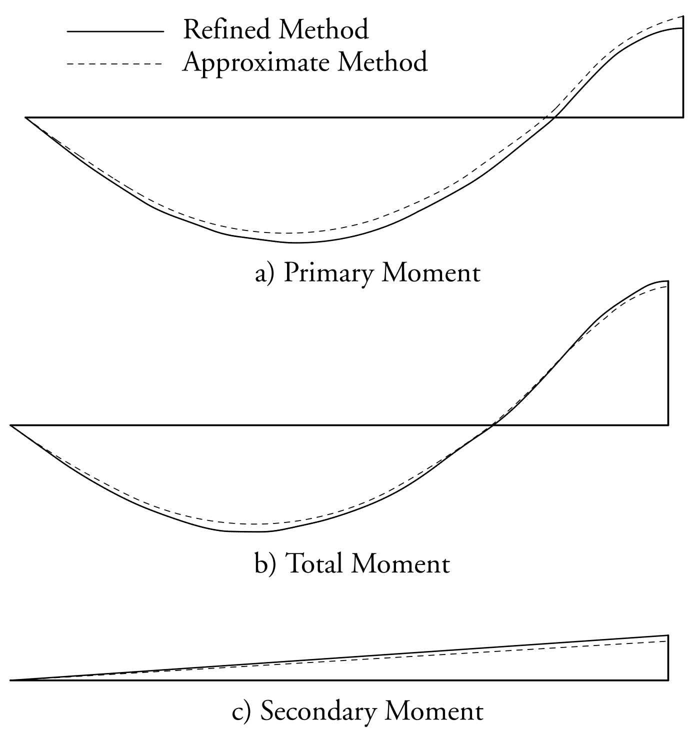
After the equivalent loads due to post-tensioning are calculated, the member should be checked for equilibrium; the sum of vertical forces and the sum of the moments about a point should be equal to zero. When the supports are placed and a continuous beam analysis is performed, the total reactions, shears and moments due to post-tensioning are obtained. The reactions obtained in this step are due to secondary effects that are intended to maintain the restraint conditions at the supports. **Table 11.6.4.2.1-1** shows the total moment using the proposed method.

The total moments are shown in **Figure 11.6.4.2.1-2b**. Subtracting the primary moments from the total moments results in the secondary moments.

Table 11.6.4.2.1-1
Post-Tensioning Effect – Refined Method

Node #	1	2	3	4	5	6	7	8	9	10	11
Distance from left end, in.	0	186	372	558	744	930	1,116	1,302	1,488	1,674	1,860
Tendon eccentricity, in.	0	-13.26	-22.73	-28.41	-30.30	-28.43	-22.80	-13.43	-0.30	16.58	25.95
Post-tensioning stress, ksi	181.4	182.9	184.4	185.9	187.4	188.3	187.7	186.3	184.8	183.4	178.9
Post-tensioning force, kips	787.3	793.8	800.3	806.8	813.3	817.2	814.6	808.5	802.0	796.0	-776.4
Equivalent loads:											
Vertical force, kips	56.59	-15.38	-15.78	-16.17	-16.84	-17.48	-16.42	-15.61	-15.12	34.79	74.83
Moment, in.-kip	0	0	0	0	0	0	0	0	0	0	20,148.3
Total Moment, in.-kip	0	-9,620.6	-16,376.8	-20,194.2	-20,998.3	-18,677.8	-13,102.1	-4,476.1	7,051.4	21,387.7	29,251.1
Primary Moment, in.-kip.	0	-10,525.6	-18,190.7	-22,921.4	-24,643.5	-23,233.6	-18,573.3	-10,858.7	-240.6	13,189.0	20,148.3
Secondary Moment, in.-kip	0	905.0	1,814.0	2,727.2	3,645.2	4,555.9	5,471.2	6,382.6	7,292.0	8,198.7	9,102.8

Figure 11.6.4.2.1-2
Bending Moment Diagrams



11.6.4.3 Design Consideration

The secondary reactions are the only external forces acting on the member due to post-tensioning. They act at the supports. These reactions must be in equilibrium. For the two span example, the reaction due to post-tensioning at the center pier is 9.79 kips downward (or pier uplift), and the reaction at each abutment is 9.79/2 = 4.89 kips upward (or downward load on abutment). Accordingly, secondary shears must be constant between supports and secondary moments must be linear between supports. If these characteristics are not observed, the calculations must be reviewed for errors. These characteristics must hold true regardless of the tendon profile and whether or not the member's cross-section properties vary along its length.

The total (primary plus secondary) effects must be used when checking service limit states, e.g., tension at bottom fibers at final loading conditions, etc. However, the primary and secondary effects must be separated before performing calculations for the strength limit state. Because post-tensioning continuous members creates a set of external loads, i.e. support reactions, these external loads must be considered in the factored load combinations in strength design. The accepted practice is to combine the factored secondary moment using a load factor of 1.0 with the moments due to factored dead and live loads, and to compare the "total factored load" moment at a section against the design flexural strength at that section.

The accuracy of using elastic analysis to calculate the secondary moments and of using a load factor of 1.0 at the strength limit state has occasionally been the subject of debate. However, no better approach has been adopted for standard practice.

11.6.5 Shear Limits in Presence of Post-Tensioning Ducts

In order to ensure that the concrete in the web of the beam will not crush prior to yielding of the transverse reinforcement, the *LRFD Specifications* gives an upper limit of V_n:

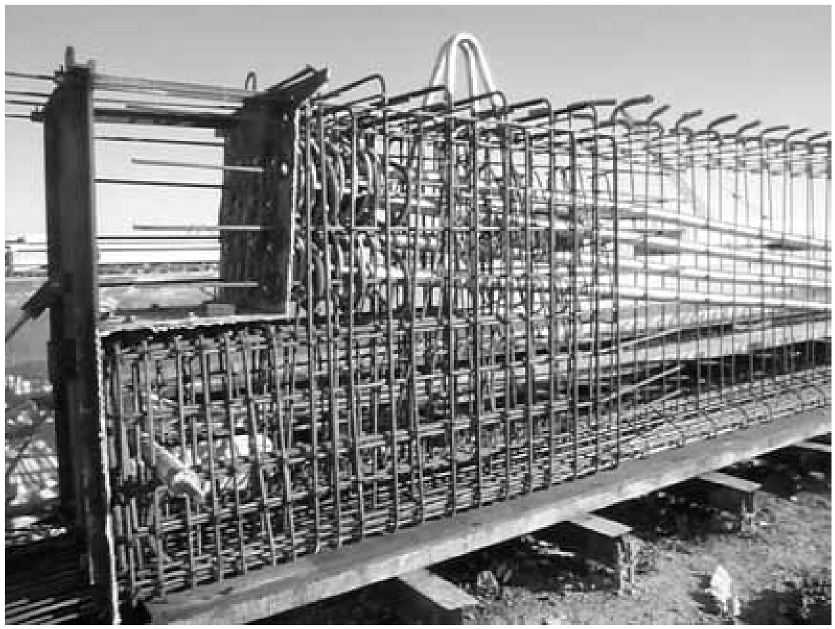
$$V_n = 0.25f'_c b_v d_v + V_p$$ [LRFD Eq. 5.8.3.3-2]

Where b_v is taken as the minimum web width within the depth d_v, modified for the presence of ducts where applicable. In determining b_v at a particular level, the diameters of the ungrouted ducts or one-half the diameters of the grouted ducts at that level must be subtracted from the web width. [LRFD Art. 5.8.2.7]

11.7 POST-TENSIONING ANCHORAGES IN I-BEAMS

Anchorage zones are designed to accommodate anchorage hardware with its associated special reinforcement and to provide adequate space for the reinforcement needed to distribute the highly concentrated post-tensioning force. Detailed guidance for the design of anchorage zones is given in the PTI publication, *Anchorage Zone Design* (2000). A design example in NCHRP Report 517 (Castrodale and White, 2004) also discusses the design of anchorage zone reinforcement. **Figure 11.7-1** shows reinforcement and anchorages in the end of a beam that has been designed with a top recess. The dapped area provides access for post-tensioning after both abutting beams are erected in place.

Figure 11.7-1
Reinforcement and Anchorages in an I-Beam End Block

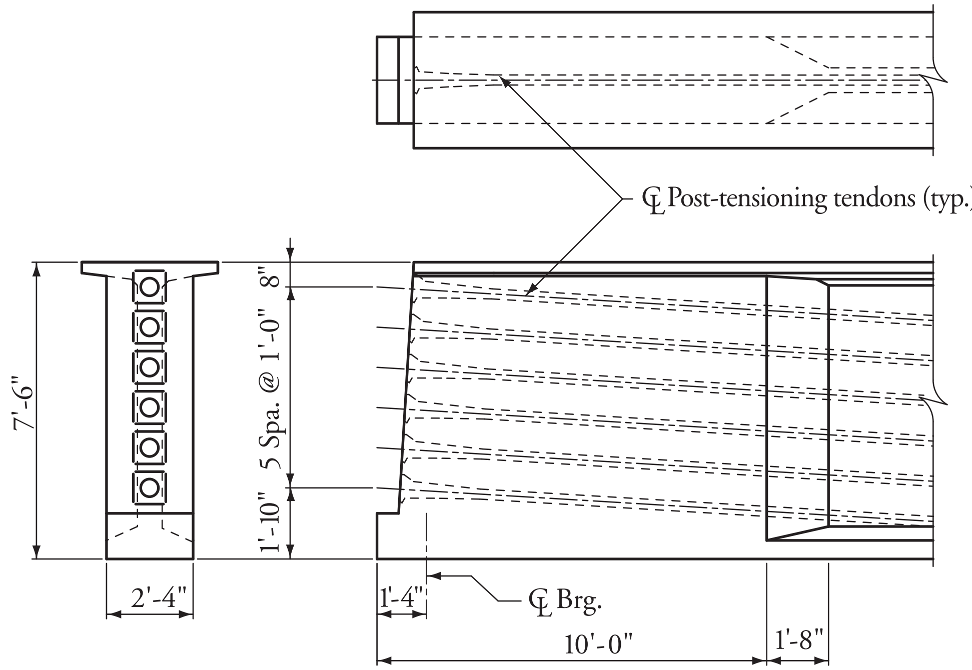


The anchorage zone is typically detailed using an end block that is the same width as the bottom flange and extends for a distance from the end of the beam of at least one beam height before a tapered section returns the cross-section to the width of the web. Typical dimensions are illustrated in **Figure 11.7-2**. The extent of the anchorage zone is based on the principle of St. Venant which proposes that the disturbed stress field introduced at the end of the beam by the concentrated forces at post-tensioning tendon anchorages extends approximately a beam height into the beam (see the discussion in Section 11.4.10 and LRFD Fig. C5.10.9.1-1).

Based on this principle, the cross-section in the anchorage zone (end block) has generally been held constant until the stress distribution from the anchorage forces becomes more uniform. If the cross-section were also decreased within the disturbed region, it is believed that this could compound the stress disturbance and lead to increased cracking.

Some research has indicated that a much smaller anchorage zone may be adequate. It has been proposed that the concrete should be the minimum size necessary to house the anchorage hardware and to provide for cover over reinforcement. It is suggested that large concrete dimensions in the anchorage zone are unnecessary and possibly counterproductive, as they may require large amounts of reinforcement to control cracks. A research project by Tadros and Khalifa (1998) for the Federal Highway Administration and the Nebraska Department of Roads tested full-size beams with two concepts for anchorage zones with significally reduced cross-sections. The new details have been adopted and used

Figure 11.7-2
Typical I-Beam End Block Dimensions



on several projects in Nebraska and other areas such as project shown in **Figure 11.7-3**. A paper by Ma, et al (1999), discusses the design of this post-tensioned anchorage zone in accordance with the *LRFD Specifications* using strut-and-tie modeling. The paper includes a design example. Experimental testing of post-tensioning anchor zones has been reported by Breen, et al (1994) and Ma, et al (1999).

In Washington State, alternative details were used on the Rock Cut Bridge for Stevens and Ferry Counties (Nicholls and Prussack, 1997). This project included casting the end blocks in a secondary cast after the prismatic beams were stripped from the form. This can result in cost savings by not having to use special beam forms to accommodate the widened end block section. **Figure 11.7-3a** shows a workman tying bars and forming the short end block. **Figure 11.7-3b** shows the finished secondary cast. Additional details of the project are discussed in Section 11.5.2.

Figure 11.7-3
Rock Cut Bridge End Block

a) Forming and Tying Steel for End Block

b) Completed End Blocks

11.8 DESIGN EXAMPLE: TWO-SPAN BEAM SPLICED OVER PIER

11.8.1 Introduction

This example is similar to Examples 9.5 and 9.6. It will provide a comparison in design calculations when post-tensioning is employed for a very common superstructure system. It will also illustrate the increased span length when post-tensioning is used to establish continuity over a pier.

Many of the fundamental calculations in this example are not shown or are not explained in detail. Frequently, the applicable AASHTO Specifications references are not given. These details are provided in Chapter 8 and in the design examples in Chapter 9.

This example selects the same 72-in.-deep bulb tee (BT-72) used in Examples 9.5 and 9.6. However, for this example, two, 155-ft spans will be used instead of the 110-ft end spans and the 120-ft center span of the three-span bridge designed in those examples. The full span beam segments are spliced over the pier with post-tensioning and are made composite with the deck. The design uses the *LRFD Specifications*, the same as used for Example 9.6. Some of the details already discussed in Example 9.6 are not repeated here. Analysis for post-tensioning effects is emphasized.

Figures 11.8.1-1 and **11.8.1-2** show the longitudinal section and cross-section of the bridge. The cross-section has four beams spaced at 12'-0". AASHTO-PCI Bulb Tees are modified by widening the section 1-in. to accommodate post-tensioning ducts. The beams are designed to act compositely with the cast-in-place concrete deck slab. The 8-in.-thick slab includes a ½-in. integral wearing course. Therefore, the full 8-in. thickness is used in load calculations but 7.5-in. is used for the deck to compute composite section properties. A haunch over the top flange averaging ½-in. thick is considered in the load and stress analysis. Design live loading is HL-93.

Figure 11.8.1-1 Longitudinal Section

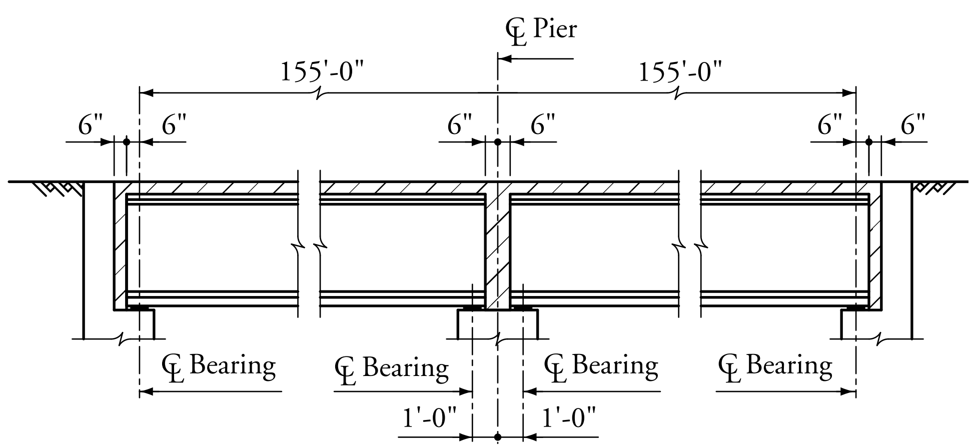
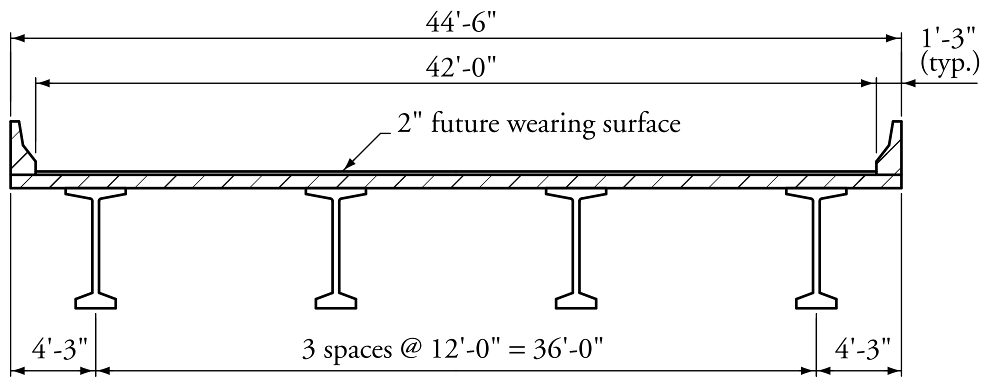


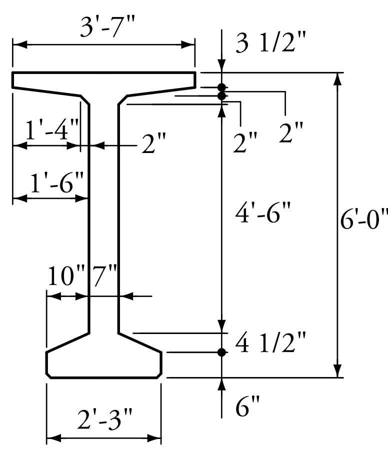
Figure 11.8.1-2
Cross-Section



11.8.2
Materials and Beam Cross-Section

The cross-section of the modified AASHTO-PCI Bulb Tee (BT-72) is shown in **Figure 11.8.2-1**. The width of the beam was increased 1 in. to provide a 7-in.-wide web to accommodate post-tensioning ducts.

Figure 11.8.2-1
Modified 72-in.
AASHTO-PCI Bulb Tee



Cast-in-place slab:	Total thickness, t_s = 8.0 in.
	Structural thickness = 7.5 in.
	Concrete strength at 28 days, $f'_c = 4.0$ ksi
	Concrete unit weight, $w_c = 0.150$ kcf
	$E_c = 33{,}000(0.15)^{1.5}\sqrt{4} = 3{,}834$ ksi
Precast beams:	Concrete strength at transfer, $f'_{ci} = 5.5$ ksi
	Concrete strength at 28 days, $f'_c = 7.0$ ksi
	Concrete unit weight, $w_c = 0.150$ kcf
	$E_c = 33{,}000(0.15)^{1.5}\sqrt{7} = 5{,}072$ ksi

From **Figure 11.8.1-1**, the design span is assumed to be 154.0 ft when the beam is supported on its bearing pads before it is made continuous, and 155.0 ft after the pier diaphragm concrete is cured and the beam becomes continuous.

Pretensioning strands:

½-in.-dia, low-relaxation

Area of one strand = 0.153 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Stress limits for pretensioning strands: [LRFD Table 5.9.3-1]

before transfer, $f_{pi} \leq 0.75f_{pu}$ = 202.5 ksi

Post-tensioning strands:

0.6-in.-dia, low-relaxation

Area of one strand = 0.217 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Stress limits for post-tensioning strands: [LRFD Table 5.9.3-1]

prior to seating, $f_s \leq 0.9f_{py}$ = 218.7 ksi

immediately after anchor set,

$(f_{pt} + \Delta f_{pES} + \Delta f_{pA}) \leq 0.7f_{pu}$ = 189 ksi

at end of the seating loss zone immediately after anchor set,

$(f_{pt} + \Delta f_{pES} + \Delta f_{pA}) \leq 0.74f_{pu}$ = 199.8 ksi

A maximum of three tendons, each with up to 15 strands, for a total of 45 strands, will be assumed.

P-T Tendon Duct:

Rigid galvanized steel duct with outside diameter 3.75-in.

Reinforcing bars:

Yield strength, f_y = 60 ksi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Future wearing surface:

An additional weight of 0.025 ksf for a future 2-in. concrete wearing surface is included. Unit weight, w_c = 0.150 kcf

New Jersey-type barriers:

Two weighing 0.300 kip/ft per barrier are assumed to be distributed equally to all beams

11.8.3 Cross-Section Properties

11.8.3.1 Non-Composite Section

Standard section properties for PCI BT-72 are modified to reflect the 1-in. increase in width.

A = cross-sectional area of beam = 767 + 72 = 839 in.^{2}

h = overall depth of beam = 72 in.

I = moment of inertia about the centroid of the non-composite precast beam = 577,022 in.^{4}

y_b = distance from centroid to extreme bottom fiber of the non-composite precast beam = 36.55 in.

y_t = distance from centroid to extreme top fiber of the non-composite precast beam = 35.45 in.

S_b = section modulus for the extreme bottom fiber of the non-composite precast beam = 15,789 in.^{3}

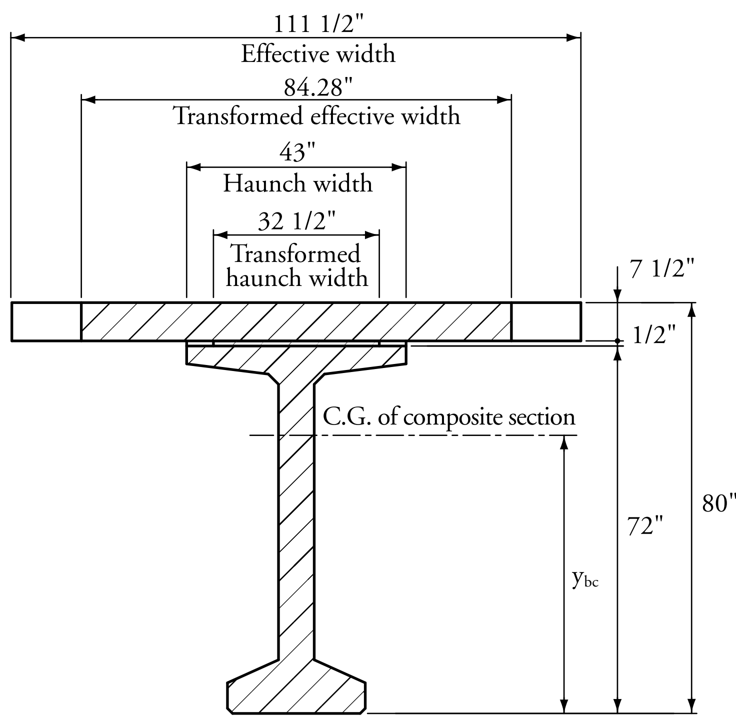
S_t = section modulus for the extreme top fiber of the non-composite precast beam = 16,276 in.^{3}

Beam weight = 0.874 kip/ft.

11.8.3.2
Composite Section

The composite section properties are calculated according to the *LRFD Specifications*. **Figure 11.8.3.2-1** shows the cross-section of the composite section.

Figure 11.8.3.2-1
Composite Transformed Section



n = modular ratio of deck and girder concretes = 3,834/5,072 = 0.7559

A_c = total area of composite section = 1,487 in.^{2}

h_c = overall depth of the composite section = 72 + 7.5 + 0.5 = 80 in.

I_c = moment of inertia of the composite section = 1,153,760 in.^{4}

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = $(\sum Ay_b / A_c) = 80{,}038/1{,}487 = 53.81$ in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 72 − 53.81 = 18.19 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the slab = 80 − 53.81 = 26.19 in.

S_{bc} = composite section modulus for the extreme bottom fiber of the precast beam = 1,153,760/53.81 = 21,441 in.^{3}

S_{tg} = composite section modulus for the top fiber of the precast beam = 1,153,760/18.19 = 63,428 in.^{3}

S_{tc} = composite section modulus of extreme top fiber of the slab =

$$\left(\frac{1}{n}\right)\left(\frac{I_c}{y_{tc}}\right) = \left(\frac{1}{0.7559}\right)\left(\frac{1,153,760}{26.19}\right) = 58,279 \text{ in.}^3$$

11.8.4 Shear Forces and Bending Moments

The weight of the beam acts on the non-composite, simple-span beam. The staging of post-tensioning (see Sect. 11.8.6.1) allows the weight of the slab and haunch to act on the non-composite, continuous span beam. The weight of the barriers and the future wearing surface, and the live load act on the composite, continuous-span beam.

The values of shear forces and bending moments for a typical interior beam, under self-weight of beam, weight of slab and haunch are computed, similar to Example 9.6. These are listed in **Table 11.8.4-1**. The two-span structure was analyzed using a continuous beam program that has the capability to generate live load shear force and bending moment envelopes for a "lane" of HL-93 live loading according to the *LRFD Specifications*. The span lengths used are the continuous bridge with span lengths of 155 ft.

Table 11.8.4-1
Unfactored Shear Forces and Bending Moments for a Typical Interior Beam

Distance X	Section	Girder Weight (Simple Span)		Slab+Haunch Weight (Continuous Span)		Barrier Weight (Continuous Span)		Wearing Surface (Continuous Span)		HL-93 Live Load Envelope (Continuous Span)	
		Shear	Moment M_g	Shear	Moment M_s	Shear	Moment M_b	Shear	Moment M_{ws}	Shear V_{LL+I}	Moment M_{LL+I}
ft	X/L	kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips	kips	ft-kips
0.0	0.000	67.7	0.0	72.0	0.0	8.7	0.0	15.3	0.0	146.2	1,981.7
15.5	0.100	54.2	944.9	53.0	969.6	6.4	117.1	11.2	205.3	122.1	3,398.6
31.0	0.200	40.6	1,679.8	33.9	1,641.1	4.1	198.2	6.7	347.5	99.7	4,274.2
46.5	0.300	27.1	2,204.8	14.1	2,014.5	1.7	243.3	3.1	426.5	79.4	4,663.8
62.0	0.400	13.5	2,519.7	5.0	2,089.0	0.6	252.3	1.1	442.3	61.1	4,585.9
77.5	0.500	0.0	2,624.7	24.0	1,864.7	2.9	225.2	5.1	394.9	78.8	4,079.0
93.0	0.600	13.5	2,519.7	43.1	1,343.0	5.2	162.2	9.2	284.3	97.6	3,146.6
108.5	0.700	27.1	2,204.8	62.9	522.5	7.6	63.1	13.3	110.6	116.8	–2,541.8
124.0	0.800	40.6	1,679.8	82.0	–597.0	9.9	–72.1	17.3	–126.3	136.3	–3,209.6
139.5	0.900	54.2	944.9	101.0	–2,014.5	12.2	–243.3	21.4	–426.5	155.8	–3,209.6
[1]147.9	0.954	61.5	458.0	111.8	–3,558.7	13.5	–429.8	23.6	–753.6	166.1	–4,279.9
155.0	1.000	67.7	0.0	120.1	–3,730.1	14.5	–450.5	25.4	–789.9	174.8	–4,455.4

Note: Shear is given in absolute values

[1] *Section designed in shear*

For all limit states except the Fatigue Limit State, for two or more lanes loaded, the distribution factor for moment (DFM) = 0.849 lanes/beam. For one design lane loaded, DFM = 0.550 lanes/beam. Therefore, the case of the two design lanes loaded controls.

For two or more lanes loaded, distribution factor for shear (DFV) = 1.082 lanes/beam [LRFD Table 4.6.2.2.3a-1]. For one design lane loaded, DFV = 0.840 lanes/beam. Therefore, the case of two design lanes loaded controls. Values of V_{LL+I} and M_{LL+I} at various points along the span are given in **Table 11.8.4-1**.

11.8.5 Required Pretensioning

The number of pretensioning strands is selected to resist at least 120 percent of the beam weight. This would allow for a slight camber at prestress transfer and for additional safety during handling and shipping.

Using the value of bending moment from **Table 11.8.4-1**, the bottom tensile stress at midspan (0.5L), due to 1.2 times beam weight is:

$$f_b = -\frac{1.2(2{,}624.7)(12)}{15{,}789} = -2.394\,\text{ksi}$$

Tensile stress limit at service loads = $-0.19\sqrt{f'_c} = -0.503$ ksi [LRFD Table 5.9.4.2.2-1]

The required precompressive stress at bottom fiber of the beam is the difference between bottom tensile stress due to the applied loads and the concrete tensile stress limit:

$f_{pb} = 2.394 - 0.503 = 1.891$ ksi.

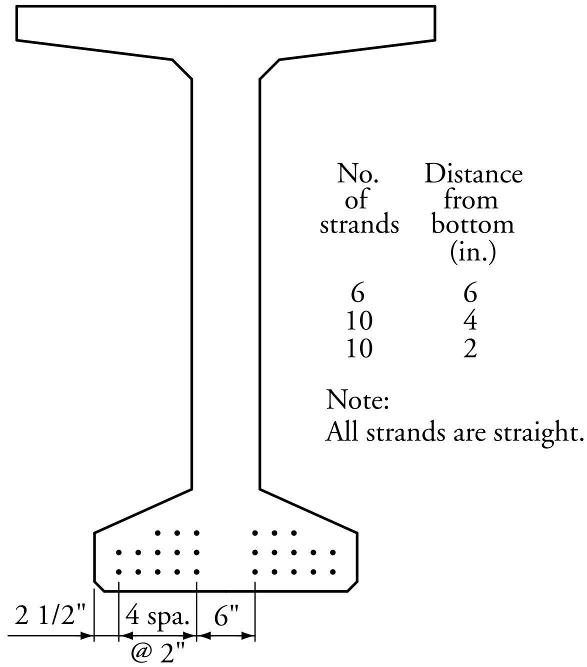
Similar to Example 9.6, assume the distance from the center of gravity of strands to the bottom fiber of the beam, y_{bs}, is equal to 7 percent of the beam depth, or, $y_{bs} = 0.07h = 0.07(72) = 5.04$ in. Then, strand eccentricity at midspan, e_c, equals $y_b - y_{bs} = 36.55 - 5.04 = 31.51$ in.

The minimum required effective prestress force, P_{pe}:

$$1.891 = \frac{P_{pe}}{839} + \frac{P_{pe}(31.51)}{15{,}789}$$

Therefore, $P_{pe} = 593.2$ kips. Assuming a total prestress loss of 25 percent, the prestress force per strand after all losses = (0.153)(202.5)(1 − 0.25) = 23.2 kips. The number of strands required is (593.2/23.2) = 25.6 strands. Use 26, ½-in.-dia, 270 ksi, low-relaxation strands. The assumed strand pattern for the 26 strands at midspan is shown in **Figure 11.8.5-1**. Each available location, with allowance for post-tensioning ducts, was filled beginning with the bottom row.

Figure 11.8.5-1
Pretensioning Strand Pattern at Midspan



The distance between the center of gravity of strands and the bottom fiber of the beam,

y_{bs} = [10(2) + 10(4) + 6(6)]/(26) = 3.69 in. Strand eccentricity at midspan, $e_c = y_b - y_{bs}$ = 36.55 – 3.69 = 32.86 in.

Before continuing with post-tensioning calculations, the designer should investigate if analysis is warranted for slender member stability (see Section 8.10) or for stresses at prestress transfer. In most cases, these two design considerations do not control.

11.8.6 Modeling of Post-Tensioning

In continuous structures, the moments due to post-tensioning may not be proportional to the tendon eccentricity. The difference occurs because the deformations imposed by the post-tensioning are resisted by the continuity of the members at the piers. The moments resulting from the restraint to the post-tensioning deformations are called secondary moments. Also, see Section 11.6.4.

11.8.6.1 Post-Tensioning Profile

The post-tensioning is applied in two stages. In the first stage, two of three equal tendons are post-tensioned before the beams are made composite with the deck. The second stage post-tensioning is applied through one tendon to the composite section. This two-thirds, one-third division of post-tensioning allows for the deck to be pre-compressed for crack control, yet not compressed enough to require extensive analysis for effects of future deck removal and replacement.

Stage 1:

Place two tendons with two-thirds of the total number of post-tensioning strands in the precast continuous member. Assume an initial post-tensioning force equal to 1,000 kips.

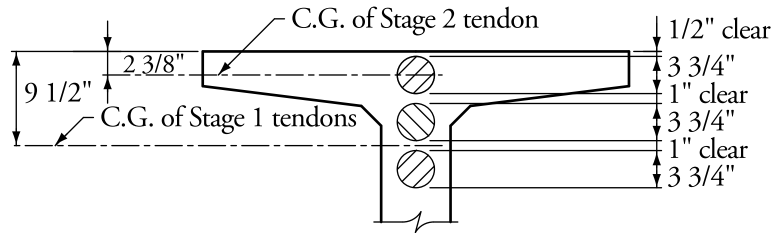
Stage 2:

Place one tendon with one-third of the total number of the post-tensioning strands in the composite member. Assume an initial post-tensioning force equal to 500 kips.

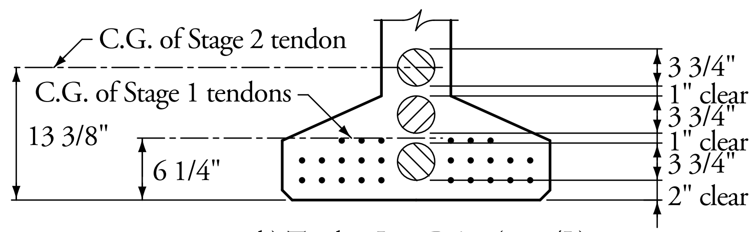
Once the total required post-tensioning force is determined based on various design criteria, the effects of the 1,500 kips (1,000 + 500) are linearly factored to correspond to the calculated force and analysis continues.

Figure 11.8.6.1-1 shows the positions of the tendons in a cross-section of the beam. Note that the clear spacing between ducts is taken as 1 in. This is a good practice as long as maximum aggregate size is not larger than 3/4 in. The *LRFD Specifications*, Article 5.10.3.3.2, states that up to three ducts may be bundled as long as they are splayed out in the anchorage area for 3 ft, at a spacing of 1.5 in. or 1.33 times the maximum aggregate size.

Figure 11.8.6.1-1
Duct Locations



a) Tendon High Point (at Interior Pier)



b) Tendon Low Point (at 0.4L)

Figure 11.8.6.1-2 shows the post-tensioning tendon profile for both stages. **Tables 11.8.6.2-1** and **11.8.6.2-2**, found in the next section, show tendon eccentricities at various locations. In a detailed analysis, the difference between the centroid of the tendon and the center of the ducts may be accounted for in accordance with LRFD Article C5.9.1.6. The difference between the two centers is due to the fact that the strands cluster near the top of the duct in the low segments of the duct profile and cluster near the bottom in the high, negative moment areas of the duct profile. This minor effect is ignored in the calculations of this example.

Figure 11.8.6.1-3 illustrates the equation used to calculate the eccentricity of the post-tension profile at any point of the span. For a tendon geometry to be fully defined, two conditions are required for a straight-line tendon and three conditions for a second-degree curve.

Figure 11.8.6.1-2
Post-Tensioning Tendon Profiles

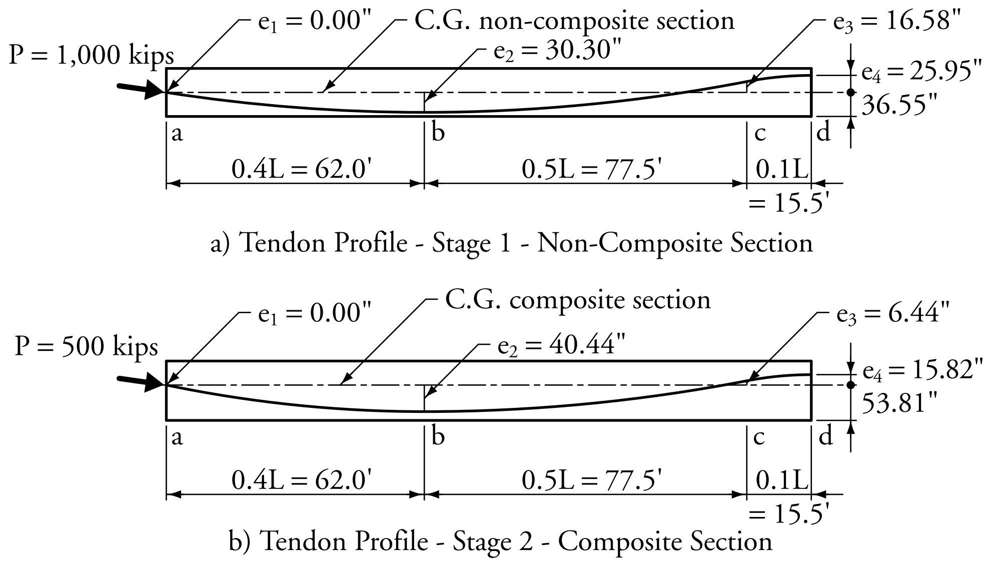
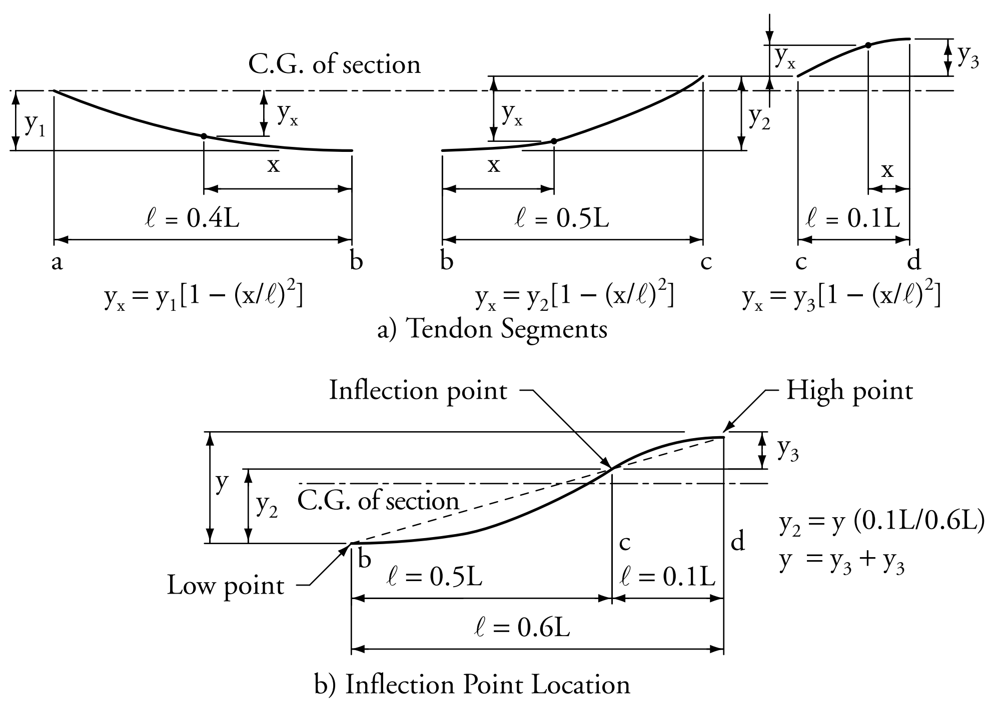


Figure 11.8.6.1-3
Eccentricity of the Tendon Profile



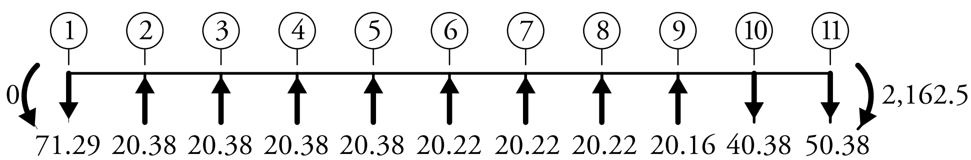
It is common practice is to assume a parabolic profile, defined by three parabolas in the end span of a continuous beam. The first has zero eccentricity at beam-end and has the maximum allowed bottom position at 0.4L with zero slope (or horizontal tangent) at that point. The second parabola has the same eccentricity and tangential slope at 0.4L and a common tangent and eccentricity as it joins the third parabola. The third parabola is a small curve dictated by the specification limits of tendon curvature. Generally, it has common eccentricity and is tangent with the second parabola. It has zero slope (horizontal tangent) over the pier centerline and the maximum possible eccentricity. The point of common tangent between the second and third parabolas has traditionally been taken as 0.1L from centerline of support. However, other locations should be examined in an optimization of the tendon profile. The assumptions made for the three parabolas allow the tendon geometry to be fully defined when the eccentricities at the abutment (i.e., 0.0L), 0.4L, 0.9L, and pier (i.e., 1.0L) are given.

For bridges with interior spans, similar assumptions may be made, namely, horizontal tangents at the ends and at 0.5L, and common tangents at 0.1L and 0.9L.

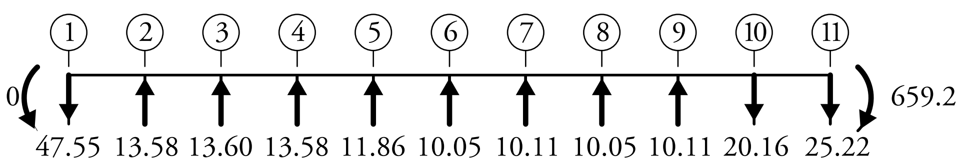
11.8.6.2 Equivalent Loads

When equivalent loads are placed on the continuous beam, and structural analysis is performed, the resulting moments, shears and reactions are the total effects. The secondary moments are the total moments minus the primary moments, which are defined as the products of the prestress force and the eccentricity at any given section. **Figure 11.8.6.2-1** shows the equivalent loads for the tendon profiles shown in **Figure 11.8.6.1-2**.

Figure 11.8.6.2-1 Post-Tensioning Equivalent Vertical Loads and Moments (Refined Method)



a) Stage 1 Tendons - P = 1,000 kips
(Force - kips, Moment - ft-kip)



b) Stage 2 Tendon - P = 500 kips
(Force - kips, Moment - ft-kip)

Table 11.8.6.2-1 shows the total moments, primary moments and secondary moments at tenth-span points due to Stage 1 post-tensioning. **Table 11.8.6.2-2** shows the same quantities for Stage 2 post-tensioning.

Table 11.8.6.2-1
Loads Applied by Post-Tensioning in Stage 1 (P = 1,000 kips)

Point along Span	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Distance from Top of Girder, in.	35.45	48.71	58.18	63.86	65.75	63.88	58.25	48.88	35.75	18.88	9.50
Tendon Eccentricity, in.	0.00	–13.26	–22.73	–28.41	–30.30	–28.43	–22.80	–13.43	–0.30	16.58	25.95
Equivalent Loads:											
Vertical Force, kips	71.29	–20.38	–20.38	–20.38	–20.22	–20.22	–20.22	–20.22	–20.16	40.38	50.38
Moment, ft-kips	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2,162.5
Total Moment, ft-kips	0	–1,013.6	–1,711.3	–2,093.2	–2,159.3	–1,912.0	–1,351.4	–479.2	706.4	2,204.5	3,076.8
Primary Moment, ft-kips	0	–1,105.0	–1,894.2	–2,367.5	–2,525.0	–2,369.2	–1,900.0	–1,119.2	–25.0	1,381.7	2,162.5
Secondary Moment, ft-kips	0	91.4	182.9	274.3	365.7	457.2	548.6	640.0	731.4	822.8	914.3

Table 11.8.6.2-2
Loads Applied by Post-Tensioning in Stage 2 (P = 500 kips)

Point along Span	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Distance from Top of Girder, in.	18.19	35.88	48.52	56.10	58.63	56.75	51.13	41.75	28.63	11.75	2.38
Tendon Eccentricity, in.	0.00	–17.69	–30.33	–37.91	–40.44	–38.56	–32.94	–23.56	–10.44	6.44	15.82
Equivalent Loads:											
Vertical Force, kips	47.55	–13.58	–13.60	–13.58	–11.86	–10.05	–10.11	–10.05	–10.11	20.16	25.22
Moment, ft-kips	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	659.0
Total Moment, ft-kips	0	–629.8	–1,049.2	–1,257.8	–1,256.0	–1,071.4	–728.9	–230.8	423.1	1,233.7	1,731.8
Primary Moment, ft-kips	0	–737.1	–1,263.8	–1,579.6	–1,685.0	–1,606.7	–1,372.5	–981.7	–435.0	268.3	659.2
Secondary Moment, ft-kips	0	107.3	214.6	321.8	429.0	536.3	643.6	750.9	858.1	965.4	1,072.6

11.8.7 Required Post-Tensioning

At this stage of analysis, the post-tensioning forces are not yet known and prestress losses must be initially assumed. **Table 11.8.7-1** shows the assumed prestressing levels at each of the construction stages.

Table 11.8.7-1
Assumed Effective Prestress at Various Construction Stages

Construction Stage	Stress in Pretensioning Strand	Stress in Post-Tensioning Strand, Stage 1	Stress in Post-Tensioning Strand, Stage 2
Pretensioning	$0.92(0.75)f_{pu}$ = 186.3 ksi	–	–
Post-Tensioning Stage 1	$0.87(0.75)f_{pu}$ = 176.2 ksi	$0.92(0.78)f_{pu}$ = 193.8 ksi	–
Post-Tensioning Stage 2	$0.87(0.75)f_{pu}$ = 176.2 ksi	$0.87(0.78)f_{pu}$ = 183.2 ksi	$0.92(0.78)f_{pu}$ = 193.8 ksi
Service Loads	$0.82(0.75)f_{pu}$ = 166.1 ksi	$0.82(0.78)f_{pu}$ = 172.7 ksi	$0.82(0.78)f_{pu}$ = 172.7 ksi

Elastic pretension loss, the total loss at time of post-tensioning and the total loss at final service time are assumed to be 8, 13 and 18 percent, respectively, of initial prestress which is assumed to be 75 percent of the specified ultimate strength. Once the prestress forces are determined, primarily based on concrete tension limits at service load conditions, then a detailed analysis of prestress loss should be conducted and the prestress force revised if needed. The process of calculating prestress losses is covered in detail in Chapters 8 and 9.

The pretensioning forces are:

Immediately following transfer,

26(0.153)(186.3) = 741.1 kips

At Stage 1 (assumed to be the same as at Stage 2 post-tensioning),

26(0.153)(176.2) = 700.8 kips

At service,

26(0.153)(166.1) = 660.5 kips

Several factors affect post-tensioning losses. In addition to friction and anchor set described in Section 11.6.2, post-tensioning at any stage affects the prestressing tendons stressed in preceding stages. At this stage of analysis, post-tensioning losses are estimated, as given in **Table 11.8.7-1**, to be verified later with a detailed analysis after the post-tensioning forces are finalized.

Using 0.6-in.-dia strand (area per strand = 0.217 in.^{2}), the post-tensioning force per strand is: For Stage 1 strands, following jacking, 193.8(0.217) = 42.04 kips, and at the time of Stage 2 post-tensioning, 39.76 kips, and at service = 37.47 kips. For Stage 2 strands, immediately following jacking, 42.04 kips, and at service, 37.47 kips.

11.8.7.1 Stress Limits for Concrete

[LRFD Art. 5.9.4.2]

The concrete compressive stress limit for the Service I load combination due to weight of beam, slab, future wearing surface and barriers, is $0.45f'_c$. For the precast beam alone, the limit is 0.45(7.0) = +3.15 ksi, and for the slab, 0.45(4.0) = +1.80 ksi. Due to dead loads plus live loads, for Service I load combination, the limit is $0.60f'_c$, or +4.200 ksi for the precast beam and +2.400 ksi for the slab. For the Service III load combination, the tension limit is $-0.19\sqrt{f'_c} = -0.19\sqrt{7.0} = -0.503$ ksi.

The post-tensioning is calculated to satisfy tensile stresses at final service conditions due to full loads. The maximum negative section at the pier and the maximum positive moment section at 0.4L from the abutment are the two sections used for this analysis. Once the amount of post-tensioning is estimated, a detailed prestress loss calculation is made and all other design criteria are verified, the post-tensioning is adjusted as needed.

11.8.7.2 Positive Moment Section

The values of the bending moments due to various cases of loading are given in **Table 11.8.4-1**. The critical positive moment section is assumed to be at 0.4L. The moments shown are:

M_g, due to beam weight = 2,519.7 ft-kips

M_s, due to deck weight = 2,089.0 ft-kips

M_b, due to barrier weight = 252.3 ft-kips

M_{ws}, due to wearing surface weight = 442.3 ft-kips

M_{LL+I}, due to live load and impact = 4,585.9 ft-kips

The pretensioning force at 0.4L, P_{pe} = 660.5 kips, and its eccentricity is 32.86 in. The total moments due to post-tensioning are shown in **Tables 11.8.6.2-1** and **11.8.6.2-2**, for assumed values of Stage 1 and Stage 2 post-tensioning. Allowing for prestress losses:

$$\text{Stage 1 total moment} = \left(\frac{2(n)37.47}{1{,}000}\right)(-2{,}159.3)$$

$$= (0.075)(n)\ (-2{,}159.3) = -161.95(n)\ \text{ft-kips}$$

$$\text{Stage 2 total moment} = \frac{(n)37.47}{500}(-1{,}256.0)$$

$$= -94.20(n)\ \text{ft-kips}$$

Where (n) is the number of 0.6-in.-dia strands per tendon. Stage 1 has two post-tensioned tendons, i.e., 2(n) strands, and stage 2 has one tendon, i.e., (n) strands.

The bottom fiber stress due to each of the effects is given as:

$$-\frac{(M_g + M_s)}{S_b} - \frac{(M_{ws} + M_b) + 0.8M_{LL+I}}{S_{bc}} =$$

$$-\frac{(2{,}519.7 + 2{,}089.0)(12)}{15{,}789} - \frac{(442.3 + 252.3)(12) + 0.8(4{,}585.9)(12)}{21{,}441} = -5.945\ \text{ksi}$$

$$\frac{(P_{pe})}{A} + \frac{(P_{pe})e}{S_b} = \frac{660.5}{839} + \frac{660.5(32.86)}{15{,}789} = +0.787 + 1.375 = +2.162\ \text{ksi}$$

$$\frac{(P_{PT})_{Stage\,1}}{A} + \frac{(M_{Total})_{Stage\,1}}{S_b} = \frac{2(n)(37.47)}{839} + \frac{161.95(n)(12)}{15{,}789} = +0.212(n)\ \text{ksi}$$

$$\frac{(P_{PT})_{Stage\,2}}{A_c} + \frac{(M_{Total})_{Stage\,2}}{S_{bc}} = \frac{(n)(37.47)}{1{,}487} + \frac{(n)(94.20)(12)}{21{,}441} = +0.078(n)\ \text{ksi}$$

Therefore, the total stress is = −5.945 + 2.162 + 0.212(n) + 0.078(n) ksi

By setting this value equal to the stress limit, −0.503 ksi, a value of n = 12 strands per tendon, or a total of 36 strands total, is found to be required.

11.8.7.3 Negative Moment Section

The section at the centerline of the pier will be used for analysis of negative moment stresses. For the refined analysis, the critical section should be selected at the face of the diaphragm. It is interesting to note here that the requirement for checking tensile stresses in the negative moment zone is not enforced by designers of bridges made continuous without post-tensioning (using conventional reinforcement in the deck slab). This added check has good value in that it controls top cracking in the pier area and therefore increases the performance of the bridge. The claim can be made that the owner is receiving higher value by using a post-tensioning system compared to a conventionally-reinforced one.

From **Table 11.8.4-1**, the values of the bending moments are:
M_g = 0.0 ft-kips; M_s = −3,730.1 ft-kips; M_b = −450.5 ft-kips;
M_{ws} = −789.9 ft-kips; M_{LL+I} = −4,455.4 ft-kips

The post-tensioning force at this section is:

Stage 1, P_{PT} = 74.94(n) kips. The total moment per 1,000 kips of post-tensioning force = 3,076.8 ft-kips. This corresponds to (0.075)(n)(3,076.8) = 230.76(n) ft-kips, where (n) is the number of strands per tendon in the two-tendon Stage 1 post-tensioning.

Stage 2, P_{PT} = 37.47(n) kips. The total moment per 500 kips of Stage 2 post-tensioning = 1,731.8ft-kips. This corresponds to (0.075)(n)(1,731.8) = 129.89(n) ft-kips.

Solving for the total stress at the top fibers of the beam:

$$-\frac{(M_g + M_s)}{S_t} - \frac{(M_{ws} + M_b) + 0.8M_{LL+I}}{S_{tc}} + \frac{(P_{PT})_{Stage\,1}}{A} + \frac{(M_{Total})_{Stage\,1}}{S_t}$$

$$+\frac{(P_{PT})_{Stage\,2}}{A_c} + \frac{(M_{Total})_{Stage\,2}}{S_{tc}}$$

$$= -2.750 - 0.909 + \frac{74.94(n)}{839} + \frac{230.76(n)(12)}{16,276} + \frac{37.47(n)}{1,487} + \frac{129.89(n)(12)}{63,428}$$

$$= -3.659 + (0.089 + 0.170 + 0.025 + 0.025)(n) = -0.503 \text{ ksi}$$

Solving for (n), the minimum number of 0.6-in.-dia strands per post-tensioning tendon is 10.21, or, rounding, a total of 33 strands for the three tendons.

The positive moment section requires three more post-tensioning strands than the negative moment section. In order to optimize, the P/T required for the negative moment region will be used for the entire beam. An attempt will be made to increase the positive moment capacity by adding pretensioning. However, due to the limitations of the AASHTO-PCI Bulb Tee, the maximum number of strands that can be placed in the bottom flange, outside of the web area (which is reserved for post-tensioning), is 28. So, the design will be attempted using 28, ½-in.-dia pretensioning strands and 3 post-tensioning tendons of 11, 0.6-in.-dia strands.

The pretensioning force at this section, P_{pe} = 711.2 kips. Strand eccentricity at midspan, $e_c = y_b - y_{bs}$ = 36.55 − [(10(2) + 10(4) + 6(6) + 2(8)]/(28) = 32.55 in.

$$\text{Positive moment stress} = -5.945 + \frac{711.2}{839} + \frac{711.2(32.55)}{15,789} + 0.212(11) + 0.078(11)$$

$$= -5.945 + 0.848 + 1.466 + 3.190 = -0.441 \text{ ksi}$$

Negative moment stress = −3.659 + (0.089+0.170 + 0.025 + 0.025)(11) = −0.260 ksi

Both values are within −0.503 ksi allowable tension for load combination, Service III.

11.8.8 Prestress Losses

11.8.8.1 Prediction Method

As previously stated, the recommended method for calculation of prestress losses is the one given in this manual in Section 8.6. It more accurately accounts for level of prestress, concrete strength and environmental conditions than the Detailed Method of the *LRFD Specifications*. It also allows for the long-term losses to be broken into increments representing the significant construction stages present in this type of superstructure. All calculations are made for the maximum positive moment section and the resulting elastic shortening and long-term losses are assumed to be constant along the entire length of the member.

11.8.8.2 Time-Dependent Material Properties

For the calculation of prestress losses, the bridge is assumed to be located where the average ambient relative humidity is 70 percent. The following construction schedule has been assumed:

- Pretensioning is transferred one day after beam concrete placement
- Stage 1 post-tensioning occurs at 30 days
- Deck slab concrete is placed at 60 days
- Stage 2 post-tensioning and superimposed dead loads are applied shortly after the deck is placed

To simplify the time-dependent calculations, it is reasonable to calculate the creep and shrinkage coefficients for three time periods: 1 to 30 days, 30 to 60 days and 60 to 20,000 days. The selection of 20,000 days is arbitrary and represents a bridge life of 55 years. Creep and shrinkage essentially cease after several years so this assumption is inconsequential. It is further assumed that no time-dependent effects take place between deck placement, Stage 2 post-tensioning, and application of superimposed dead load.

Using the procedures of Section 8.6.5, the beam creep coefficient for a loading age of one day and a loading duration of 29 days is 0.691. Assuming a loading age of 30 applied to all concrete stress components introduced between 1 and 30 days, and a loading duration of 30 days (60 − 30), the creep coefficient is 0.507. For concrete loaded at 60 days and for a loading duration of 20,000 days (≈ 20,000 − 60), the creep coefficient is 1.00. The corresponding shrinkage strains of the beam are 143×10^{-6}, 65×10^{-6}, and 167×10^{-6} in./in. The strand relaxation is a minor contributor to the prestress losses and it is assumed equal to 1.2 ksi between 1 and 30 days, 1.2 ksi between 30 and 60 days, and zero thereafter. The modulus of elasticity was calculated earlier. These are E_c (deck) = 3,834 ksi; E_c (beam) at one day = 4,496 ksi and at 30 days (assumed the same as at 28 days) = 5,072 ksi.

11.8.8.3 Loss Increments

At day one, elastic shortening loss is calculated for pretensioning and beam self-weight. The initial prestress, just before transfer is assumed equal to $0.75f_{pu}$ = 202.5 ksi. When the corresponding force and the self-weight moment are introduced to a transformed precast concrete section, with the steel area transformed by the factor, $n_i = E_p/E_{ci}$, the resulting concrete stress is the true stress in the section. The concrete stress at the centroid of the steel, multiplied by the modular ratio results in an elastic shortening loss = 5.458 ksi.

The concrete stress at the centroid of the pretensioning steel is used to calculate the creep loss between 1 and 30 days. With the relaxation loss added, the total long-term loss between 1 and 30 days is 7.987 ksi.

The concrete stress at the centroid of the pretensioning steel due to Stage 1 post-tensioning allows determination of elastic loss due to that stage of post-tensioning. The post-tensioning force is the applied load. The stress in the post-tensioning steel is calculated with due consideration of friction losses as given in Section 11.6.2 to be 187.18 ksi. The corresponding force is introduced to a transformed precast section with the pretensioning steel area transformed by the factor $n = E_p/E_c$. The resulting elastic shortening loss is 13.744 ksi. The net concrete stress at the pretensioning steel level immediately after application of Stage 1 post-tensioning is then used to calculate creep loss between 30 and 60 days. The corresponding long-term loss in the pretensioning steel is 8.922 ksi.

The elastic gain due to deck weight is calculated using the same section properties as for the Stage 1 post-tensioning. It is found to be = 7.102 ksi.

Transformed composite section properties are used for all effects that follow deck placement. The deck slab is transformed to beam concrete using E_c(deck)/E_c(beam). The pretensioning steel and the post-tensioning steel are transformed using E_p/E_c(beam). The elastic loss due to Stage 2 post-tensioning, and the elastic gain due to superimposed dead load are calculated and the net concrete stress at centroid of pretensioning steel is determined. Combined with the creep and shrinkage properties between 60 days and 20,000 days, the long-term loss is calculated. It is found to be 17.682 ksi.

Similar calculations are carried out for post-tensioning Stage 1, except that concrete stresses are calculated at the centroid of the post-tensioning steel at that stage. Also, note that the transformed section should not include the area of the post-tensioning steel until after that steel is anchored to the concrete and grouted. For purposes of loss calculations, grouting is assumed to be completed immediately after a tendon is post-tensioned.

Stresses in the prestressed reinforcement are summarized in **Table 11.8.8.3-1** and are plotted in **Figure 11.8.8.3-1**.

Figure 11.8.8.3-1
Stresses in the Prestressed Reinforcement

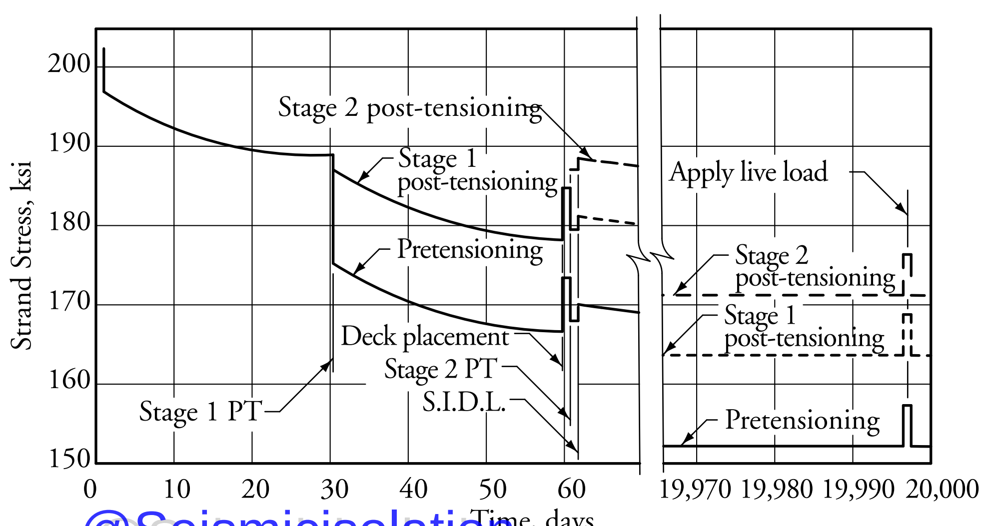


Table 11.8.8.3-1
Prestress Levels at Loading Stages using PCI Loss Methods

Loading	Construction Schedule, days	Stress, ksi		
		Pretensioning	Stage 1 Post-Tensioning	Stage 2 Post-Tensioning
Pretensioning plus Girder Weight	1	197.042	–	–
Long-Term	1-30	189.055	–	–
Stage 1 Post-Tensioning	30	175.311	187.18	–
Long-Term	30-60	166.389	178.276	–
Deck Weight	60	173.490	184.861	–
Stage 2 Post-Tensioning	60	168.041	179.59	187.18
Superimposed Dead Load	60	169.822	181.287	188.618
Long-Term	60-20,000	152.140	163.646	171.245
Live Load	20,000	163.911	174.867	180.724
80% Live Load (Service III)	20,000	161.557	172.623	178.828

11.8.9 Service Limit State at Section 0.4L

11.8.9.1 Stress Limits for Concrete

[LRFD Art. 5.9.4.2]

Compression:

Due to Service I, permanent load combination:

for the precast beam: $0.45f'_c = 0.45(7.0) = +3.15$ ksi

for the deck slab: $0.45f'_c = 0.45(4.0) = +1.80$ ksi

Due to Service I, full load combination:

for the precast beam: $0.60f'_c = 0.60(7.0) = +4.200$ ksi

for the deck slab: $0.60f'_c = 0.60(4.0) = +2.400$ ksi

Tension:

For Service III, full load combination:

for the precast beam: $-0.19\sqrt{f'_c} = -0.19\sqrt{7.0} = -0.503$ ksi

The conditions at the time of Stage 1 and Stage 2 post-tensioning are considered temporary and tension and compression limits should be the same as for the conditions of Service III.

11.8.9.2 Stage 1 Post-Tensioning

The maximum positive moment section, located at 0.4L, is checked.

- The pretensioning force, $P_{pe} = 28(0.153)(175.311) = 751.032$ kips
- Post-tensioning, Stage 1, $P_{PT} = 22(0.217)(187.18) = 893.597$ kips

$$f_{tg} = \frac{(M_g + M_s)}{S_t} + \frac{P_{pe}}{A} - \frac{P_{pe}(e)}{S_t} + \frac{(P_{PT})_{Stage\,1}}{A} - \frac{(M_{Total})_{Stage\,1}}{S_t}$$

$$= +\frac{(2{,}519.7 + 2{,}089.0)12}{16{,}276} + \frac{751.032}{839} - \frac{751.032(32.55)}{16{,}276}$$

$$+\frac{893.597}{839} - \frac{893.597(2{,}159.3)}{1{,}000}\frac{(12)}{16{,}276}$$

$$= +3.398 + 0.895 - 1.502 + 1.065 - 1.424 = +2.432 \text{ ksi} < +3.15 \text{ ksi} \quad \text{OK}$$

$$f_b = -\frac{(M_g + M_s)}{S_b} + \frac{P_{pe}}{A} - \frac{P_{pe}(e)}{S_b} + \frac{(P_{PT})_{Stage\,1}}{A} + \frac{(M_{Total})_{Stage\,1}}{S_b}$$

$$= -\frac{(2{,}519.7 + 2{,}089.0)(12)}{15{,}789} + \frac{751.032}{839} - \frac{751.032(32.55)}{15{,}789}$$

$$+\frac{893.597}{839} + \frac{893.597(2{,}159.3)}{1{,}000}\frac{(12)}{15{,}789}$$

$$= -3.503 + 0.895 + 1.548 + 1.065 + 1.466 = +1.471 \text{ ksi} < +3.15 \text{ ksi} \quad \text{OK}$$

11.8.9.3 Stage 2 Post-Tensioning

The maximum positive moment section, located at 0.4L, is checked.

Immediately after Stage 2 post-tensioning, the following prestressing forces act on the cross-section:

Pretensioning force, $P_{pe} = 28(0.153)(168.04) = 719.883$ kips

Post-tensioning, Stage 1, $P_{PT} = 22(0.217)(179.59) = 857.363$ kips

Post-tensioning, Stage 2, $P_{PT} = 11(0.217)(187.18) = 446.8$ kips

$$f_{tg} = \frac{(M_g + M_s)}{S_t} + \frac{P_{pe}}{A} - \frac{P_{pe}(e)}{S_t} + \frac{(P_{PT})_{Stage\,1}}{A} + \frac{(M_{Total})_{Stage\,1}}{S_t}$$

$$+\frac{(P_{PT})_{Stage\,2}}{A_c} + \frac{(M_{Total})_{Stage\,2}}{S_{tg}}$$

$$= \frac{(2{,}519.7 + 2{,}089.0)(12)}{16{,}276} + \frac{719.883}{839} - \frac{719.883(32.55)}{16{,}276}$$

$$+\frac{857.363}{839} - \frac{857.363(2{,}159.3)}{1{,}000}\frac{(12)}{16{,}276} + \frac{446.8}{1{,}487} + \frac{446.8(1{,}256.0)}{500}\frac{(12)}{63{,}428}$$

$$= +3.398 + 0.858 - 1.440 + 1.022 - 1.365 + 0.300 - 0.212$$

$$= +2.561 \text{ ksi} < +3.15 \text{ ksi} \quad \text{OK}$$

$$f_b = -\frac{(M_g + M_s)}{S_b} + \frac{P_{pe}}{A} + \frac{P_{pe}(e)}{S_b}$$

$$+\frac{(P_{PT})_{Stage\,1}}{A} + \frac{(M_{Total})_{Stage\,1}}{S_b} + \frac{(P_{PT})_{Stage\,2}}{A_c} + \frac{(M_{Total})_{Stage\,2}}{S_{bc}}$$

$$= -\frac{(2{,}519.7 + 2{,}089.0)(12)}{15{,}789} + \frac{719.883}{839} + \frac{719.883(32.55)}{15{,}789}$$

$$+\frac{857.363}{839} + \frac{857.363(2{,}159.3)}{1{,}000}\frac{(12)}{15{,}789} + \frac{446.8}{1{,}487} + \frac{446.8(1{,}256.0)}{500}\frac{(12)}{21{,}441}$$

$$= -3.503 + 0.858 + 1.484 + 1.022 + 1.407 + 0.300 + 0.628$$

$$= +2.196 \text{ ksi} < +3.15 \text{ ksi} \quad \text{OK}$$

11.8.9.4 Compression Due to Service I Loads

To check compressive stress at the top fiber of the beam, two cases are checked:

1. Under permanent load, Service I:

At long term without the live load effect, the following prestressing forces act on the cross-section:

- Pretensioning force, $P_{pe} = 28(0.153)(152.14) = 651.768$ kips
- Post-tensioning, Stage 1, $P_{PT} = 22(0.217)(163.646) = 781.246$ kips
- Post-tensioning, Stage 2, $P_{PT} = 11(0.217)(171.245) = 408.762$ kips

Using bending moment values given in **Table 11.8.4-1**, concrete stress at the top fiber of the beam is:

$$f_{tg} = +\frac{(M_g + M_s)}{S_t} + \frac{(M_{ws} + M_b)}{S_{tg}} + \frac{P_{pe}}{A} - \frac{P_{pe}(e)}{S_t} + \frac{(P_{PT})_{Stage\,1}}{A} - \frac{(M_{Total})_{Stage\,1}}{S_t}$$

$$+\frac{(P_{PT})_{Stage\,2}}{A_c} - \frac{(M_{Total})_{Stage\,2}}{S_{tg}}$$

$$= +\frac{(2{,}519.7 + 2{,}089.0)(12)}{16{,}276} + \frac{(442.3 + 252.3)(12)}{63{,}428} + \frac{651.768}{839} - \frac{651.768\,(32.55)}{16{,}276}$$

$$+\frac{781.246}{839} - \frac{781.246(2{,}159.3)}{1{,}000}\frac{(12)}{16{,}276} + \frac{408.762}{1{,}487} - \frac{408.762(1{,}256.0)}{500}\frac{(12)}{63{,}428}$$

$$= 3.398 + 0.131 + 0.777 - 1.303 + 0.931 - 1.244 + 0.275 - 0.194$$

$$= +2.771 \text{ ksi} < +3.15 \text{ ksi} \quad \text{OK}$$

11.8.9.5 Tension Due to Service III Loads

At long term with 80 percent live load effect, the following prestressing forces act on the cross-section:

Pretensioning force, $P_{pe} = 28(0.153)(161.557) = 692.110$ kips

Post-tensioning, Stage 1, $P_{PT} = 22(0.217)(172.623) = 824.102$ kips

Post-tensioning, Stage 2, $P_{PT} = 11(0.217)(178.828) = 426.862$ kips

$$f_b = -\frac{(M_g + M_s)}{S_b} - \frac{(M_{ws} + M_b) + 0.8(M_{LL+I})}{S_{bc}}$$

$$+\frac{P_{pe}}{A} - \frac{P_{pe}(e)}{S_b} + \frac{(P_{PT})_{Stage1}}{A} + \frac{(M_{Total})_{Stage1}}{S_b} + \frac{(P_{PT})_{Stage2}}{A_c} + \frac{(M_{Total})_{Stage2}}{S_{bc}}$$

$$= -\frac{(2{,}519.7 + 2{,}089.0)(12)}{15{,}789} - \frac{(442.3 + 252.3 + 0.8(4{,}585.9))(12)}{21{,}441}$$

$$+\frac{692.110}{839} + \frac{692.110(32.55)}{15{,}789} + \frac{824.102}{839} + \frac{824.102(2{,}159.3)}{1{,}000}\frac{(12)}{15{,}789}$$

$$+\frac{426.862}{1{,}487} + \frac{426.862(1{,}256.0)}{500}\frac{(12)}{21{,}441}$$

$$= -3.503 - 2.442 + 0.825 + 1.427 + 0.982 + 1.352 + 0.287 + 0.600$$

$$= -0.472 \text{ ksi} > -0.503 \text{ ksi} \quad \text{OK}$$

The above process should be repeated for several sections along the span, usually 1/10th span points.

11.8.10 Stresses at Transfer of Pretensioning Force

11.8.10.1 Stress Limits for Concrete

[LRFD Art. 5.9.4.1]

Compression: $0.6f'_{ci} = 0.6(5.5) = 3.300$ ksi

Tension without bonded auxiliary reinforcement:
$-0.0948\sqrt{f'_{ci}} = -0.0948\sqrt{5.5} = -0.222 \text{ ksi} \leq -0.200 \text{ ksi}$

Therefore, –0.200 ksi controls.

Tension with bonded auxiliary reinforcement which is sufficient to resist the tension force in the concrete: $-0.24\sqrt{f'_{ci}} = -0.24\sqrt{5.5} = -0.563$ ksi

11.8.10.2 Stresses at Transfer Length Section

Stresses at the end of the transfer length must be checked at time of transfer. This stage usually governs design. However, the magnitude of losses will lessen with time, rendering the concrete stresses less critical.

$P_i = (28)(0.153)(189.055) = 809.911$ kips

Transfer length = 60(Strand diameters) = 60(0.5) = 30 in. = 2.5 ft [LRFD Art. 5.8.2.3]

Bending moment at a distance 2.5 ft from the end of the beam due to beam self-weight: M_g = (0.5)(0.874)(2.5)(155 – 2.5) = 166.6 ft-kips.

Stress at the top fiber of the beam:

$$f_t = \frac{P_i}{A} - \frac{P_i e_c}{S_t} + \frac{M_g}{S_t} = \frac{809.911}{839} - \frac{(809.911)(32.55)}{16,276} + \frac{(166.6)(12)}{16,276}$$

$$= 0.965 - 1.620 + 0.123 = -0.532 \text{ ksi} > -0.563 \text{ ksi} \quad \text{OK}$$

Stress at the bottom fiber of the beam:

$$f_b = \frac{P_i}{A} + \frac{P_i e}{S_b} - \frac{M_g}{S_b} = \frac{809.911}{839} + \frac{809.911(32.55)}{15,789} - \frac{(166.6)(12)}{15,789}$$

$$= 0.965 + 1.670 - 0.127 = +2.508 \text{ ksi} < 3.300 \text{ ksi} \quad \text{OK}$$

Tensile stress does not satisfy the limit for concrete with bonded reinforcement (–0.516 ksi). If four strands are debonded at the end of the member for six feet, the limit can be shown to be satisfied. Adding crack control conventional reinforcement at the top of the precast beam is required to satisfy the *LRFD Specifications*. Refer to Chapters 8 and 9 of this manual for additional details. Compressive stress is within the limit of +3.300 ksi.

11.8.10.3 Stresses at Midspan

Bending moment due to the weight of the beam at midspan:
M_g = (0.5)(0.874)(77.5)(155 – 77.5) = 2,624.7 ft-kips

Concrete stress at the top fiber of the beam:

$$f_t = \frac{809.911}{839} - \frac{809.911(32.55)}{16,276} + \frac{(2,624.7)(12)}{16,276}$$

$$= 0.965 - 1.620 + 1.935 = +1.280 \text{ ksi} \quad \text{OK}$$

Concrete stress at the bottom fiber of the beam:

$$f_b = \frac{809.911}{839} + \frac{809.911(32.55)}{15,789} - \frac{(2,624.7)(12)}{15,789}$$

$$= 0.965 + 1.670 - 1.995 = +0.640 \text{ ksi} \quad \text{OK}$$

11.8.11 Strength Limit State

11.8.11.1 Positive Moment Section

Using the values of bending moments from **Tables 11.8.4-1**, **11.8.6.2-1** and **11.8.6.2-2**, total ultimate bending moment for Strength I is:

For Stage 1 post-tensioning, $M_{secondary} = \left(\frac{822.08}{1,000}\right)(365.7) = 300.6$ ft-kips

For Stage 2 post-tensioning, $M_{secondary} = \left(\frac{411.04}{500}\right)(429.0) = 352.7$ ft-kips

$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL{+}IM) + 1.0(M_{secondary})$

[LRFD Tables 3.4.1-1&2]

$M_u = 1.25(2{,}519.7 + 2{,}089.0 + 252.3) + 1.5(442.3) + 1.75(4{,}585.9) + 1.0(300.6 + 352.7)$
$= 15{,}418.3$ ft-kips

At this section, there are three layers of prestressing steel, as shown in **Table 11.8.11.1-1**.

Table 11.8.11.1-1
Prestressing Steel at 0.4L

Layer	A_{ps}, in.^{2}	Distance from Bottom of Section, in.
Pretensioned Strands	4.284	4.0
Post-Tensioning Stage 1	4.774	6.25
Post-Tensioning Stage 2	2.387	13.375
Total	11.445	–

This example used the strain compatibility approach to calculate the capacity of the section. For detailed information on the method, please refer to Chapter 8. The *LRFD Specifications* approximate formulas are not recommended in this type of application.

Using the strain compatibility method, the following results were found:

The distance from top of the composite section to the neutral axis, $c = 9.966$ in.

The depth of the rectangular stress block, $a = 8.3634$ in.

The nominal moment capacity, $M_n = 17{,}504.87$ ft-kips

Average stress in the pretensioning steel, $f_{ps} = 267.98$ ksi

Average stress in the post-tensioning steel, Stage 1, $f_{ps} = 267.88$ ksi

Average stress in the post-tensioning steel, Stage 2, $f_{ps} = 265.96$ ksi

Factored flexural resistance, $M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where

ϕ = resistance factor = 1.00, [LRFD Art. 5.5.4.2.1]
for flexure and tension of prestressed concrete

$M_r = 17{,}504.87$ ft-kips $> M_u = 15{,}418.3$ ft-kips OK

11.8.11.2
Negative Moment Section

Ignoring the width of the pier diaphragm and using the values of bending moments from **Table 11.8.4-1**, **Table 11.8.6.2-1** and **Table 11.8.6.2-2**, the total factored bending moment for Strength I is:

For Stage 1 post-tensioning, $M_{secondary} = \left(\dfrac{822.08}{1,000}\right)(914.3) = 751.6$ ft-kips

For Stage 2 post-tensioning, $M_{secondary} = \left(\dfrac{411.04}{500}\right)(1,072.6) = 881.8$ ft-kips

The ultimate moment is computed using load factors found in LRFD Tables 3.4.1-1 & 2. The load factor for secondary moments is determined by the factor for "EL" defined in LRFD Article 3.3.2.

$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM) + 1.0(M_{secondary})$

[LRFD Tables 3.4.1-1 & 2]

$M_u = 1.25(-3,730.1 - 450.5) + 1.5(-789.9) + 1.75(-4,455.4) + 1.0(751.6 + 881.8)$

$= -12,574.2$ ft-kips

The compression face is the bottom flange of the beam which is 27-in. wide. The deck reinforcement in the longitudinal direction is assumed to be #6 @ 6-in. top and bottom. At this section there are three layers of reinforcement as shown in **Table 11.8.11.2-1**.

Table 11.8.11.2-1
Reinforcing Steel at the Pier

Layer	Area of Steel, in.^{2}	Distance from Top of the Composite Section, in.
Reinforcement in the Deck	16.720	3.750
Post-Tensioning Stage 1	4.774	17.500
Post-Tensioning Stage 2	2.387	10.375
Total	23.881	–

Using the strain compatibility method, the depth of the compression block is large and the stress in the prestressing steel is low, causing the steel to be used inefficiently. This is due to the relatively small bottom flange of the AASHTO-PCI Bulb Tee, which was not originally developed for continuous post-tensioned applications. If this section is the only one available locally, the compression capacity of the bottom flange can be enhanced in several ways:

- increase the strength of the concrete in the precast beam.
- add compression reinforcement in the bottom flange of the precast beam if it does not interfere with the pretensioning strands already there.
- add a structural steel plate embedded in the bottom of the precast section. Make the plate composite through the use of studs, similar to the connection of plate girders to deck slabs.

In this example, if a 1-in.-thick steel plate is used, strain compatibility analysis produces the following results:

Neutral axis depth, c = 11.895 in.

Rectangular stress block depth, a = 8.327 in.

Stress in mild reinforcement, f_{ps} = 60.0 ksi

Average stress in Stage 1 post-tensioning steel, f_{ps} = 262.55 ksi

Stress in Stage 2 post-tensioning , f_{ps} = 264.19 ksi

Nominal capacity, M_n = 16,393.65 ft-kips

Factored flexural resistance, $M_r = \phi M_n$ [LRFD Eq. 5.7.3.2.1-1]

where

ϕ = resistance factor [LRFD Art. 5.5.4.2.1]

= 1.00, for flexure and tension of prestressed concrete

M_r = 16,393.65 ft-kips > M_u = 12,574.2 ft-kips OK

Further design refinement may prove that a thinner plate is adequate. It will also determine the location where the plate may be terminated.

11.8.12 Limits of Reinforcement

11.8.12.1 Positive Moment Section

This section is a prestressed reinforced concrete section. The maximum amount of reinforcement, according to LRFD Article 5.7.3.3.1, should be such that:

$$\frac{c}{d_e} \leq 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

where $$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y}$$ [LRFD Eq. 5.7.3.3.1-2]

Since $A_s = 0$, $d_e = d_p$ = 73.11 in.

$$\frac{c}{d_e} = \frac{9.966}{73.11} = 0.136 < 0.42 \quad \text{OK}$$

According to LRFD Article 5.7.3.3.2, the minimum amount of prestressed and non-prestressed tensile reinforcement should be adequate to develop a factored flexural resistance, M_r, equal to the lesser of 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, and 1.33 times the factored moment required by the applicable strength load combination.

At midspan:

The following equation, adapted from the *Standard Specifications*, Article 9.18.2.1, is used for computing the cracking moment:

$$M_{cr} = (f_r + f_{pb})S_b - M_{d/nc}(S_{bc}/S_b - 1)$$

where

f_r = modulus of rupture = $0.24\sqrt{f'_c} = 0.24\sqrt{7.0} = 0.635$ ksi

[LRFD Art. 5.4.2.6]

f_{pb} = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of the section where tensile stress is caused by externally applied loads

$$= \left(\frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b}\right)_{Pretensioning} + \left(\frac{P_{pe}}{A} + \frac{M_{Total}}{S_b}\right)_{P/T\,Stage\ 1} + \left(\frac{P_{pe}}{A} + \frac{M_{Total}}{S_{bc}}\right)_{PT\ Stage\ 2}$$

$$= \left(\frac{651.768}{839} + \frac{651.768(32.55)}{15{,}789}\right) + \left(\frac{781.246}{839} + \frac{1{,}686.94(12)}{15{,}789}\right) + \left(\frac{408.762}{1{,}487} + \frac{1{,}026.81(12)}{21{,}441}\right)$$

= 2.211 + 2.213 + 0.850 = 5.274 ksi

$M_{d/nc}$ = moment due to non-composite dead loads

= $M_g + M_s$ = 2,519.7 + 2,089.0 = 4,608.7 ft-kips

S_{bc} = composite section modulus for the extreme fiber of the section where the tensile stress is caused by externally applied loads = 21,441 in.^{3}

S_b = non-composite section modulus for the extreme fiber of the section where the tensile stress is caused by externally applied loads = 15,789 in.^{3}

$$M_{cr} = (0.635 + 5.274)\left(\frac{15{,}789}{12}\right) - (4{,}608.7)\left(\frac{21{,}441}{15{,}789} - 1\right) = 6{,}125.0 \text{ ft-kips}$$

$1.2M_{cr}$ = 1.2(6,125.0) = 7,350.0 ft-kips

At midspan, the factored moment required by the Strength I load combination,

M_u = 15,418.3ft-kips

$1.33M_u$ = 1.33(15,433.4) = 20,506.3 ft-kips

Since $1.2M_{cr} < 1.33M_u$, $1.2M_{cr}$ controls, and, M_r = 17,504.87 ft-kips > $1.2M_{cr}$ OK

11.8.12.2 Negative Moment Section

The maximum amount of prestressed and nonprestressed reinforcement, according to LRFD Article 5.7.3.3.1, should be such that:

$$\frac{c}{d_e} \leq 0.42$$ [LRFD Eq. 5.7.3.3.1-1]

where

$$d_e = \frac{A_{ps}f_{ps}d_p + A_s f_y d_s}{A_{ps}f_{ps} + A_s f_y} \qquad \text{[LRFD Eq. 5.7.3.3.1-2]}$$

$$= \frac{[4.774(262.55)(62.5) + (2.387)(264.19)(69.62)] + (16.72)(60)(76.25)}{[4.774(262.55) + (2.387)(264.19)] + (16.72)(60)}$$

$$= 68.83 \text{ in.}$$

$$\frac{c}{d_e} = \frac{11.895}{68.83} = 0.173 \le 0.42 \quad \text{OK}$$

11.8.13 Shear Design

For an example of detailed calculations of shear design, refer to Example 9.2 of Chapter 9. The following calculation is intended to demonstrate the feasibility of this beam size in shear and the order of magnitude of the shear reinforcement required.

A section at 7.1 ft away from the pier centerline is considered. The factored shear, V_u = 482.7 kips, and the factored moment, M_u = 12,056.23 ft-kips.

Based on the *LRFD Specifications*, the effective web width for shear after the duct is grouted, is the gross web width minus 50 percent of the duct diameter:

$b_v = 7.00 - 3.75/2 = 5.13$ in. [LRFD Art. 5.8.2.7]

The effective depth for shear, d_v = 61.32 in. By conservatively assuming that $\beta = 2$ and $\theta = 45°$, the calculated V_c and V_p are 52.6 kips and 56.90 kips respectively, and the required V_s = 426.83 kips. Therefore, the required A_v/s is calculated to be 1.39 in.^{2}/ft.

Use WWR D20 with 3" spacing. The sum of V_c and V_s is 479.43 kips, which is less than maximum limit of $0.25f'_c b_v d_v$ = 528.42 kips.

11.8.14 Comments and Remaining Steps

The calculations presented in Section 11.8 cover the preliminary design steps needed to:

- check adequacy of the precast member for the selected span and spacing
- determine concrete strength
- determine the amount of pretensioning required
- determine the amount of post-tensioning required

After a solution is established, a thorough and detailed design should be performed. As a minimum, a commercially available continuous beam analysis program is needed for the detailed analysis for post-tensioning and live load effects. Specialized programs for computing moments and shears from LRFD live loads may also be used.

The following items should be considered in completing the design:

1. Calculate the prestress losses at various stages of loading and construction, specifically at pretension transfer, Stage 1 post-tensioning, Stage 2 post-tensioning and final time. The method in Section 8.6 has been used in the calculations in this example. TRB Report 496 (Tadros, et al, 2002) provides a refined method

for calculations of prestress losses. It includes creep and shrinkage parameters for high strength concrete and effects of differential creep and shrinkage between the beam and the deck. A study by Girgis, et al (2002) using the TRB Report 496 approach shows that the new method produces more realistic prestress losses than the current LRFD methods. The method in Section 8.6 provides reasonable intermediate results. It is acceptable to calculate the long-term prestress losses and the maximum positive moment cross-section in an interior beam line, and use these values throughout the member.

2. Determine the bending moments and shear forces due to pretensioning, dead loads and live loads. A spreadsheet table could be used to organize the calculations at equal span increments, say tenth points of each span.

3. Conduct service load analysis and check concrete stresses at various sections and various loading and construction stages. Modify prestressing if necessary and recycle the analysis.

4. Conduct strength analysis. Avoid the unnecessary penalties imposed by the *LRFD Specifications* described earlier in this example, by using the strain compatibility approach in Section 8.2.2.5 of this manual, including Mast's unified method variable reduction factor (see discussion by Girgis, et al, 2002). If there is strength deficiency in a positive moment area, attempt to correct it by providing additional pretensioning. If there is a deficiency in a negative moment area, attempt to correct it by providing additional deck reinforcement.

5. Calculate cambers and deflections. Use this information to determine requirements for setting the build-up over the beam top flange, and for setting the beam seat elevations to match the roadway profile. Also, check to see that live load deflection is within the optimal limits.

6. Conduct a thorough shear design. Systems of this type have a reduced equivalent web width due to the presence of post-tensioning ducts. The *LRFD Specifications* limit of $0.25f'_c b_v d_v$ on the maximum shear force controls the design in many cases. This provides a better solution compared to the previous *Standard Specifications* limit.

7. Design and detail the post-tensioning anchorage zone, as suggested in Section 11.7.

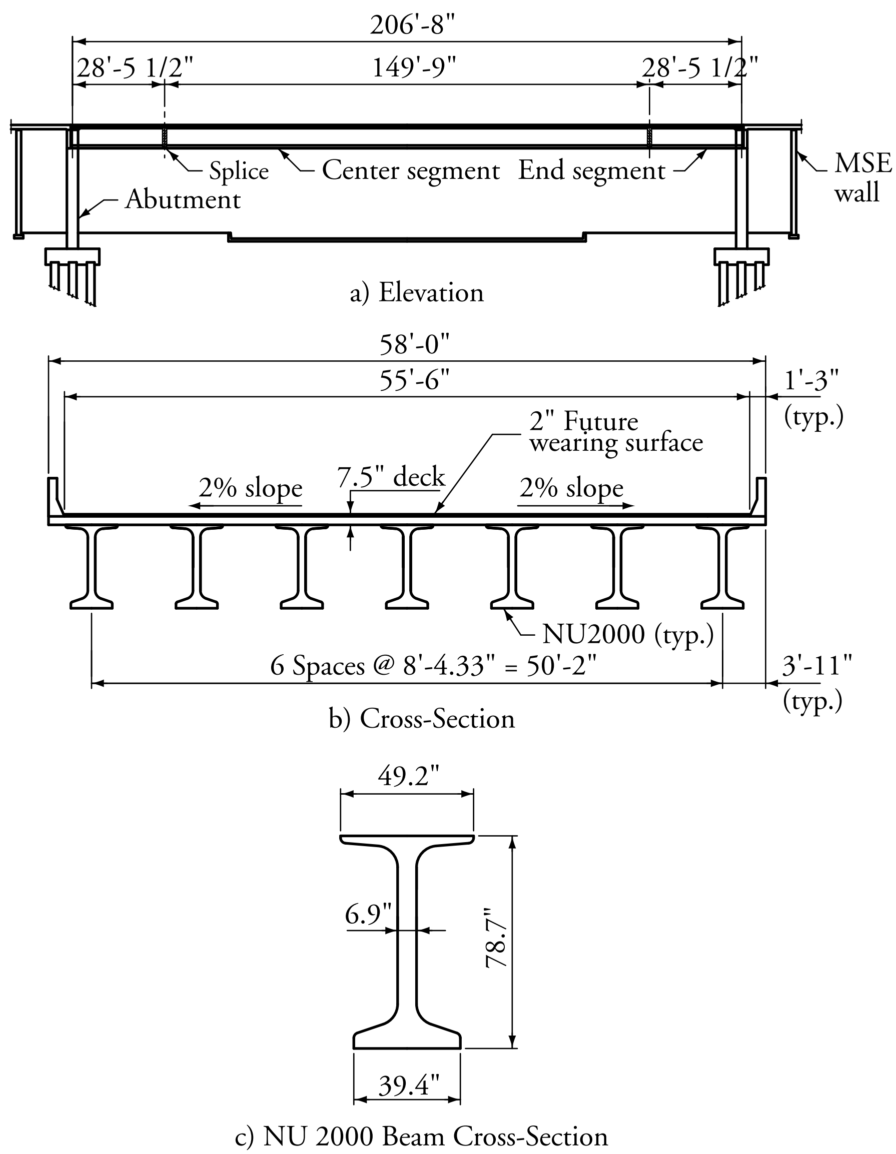
11.9 DESIGN EXAMPLE: SINGLE SPAN, THREE-SEGMENT BEAM

This example provides a summary of the calculations for a bridge constructed in 2003 and 2004 in Omaha, Nebraska, at Dodge Street (US Highway 6) and 204th Street (Nebraska Highway 31). Some of the significant considerations are presented in summary.

11.9.1 Input Data and Design Criteria

The length of this single-span bridge is 206.7 ft. The project used the NU2000PT (Nebraska) I-beam. The beam depth is 78.74 in. (2,000 mm) and web width is 6.9 in. The bridge section consists of 7 beams spaced at 8'-4.3". The bridge is 58-ft wide. Details of the bridge are shown in **Figure 11.9.1-1**.

Figure 11.9.1-1 Details of the Dodge Street Bridge



A composite, 8-in.-thick concrete slab (7.5-in. structural depth) is cast-in-place. Each beam line uses three beam segments. The end segments are approximately 28-ft long each and the center segment is nearly 149-ft long. These lengths are in addition to two, 12-in.-wide spaces for the splices. The compressive strength of the precast beam concrete is 10 ksi and 4.3 ksi for the CIP slab. The bridge is designed in accordance with *LRFD Specifications*, 2nd Edition and the 1999 and 2000 Interim Revisions. Design live load is HL-93.

11.9.2 Construction Stages

The construction stages are as follows:

Stage 1: Fabricate precast beam segments

Stage 2: Erect precast beam segments on temporary towers and abutments

Stage 3: Splice post-tensioning ducts and cast splice

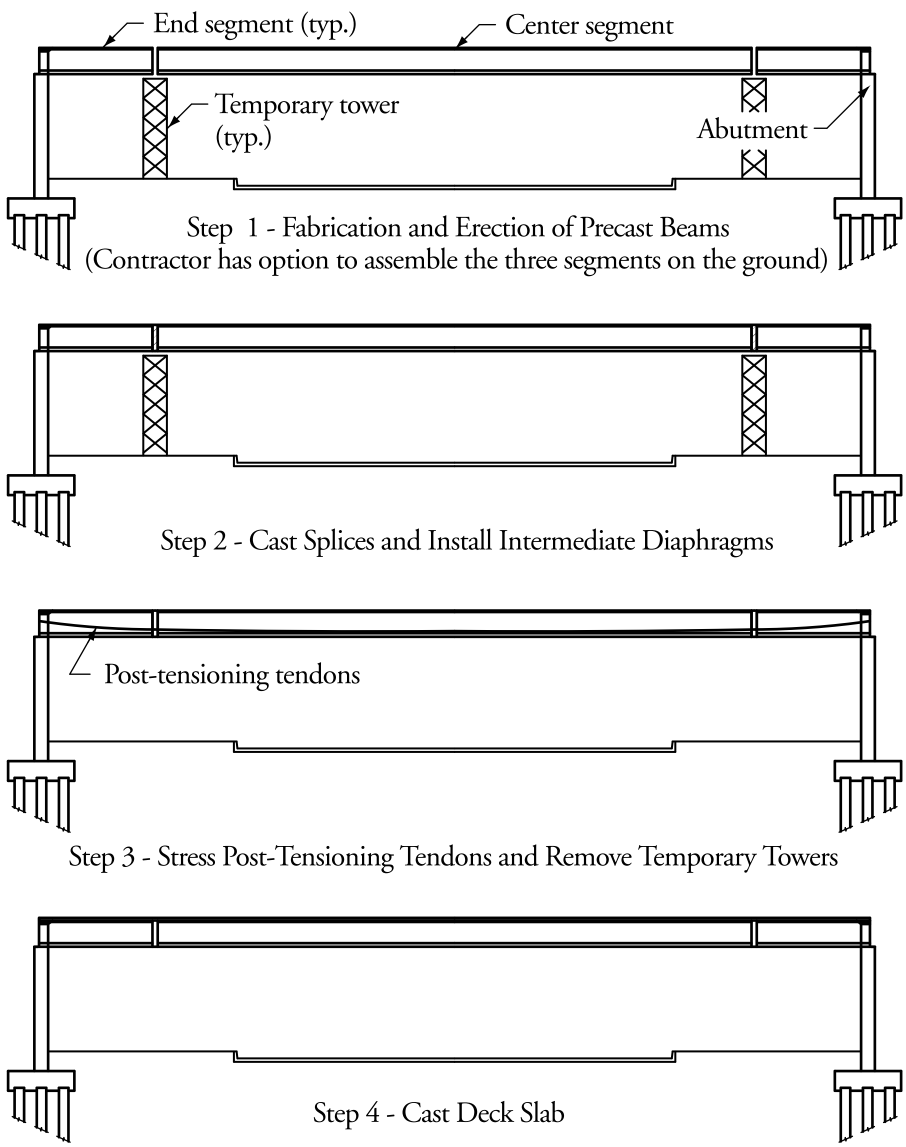
Stage 5: Stress post-tension tendons and remove temporary towers

Stage 6: Place deck slab

Stage 7: Construct barriers

The construction stages are summarized in **Figure 11.9.2-1**. This construction schedule does not provide compression in the deck, since all post-tensioning is applied before the deck is cast. This solution is also less efficient, since only the beam is post-tensioned. However, this does permit removal of the deck for replacement.

Figure 11.9.2-1
Construction Sequence of the Dodge Street Bridge



11.9.3 Flexure at Service Limit State

The critical section in flexure, after all losses, due to full loads plus effective prestress, is at midspan. For pretensioning and post-tensioning details, see **Figures 11.9.3-1** and **11.9.3-2** respectively.

Figure 11.9.3-1
Pretensioning Details

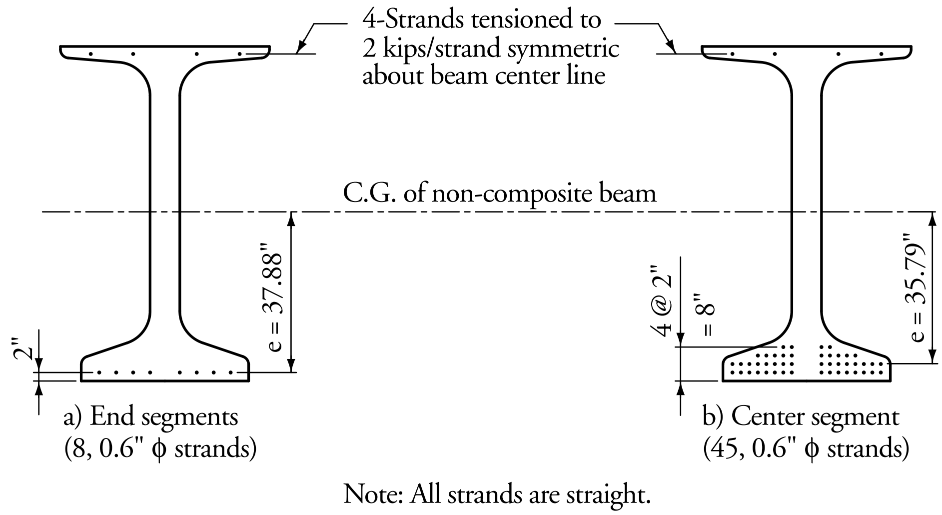


Figure 11.9.3-2
Post-Tensioning Details

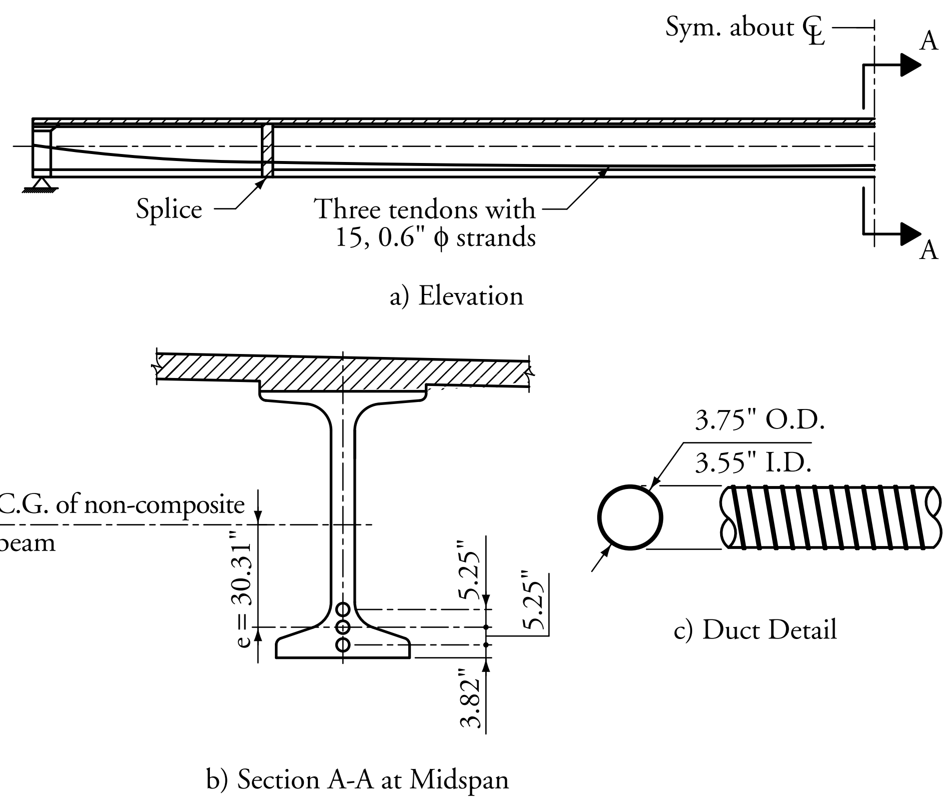


Table 11.9.3-1 provides the bending moments for an interior beam line. **Table 11.9.3-2** gives a summary of the concrete stresses at midspan. The table shows that the most critical stress is concrete compression at the top fibers of the beam due to effective prestress plus permanent loads. The stress limit in the *LRFD Specifications* is $0.45f'_c$. It required the beam concrete strength to be specified at 10 ksi.

Table 11.9.3-1
Bending Moments at Midspan

Loading	Bending Moment at Midspan Section, ft-kips
Girder Weight	2,774.54
Support Removal	2,499.20
Deck Slab	4,464.75
Wearing Surface	1,116.19
Barriers	560.76
Live Loads	5,031.50

Table 11.9.3-2
Service Load Stresses at Midspan

Location/ Load Stage	Top of Slab (ksi) Service I		Top of Beam (ksi) Service I		Bottom of Beam (ksi) Service III
	Permanent Loads	Total Loads	Permanent Loads	Total Loads	Total Loads
At Midspan	0.279	1.117	4.492	5.491	1.054
Allowable Stresses	1.935	2.580	4.500	6.000	−0.600

11.9.4 Flexure at Strength Limit State

The required factored bending moment is:

$M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM) + 1.0(M_{secondary})$

[LRFD Tables 3.4.1-1 & 2]

Since this is a statically determinate beam, there are no secondary effects from post-tensioning.

$$M_u = 1.25(2{,}774.54 + 2{,}499.20 + 4{,}464.75 + 560.76) + 1.5(1{,}116.19) + 1.75(5{,}031.5)$$
$$= 23{,}353.47 \text{ ft-kips}$$

At this section, there are seven layers of prestressing steel as shown in **Table 11.9.4-1**.

Using the strain compatibility method, the following results were found:

Neutral axis depth, c = 32.3 in.

Stress block depth, a = 24.5 in.

Stress in the pretensioning steel varied from 243.99 ksi to 247.85 ksi

Stress in the post-tensioning steel varied from 241.90 ksi to 246.47 ksi

$M_r = 27{,}317.73$ ft-kips > $M_u = 23{,}353.47$ ft-kips OK

Table 11.9.4-1
Prestressing Steel at Midspan

Layer	A_{ps}, in.^{2}	Distance from Bottom of Section, in.
Layer 1 Pretensioning Strands	3.472	2.000
Layer 2 Pretensioning Strands	3.472	4.000
Layer 3 Pretensioning Strands	2.170	6.000
Layer 4 Pretensioning Strands	0.868	8.000
First Post-Tensioning Tendon	3.255	3.875
Second Post-Tensioning Tendon	3.255	8.625
Third Post-Tensioning Tendon	3.255	13.375

11.9.5 Discussion

This is a simple, yet important application of spliced beams. Single-point urban interchanges such as the Dodge Street Bridge are becoming increasingly important. They require a single long, wide span.

Production and handling of 200-ft-long beams would be challenging in most areas, although it has been achieved, for example, on the Bow River (Sect. 11.5.4). Temporary towers were used to support the segments during construction of the Dodge Street Bridge. However, post-tensioning segments together on the ground, then lifting the full-length beam into place is also challenging and requires careful slenderness calculations and the use of larger erection equipment. Before a decision is made, all of these options should be investigated for each project. Contractors can be an excellent resource to designers in this situation and are usually willing to contribute. Many precasters have had experience furnishing these products and are also good resources. It is practically impossible to achieve the span demonstrated by this example without carefully combining pretensioning with post-tensioning.

The 10 ksi specified concrete strength for the Dodge Street Bridge beams helped keep the compressive stress due to effective prestress plus permanent loads below the *LRFD Specifications* limit of $0.45f'_c$. This strength is now achievable in most parts of the country. In addition, the strength of the deck could be increased to improve behavior at both service and strength limit states.

On bridges of this type, it is important to carefully calculate instantaneous and long-term deflections at various stages of loading and at final conditions. Net long-term deflection may be downward. To compensate for a possible sag in the span, which may be psychologically unacceptable, the elevations of the temporary tower supports can be raised to create a cambered beam.

More useful design and construction details can be found in an article by Van Lund, et al (2002) on the Twisp River Bridge in Washington State, which is similar to the bridge in this example.

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A = area

A = area of cross-section of the precast beam

A_c = total area of the composite section

A_{cp} = total area enclosed by outside perimeter of concrete cross-section

a = a length defined in **Figure 12.4.1.1-1**

B = width

b_v = effective web width

C = coefficient to compute centrifugal force

DC = dead load structural components and nonstructural attachments

DW = dead load of wearing surfaces and utilities

E = modulus of elasticity

E_c = modulus of elasticity of concrete

E_{ci} = modulus of elasticity of the beam concrete at transfer or at post-tensioning

E_{cs} = modulus of elasticity of concrete slab

E_p = modulus of elasticity of prestressing tendons

E_s = modulus of elasticity of reinforcing bars

e = eccentricity

f_b = concrete stress at the bottom fiber of the beam

f'_c = specified compressive strength at 28 days

f'_{ci} = concrete strength at transfer or at post-tensioning

f_{pc} = compressive stress in concrete at the centroid after prestress losses have occurred

f_{pc} = compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange

f_{pe} = effective stress in the prestressing steel after losses

f_{pj} = jacking stress (maximum)

f_{pu} = specified tensile strength of prestressing steel

f_{py} = yield strength of prestressing steel

f_y = specified minimum yield strength of reinforcing bars

g = gravitational acceleration

H = elevation difference between ends of a beam

h = overall depth of a member

h_c = total height of composite section

I = moment of inertia

I_c = moment of inertia of composite section

I_{clat} = moment of inertia of composite section for lateral bending

I_{lat} = moment of inertia for lateral (weak axis) bending

I_p = polar moment of inertia

I_{pc} = polar moment of inertia of composite section

IM = dynamic load allowance (impact factor)

J = torsional constant

J_c = torsional constant for composite section

L = live load
L = overall beam length or design span
L_a = arc length
L_c = chord length
M = bending moment
M_c = moment applied to cross beam
M_i = moment in inside beam
M_o = moment in outside beam
M_t = torsional moment
M_u = factored bending moment
$M(-)$ = negative moment
m = multiple presence factor
n = modular ratio between deck slab and beam materials
P = effective prestressing force
p_c = the length of the outside perimeter of the concrete section
R = radius of curvature
R_i = reaction of inside beam
R_o = reaction of outside beam
S = section modulus
S_b = section modulus for the extreme bottom fiber of the non-composite precast beam
S_{bc} = composite section modulus for extreme bottom fiber of the precast beam
S_t = section modulus for the extreme top fiber of the non-composite precast beam
S_{tc} = composite section modulus for top fiber of the deck slab
S_{tg} = composite section modulus for extreme top fiber of the precast beam
s = sagitta, arc-to-chord offset
T = unfactored torsional moment
T_u = factored torsional moment
T_{cr} = torsional cracking moment
t_s = cast-in-place deck thickness
V = shear
v = highway design speed
W = weight
w = clear width of roadway
w_c = unit weight of concrete
w_g = beam self weight per unit length
x = a length
y = distance
y_b = distance from centroid to the extreme bottom fiber of the non-composite precast beam
y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam
y_{max} = maximum distance, used in computing section modulus

y_t = distance from centroid to the extreme top fiber of the non-composite precast beam

y_{tc} = distance from the centroid of the composite section to extreme top fiber of the slab

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam

γ = grade angle expressed as a decimal

ϕ = resistance factor

θ = skew angle

ψ = an angle

Curved and Skewed Bridges

12.1 SCOPE

This chapter deals with the geometric and structural challenges for bridges with curvature in plan, or with skewed supports, and on a grade. The effects of skew and grade are primarily geometric, with some effect on shears and moments. Larger skew angles also have some effect on live load distribution. The effects of curvature are both structural and geometric. This chapter primarily describes the design of curved bridges. Structures with very sharp curvature (say 300-ft radius) such as freeway on or off ramps, may require the use of specially made box beams that are also described in ABAM (1988). Straight AASHTO I-beams and box beams are normally used on curved bridges with shorter spans, or on longer spans with larger radii, because the offset between arc and chord is small.

Curve and skew effects are described in more detail in the *LRFD Specifications* than in the *Standard Specifications*. Even if one is designing using the *Standard Specifications*, it would be prudent to consult the *LRFD Specifications* for guidance on curve and skew effects. This chapter is based on the *LRFD Specifications*.

12.2 SKEW AND GRADE EFFECTS

12.2.1 General

A skewed bridge is one in which the major axis of the substructure is not perpendicular to the longitudinal axis of the superstructure. For the work at most agencies, the skew angle (usually given in degrees) is the angle between the major axis of the substructure and a perpendicular to the longitudinal axis of the superstructure. Some agencies use a different convention. Usually, different substructure units have approximately the same skew angle.

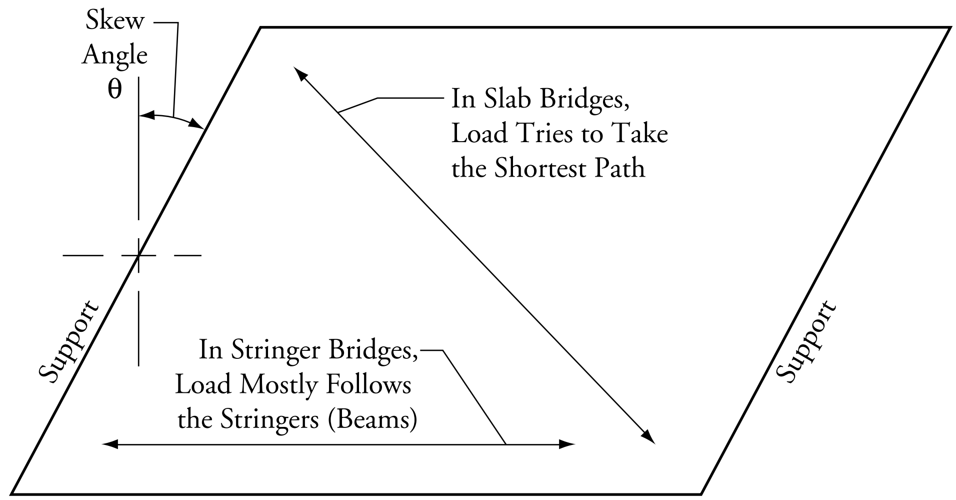
The presence of skew affects the geometry of many bridge details. Skew angles greater than 20 degrees also have an effect on bending moment, and on shear in the exterior beams. The structural response of a skewed bridge to seismic loads can be significantly altered by the skew angle of the substructure.

The effects of grade are geometric.

12.2.2 Superstructure Behavior

In stringer bridges (bridges supported by longitudinal I- or bulb-tee beams), the load tends to flow along the length of the supporting beams, and the effect of skew on the bending moments is minimized. In solid slab bridges and other bridges with high torsional rigidity, the load tends to take a “short cut” between the obtuse corners of the span, as shown in **Figure 12.2.2-1**. This reduces the longitudinal bending moments, but it increases the shear in the obtuse corners. The same effect occurs in stringer bridges, but is less pronounced. The modification factors due to skew for shear and moment are given in Section 7.5.4.

Figure 12.2.2-1
Load Distribution in Skewed Spans

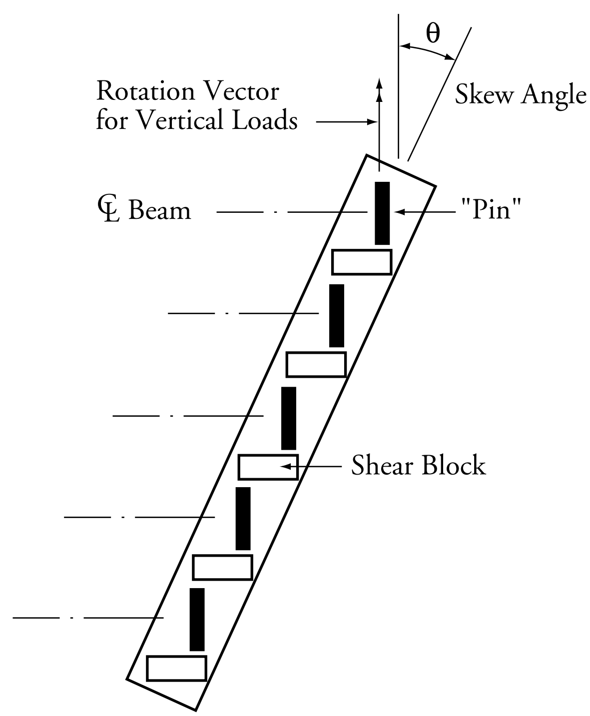


12.2.3 Substructure Behavior

The relative stiffness of the substructure about its major and minor axis is important. A substructure consisting of round columns and a cap beam is about four times as stiff when acting as a frame resisting loads along its major axis, compared to its stiffness acting as a cantilever for loads along its minor axis. For rectangular cantilever piers, the ratio of major-to-minor-axis stiffness is proportional to the dimension ratio squared. For wall piers, the major axis stiffness is almost infinite compared to the minor axis stiffness.

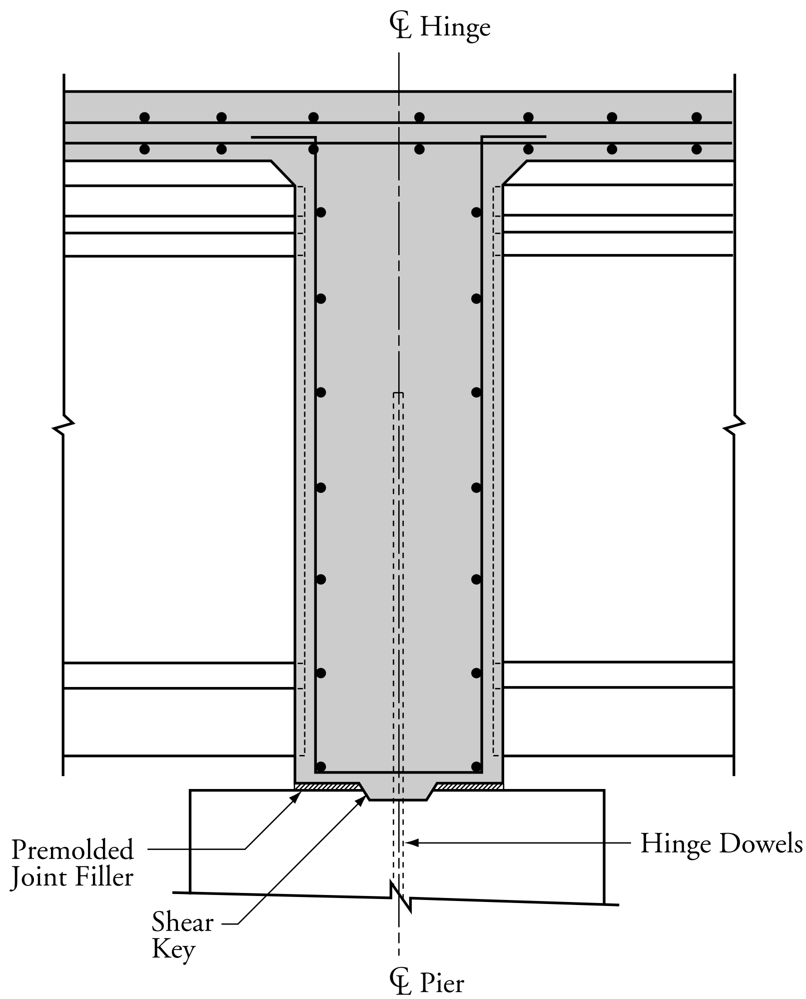
When a substructure unit deflects due to horizontal loads or superstructure deformations, the deflection is primarily along the minor axis, and the rotation vector at the top is along the major axis. When the superstructure deflects due to vertical loads, the rotation vector at the support is perpendicular to the axis of the beams, as shown in **Figure 12.2.3-1**.

Figure 12.2.3-1
Rotation Vector for Vertical Loads



This raises a question of how to orient the "pin" between superstructure and substructure. Concrete bridges are seldom supported by real pins. Bearings consisting of elastomeric pads can provide rotation capacity about all axes; this solves the problem of how to orient the pin. Continuous bridges are sometimes constructed using a concrete hinge between superstructure and substructure, as shown in **Figure 12.2.3-2** (also, see Section 3.2.3.2.2). This forces the rotation vector to lie along the major axis of the substructure, which is inconsistent with the end rotation of the superstructure beam. However, live load rotations at an interior support of a continuous bridge are small, and structures so constructed seem to perform satisfactorily.

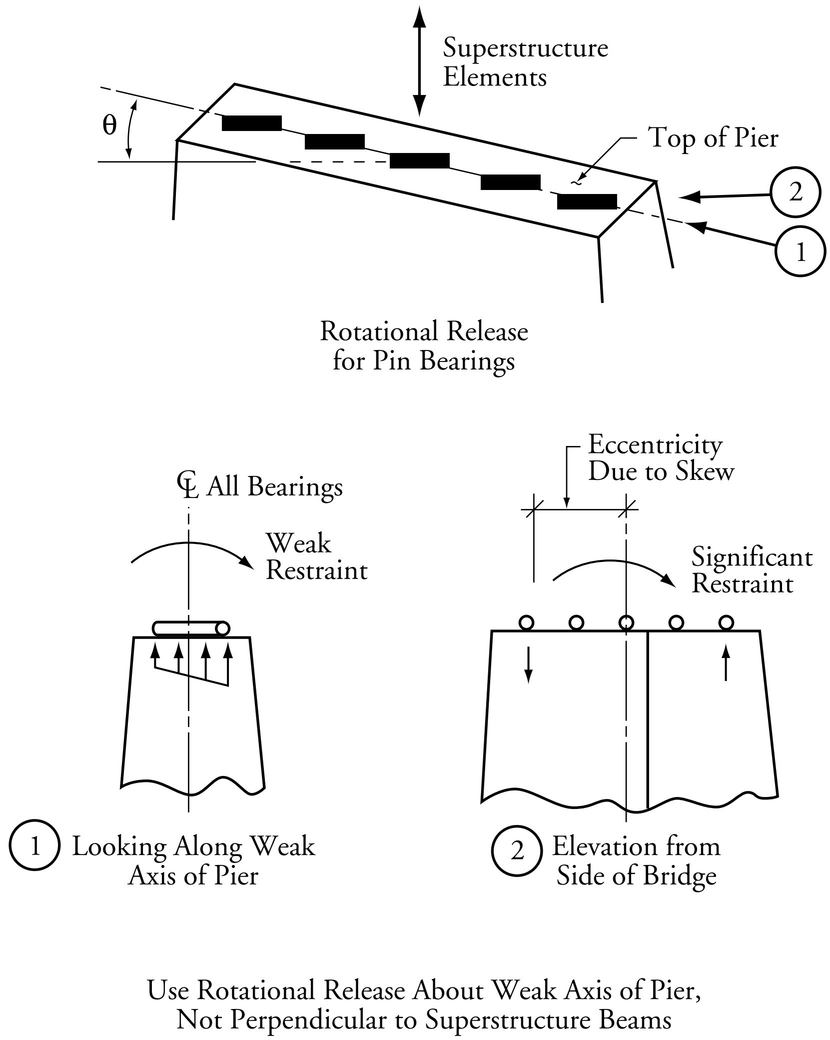
Figure 12.2.3-2
Typical Hinge Section



NOTE: Reinforcement from beams extending into diaphram and other details not shown for clarity.

A sidelight to this discussion concerns computer modeling. Orienting the rotational release vector with respect to the superstructure axes may force a component of rotation about the major axis of the substructure. This will create a fictitious moment at the top of the substructure in the computer model, as shown in **Figure 12.2.3-3**. In general, a rotational release between superstructure and substructure should be oriented with respect to the substructure axes.

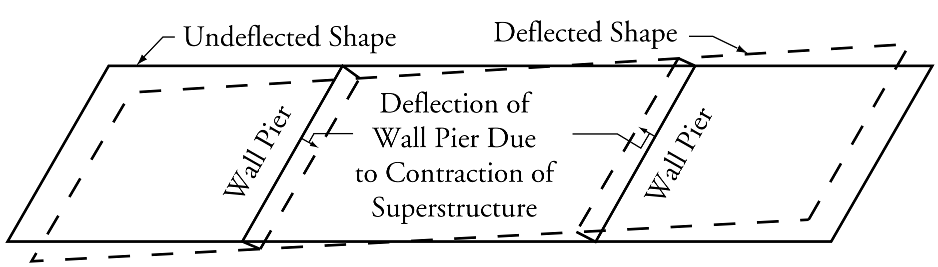
Figure 12.2.3-3
Orientation of Pins in Computer Model



12.2.4 Temperature and Volume Change Effects

The shortening of a skewed span due to shrinkage and temperature will cause the supporting substructure units to deflect, if they are connected longitudinally to the superstructure. The substructure units will tend to deflect about their minor axes, causing a rotation of the superstructure, as shown in **Figure 12.2.4-1**. If transverse shear blocks are provided at the abutments, transverse forces at the abutments can develop, as well as forces along the major axis of the piers.

Figure 12.2.4-1
Bridge Rotation Caused by Shrinkage Deflection



12.2.5 Response to Lateral Loads

Wind and seismic loads transverse to the major axis of the bridge cause both transverse and longitudinal deflection of the superstructure, as the substructure elements deflect about their weak axes. Similarly, longitudinal loads also cause both longitudinal and transverse deflections of the superstructure. This can lead to a coupling of transverse and longitudinal modes in a dynamic seismic analysis. This subject is more fully discussed in FHWA (1996 A and B).

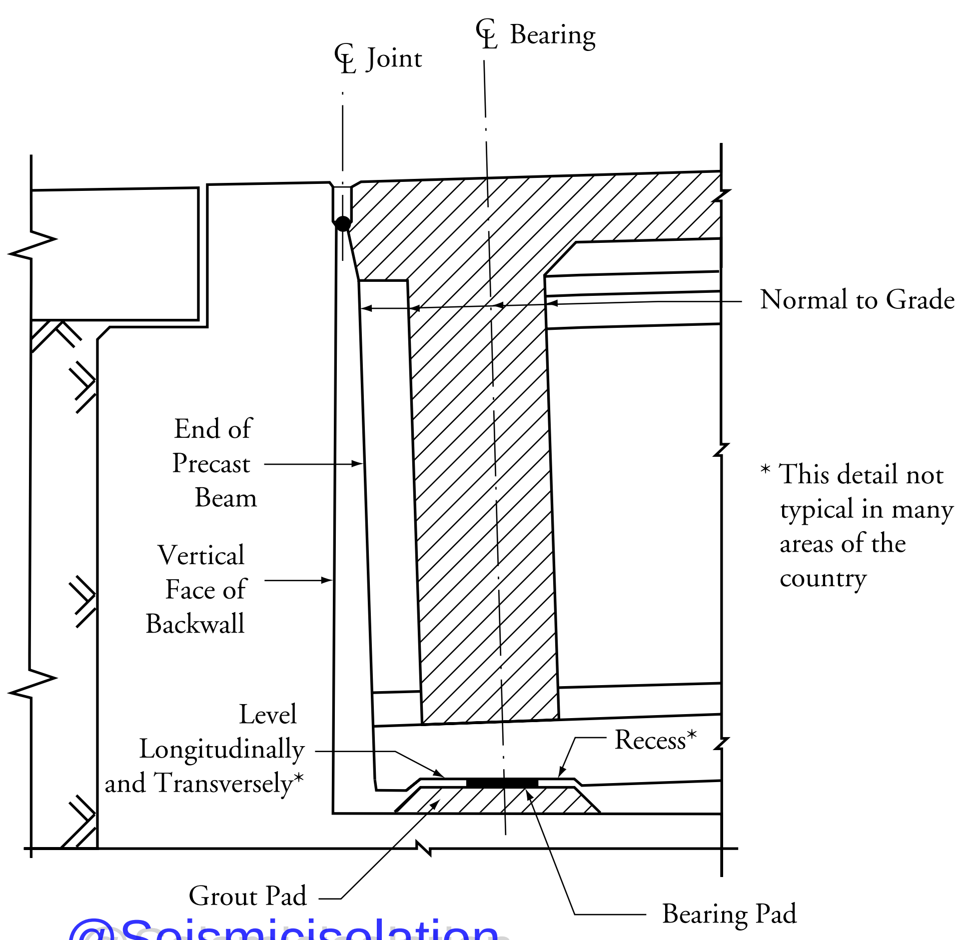
12.2.6 Detailing

12.2.6.1 Effects of Grade

Grade affects the geometry of the precast beams. The slant length is increased over the plan length by an amount $\gamma^2L/2$, where γ is the grade, expressed as a decimal. The precast beam is normally made in the shape of a rectangle, as seen in elevation. That is to say, the ends of the beam are usually square with the long axis of the beam, rather than being vertical in the final position of the beam. Similarly, the diaphragms are normally square with the axis of the beam.

Cast-in-place substructures are normally cast with vertical surfaces. This needs to be considered in the abutment detail (see **Figure 12.2.6.1-1**) in which the beam end is not vertical. The bearing pad is set on a level, horizontal surface. Recesses, shims, sloped risers or grout pads are used to compensate for the difference in planes between the beam soffit and the top of the cap beam. Sometimes, on moderate grades, the bearing pads and bearing surfaces on the abutment and on the underside of the beam are set parallel to the grade.

Figure 12.2.6.1-1
Section at Abutment Showing Effects of Grade



12.2.6.2 Skewed Beam Ends

Skewed beam ends (in plan) are sometimes provided at expansion joints. Skew angles should be grouped into standard increments because each skew angle will require a special end bulkhead to form it. At interior ends in continuous beams, the beam-ends are normally made square in plan. Some end diaphragm details may require that the ends of continuous beams be skewed. In the latter case, when using precast, prestressed box beams, a maximum skew angle of 30 degrees avoids warping or racking of the beams.

12.2.6.3 Diaphragms

Intermediate diaphragms, if used, may be perpendicular to the beam axes, or parallel to the skew. Making them parallel to the skew can have the advantage of making interior beams identical. Making them perpendicular to the beams simplifies their construction in the field.

12.2.6.4 Deck Reinforcement

For skew angles of 10 degrees or less, deck reinforcement is normally placed parallel to the skew. This simplifies detailing and the placement of reinforcement. For skew angles exceeding 20 degrees, transverse deck reinforcement should be placed normal to the axis of the beams, and shorter bars should be used in the acute corners of the deck. For skew angles between 10 and 20 degrees, practice varies concerning orientation of the deck reinforcement.

12.2.6.5 Plans

The detailing recommendations made in Section 12.2.6 are by no means universal. The plans must show the geometric effects of skew and grade. It is important to indicate which surfaces are parallel or normal to the skew, and which surfaces are parallel or normal to the beam axis. Similarly, the plans should indicate which surfaces are truly vertical and horizontal, and which surfaces are parallel or normal to the inclined beam axis for beams on a grade.

12.3 CURVED BRIDGE CONFIGURATIONS

12.3.1 Use of Chords

Curved bridge beams are usually made as a series of short straight segments, or chords, to approximate the theoretical arc. Forms for the beams are made in straight segments, with a small angle at the form joints. The exception is monorail beams in which the concrete beam surface is the running surface for the wheels of the monorail vehicle. Such monorail beams are made in an adjustable form that can be bent to form a smooth arc.

The maximum offset between an arc and its chord is equal to $L_c^2/8R$, where L_c is the chord length and R is the radius of curvature. Although this is an approximation, it is a good one. Because it is an approximation, the length may be either the arc length, L_a, or the chord length, L_c, whichever is known. The formula shows that the offset varies as the square of the chord length. For practical curve radii encountered in bridges, a curve approximated by 20-ft chords will appear to the eye as a smooth continuous curve.

The simplest way to support a curved roadway is to use straight beams beneath a curved deck. If the offset between chord and arc is too large, the appearance will be poor, and the exterior beam on the outside of the curve will be required to support too much additional load. It is desirable that the arc-to-chord offset be limited to 1.5 ft, and that the edge of the top flange of the beam be no closer than 0.5 ft to the slab edge. **Table 12.3.1-1** shows the minimum curve radii that satisfy the 1.5 ft maximum offset criterion. This limit is often exceeded, but each case should be examined for acceptability.

Table 12.3.1-1
Radii that Provide Offsets Shown for Various Straight Beam Lengths

Beam Length, ft	Radius, ft			
	Offsets, ft			
	0.5	1.0	1.5	2.0
70	1,225	613	408	306
80	1,600	800	533	400
90	2,025	1,013	675	506
100	2,500	1,250	833	625
110	3,025	1,513	1,008	756
120	3,600	1,800	1,200	900
130	4,225	2,113	1,408	1,056
140	4,900	2,450	1,633	1,225
150	5,625	2,813	1,875	1,406
160	6,400	3,200	2,133	1,600

Straight beams are by far the simplest and most cost-effective way to use precast, prestressed beams in a curved bridge; they should be used whenever appropriate. This solution is not discussed further in this chapter because the analysis is almost identical to that for a straight bridge. The only difference is in the computation of loads on the exterior beams. The “lever rule” [LRFD Art. C4.6.2.2.1] may be used in the same manner as for a straight bridge, as long as the variable overhang is accounted for. In addition, the extra span length on the outside of the curve must, of course, be used in the design of these beams.

For situations in which the offset exceeds 1.5 ft, the number of chords may need to be increased. One method is to splice I- and bulb-tee-beam segments together in the field using methods described later in this chapter and in Chapter 11. With two chords, the offset will decrease by a factor of 4; and with three chords, the offset will decrease by a factor of 9.

12.3.2 Beam Cross-Section

12.3.2.1 Box Beams Versus I-Beams

Full-span-length, chorded, curved beams may be made in the plant, using post-tensioning. Torsional stresses and handling considerations will usually cause a closed box section to be preferred for full-length curved beams.

Segmental construction may be used with conventional I-beams. Two or three straight segments may be supported on temporary shores, and post-tensioned in the field after constructing diaphragms at the segment joints. Refer to details in Chapter 11, as well.

12.3.2.2 Box Section Configuration

Box sections will often require a new form, as standard box sections of the size needed do not exist in many localities. The precast box beam needs to be closed at the top, in order to have sufficient torsional resistance.

The sides of box beams may be vertical or sloped. Vertical sides are somewhat easier to form. Sloping sides are generally thought to have a better appearance.

The maximum span of box beams is often limited by shipping weight. Field-splicing of shorter segments may be used to minimize weight of individual segments. In order to minimize the thickness of webs and flanges, consideration should be given to the use of “external” post-tensioned tendons inside the box section.

12.3.2.3 I-Beam Configuration

The use of post-tensioning requires webs thicker than the 6-in. webs of AASHTO-PCI Bulb-tees and other standard I-beams. To accommodate post-tensioning ducts and reinforcement, the minimum web thickness should be 7 to 8 inches. Thicker webs can often be obtained by spreading the side forms of standard shapes by 1 or 2 in.

12.3.2.4 Continuity

Continuity is very desirable in curved bridges. In addition to the benefits that continuity provides for straight bridges, there are two additional benefits for curved bridges. Continuity greatly reduces torsion resulting from applied loads, and it reduces the excess load on the exterior beam on the outside of the curve.

12.3.2.5 Crossbeams

Transverse members spanning between beams within a span (intermediate diaphragms) are often omitted on straight bridges (see Section 3.7). However, in curved bridges, the transverse members, which will be referred to as crossbeams in this chapter due to their unique role, are required to counteract both the effects of torsion and the lateral forces resulting from curvature. The crossbeams should also be deep enough to brace the bottom flange.

12.3.2.6 Superelevation

Standard practice is to keep the beam cross-section vertical, and provide a "haunch" or "pad" of cast-in-place deck concrete to fill the space between the sloping deck and the horizontal top flange.

12.4 PRELIMINARY DESIGN

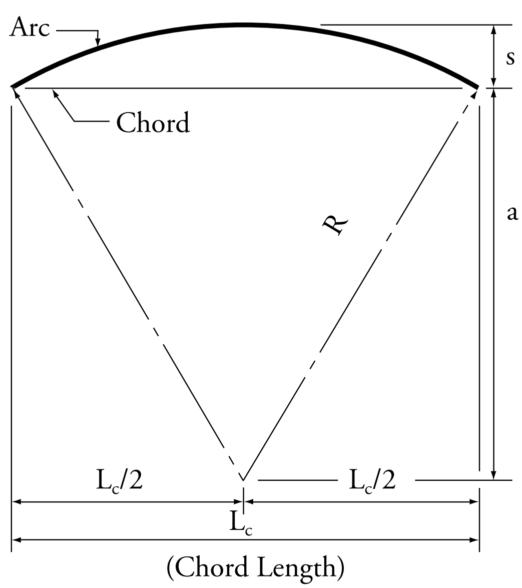
12.4.1 Useful Geometric Approximations

Despite the immense computing power now available, simple approximations remain very useful for preliminary design. They are quick to use, and they give the designer a "feel" for how a change in one parameter affects other parameters.

12.4.1.1 Arc Offset from Chord

The maximum offset between arc and chord is called the middle ordinate or the "sagitta" (sagitta is Latin for "arrow") and represented by the symbol, s. As noted in Section 12.3.1, the sagitta is approximately equal to $L_c^2/8R$. The derivation is simple and is shown in **Figure 12.4.1.1-1**. Once again, since these are approximations, it is unimportant whether the arc length or chord length is used.

Figure 12.4.1.1-1
Arc Offset from Chord



By Pythagorean Theorem:

$a^2 + (L_c/2)^2 = R^2$

Also:

$a = R - s$

$\cancel{R^2} - 2Rs + s^2 + L_c^2/4 = \cancel{R^2}$

But s is small compared to R and L_c. Therefore, ignore the term s^2 and solve for s.

$$s = \frac{L_c^2}{8R}$$

The formula slightly underestimates the distance, s. The approximation is slightly better if the length is taken as the arc length, L_a.

12.4.1.2
Excess of Slant Length over Plan Length

The slant length of a beam on a grade is longer than the plan length by an amount $H^2/2L$, where H is the difference in elevation of the two ends of the beam. This is a well-known formula, and is identical to the $\gamma^2L/2$ formula given in Section 12.2.6.1 (γ is equal to H/L). The derivation is similar to that for the arc-chord offset. The Pythagorean theorem is used, neglecting a small second-order quantity.

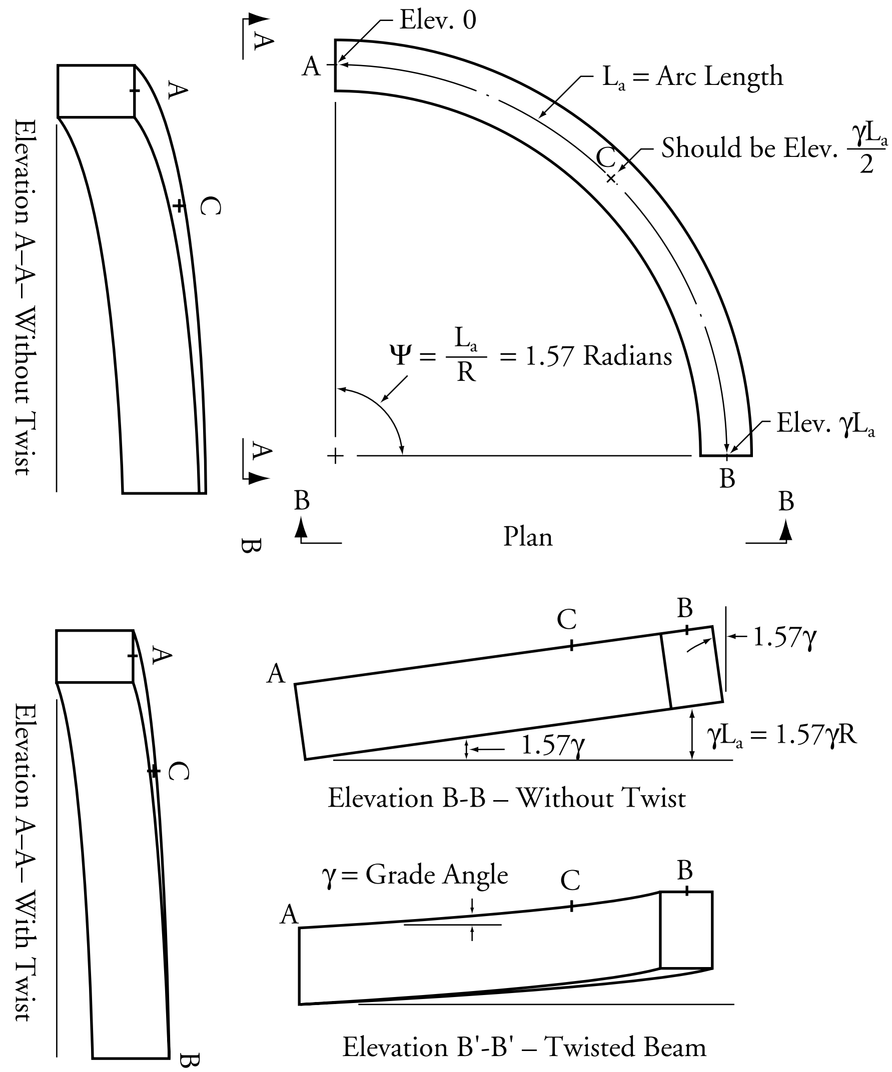
12.4.1.3
Excess of Arc Length over Chord Length

The length of an arc is longer than its chord by an amount $8s^2/3L_c$, where s is the arc-chord offset and L_c the chord length. The excess length may also be expressed as $L_c^3/24R^2$. This formula is derived by approximating the arc length as a series of short chords, then taking the limit as the chord length approaches zero.

12.4.1.4
Twist Resulting from Grade

The shape of a curved beam on a grade is a helix. It has the same shape as the railing on a "spiral" (more correctly, helical) stair. Such a railing is twisted. If a section were cut out of the railing and laid flat, the twist would be apparent.

Figure 12.4.1.4-1
Twist Resulting from Grade Change



To understand more fully the twist in a curved beam caused by grade, consider a beam curved 90 degrees (1.57 radians) in plan, made without twist, with square ends as illustrated in the Plain View of **Figure 12.4.1.4-1**. The bearing at Point B is elevated higher than at Point A by an amount $1.57\gamma R$ as shown in Elevation B-B. Therefore, the beam will be tipped by an angle of 1.57γ. At Point B, the sides of the beam will not be plumb; they will be tipped by an angle 1.57γ. Also, note that at Point C, the midpoint of the beam, the elevation of the beam will not be half of $1.57\gamma R$, as it should be.

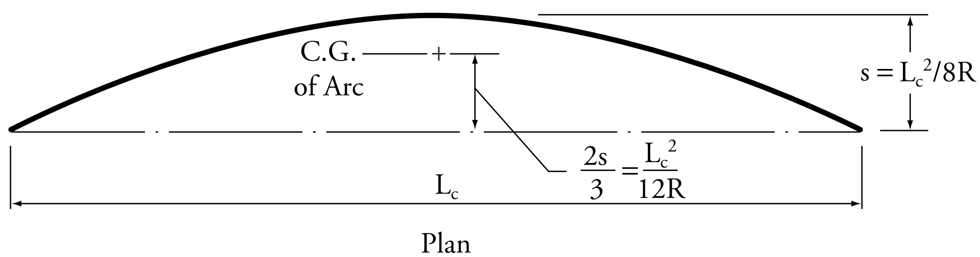
Elevation B'-B', **Figure 12.4.1.4-1**, shows the elevation of the beam fabricated to a true helix. The ends and sides of the beam will be plumb at Points A and B, and the elevation at C will be correct. The beam must be twisted by an amount 1.57γ. Generalizing for angles other than 1.57 radians, the amount of twist is $\psi\gamma$, or $(L_a/R)\gamma$ where L_a is the arc length.

The approximation is this: The twist angle is normally small enough to be ignored in beam fabrication, except for monorail beams. If the twist is ignored in beam fabrication, it should be realized that when the beam is set in the field, it will not be possible for both ends to be perfectly plumb. If the apparent twist is large enough to be measurable, the beam should be set "splitting the difference" of the out-of-plumbness at the two ends. This will also result in the midpoint of the beam being at proper elevation (not including the effects of camber).

12.4.1.5 Center of Gravity of an Arc

The center of gravity of an arc (and of a load applied along the arc) is offset from the chord by $2s/3$, or $L_c^2/12R$. See **Figure 12.4.1.5-1**.

Figure 12.4.1.5-1
Center of Gravity of Arc



12.4.1.6 Curved Surfaces

The area of a curved surface with radial ends, such as a bridge deck, is equal to BL_a, where B is the width and L_a is the arc length along the centerline. See **Figure 12.4.1.6-1**.

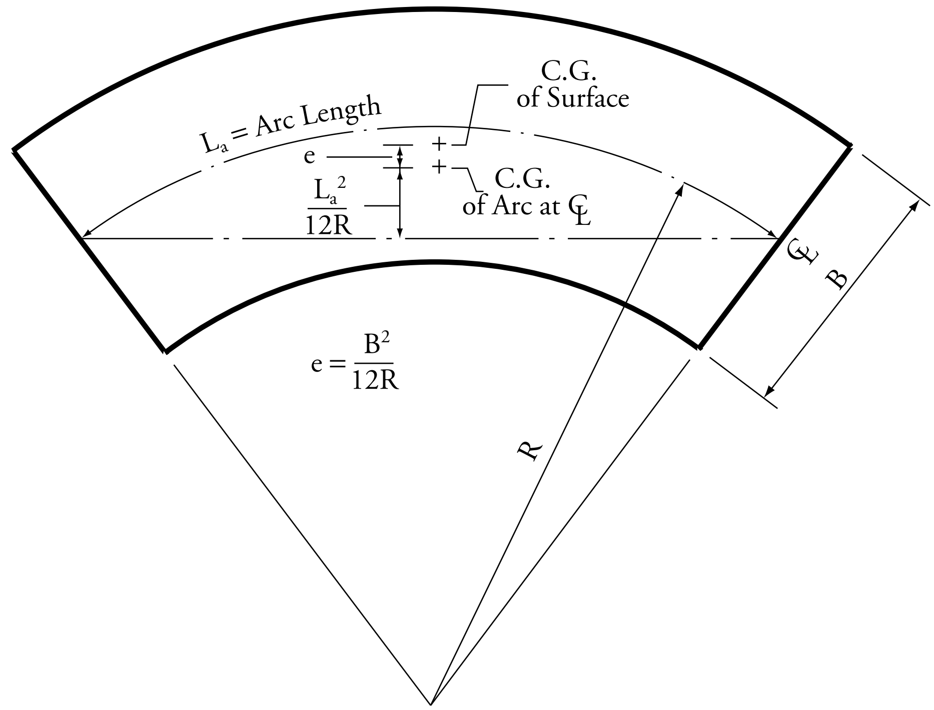
The center of gravity of a curved surface lies outside of the center of gravity of the centerline arc, because there is more area outside of the centerline than inside. This additional eccentricity, e, is equal to $B^2/12R$. The total offset from the chord to the center of gravity of the surface is therefore $(L_c^2 + B^2)/12R$. Where ends of the bridge are not radial, a more detailed calculation is required for area and centroid of the surface.

12.4.2 Useful Structural Approximations

12.4.2.1 Analysis as a Straight Beam

The bending moments in a curved beam due to vertical loads may be analyzed by considering the beam to be a straight beam of span equal to the arc length of the curved beam. This approximation is very good, and sufficiently accurate for preliminary design.

Figure 12.4.1.6-1 Properties of a Curved Planar Surface



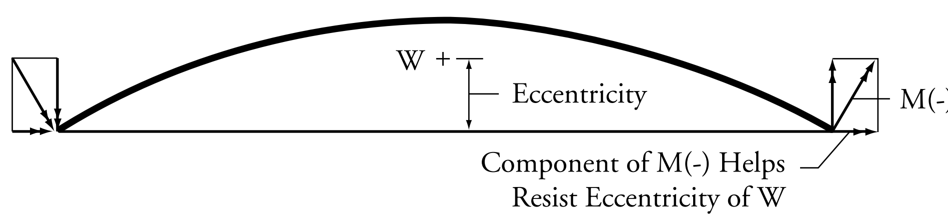
12.4.2.2 Torsion

Although flexural moments may be estimated by analyzing a straight beam of length equal to the arc length of the curved beam, the same cannot be said for torsional moments. Torsional moments are necessary for equilibrium of a curved beam. **Figure 12.4.1.5-1** shows that, as noted in Section 12.4.1.5, the center of gravity of an arc (and of loads applied along that arc) is offset from a line through the supports of a simple span beam by an amount equal to $L_c^2/12R$. The moment of weight, W, about the supports is $WL_c^2/12R$. This is resisted by torsional moments at each beam end, approximately equal to $WL_c^2/24R$. Again, because these are approximations, a known value of L_a may be used in lieu of L_c.

12.4.2.3 End Moments and Torque

The presence of end moments in continuous beams significantly reduces the torsional moments at the support. As shown in **Figure 12.4.2.3-1**, end moments have a component that helps resist the eccentricity of the weight, W, applied to the arc.

Figure 12.4.2.3-1 Negative End Moments Counteract Torsion in Continous Beams



For a uniformly loaded, fixed-ended beam, the end moment of $WL_a/12$ reduces the torsional moment at the support to (approximately) zero. For continuous beams, the torsional moment at the support will not be zero, but it will usually be less than half of the simple span torsional moment at the support. This is discussed in more detail in Section 12.5.2.

12.4.3 Design Charts

Design charts for continuous, curved box beams are given in ABAM (1988). These charts are useful for preliminary sizing of curved box beams.

12.5 STRUCTURAL BEHAVIOR OF CURVED-BEAM BRIDGES

12.5.1 Longitudinal Flexure

12.5.1.1 Analysis as a Straight Beam

As previously noted, the bending moments from longitudinal flexure are virtually the same as those for a straight beam of the developed length. However, the distribution of loads to the beams will be different in a curved bridge.

12.5.1.2 Loads on Outside Beam

The shears and moments in the exterior beam on the outside of the curve are substantially larger than for other beams in the bridge. This is caused by the following factors:

- The arc length on the outside of the curve is longer than the nominal length at the centerline of the bridge. This increases bending moments in the outer beam by (approximately) the square of the ratio of the arc lengths.
- The overhang at mid-arc may be increased by an amount equal to the arc-to-chord offset.
- Other beams will shed some of their torsional moment by shifting load toward the next beam to the outside. The outermost beam is the final resting place for this shifted load.

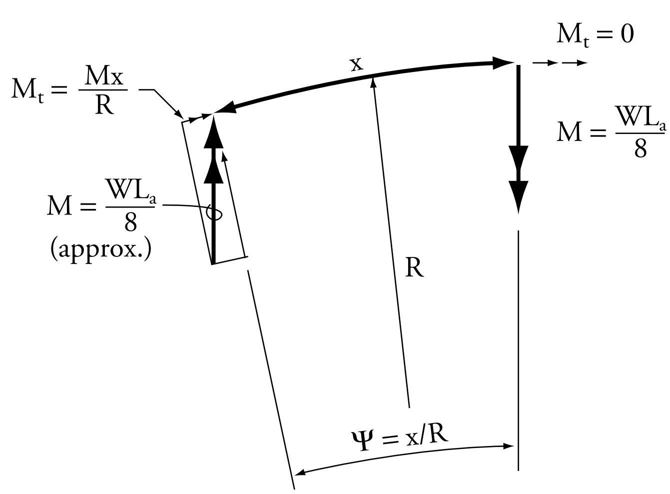
12.5.2 Torsion

It is useful to look in more detail at how torsional moments develop in a curved beam. It will be shown that torsional moments are related to the flexural moment M divided by the radius of curvature R.

12.5.2.1 Torsion in Simple-Span Beams

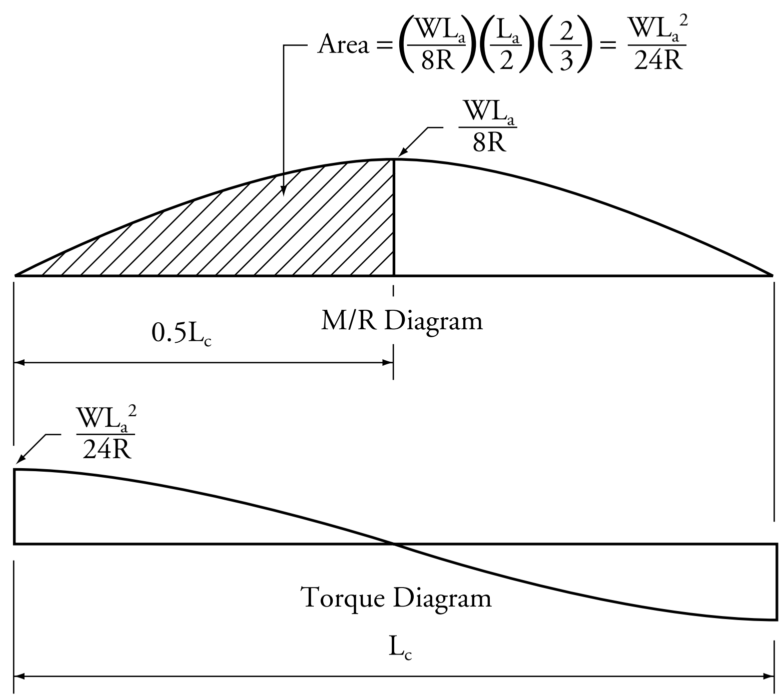
The development of torsional moments in a curved beam may be thought of in the following way. Consider a short segment near midspan of the simple-span curved beam shown in **Figure 12.5.2.1-1**.

Figure 12.5.2.1-1 Torsion and Curvature



At midspan, the bending moment is $WL_a/8$, and the torsional moment is zero (by symmetry). At a small angle, ψ, away from midspan, the bending moment must "turn" through the angle, ψ, and a torsional moment (approximately) equal to $xWL_a/8R$ is necessary for equilibrium. Following around the curve to the support, the torsional moment increases by increments of xM/R. However, M changes between midspan and the support. Integrating the M/R diagram from midspan to support, as shown in **Figure 12.5.2.1-2**, a torsional moment of $WL_a^2/24R$ is obtained. This is identical to that obtained from equilibrium in Section 12.4.2.2.

Figure 12.5.2.1-2
Torque in a Simple-Span Curved Beam



12.5.2.2
Torsion in Continuous Beams

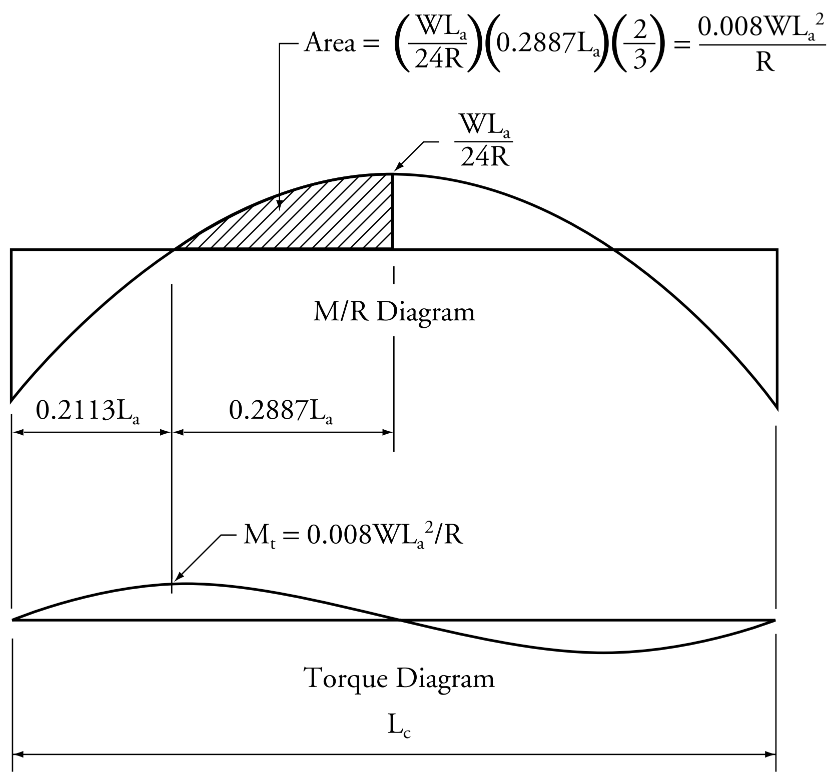
Torsion in continuous beams may be understood by first examining torsion in a fixed-ended beam. **Figure 12.5.2.2-1** shows the M/R diagram for a fixed-ended beam.

Because the area under the M/EI diagram for a fixed-ended beam must integrate to zero, the area under the M/R diagram will also integrate to zero, given constant EI and R. Thus, the torsion at the support will be zero. The maximum torque occurs at the inflection point, and is 19 percent of the maximum torque in a simple-span beam.

Continuous beams are intermediate between simple-span and fixed-ended beams. Interior spans resemble the fixed case more closely, and the free end of exterior spans may be closer to the simple-span case.

Continuity can significantly reduce torsional moments.

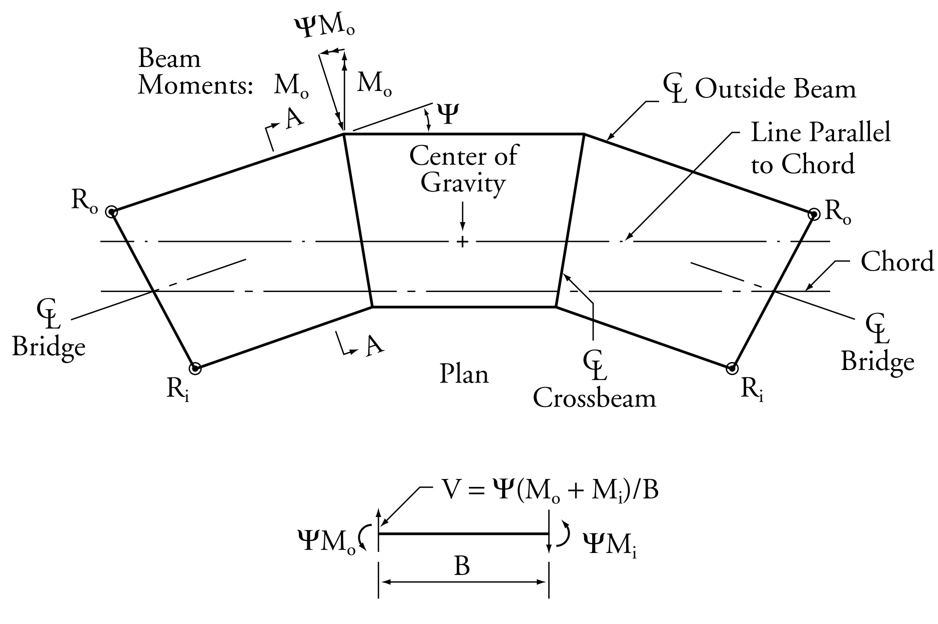
Figure 12.5.2.2-1
Torque in a Fixed-Ended Curved Beam



12.5.2.3
Behavior of Beam Gridworks

Beam gridworks composed of straight beam segments can resist eccentric loads without torsion. **Figure 12.5.2.3-1** shows a simple two-beam, three-segment gridwork.

Figure 12.5.2.3-1
Simple Gridwork



The beam moment at a joint must "turn the corner." In this case, equilibrium is supplied by a bending moment in the crossbeam. This bending moment in the crossbeam is equal to the angle (in radians) between the two beam segments multiplied by the bending moment in the main beam.

An equilibrium sketch of the crossbeam is shown in **Figure 12.5.2.3-1**. The moments at the two ends of the beam are equilibrated by shear forces, which transfer load from the inner to outer beam.

Note that for a two-beam gridwork, the reactions may be determined by statics, because the resultant of the reactions at each end must lie on a line through the resultant location of the loads. For multiple beam gridworks, reactions may be estimated by assuming a straight-line distribution of reactions that produces the correct location of the resultant. A procedure similar to that described in the *LRFD Specifications* Commentary [Article C4.6.2.2.2d], may be used. This is illustrated in the Design Example, Section 12.9.5.2.

After estimating the end reaction of the outside beam, one may estimate the bending moment in the outside beam. This is done by comparison to the bending moment in a straight beam of length equal to the arc (or chord) length of the centerline of the bridge. Two correction factors are then applied to this bending moment. The first correction is the ratio of the estimated end reaction in the beam grid work of the curved bridge to that in a straight bridge. A simplifying assumption is made that the bending moment is proportional to the end reaction multiplied by the length, giving the second correction factor, the ratio of the length of the outside beam to the centerline length. The bending moment of a straight beam of length equal to the centerline length of the bridge is then multiplied by these two factors to obtain the estimate of bending moment in the outer beam.

Loads applied after the gridwork is completed can theoretically be supported without torsion. Although equilibrium could be obtained without torsion, an analysis will show a small amount of compatibility torsion. If the factored compatibility torsion is below that given in the *LRFD Specifications* [Equation 5.8.2.1-3], the torsion may be safely ignored.

12.5.3 Crossbeams

Diaphragms in straight bridges, if used at all, are usually designed empirically, i.e., the design is not based on calculated shears and moments. In curved bridges, crossbeams must be designed for the shears and moments resulting from the change in direction of the primary bending moment in the stringer at the location of the crossbeams. The longitudinal forces in the bottom flange have a transverse component at the location of the crossbeam. The crossbeam must be deep enough to brace the bottom flange to resist this component.

12.6 DETAILED DESIGN

The loading stages given in Sections 12.6.2 and 12.6.3 are for simple spans. For continuous spans, more complex loading stages may be required.

12.6.1 Validity of Approximations

Detail design is done using a beam gridwork computer model. For mathematical consistency, it is better to use "exact" plan geometry instead of the approximations used in preliminary design. The computer model may be created in a horizontal plane, ignoring grade and superelevation. The extra weight in the "haunch" (or "pad") caused by superelevation should be taken into account, however.

12.6.2 Loading Stages – Box Beams

12.6.2.1 Bare Beam

An initial stage of plant post-tensioning is applied to the bare simple span beam to assemble the beam segments into a curved beam. This effectively applies the post-tensioning and the self-weight bending moment at the same time. After erection, crossbeams are cast, and their weight is applied to the bare beam.

12.6.2.2 Non-Composite Gridwork

The weight of the deck is applied to a non-composite gridwork, assuming unshored construction.

12.6.2.3 Composite Gridwork

The weights of future wearing surface, barriers, live load plus impact, and centrifugal force are applied to the composite gridwork. The simplifying assumptions for distribution of these loads in straight bridges cannot be used for curved bridges.

Additional field post-tensioning could be applied after casting the deck to partially compensate for the weight of the deck. This should not be done if future replacement of the deck is anticipated.

12.6.3 Loading Stages – I-Beams

A three-segment, simple-span I-beam is considered.

12.6.3.1 Individual Segments

The segments are pretensioned in the plant to compensate for self-weight bending of the individual segment.

12.6.3.2 Shoring Loads

The individual segments are erected in the field, supported by final bearings and by shores at intermediate locations. Post-tensioning ducts are spliced and crossbeams are cast.

During this loading stage, stresses in the beams do not change. Loads are added to the shoring.

12.6.3.3 Non-Composite Gridwork

Post-tensioning is applied to the non-composite gridwork after the crossbeams have cured sufficiently. This lifts the beams from the shores. The load that was present in the shores becomes a load applied to the non-composite beam gridwork.

The post-tensioning is best modeled as a set of external loads. That is, all the forces applied to the concrete by the tendons and their anchors are applied as external loads to the model. It is important that the transverse forces at the crossbeams not be overlooked. These forces are caused by the tendons that change direction (in plan) at the crossbeams.

The weight of the deck and haunch is also applied to the non-composite gridwork.

12.6.3.4 Composite Gridwork

The weights of the future wearing surface, barriers, live load plus impact, and centrifugal force are applied to the composite gridwork. See **Section 12.6.2.3** for additional considerations.

12.6.4
Other Design Checks

Checking allowable stresses, deflection and camber, prestress losses and ultimate strength is generally similar to that for a straight bridge, keeping in mind the differences between post-tensioning and pretensioning.

Torsion is an additional consideration. For segmental I-beam curved bridges, the torsion will often be below the limit for which the *LRFD Specifications* [Eq. 5.8.2.1-3] permit torsion to be neglected. Full-span box beams have higher torsion from self-weight, and a torsion analysis will be needed. Fortunately, for box sections, the torsional shear may be added directly to the vertical shear [*LRFD Specifications*, Eq. 5.8.3.6.2-3], and the analysis is similar to that done for vertical shear only.

12.7 FABRICATION

It is generally more economical to ship a full-span beam to the site instead of assembling segments on site. However, curved I-beams seldom have a sufficient torsional strength to permit this; thus, segments are used.

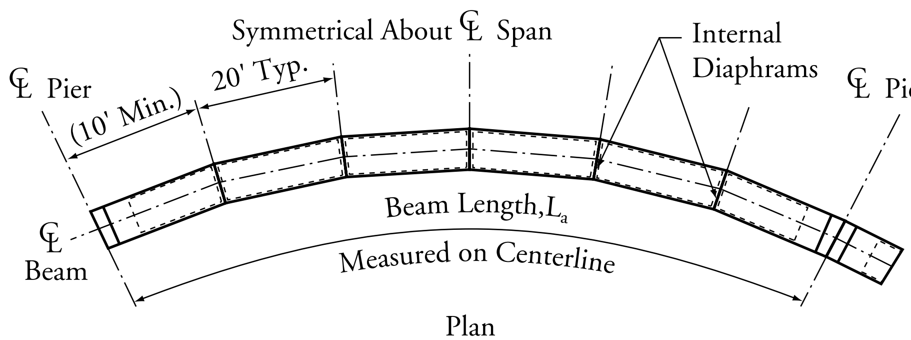
12.7.1 Box Beams

Box beams usually have enough strength to permit shipping a full-span beam. For box beams, segments would only be shipped if the full-span beam is too large to be shipped. The "bathtub" beam segments could be erected on shores, like that which is done for I-beam segments.

12.7.1.1 Chord Lengths

Chord lengths of 20 feet are suggested for curved box beams as shown in **Figure 12.7.1.1-1**. This produces a maximum arc-to-chord offset of 1 in. on a 600-ft radius and 2 in. on a 300-ft radius.

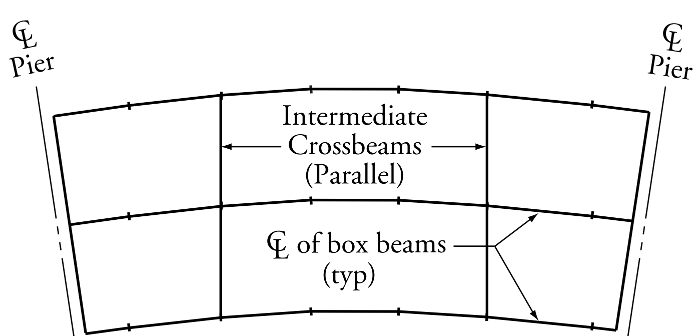
Figure 12.7.1.1-1
Precast Beam
Chorded Geometry



12.7.1.2 Bridge Layout

Using 20-ft chords, lay out the bridge so that the chords at each end are between 10-ft and 20-ft long. Lay out the crossbeams parallel to each other, so that they intersect the main beams at a form joint. These considerations will simplify beam forming and fabrication. See **Figure 12.7.1.2-1**.

Figure 12.7.1.2-1
Box-Beam-Bridge Layout



12.7.1.3 Forms

In some localities, prestressed concrete "bathtub" beams of open trapezoidal shape are used. The forms for such beams could possibly be adapted, but the top would need to be closed in order to provide a torsion box.

Side forms can be erected on a steel table, as shown in **Figure 12.7.1.3-1**. The table must be wide enough to accommodate the curvature. The 20-ft chorded side forms are secured to the table, to the desired geometry.

Another forming method is the use of form sections that form both the sides and the soffit. This is described in ABAM (1988). See **Figure 12.7.1.3-2**.

Figure 12.7.1.3-1
Chorded Forms on Flat Table

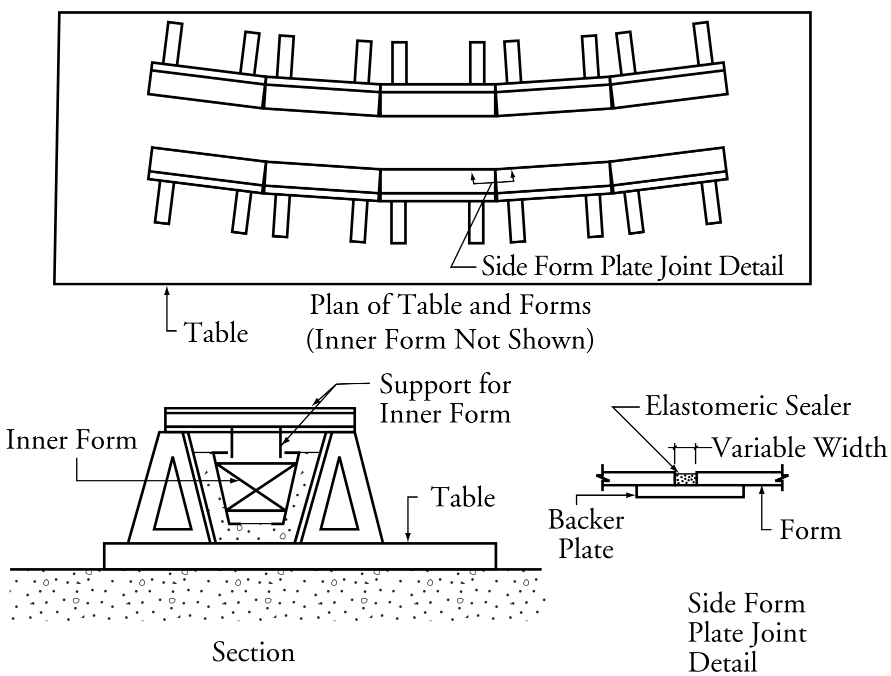
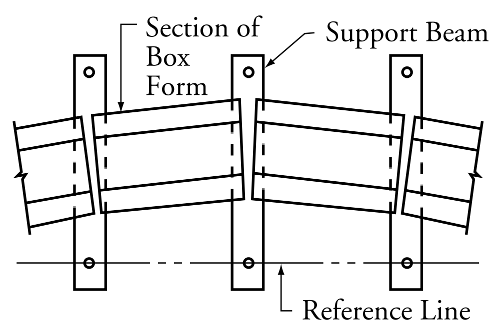


Figure 12.7.1.3-2
Plan View of Forming



12.7.1.4 Casting

Using an inner form, the beam is cast up to the underside of the top flange (top of the web). After the concrete hardens, the inner form is removed, and a stay-in-place form is used to form the top flange which is cast in a second-stage operation. Alternatively, stay-in-place void forms can be used if voids are properly anchored to prevent movement during placement of concrete and if thorough consolidation is attained under the form.

12.7.1.5 Post-Tensioning

If the complete curved box beam is prefabricated, the beam is post-tensioned and the ducts and anchorages are grouted at the plant. This may be done as a two-stage operation, with the first stage of post-tensioning done at an early age, and the final stage done after concrete design strength is achieved. Where segments of curved beams are spliced in the field due to haul limitations, additional post-tensioning will be required. If curved beams are made continuous over piers, additional post-tensioning near the piers or of the entire structure may be required in the field.

12.7.2 I-Beams and Bulb-Tee Beams

12.7.2.1 Chord Lengths

Chorded segment lengths should be made as long as is permissible (see Section 12.3.1), in order to minimize the field joints in the segmented beam. Generally two, three, or four segments should be used.

12.7.2.2 Bridge Layout

In contrast to the box beam bridge layout, it is recommended that crossbeams be on radial lines. This will result in a more consistent geometry, and the variation in length of beam segments will not cause forming problems.

12.7.2.3 Forms

Standard beam forms may be used. It is usually necessary to widen the webs to accommodate post-tensioning ducts. This can often be done by spreading the side forms. A new pallet or pan, as well as new end bulkheads, may be required.

If post-tensioning tendons are anchored at the ends of the beams, as is frequently done, end blocks will be required. End blocks are often cast with the segment but may be added later as a secondary pour. End blocks will be needed only at one end of each end segment, so odd lengths can be accommodated by adjusting the bulkhead location at the opposite end. In some cases, end blocks may be eliminated by placing post-tensioning anchorages in the end walls or end diaphragms.

12.7.2.4 Casting

The beam segments are cast in the usual manner with the addition of post-tensioning ducts and anchorages. Splices between segments are generally wet cast, so match-casting is not required.

12.7.2.5 Pretensioning

The beam segments may have a small amount of pretensioning to compensate for self-weight bending of the individual segments.

12.8 HANDLING, TRANSPORTATION AND ERECTION

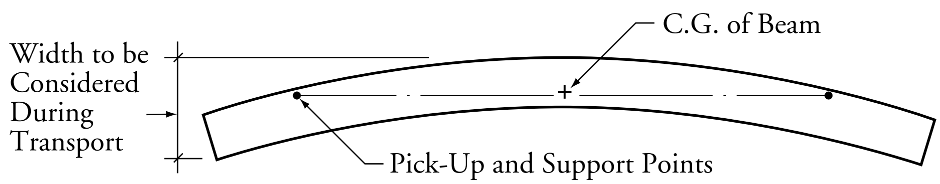
12.8.1 Box Beams

This section addresses handling considerations for curved box beams. Handling for individual straight box beam segments to be assembled in the field is similar to the handling of I-beam segments addressed in the following sections.

12.8.1.1 Handling

Pickup and support points must be located on a line through the center of gravity (in plan) of the curved beam. Pickup and temporary support points may be located inward from the beam ends, if the curvature is too great for the beam to be stable when supported at the ends. Of course, the beam stresses must be checked for the pickup and support point location. See **Figure 12.8.1.1-1**.

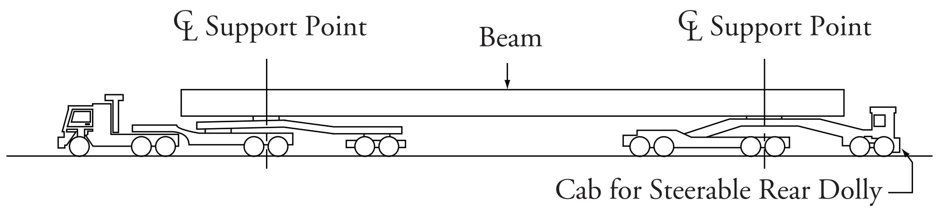
Figure 12.8.1.1-1
Pick-Up and Support Points for Curved Beam



12.8.1.2
Transportation

Long-span box beams are very heavy. The maximum span may be governed by the maximum practical transportable weight or transportable width instead of design considerations. Curved box beams may also be spliced in the field if weight or width limitations restrict transportable length. Special transporters will usually be required, as illustrated schematically in **Figure 12.8.1.2-1**, to accommodate weight and long overhangs from support points.

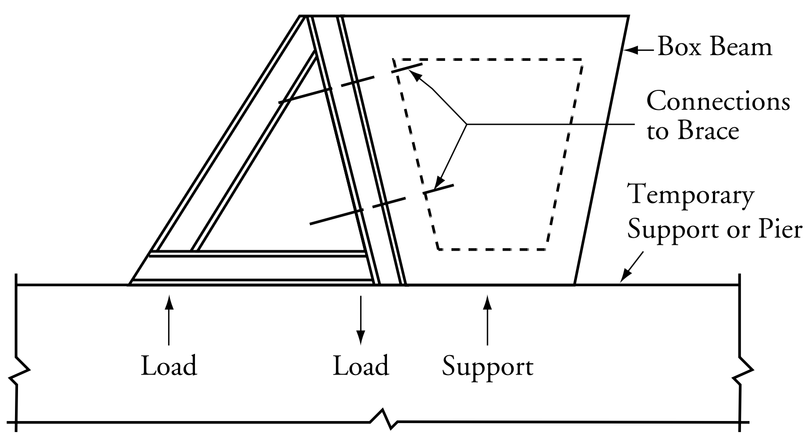
Figure 12.8.1.2-1
Beam Transporter



12.8.1.3
Erection

A temporary brace will probably be needed to stabilize the beam after erection, as shown in **Figure 12.8.1.3-1**. This brace needs to remain in place until the end and intermediate diaphragms are cast.

Figure 12.8.1.3-1
Schematic of Temporary Brace to Stabilize Beam



12.8.2
I-Beams and Bulb-Tee Beams

12.8.2.1
Handling and Transportation

The I-beam segments present no unusual difficulties in handling and transportation.

12.8.2.2
Erection and Post-Tensioning

The I-beam segments are typically erected on shoring located at the interior cross-beam closure pours. The beams generally will rise off of the shores as they are post-tensioned.

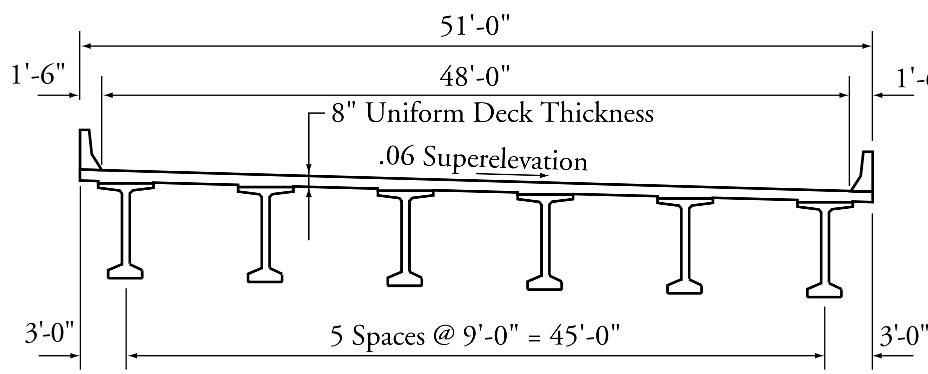
12.9 DESIGN EXAMPLE

12.9.1 Introduction

This design example demonstrates the preliminary design of a 120-ft, simple-span, bulb-tee-beam bridge on a 600-ft radius curve. Except for changes brought about by the curvature, the bridge is the same as that designed in Section 9.4. The 120-ft span is measured along the arc at the centerline of the bridge. The bridge is superelevated 6 percent, and the design speed is 40 mph. The splices, intermediate diaphragms and piers are all arranged radial to the curve.

The superstructure consists of six beams spaced at 9'-0" centers, as shown in **Figure 12.9.1-1**. Beams are designed to act compositely with the 8-in. cast-in-place deck to resist all superimposed dead loads, live loads and impact. A 1/2-in. wearing surface is considered an integral part of the 8-in. deck. The design is accomplished in accordance with the *LRFD Specifications*, 2nd Edition, 1998. Design live load is HL-93.

Figure 12.9.1-1 Bridge Cross-Section at Midspan



12.9.1.1 Plan Geometry

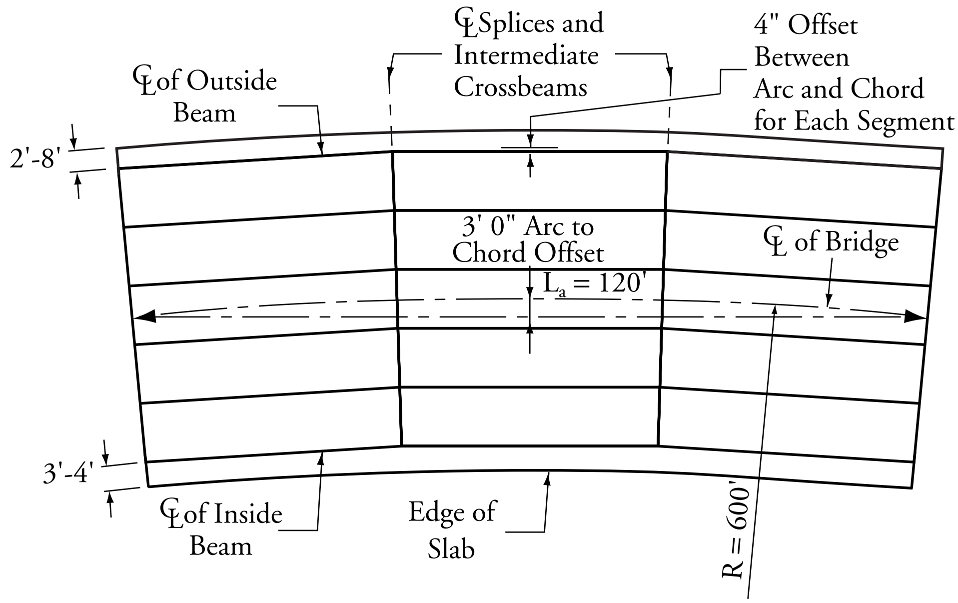
Check to see if straight beams might be used. The arc-to-chord offset is $L_a^2/8R = (120)^2/(8 \times 600) = 3$ ft. This exceeds the maximum recommended offset of 1.5 ft. If the beam is subdivided into three chords, the maximum offset will be reduced by a factor of $(3)^2$, producing an offset of 4 in. at the center of each chord. This will be barely detectable visually and will be acceptable. In order to minimize the overhang on the outside of the curve, the 3-ft overhang will be set at the middle of each chorded segment. At the ends of the chorded segments, the overhang from beam centerline on the outside will be 2'-8", and 3'-4" on the inside. **Figure 12.9.1.1-1** shows the plan geometry.

12.9.1.2 Construction

With a nominal chord offset of 3 ft for the span, the torsion will be too great for a plant post-tensioned, full-length beam. Therefore, segmental (spliced) construction will be used. Each 40-ft (nominal length) straight segment will be precast with enough pretension to compensate for self-weight on the 40-ft span. Shoring will be erected at the 1/3 points of the 120-ft span. The 40-ft segments will then be set on the shores and on the end bearings at the abutments. Crossbeams, 12-in.-thick by 66-in.-deep, will be cast at the ends. Crossbeams with the same cross-section will be cast at the 1/3 points (splice locations). The beams will then be post-tensioned and the shores removed.

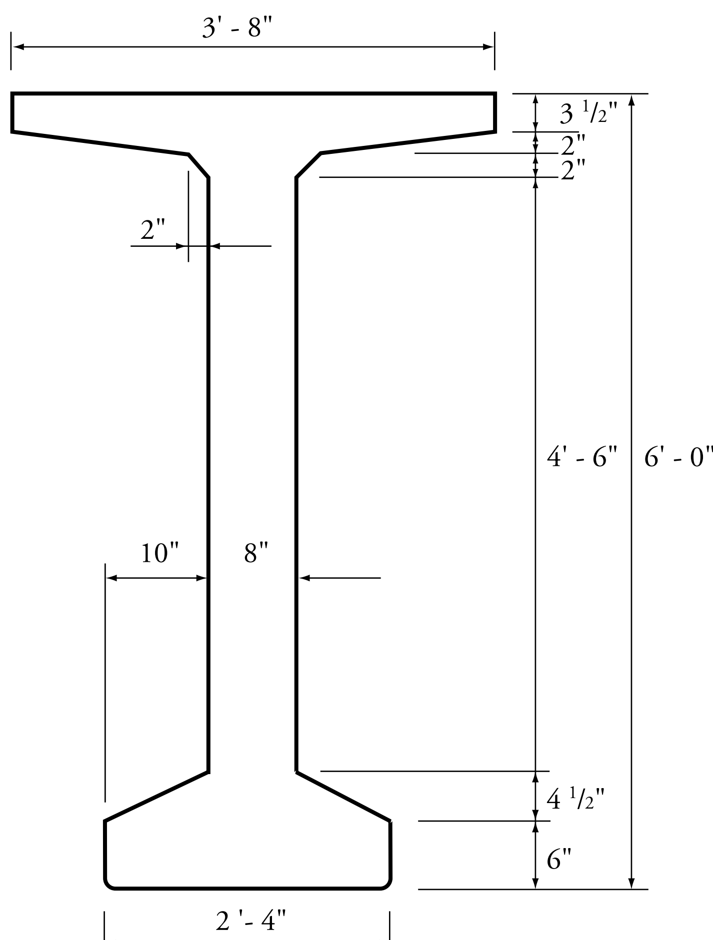
The deck is generally cast after post-tensioning. This procedure makes it feasible to replace the deck in the future, should that become necessary.

Figure 12.9.1.1-1
Beam Framing Plan Geometry



Because the girders are post-tensioned, a thicker web will be used to provide necessary cover over the ducts. This may be accomplished by spreading the side forms for an AASHTO-PCI BT-72 by 2 in., creating an 8-in.-thick web. See **Figure 12.9.1.2-1** for modified section dimensions.

Figure 12.9.1.2-1
AASHTO-PCI BT-72
Dimensions with 2 in.
Added to Width



12.9.2 Materials

These are almost identical to those used in the Section 9.4 example.

Cast-in-place slab: Actual thickness, t_s = 8.0 in.

Structural thickness = 7.5 in.

Note that a 1/2-in. wearing surface is considered an integral part of the 8-in. deck.

Concrete strength at 28 days, f'_c = 4.0 ksi

Precast beams: AASHTO-PCI Bulb-tee with 2-in.-added width as shown in **Figure 12.9.1.2-1**

Concrete strength of beam at post-tensioning, f'_{ci} = 6.5 ksi

Concrete strength at 28 days, f'_c = 6.5 ksi

Concrete unit weight, w_c = 0.150 kcf

Design span = 120.0 ft (Arc length at centerline of bridge)

Post-tensioning strands: 0.6-in. dia, seven-wire, low-relaxation

Area of one strand = 0.217 in.^{2}

Ultimate strength, f_{pu} = 270.0 ksi

Yield strength, $f_{py} = 0.9f_{pu}$ = 243.0 ksi [LRFD Table 5.4.4.1-1]

Stress limits for post-tensioning strands: [LRFD Table 5.9.3-1]

at jacking: $f_{pj} = 0.80f_{pu}$ = 216.0 ksi

at service limit state (after all losses):
$f_{pe} < 0.80f_{py}$ = 194.4 ksi

Modulus of elasticity, E_p = 28,500 ksi [LRFD Art. 5.4.4.2]

Reinforcing bars: Yield strength, f_y = 60 ksi

Modulus of elasticity, E_s = 29,000 ksi [LRFD Art. 5.4.3.2]

Future wearing surface: 2 in. additional concrete, unit weight = 0.150 kcf

New Jersey-type barrier: Unit weight = 0.300 kip/ft/side

12.9.3 Cross-Section Properties

12.9.3.1 Non-Composite Sections

A = area of cross-section of precast beam = 911 in.^{2}

h = overall depth of beam = 72 in.

I = moment of inertia about the centroid of the precast beam = 608,109 in.^{4}

y_b = distance from centroid to extreme bottom fiber of the precast beam = 36.51 in.

y_t = distance from centroid to extreme top fiber of the precast beam = 35.49 in.

S_b = section modulus for the extreme bottom fiber of the precast beam = 16,657 in.^{3}

S_t = section modulus for the extreme top fiber of the precast beam = 17,134 in.^{3}

I_{lat} = lateral moment of inertia of precast beam = 46,014 in.^{4}

w_g = beam self-weight per unit length = 0.949 kip/ft

E_c = modulus of elasticity, ksi = $33{,}000(w_c)^{1.5}\sqrt{f'_c}$ [LRFD Eq. 5.4.2.4-1]

where

w_c = unit weight of concrete = 0.150 kcf

The *LRFD Specifications*, Commentary C5.4.2.4, indicates that the unit weight of normal weight concrete is 0.145 kcf. However, precast concrete mixes typically have a relatively low water/cementitious materials ratio and high density. Therefore, a unit weight of 0.150 kcf is used in this example. For high strength concrete, this value may need to be increased based on test results.

f'_c = specified strength of concrete, ksi

Therefore, the modulus of elasticity for:

cast-in-place slab, $E_{cs} = 33{,}000(0.150)^{1.5}\sqrt{4.0} = 3{,}834$ ksi

precast beam at transfer of post-tensioning (at 28 days minimum)

$$E_{ci} = 33{,}000(0.150)^{1.5}\sqrt{6.50} = 4{,}888 \text{ ksi}$$

precast beam at service loads, $E_c = 33{,}000(0.150)^{1.5}\sqrt{6.50} = 4{,}888$ ksi

The torsional constant, J, is estimated in accordance with *LRFD Specifications*, and Section 7.6.5.

$$J \approx \frac{A^4}{40.0 I_p}$$

The polar moment of inertia I_p is equal to the sum of I and I_{lat}. $I_p = 654{,}123$ in.^{4}

$$J \approx \frac{911^4}{40(654{,}123)} = 26{,}324 \text{ in.}^4$$

Properties of the 12- by 66-in. crossbeam:

$A = 792$ in.^{2}

$I = 287{,}496$ in.^{4}

$I_{lat} = 9{,}504$ in.^{4} (for lateral bending)

$J = 33{,}120$ in.^{4}

$w_g = 0.825$ kip/ft

12.9.3.2 Composite Sections

12.9.3.2.1 Effective Flange Width

Because this is a preliminary design it is reasonable to assume the same properties for interior and exterior beams. Therefore, the properties for a typical interior beam are used. Final designs will require more thorough calculations.

Effective flange width for interior beams shall be the lesser of: [LRFD Art. 4.6.2.6.1]

- (1/4) span = (120 x 12)/4 = 360 in.
- $12t_s$ plus greater of web thickness or 1/2 beam top flange width
  = (12 x 7.5) + (0.5 x 44) = 112 in.
- average spacing between beams = (9 x 12) = 108 in.

Therefore, the effective flange width is 108 in. for the beam.

For the interior crossbeams, the effective flange width is (12 x 7.5) + 12 = 102 in.

Note that the crossbeam in a curved bridge is not an ordinary beam spanning between main beams (9 ft in this case). Rather, it transfers load all the way across the bridge from inside to outside beams.

12.9.3.2.2 Modular Ratio

Modular ratio between slab and beam materials, $n = \dfrac{E_{cs}}{E_c} = \dfrac{3{,}834}{4{,}888} = 0.7845$

12.9.3.2.3 Transformed Section Properties

Transformed flange width for interior beams = n (effective flange width)

= (0.7845)(108) = 84.73 in.

Transformed flange area for interior beams = n(effective flange width)(structural thickness)

= (0.7845)(108)(7.5) = 635.45 in.^{2}

Note: Only the structural thickness of the deck, 7.5 in., is considered.

A minimum haunch thickness of 1/2 in. at midspan is considered in the structural properties of the composite section. The superelevation will cause the average thickness of the haunch to be greater than 1/2 in. The extra weight will be accounted for, but the extra thickness caused by superelevation will conservatively be neglected in computing composite section properties. In addition, the width of haunch must be transformed.

Transformed width of haunch = (0.7845)(44) = 35.52 in.

Transformed area of haunch = (0.7845)(44)(0.5) = 17.26 in.^{2}

Note that the haunch should only be considered to contribute to section properties if it is required to be provided in the completed structure. Therefore, some designers neglect its contribution to the section properties.

A_c = total area of the composite section = 1,564 in.^{2}

h_c = overall depth of the composite section = 80 in.

I_c = moment of inertia of the composite section = 1,208,734 in.^{4}

y_{bc} = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = 53.05 in.

y_{tg} = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 18.95 in.

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the deck = 26.95 in.

S_{bc} = composite section modulus for the extreme bottom fiber of the precast beam = 22,784 in.^{3}

S_{tg} = composite section modulus for the top fiber of the precast beam = 63,792 in.^{3}

S_{tc} = composite section modulus for the extreme top fiber of the deck slab = 57,176 in.^{3}

I_{clat} = moment of inertia of composite section for lateral bending = 666,423 in.^{4}

For computing J_c, the torsional constant for the composite beam, half the composite flange width is used to compute the area A_c and the polar moment of inertia I_{pc} for substitution in Eq. C4.6.2.2.1-2 in the *LRFD Specifications*. The transformed area A_c is 1,246 in.^{2} and I_{pc} is 1,118,680 in.^{4} This results in a value of J_c of 53,865 in.^{4}

Composite properties of interior crossbeams:

A_c = 1,397 in.^{2}
I_c = 765,432 in.^{4}
I_{clat} = 529,860 in.^{4} for lateral bending
J_c = 54,204 in.^{4}

12.9.4 Loads

For a first approximation, all loads except the truck load will be assumed to be distributed over the area of the deck. Later, after a beam gridwork model is created, the computer program will generate member self-weights.

12.9.4.1 Dead Loads

12.9.4.1.1 Dead Loads Acting on the Non-Composite Structure

Beam and crossbeam weight:
Beams = (6)(120 ft)(0.949 kip/ft) = 683 kips
Crossbeams = (4)(45 ft)(0.825 kip/ft) = 149 kips
Total weight of beams and crossbeams = 683 + 149 = 832 kips

Deck weight:
Gross area of deck = (120 ft)(51 ft) = 6,120 ft^2
Actual thickness = 8 in.
Deck weight = [8 in./(12 in./ft)](0.150 kcf)(6,120 ft^2) = 612 kips

For a minimum haunch thickness of 0.5 in., the superelevation of 0.06 will cause the average haunch thickness to be 0.5 in. + 0.06(22 in.) = 1.82 in., say 2 in. The haunch weight is 0.150 kcf (2 in.)(44 in.)/(144 in.^{2} /ft^2) = 0.092 kip/ft/beam

Haunch weight = (6)(120 ft)(0.092 kip/ft) = 66 kips
Weight of deck, including haunch = 678 kips

12.9.4.1.2 Dead Loads Acting on the Composite Structure

Barrier weight is given as 0.3 kip/ft/side
Barrier weight = (2)(120 ft)(0.3 kip/ft) = 72 kips
Future wearing surface is 0.025 ksf
(0.025 ksf)(120 ft)(48 ft) = 144 kips
Dead load on composite structure = 72 + 144 = 216 kips

12.9.4.1.3 Total Dead Load

Total dead load = 832 + 678 + 216 = 1,726 kips

12.9.4.2 Live Loads

Design live loading is HL-93 which consists of a combination of: [LRFD Art. 3.6.1.2.1]

1. Design truck or design tandem with dynamic allowance.
   - The design truck is the same as the HS20 design truck specified by the *Standard Specifications*. [LRFD Art. 3.6.1.2.2]
   - The design tandem consists of a pair of 25.0-kip axles spaced at 4.0 ft apart. [LRFD Art. 3.6.1.2.3]
2. Design lane load of 0.64 kip/ft without dynamic allowance. [LRFD Art. 3.6.1.2.4]

IM = 33% [LFRDTable 3.6.2.1-1]

where IM = dynamic load allowance, applied to design truck or design tandem only

The number of design lanes is computed as:

Number of design lanes = the integer part of the ratio of w/12, where w is the clear roadway width, ft, between the curbs: [LRFD Art. 3.6.1.1.1]

w = 48 ft

Number of design lanes = integer part of (48/12) = 4 lanes

Multiple presence factor, m: [LRFD Table 3.6.1.1.2-1]

For 4 lanes, m = 0.65.

Stresses from truck and lane loads obtained from refined analysis will be multiplied by 0.65.

12.9.4.2.1 Lane Loading

The lane load is positioned over a 10-ft width within the 12-ft design lane. [LRFD Art. 3.6.1.3.1]

To maximize the effect of the live load, the 10-ft loaded width is shifted to the left within each design lane. This causes the lane load to have an eccentricity of 1 ft relative to the lane centerline, and the four lane loads have an eccentricity of 1 ft relative to the bridge centerline. The average arc length increases by the ratio of 601-ft radius/600-ft radius, to 120.2 ft.

The total lane loading for the four design lanes is (4)(120.2 ft)(0.64 kip/ft)(0.65) = 200.0 kips.

The 0.65 factor above is the factor, m.

12.9.4.2.2 Truck Loading

The total weight of the design truck is 8 + 32 + 32 = 72 kips.

Including 33% impact, 1.33 x 72 = 95.76 kips.

For 4 trucks, including the multiple presence factor, m:

4(95.76)(0.65) = 249.0 kips

Note that because this is a preliminary design of the main members of a 120-ft span, the tandem load need not be considered at this time.

12.9.4.2.3
Total Live Load

Total live load = 200.0 + 249.0 = 449.0 kip

12.9.4.2.4
Centrifugal Force

[LRFD Art. 3.6.3]

The design speed is 40 mph. The centrifugal force coefficient is given by:

$$C=\left(\frac{4}{3}\right)\frac{v^2}{gR}$$ [LRFD Eq. 3.6.3-1]

where

C = coefficient to compute centrifugal force

v = highway design speed, ft/sec

g = gravitational acceleration, 32.2 ft/sec^2

R = radius of curvature of traffic lane, ft

The design speed in ft/sec = 40 mph/0.682 = 58.65 ft/sec

$$C=\left(\frac{4}{3}\right)\frac{(58.65)^2}{(32.2)(600)}=0.2374$$

This is applied to the truck axle loads only, without the dynamic load allowance, and with the factor, m. The centrifugal force for four trucks is 4(72 kip)(0.2374)(0.65) = 44.4 kips.

12.9.5
Correction Factors

The bending moments in the exterior beam on the outside of the curve will be greater than in a straight bridge for three reasons:

1. The additional span length on the outside of the curve.
2. The center of gravity of the curved centerline lies outside of a line through the centerline of the supports.
3. The center of gravity of an area load is further shifted outward, because there is more area outside of the centerline than inward of the centerline.

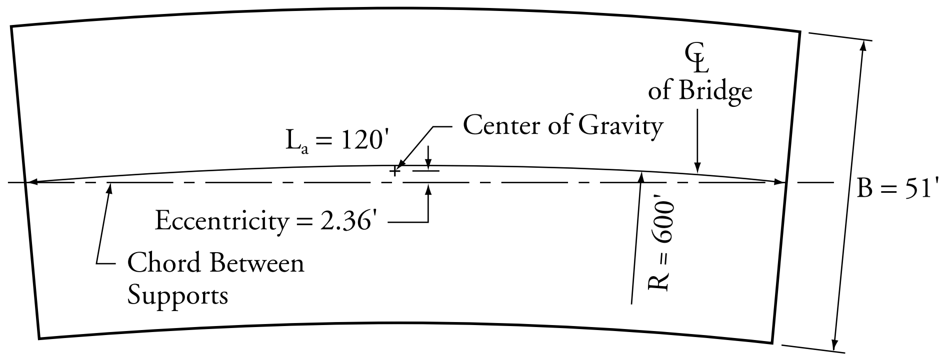
12.9.5.1
Additional Span Length Factor

The outside beam is on a radius of 622.5 ft. This increases the span length by a factor of 622.5/600 = 1.0375.

12.9.5.2
Shift in Center of Gravity

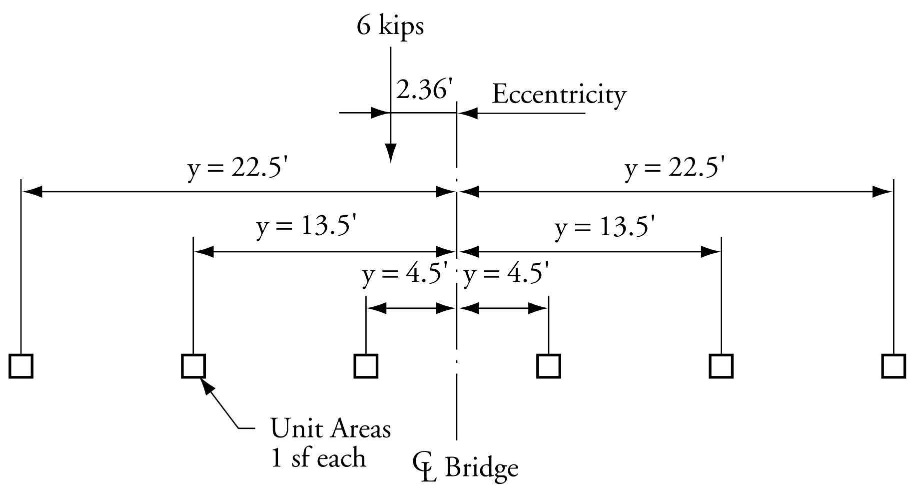
The center of gravity (in plan) of the centerline arc is offset from a line through the center of the bearings by an amount equal to 2/3 of the arc-to-chord offset (2/3)(3 ft) = 2 ft. The additional eccentricity caused by the extra area outside the centerline is equal to $B^2/12R = (51\text{ ft})^2/(12)(600) = 0.36$ ft, as shown in **Figure 12.9.5.2-1**. For the initial simplification that all dead load is an area load, the eccentricity of the dead load is 2.36 ft.

Figure 12.9.5.2-1
Center of Gravity of Curved Area



The next step is to find how much the load on the outside beam is increased because of this eccentricity. The procedure is analogous to one described in the LRFD Commentary [Article C4.6.2.2.2d] (see **Figure 12.9.5.2-2**). For six unit areas at 9-ft spacing, the moment of inertia is 1,417.5 ft^4 and the section modulus is 63 ft^3. For an arbitrary load of 1 kip per bearing, or 6 kips, at 2.36 ft eccentricity, $P/A + Pe/S = 1 + 6(2.36)/63 = 1.2248$. This is the increase in load on the outside exterior beam caused by the eccentricity of the load. The total correction factor for bending moment due to dead load is (1.0375)(1.2248) = 1.271.

Figure 12.9.5.2-2
Properties of Group of Beam Supports



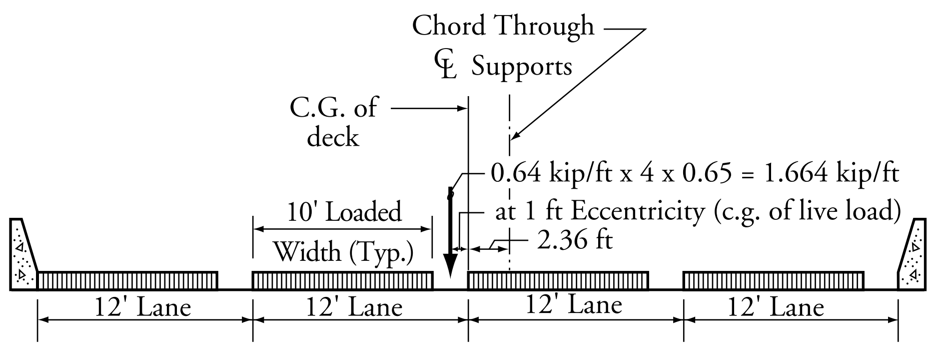
$A = 6\text{ ft}^2$

$I = \Sigma[(\text{Unit Area})(y^2)] = 1{,}417.5\text{ ft}^4$

$S = I/y_{max} = 63\text{ ft}^3$

For the lane loading, the LRFD requirement to place the load off-center of the lane adds 1 ft to the eccentricity. See **Figure 12.9.5.2-3**. For a 6-kip load at 3.36-ft eccentricity, the load on the outside beam is 1 + 6(3.36)/63 = 1.32. The total correction factor for lane loading is (1.0375)(1.32) = 1.370.

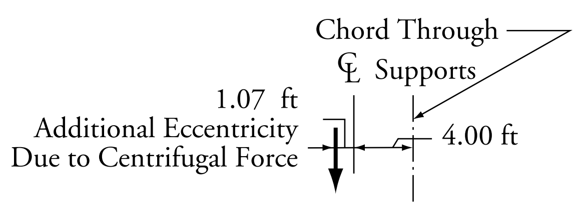
Figure 12.9.5.2-3
Lane Load Eccentricity



Lane Loading [LRFD Article 3.6.1.2.4]

For the truck loading, LRFD Article 3.6.1.3.1 specifies that the center of the wheel load be placed 2 ft from the curb. This causes the center of the vehicle to be 5 ft from the curb (also the lane edge), so the eccentricity from the centerline of the lane is 1 ft. The trucks are in the center of the bridge, which has a 3-ft eccentricity with respect to the supports. Thus, the vertical truck loading has an eccentricity of 4 ft as shown in **Figure 12.9.5.2-4**.

Figure 12.9.5.2-4
Truck Load Eccentricity



Truck Loading [LFRD Articles 3.6.1.3.1 and 3.6.3]

The effects of centrifugal force must also be taken into account. The total centrifugal force of 44.4 kips acts at a height of 6 ft [LRFD Art. 3.6.3]. The vertical truck loading is 249 kips. The horizontal force acting at 6 ft increases the eccentricity of the vertical load by (44.4/249)(6 ft) = 1.07 ft. The total eccentricity of the vertical truck load is 5.07 ft, and the correction is 1 + 6(5.07)/63 = 1.483, as shown in **Figure 12.9.5.2-4** The total correction factor due to centrifugal force and truck loading is (1.0375)(1.483) = 1.538.

12.9.6 Bending Moments – Outside Exterior Beam

The bending moments in the outside exterior beam may now be estimated. For all loads, the bending moment may be estimated as that for a 120-ft straight beam multiplied by the correction factor. For all loads except the truck loadings, the 120-ft straight beam bending moment is WL/8 divided by six beams in the bridge. For the truck loading, the bending moment is scaled from that for a standard truck on a 120-ft straight span. **Table 12.9.6-1** is a summary of the estimated midspan bending moments for the outside exterior beam.

Table 12.9.6-1
Estimated Bending Moments in Outside Beam

	Total Weight W, kips	Moment for 120-ft Straight Beam ft-kips	Correction Factor	Moment for Curved Beam ft-kips	Interior Beam, Straight Bridge, ft-kips*
Beam & Crossbeam	832	2,080	1.271	2,644	1,438
Deck & Haunch	678	1,695	1.271	2,154	1,660
Barrier	72	180	1.271	229	180
Wearing Surface	144	360	1.271	458	360
Truck Loading, w/impact	249	1,080	1.538	1,662	1,830
Lane Loading	200	500	1.370	685	843
Total	–	**5,895**	–	**7,830**	**6,311**

* *Bending moments in the right column are taken from Table 9.4.4-1.*

Comparing these estimates to the values in the right column taken from Table 9.4.4-1, it may be seen that the dead load moments are substantially increased, compared to the interior beam of a straight bridge. However, the live loads are decreased somewhat, because of the factor, m [LRFD Art. 3.6.1.1.2], which is not used in the approximate distribution method. It should also be noted that the curved beam is almost 20% heavier than the straight beam.

12.9.7 Stresses – Outside Exterior Beam

The next step is to verify that the chosen beam section is adequate. It is assumed that the bottom fiber stress due to the weight of the beams and crossbeams can be compensated by the prestensioning.

Table 12.9.7-1
Estimate of Bottom Fiber Stress

LOAD	Bending Moment, ft-kips	S_b or S_{bc}, in.^{3}	Bottom Fiber Stress, ksi
1. Self Weight of Beams and Crossbeams (Compensated by pretensioning)	2,644	16,657	1.905
2. Deck and Haunch	2,154	16,657	1.552
3. Superimposed Dead Load	687	22,784	0.362
4. Live Load (0.8)(2,347) =	1,878	22,784	0.989
5. Sum of 2 + 3 + 4			**2.903**
Allowable stress at transfer of post-tensioning = $(0.60)f'_{ci}$ = (0.6)(6.5) [LRFD Art. 5.9.4.1.1]			3.900

Table 12.9.7-1 shows the bottom fiber stress caused by deck weight, superimposed dead load, and live load. For service load tensile stress checks, the live load may be taken as 80 percent of the computed live load [LRFD Art. 3.4.1, limit state Service III]. The bottom fiber stress for these loads applied to the beams and crossbeams in **Table 12.9.7-1** is 2.903 ksi. The allowable temporary stress after post-tensioning (before time-dependant losses) is 3.90 ksi. Therefore, because there is sufficient margin between the actual stress after losses and the allowable stress before losses, the beam section should be adequate and the computer model may be constructed using this beam.

12.9.8 Beam Gridwork Computer Models

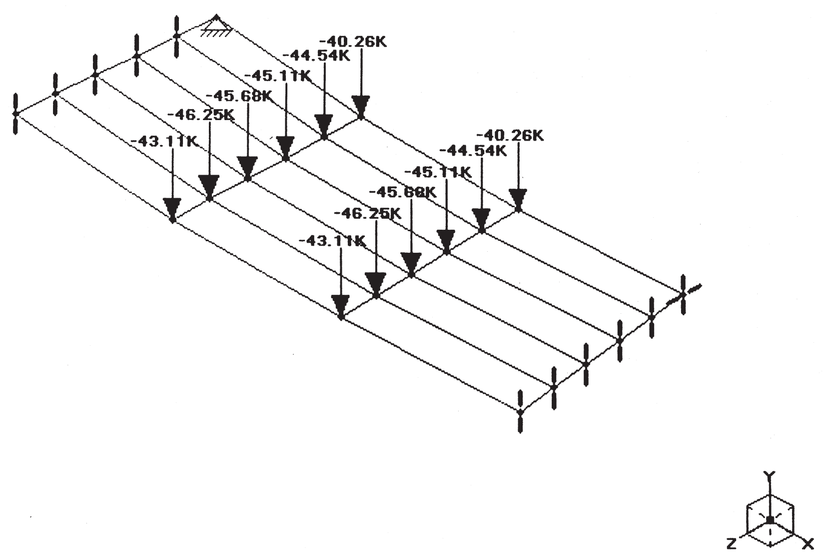
12.9.8.1 Model 1 – Beam Segments on Shores

This model (not shown) is a variation of Model 2 (see below). The ends of the members are released in bending to model the situation of simple span beam segments supported on shores. The simple span length is conservatively taken as the center-to-center distance of the crossbeams (40 ft). The reactions at each shoring location are computed. These loads are then applied to the Model 2 beam gridwork to represent removal of the shores. These loads are shown on **Figure 12.9.8.2-1**.

12.9.8.2 Model 2 – Shore Loads

Model 2 is the non-composite beam gridwork on the nominal 120-ft span. The loads applied to Model 2 are the loads that previously existed in the shores, as determined in Model 1. When the shores are removed, the loads previously existing in the shores are loads that are applied to Model 2. These loads are shown in **Figure 12.9.8.2-1**. The analysis done using Model 1 could be skipped, and the self-weight loads applied directly to Model 2. The difference in total self-weight bending moment in the outside exterior beam is less than 0.1 percent. However, it must be remembered that the moment consists of two parts, that applied to the 40-ft nominal span, and that applied to the 120-ft nominal span.

Figure 12.9.8.2-1
Non-Composite Model 2 – Shore Loads



12.9.8.3 Model 3 – Weight of Deck and Haunches

The total weight of the deck and the haunches between the deck and the top flanges of the beam was calculated to be 678 kips in Section 12.9.4.1.1. This load is assumed to be applied as a uniform load of 110.8 psf over the 6,120 sf gross area of the deck.

The model for deck weight is shown in **Figure 12.9.8.3-1**. The finite elements are only used as a means of applying a uniform load. The structural properties of the deck are zeroed out, because this is a non-composite model. The beam gridwork is the same as in Model 2.

12.9.8.4 Model 4 – Weight of Barriers and Future Wearing Surface

Model 4 represents the composite structure. Composite section properties are used in the beam gridwork. The general appearance of the model is the same as Model 3 (see **Figure 12.9.8.3-1**). The 0.025 ksf uniform load is applied over the entire 51-ft width of the deck. A net barrier load of 0.263 kip/ft (0.3 kip/ft less the 0.025 ksf acting over the 1.5-ft width occupied by the barrier) is applied as a line load along the longitudinal edges of the model.

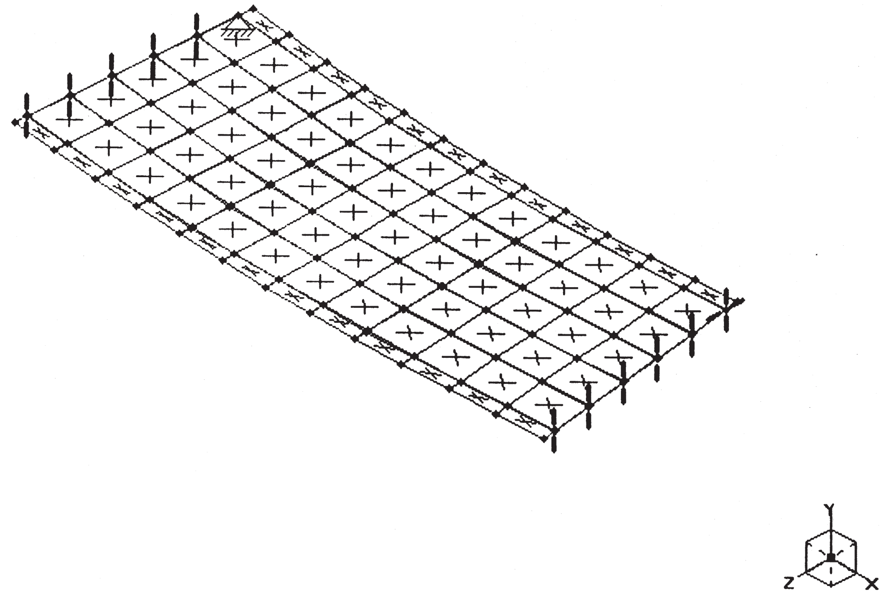


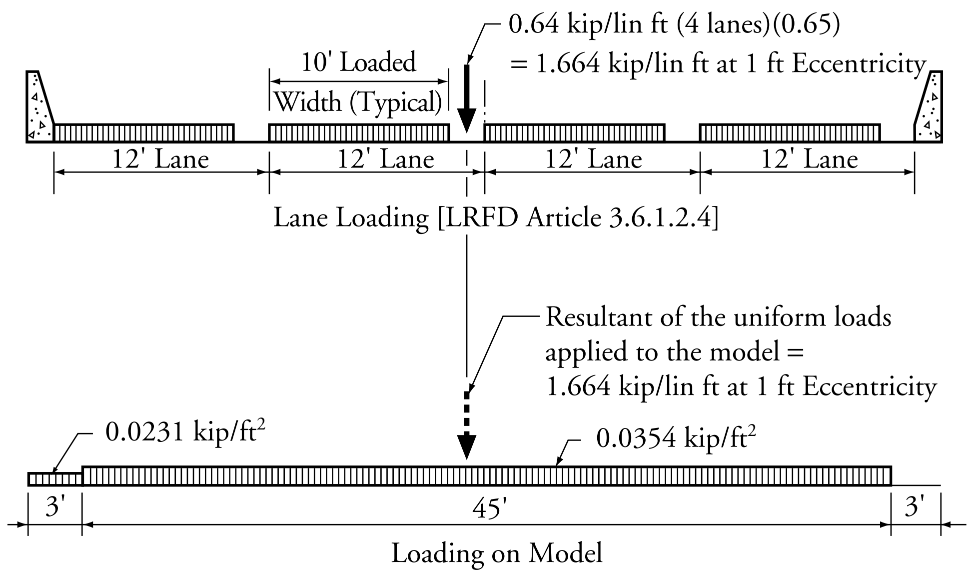
Figure 12.9.8.3-1
Non-Composite Model 3 – Deck Weight

12.9.8.5 Model 5 – Lane Loading

As noted in Section 12.9.4.2.1, LRFD Article 3.6.1.3 specifies that the design lane load be applied over a 10-ft width within the design lane width (of 12 ft in this case). This causes the resultant of the lane loads to be shifted 1 ft towards the outside of the curve.

The upper part of **Figure 12.9.8.5-1** shows the specified location of the lane loads in a cross-section through the bridge. The lower part of **Figure 12.9.8.5-1** shows the actual loads applied to the model. The loads were chosen so that deck elements would be loaded uniformly and the total load would have the correct location of the resultant load.

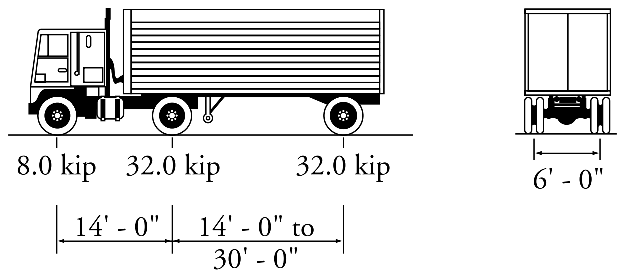
Figure 12.9.8.5-1
Lane Loading



12.9.8.6 Model 6 – Truck Loading with Centrifugal Force

The design truck is shown in **Figure 12.9.8.6-1**, which is Fig. 3.6.1.2.2-1 from the *LRFD Specifications*. For maximum positive moment, the minimum rear axle spacing of 14 ft controls. The maximum bending moment occurs with the middle axle load placed 2.33 ft from midspan.

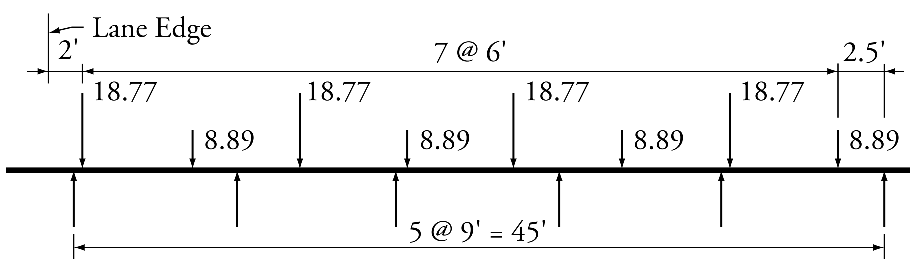
Figure 12.9.8.6-1 Characteristics of the Design Truck



The main axle wheel loads are 16 kips each, plus a 33 percent dynamic allowance, or 21.28 kips. For the design speed of 40 mph, the centrifugal force is 0.2374 of the truck weight (without dynamic allowance). This force acts 6 ft above the roadway. The overturning moment per main axle is 0.2374 times 32 kips times 6 ft, or 45.58 ft-kips. Dividing by the 6-ft wheel spacing, the wheel loads due to centrifugal force are ±7.6 kip. The total main axle wheel loads, including the 0.65 factor, m, are 0.65 (21.28 ±7.6) = 18.77 kips and 8.89 kips. The front axle wheel loads are 1/4 of this, or 4.69 kips and 2.22 kips.

The wheel loads are placed on fictitious, pin-ended members in order to transfer the loads to the main beams, as shown in **Figure 12.9.8.6-2** for the heavier axles.

Figure 12.9.8.6-2 Wheel Load Placement across Model for Heavy Axles



The added pin-ended members and loads that represent the truck loading for the condition producing maximum moment are shown in **Figure 12.9.8.6-3**.

12.9.8.7 Summary of Bending Moments

The bending moments for each of the six beams from the six loading models are summarized in **Table 12.9.8.7-1**. Pretensioning counteracts the moments from Model 1 while post-tensioning is used to counteract the moments from Models 2 through 6. The *LRFD Specifications* [Article 3.4.1 and Table 3.4.1.1], states that for checking tension in prestressed members at service load, the Service III load combination may be used. This combination is 1.00 (DC + DW) + 0.8(L + IM).

Figure 12.9.8.6-3
Truck Loading on Model 6

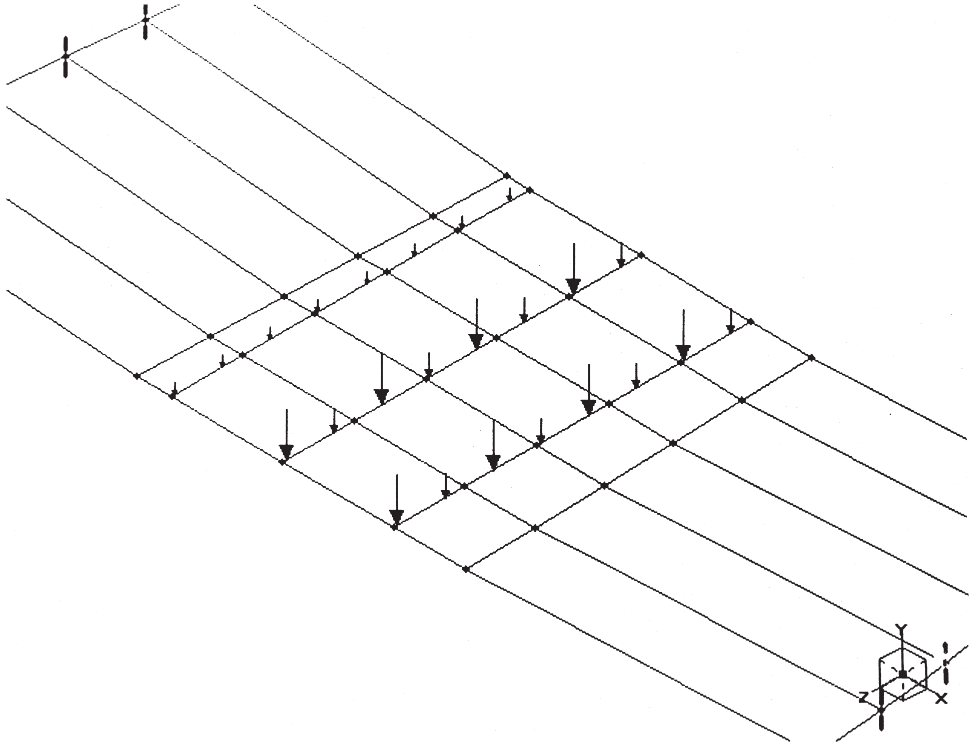


Table 12.9.8.7-1
Bending Moments in Each Beam

	Maximum Bending Moments, ft-kips					
	Beam Number					
	Outside					**Inside**
Load	**1**	**2**	**3**	**4**	**5**	**6**
Model 1 – Segments on Shores	204	199	193	187	181	176
Model 2 – Shore Loads	2,249	2,067	1,883	1,694	1,491	1,270
Model 3 – Deck & Haunch	2,119	1,973	1,823	1,662	1,479	1,286
Model 4 – Barrier & Surfacing	720	610	565	513	444	446
Model 5 – Lane Loading	649	605	551	491	420	341
Model 6 – Truck Loading	1,468	1,324	1,204	1,045	917	603
(0.8)Live Load = (0.8)(Models 5 + 6)	1,694	1,543	1,404	1,229	1,070	755
Models 2 + 3 + 4 + (0.8)(5 + 6)	6,782	6,193	5,675	5,098	4,484	3,757

12.9.9 Selection of Prestress Force

12.9.9.1 Pretensioning

The maximum self-weight bending moment for a beam segment is 204 ft-kips. The bottom fiber stress is M/S_b, or (204)(12)/(16,657) = 0.147 ksi. For y_b = 36.51 in., the eccentricity, e, is 34.51 in for strands centered at 2 in. from bottom of the beam. Try (4) 1/2 in.-dia strands with a force of 25 kips each.

Table 12.9.9.1-1 Stress Due to Pretensioning

Stress	Fiber Stress, ksi	
	Top	Bottom
P/A = 100/911	0.110	0.110
Pe/S = (100)(34.51)/S	-0.201	0.207
M/S = (204)(12)/S	0.143	-0.147
Pretension & Self-Weight	0.052	0.170

Because this is a temporary condition, a check for minimum reinforcement is not necessary.

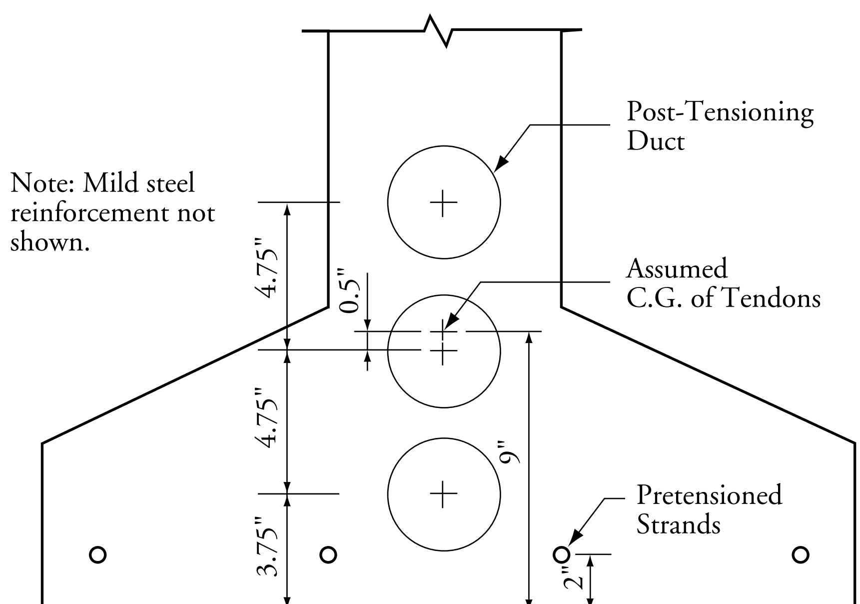
12.9.9.2 Post-Tensioning

Table 12.9.9.2-1 shows the stress to be resisted by post-tensioning. Assuming three tendons, the maximum eccentricity is estimated to be y_b less 9 in. (at the location of maximum moment) as seen in **Figure 12.9.9.2-1**.

Table 12.9.9.2-1 Bottom Fiber Stresses in Outside Beam at Location of Maximum Moment

Load	M/S	Bottom Stress, ksi
Shore Loads	(2,249)(12)/16,657 =	1.620
Deck & Haunch	(2,119)(12)/16,657 =	1.527
Barrier & Surfacing	(720)(12)/22,784 =	0.379
(0.8)Live	(1,694)(12)/22,784 =	0.892
Total Stress to be Compensated by Post-Tensioning, f_b =		**4.418**

Figure 12.9.9.2-1 Bottom Flange Detail at Maximum Moment Location



For preliminary design, assume zero tension in the bottom fiber. The required final force, P, is computed below, using non-composite section properties because the tensioning is assumed to be completed before casting the deck.

$P = f_b/(1/A + e/S_b)$

$P = 4.418/(1/911 + 27.51/16{,}657)$

$P = 1{,}607$ kips

Try (48) 0.6-in.-dia strands at 162 ksi ($0.6f_{pu}$)

$P = (48)(0.217)(162) = 1{,}687$ kips

A review of the total bending moments in **Table 12.9.8.7-1** indicates that the post-tensioning should be reduced in the other beams. Try 44 strands in Beam 2, 40 in Beam 3, 36 in Beam 4, 32 in Beam 5, and 28 in Beam 6.

12.9.9.3
Model 7 – Post-Tensioning

For the preliminary design, the post-tensioning trajectory is simplified to be three straight segments, with horizontal and vertical angle changes at the interior diaphragms. The tendons are modeled as bar elements, with a thermal coefficient equal to $1/E_p$. The tensioning of the model is done by applying a negative temperature change equal to effective prestress.

Figure 12.9.9.3-1
Model - 7 Post-Tensioning

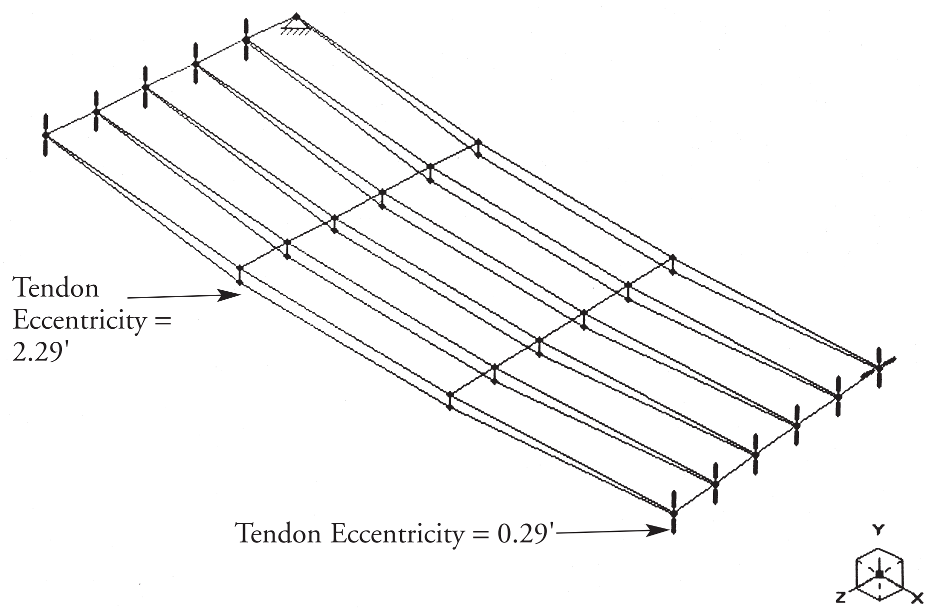


Figure 12.9.9.3-1 shows the post-tensioning model. Short, rigid stubs are used to connect the tendons to the beam gridwork. The length of these stubs is equal to the tendon eccentricity, 0.29 ft at the ends and 2.29 ft at the interior crossbeams. These stubs will also resist the transverse forces caused by the angle change of the tendons at the crossbeams.

For the middle third of the outer beam, the axial force is found to be 1,663 kips, and the bending moment due to post-tensioning is 3,650 ft-kips which agrees well with our assumptions. The tendon profiles will be held constant but the post-tensioning force changed for the remaining beams.

12.9.10 Results

12.9.10.1 Stresses in Outside Exterior Beam

Table 12.9.10.1-1 summarizes the stress history of the outside beam for the stages of service loads. The stresses are within the limits. As stated in LRFD Article 3.4.1, the Service I load combination is used to check compressive stresses, and the Service III load combination is used to check tensile stresses. Service I uses a load factor of 1.0 for live loads, whereas Service III uses a load factor of 0.8 for live loads.

Table 12.9.10.1-1 Stress Summary for Outside Beam

Load	Stresses, ksi		
	Top Slab	Top Beam	Bottom Beam
1. Pretensioning + Beam Segment Self Weight		0.152	0.170
2. Post-Tensioning: P/A = 1,663/911		1.825	1.825
3. Post-Tensioning: M/S = (3,650)(12)/S		-2.556	2.630
4. Shore Loads: M/S = (2,249)(12)/S		1,575	-1.620
5. Stress after Losses*		0.996	3.005
6. Deck & Haunch: M/S = (2,119)(12)/S		1.484	-1.527
7. Barrier & Surface: M/S = (720)(12)/S	0.151	0.135	-0.379
8. Dead Load**	0.151	2.615	1.099
9a. 0.8 Live Load: M/S = (1,694)(12)/S			-0.892
9b. 1.0 Live Load: M/S = (2,117)(12)/S	0.445	0.399	
	0.596	3.014	0.207
Stress Limits $0.60f'_c$ Compression & 0 Tension	2.400	3.900	0 Tension

* The stress before losses should also be checked. The allowable temporary compressive stress for this condition is $(0.60)f'_{ci} = (0.60)(6.5) = 3.90$ ksi. [LRFD Art. 5.9.4.2.1]

It appears, by inspection, that this stress should be OK.

** The allowable compressive stress in the beam under full dead load is $(0.45)f'_c = 2.925$ [LRFD Art. 5.9.4.2.1] OK.

12.9.10.2 Strength Limit State

The check for the strength limit state is done in the same manner as that presented in Section 9.4.9 for a straight beam. For the straight beam, the provided strength was 22 percent in excess of that required, and a similar amount of excess strength would be found for the beams in the curved bridge.

12.9.10.3 Crossbeams

The diaphragms function as crossbeams in the beam gridwork. They transfer load from the inside to the outside of the curve. This load transfer maintains equilibrium without the necessity of large torsional moments.

Figure 12.9.10.3-1 Model 2 - Crossbeam Shears and Moments

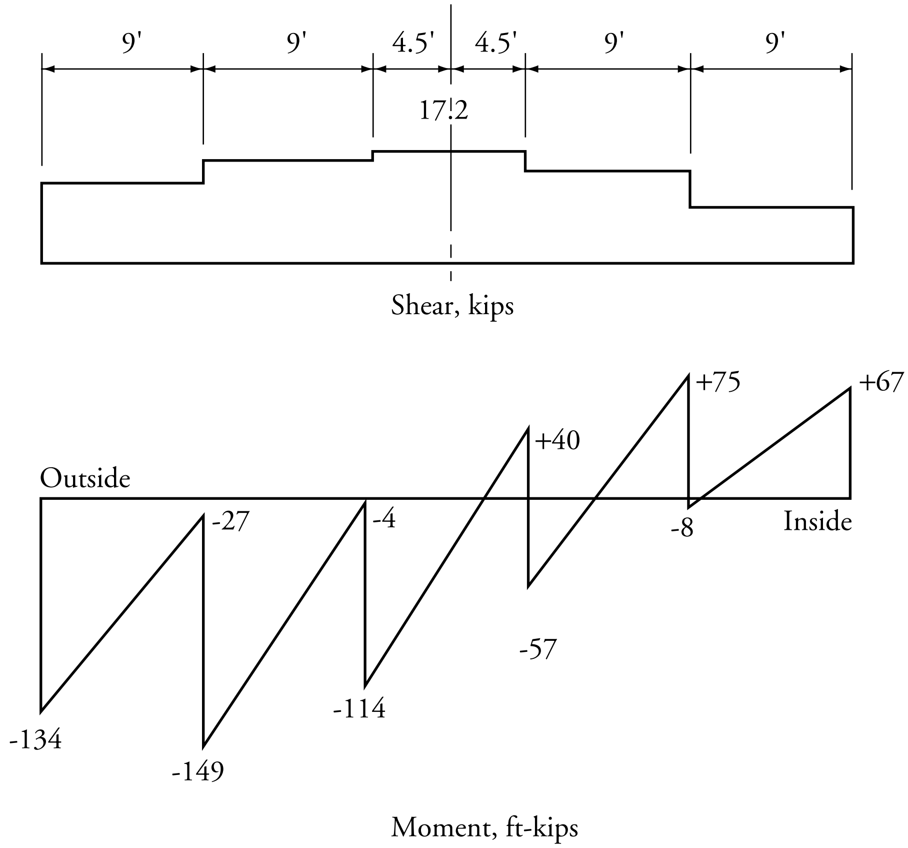


Figure 12.9.10.3-1 shows the shear and moment curves for an interior crossbeam for Model 2. The shear is relatively constant, transferring load to the outside. The crossbeam is also loaded by bending moments at each stringer. These moments equilibrate the primary bending moments in the stringers as they turn through an angle at the joint with the crossbeam.

Table 12.9.10.3-1 Factored Bending Moments in Crossbeam and at First Interior Beam

Load	M ft-kips	Load Factor	M_u ft-kips
Model 2 – Beams	-149	1.25	-186
Model 3 – Deck	-130	1.25	-163
Model 4 – Barrier & Surface	-138	1.50	-207
Model 5 + 6 – Live Loading	-110	1.75	-193
Model 7 – Prestress	-34	1.25	- 42
Total			-791

The maximum bending moment occurs at the first interior beam. **Table 12.9.10.3-1** shows the factored bending moments at this location. The *LRFD Specifications* do not give a load factor for prestressing. Because the bending from prestressing is additive to that from loads, a load factor of 1.25 (the same as for dead load) was conservatively used. The bending moments are well within the capacity of a non-prestressed beam. Although the crossbeam could be post-tensioned, the simple solution is to use a conventionally reinforced (non-prestressed) member.

12.9.10.4 Behavior Check

The behavior of the beam gridwork may be checked manually by observation of the bending moments applied to the crossbeam. The beams are bent through an angle ψ of 0.0667 radians at the crossbeam. The crossbeams must resist a moment of 0.0667 times the flexural bending moment in the beam.

Table 12.9.10.4-1 Beam Gridwork Behavior Check

Beam Number	Beam Bending Moment, ft-kips	M x ψ, ft-kips	Moment on Crossbeam, M_c ft-kips	Torque, M_t in Beam, ft-kips	$M_c + M_t$ ft-kips
1 (outside)	2,255	150	134	16	150
2	2,067	138	122	16	138
3	1,883	126	110	16	126
4	1,694	113	97	16	113
5	1,491	99	83	16	99
6 (inside)	1,264	84	67	17	84

Table 12.9.10.4-1 shows the flexural bending moment in each of the beams for Model 2, Shore Loads. The third column shows the bending moments multiplied by the angle ψ. The fourth column shows the moments in the crossbeams, from the gridwork analysis. The difference is resisted by torsion in the beams. This is compatibility torsion, caused by the fact that the bridge tilts slightly toward the outside of the curve.

12.9.10.5 Shear and Torsion

The beam gridwork is stable without torsional moments in its members. However, some torsion occurs due to the deformations of the gridwork. LRFD Article 5.8.2.1 requires torsion to be investigated when:

$$T_u > 0.25\phi T_{cr} \qquad \text{[LFRD Art. 5.8.2.1-3]}$$

where

T_u = factored torsional moment, in.-kips

ϕ = resistance factor [LRFD Article 5.5.4.2]

T_{cr} = torsional cracking moment, in.-kips, and where:

$$T_{cr} = 0.125\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_c}\right)\sqrt{1+\frac{f_{pc}}{0.125\sqrt{f'_c}}} \qquad \text{[LRFD Eq. 5.8.2.1-4]}$$

where

A_{cp} = total area enclosed by outside perimeter of concrete cross-section, in.^{2}

p_c = the length of the outside perimeter of the concrete section, in.

f_{pc} = compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange, ksi

For consistency, the transformed section is used to compute A_{cp}, p_c, and the average f_{pc} on the transformed section.

$A_{cp} = A_c = 1{,}564$ in.^{2}

$p_c = 400$ in.

$$f_{pc} = P/A_c = 1{,}663/1{,}564 = 1.063 \text{ ksi}$$

$$T_{cr} = 0.125\sqrt{6.5}\left(\frac{(1{,}564)^2}{400}\right)\sqrt{1+\frac{1.063}{0.125\sqrt{6.5}}} = 4{,}058 \text{ in.-kips} = 338 \text{ ft-kips}$$

Check if $T_u < 0.25\phi T_{cr} = 0.25(0.9)(338) = 76.1$ ft-kips

Torsion may be neglected if the ultimate torque is less than 76.1 ft-kips. Examine torsion in the outside exterior beam.

Table 12.9.10.5-1 Torsional Moments in Outside Beam

Load	T ft-kips	Load Factor	T_u ft-kips
Model 2 – Beams	-16.1	1.25	-20.1
Model 3 – Deck	-15.9	1.25	-19.9
Model 4 – Barrier & Surface	-9.5	1.50	-14.3
Models 5 + 6: Live Loads	-15.8	1.75	-27.6
Model 7 – Prestress	+21.7	0.9*	+19.5
Total			62.4

*Because the prestress acts to oppose the other torsional moments, a load factor of 0.9 was conservatively assumed.

Table 12.9.10.5-1 shows that T_u is less than 76.1 ft-kips. Therefore, torsion may be neglected.

The shear design is performed in a manner similar to that shown in Section 9.4.11 for a straight beam. Note that for post-tensioned beams, LRFD Article 5.8.2.7 requires that the effective web width, b_v, be computed deducting one half of the diameter of grouted ducts. The actual web width is 8 in., but the effective width, b_v, will be approximately 6 in., similar to that for the straight beam.

12.9.11 Comparison To Straight Bridge

Compared to the straight bridge of Section 9.4, the additional cost items for this curved bridge are as follows:

1. Additional design cost.
2. The cost and inconvenience of shoring. This may be at least partially offset by the reduced shipping and erection costs for the beam segments, as compared to full-length beams.
3. The additional cost of concrete for 2 in.increase in width of beams (1 cf/lf of added concrete) due to addition of post-tensioning.
4. The cost of intermediate crossbeams (not required for straight bridge).
5. Additional cost of post-tensioning compared to pretensioning.
6. The cost of additional strand. Less strand is used in the other five beams, but the total strand area (including pretensioned strands) for the six beams is about 20 percent greater than for the straight bridge.
7. A wider cap beam may be necessary to allow clearance for the post-tensioning jacks between the ends of the beams.

12.10 DETAILED DESIGN

The detailed final design of the curved beam bridge will generally follow the design for a similar straight bridge, as described in Section 9.4. Some special points relating to the post-tensioned curved bridge are given below.

12.10.1 Loss of Prestress

The calculation of prestress losses for post-tensioned beams is somewhat different from that for pretensioned beams. Refer to *LRFD Specifications*, Article 5.9.5.

12.10.2 Computer Models

The computer models used in the preliminary design to analyze the effect of vertical loads are adequate for use in the detailed final design. The model for the post-tensioning (Model 7) should be refined, using more realistic tendon trajectories and accurate estimates of the initial and final prestress forces. In addition, the optimum prestress levels for all six beams needs to be investigated more thoroughly. This is a trial-and-error process.

12.10.3 Crossbeam Details

The detailing of crossbeams between the beam segments is similar to that described in Chapter 11 for spliced beams. Refer to Chapter 11.

Initial stresses in the beams at the crossbeam location need to be calculated in order to determine the required initial concrete strength, f_{ci}, for the crossbeam concrete at the time the beams are post-tensioned.

The post-tensioning tendons undergo an angle change at the crossbeams. This creates an inward radial force equal to the tension in the tendon multiplied by the angle change in radians. At the exterior beam on the inside of the curve, reinforcement must be provided to tie this force back into the crossbeam. See Podolny 1985 for a further discussion of this problem.

12.10.4 Post-Tensioning Anchorages

Post-tensioned beams will generally be detailed with end blocks to contain the tendon anchors. The design of post-tensioned anchorage zones is given in *LRFD Specifications*, Article 5.10.9. An alternate method is to place anchorages in the end walls to eliminate the need for end blocks on the beams.

12.11 REFERENCES

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SEISMIC DESIGN

A = total section area
A = seismic acceleration coefficient
A_c = area of pile core measured to the outside of the transverse spiral reinforcement
A_g = gross area of pile
A_h = area of hoop reinforcement
A_i = area of a segment in a bent cap section
A_{jv} = vertical reinforcement to be placed over a distance of $h_b/2$ from the column face
A_{ps} = area of prestressing steel
ARS = acceleration response spectrum
A_s = area of reinforcing steel passing through shear plane including prestressing steel
A_{sc} = total area of longitudinal reinforcement in column section
A_{vi} = interior vertical joint stirrup area
b_b = bent cap width parallel to the longitudinal axis of the bridge
b_{je} = effective width of bent cap
C_b = compression force
C_c = column compression force
$C_{s(long)}$ = elastic seismic response coefficient in the longitudinal direction
$C_{s(tr)}$ = elastic seismic response coefficient in the transverse direction
D = column diameter
D' = core diameter of spirally confined column
D_i = force in diagonal compression strut within the superstructure/substructure joint where i = 1 through 3
E_{cc} = modulus of elasticity of deck and column concrete
E_{cs} = modulus of elasticity of concrete in the beam and bent cap
E_s = modulus of elasticity of nonprestressed reinforcement
F = bent cap prestressing force after all losses
F_i = force in each quadrant
f'_c = specified compressive strength of concrete
f^o_{yc} = over-strength stress of column reinforcement including strain hardening
f_h = average horizontal stress (due to prestress) in the horizontal direction
f_v = average joint axial stress in the vertical direction
f_y = specified yield strength of non-prestressed reinforcement
f_{yh} = yield strength of hoop or spiral reinforcement
f_{yv} = yield strength of joint vertical reinforcement
g = gravitational acceleration (32.2 ft/sec/sec)
H = average column height in frame
h_b = cap beam section depth
h_c = column length from top of footing to center of gravity of the superstructure
IC = importance classification

I_c = moment of inertia of column
I_s = moment of inertia of superstructure cross-section about vertical axis through centroid
K = equivalent transverse stiffness
K_c = column shear stiffness
L = span length
L = length of longitudinal frame between expansion joints
l_a = assumed length of column anchorage reinforcement in joint
M = total contributory mass of superstructure and column
M^o_{bot} = column over-strength moment capacity at column bottom
$M^o_{i,bentcap}$ = moment at middepth of bent cap
M_L = longitudinal moment
M_T = transverse moment
M^o_{top} = column over-strength moment capacity at column top
N = minimum abutment support length
n = modular ratio
P = axial force
$P_{DL,BOT}$ = axial force due to dead load at bottom of column
$P_{DL,TOP}$ = axial force due to dead load at top of column
P_e = axial compression load on the pile
p_t = principal tensile stress
q = uniformly distributed load
R = response modification factor
R_{col} = column seismic shear force
RSA = response spectrum analysis
S = angle of skew (degrees) measured from a line normal to the span
S = site coefficient
SPC = seismic performance category
s_{reqd} = required spacing of hoop reinforcement
T = external torsion
T_c = partial tension force in column
$T_{c'}$ = partial tension force in column
$T_{(long)}$ = period of structure in the longitudinal direction
$T_{(tr)}$ = period of structure in the transverse direction
V_c = column shear force
$V^o_{i,column}$ = horizontal shear force at top of column
V_{jh} = horizontal joint shear force
v_{jh} = nominal horizontal shear stress in the joint
V_L = longitudinal shear force
V_T = transverse shear force
$V_{T(Abutment)}$ = abutment transverse shear force

V_V = vertical shear force
W = total contributory weight of superstructure and column
x_i, y_i = distances defining location of quadrant force, F_i
$\overline{Y}_b$ = distance to centroid of superstructure cross-section from extreme bottom fiber
Z = force reduction factor
Δ = longitudinal or transverse superstructure displacement at intermediate support(s)
Δ_1 = longitudinal or transverse superstructure displacement at intermediate support(s)
Δ_2 = longitudinal or transverse superstructure displacement at intermediate support(s)
μ = coefficient of friction over the interface
ρ_s = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spiral)
$\rho_{s,min}$ = minimum required value of ρ_s
τ = shear friction stress

SEISMIC DESIGN

15.1 INTRODUCTION

15.1.1 Seismic Activity

The threat of seismic hazard is often thought to be limited to California and a few other western states. However, the discovery of new fault zones and an increased understanding of their activity have prompted many other states to include some form of seismic design requirements in their bridge design specifications. Although most states have not had significant levels of earthquake activity during recent history, the occurrence of a few notable earthquakes indicates that there may be a significant earthquake hazard in many states. For example, the most notable earthquake affecting South Carolina was the one that shook the Charleston-Summervale area in 1886 causing loss of life and considerable damage. Small earthquakes still occur in the region and seismologists indicate the potential for another damaging earthquake.

Other notable sources of earthquakes include the New Madrid Seismic Zone, the Central Virginia Seismic Zone; the Giles County, Virginia, Seismic Zone; and the Eastern Tennessee (or Southern Appalachian) Seismic Zone. Low seismic wave attenuation in the Eastern United States has the potential to cause significant shaking over broad areas, sometimes covering several states. The 1811-1812 New Madrid earthquakes, for example, caused seismic shaking of Intensity VI on the Modified Mercalli Intensity scale as far away as South Carolina. **Figure 15.1.1-1**, from the *Standard Specifications*, shows contours of current estimates of peak ground accelerations, expressed in terms of the gravitational acceleration coefficient, g. The accelerations shown have a 10% probability of being exceeded in a 50-year period.

15.1.2 Seismic Design Criteria

15.1.2.1 Background

The first United States highway bridge design standard was published in 1931 by American Association of State Highway Officials (AASHO), predecessor to the American Association of State Highway and Transportation Officials (AASHTO). Neither the first edition nor subsequent editions of the standard published prior to 1941 addressed seismic design. The editions published in the 1940s mentioned seismic loading only to the extent that bridge structures must be proportioned for earthquake stresses.

The California State Department of Transportation (Caltrans) has been at the forefront in the development of specific seismic criteria for bridges. The first general requirements for seismic design of bridges were formulated in 1940. Specific force level recommendations for earthquake design were established in 1943.

The collapse of several California freeway structures during the 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971, AASHTO and Caltrans specifications

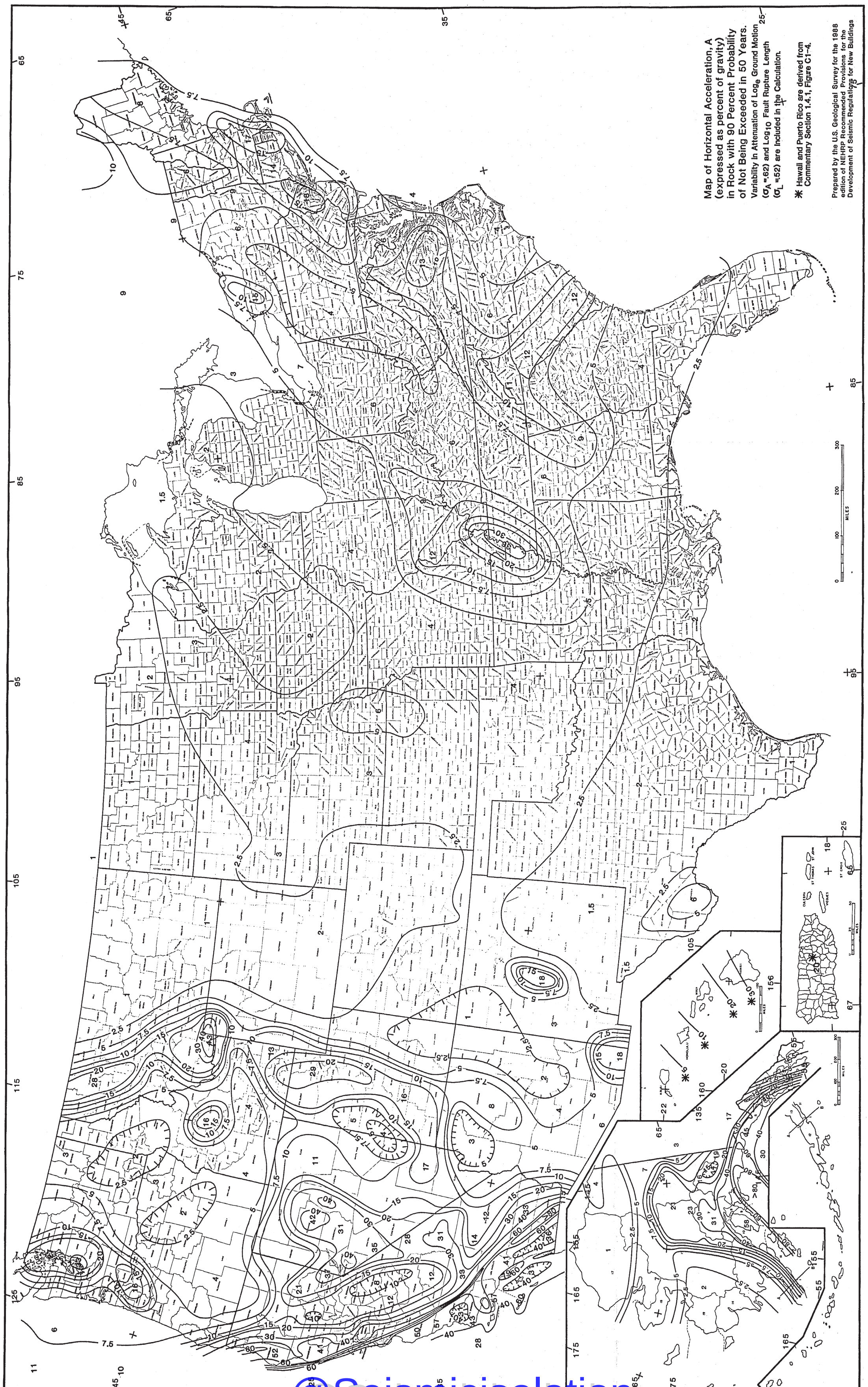


Figure 15.1.1-1
Acceleration Coefficient, A, for the United States (g's)

for seismic design of bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California. In 1973, Caltrans developed a specification based on research that considered the relationship of the site to active faults, seismic response of the soils at the site and dynamic response characteristics of the bridge. In 1975, AASHTO adopted interim specifications that were slightly modified from the 1973 Caltrans provisions.

The 1971 San Fernando earthquake stimulated research activity by the Federal Highway Administration (FHWA), which, in 1978, funded a major research project headed by the Applied Technology Council (ATC). This effort focused on the development of improved seismic design guidelines for highway bridges that would be applicable to all regions of the United States. It culminated in the publication of Report No. ATC-6 entitled *"Seismic Design Guidelines for Highway Bridges"* (ATC, 1982). These guidelines incorporated an elastic Response Spectrum Analysis (RSA), with R and Z factors to account for redundancy in the structure, ductility of the structural components and risk. These guidelines emphasized detailing for ductile behavior and prevention of collapse even after significant structural damage occurs.

15.1.2.2 Performance Objectives

Acceptable seismic performance criteria for bridge structures must satisfy both safety and economic conditions. Clearly, requiring all bridges to be serviceable immediately after an earthquake may not be economically feasible. At the same time, it is well recognized that preventing bridge collapse and possible loss of life can and must be achieved. The principles used in the development of AASHTO provisions were:

1. The design ground motion must have a low probability of being exceeded during the normal lifetime of the bridge (10% probability of being exceeded in 50 years or a 475-year return period).
2. The bridge must have a low probability of collapse due to the design ground motion.
3. Structural damage is acceptable as long as it does not result in collapse or loss of life; and, where possible, damage that does occur should be readily detectable and accessible for inspection and repair. Small and moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
4. Functionality of essential bridges must be maintained.
5. The provisions must be applicable to all regions of the United States.

15.1.2.3 Current Design Specifications

15.1.2.3.1 Standard Specifications

AASHTO adopted the ATC-6 recommendations as a guide specification in 1983, a standard specification in 1990, and incorporated them into the *Standard Specifications* in 1992. Subsequent to the earthquakes in Loma Prieta in California (1989), Costa Rica (1991) and the Philippines (1991), AASHTO requested that the Transportation Research Board review the provisions and prepare revised specifications as appropriate. Funded by the National Cooperative Highway Research Program (NCHRP), the National Center for Earthquake Engineering Research prepared the seismic design provisions that are included in the *Standard Specifications*.

15.1.2.3.2
Caltrans Specifications

Caltrans bridge engineering practice generally embraced deterministic ground motion hazards as established based on Caltrans 1996 California Seismic Hazard Map. A technical report published with the 1996 Map can be reviewed for further information on the considered seismic sources. Caltrans uses the mean event for standard practice and refers to it as the Maximum Credible Earthquake (MCE). Caltrans Seismic Design Criteria (SDC) V1.3, 2004, documents the current state of the practice. These criteria are intended to achieve a "No Collapse" condition for standard ordinary bridges using one level of Seismic Safety Evaluation.

Design spectra for three magnitudes of earthquake established in ATC 32 is used for the one level Seismic Safety Evaluation unless site-specific spectrum is recommended according to SDC V1.3. Design spectrum adjustment for long period structures and proximity to a fault is prescribed in SDC V1.3. The SDC uses a strictly displacement approach that compares displacement demands obtained from an elastic analysis to displacements capacities obtained from inelastic static analysis commonly referred to as "Push Over Analysis". The Engineer is referred to SDC V1.3 for a thorough description of the displacement approach adopted and practiced in California.

15.1.2.3.3
LRFD Specifications

The AASHTO *LRFD Specifications* incorporates many of the seismic provisions of the 1992 *Standard Specifications*, but has updated them in light of new research developments. The principal areas where provisions were updated are:

1. The introduction of separate soil profile site coefficients and seismic response coefficients (response spectra) for soft soil conditions.
2. Definition of three levels of importance, namely *critical, essential* and *other* as opposed to the two defined in previous AASHTO provisions. The importance level is used to specify the degree of damage permitted by the use of appropriate Response Modification Factors (R factors) in the seismic design procedure.

15.1.2.4
Effect of Local Geology and Soil Conditions

As more is learned about the effect of soil-structure interaction (SSI), new guidelines and procedures continue to be developed to enhance the accuracy of predictions of the bridge response to seismic loading. However, practical limitations prevent detailed incorporation of SSI effects in every project. Where a situation warrants the development of a site-specific spectra, extra effort in site investigation, laboratory testing and modeling may be required. On very long bridges, the subsurface conditions may vary to the extent that a single-response spectra is not an accurate representation of the soil conditions. In these cases, multiple-support excitations may be specified. Multiple-support excitation requires the use of time history analysis, i.e., RSA cannot be used.

On large, important structures where the presence of large piles or drilled shafts can significantly influence the soil response, free-field response spectra may not be accurate. In these exceptional situations, state-of-the-art knowledge in the area of SSI should be utilized to improve prediction accuracy.

In addition to SSI analyses, site stability issues should be addressed. These issues include soil liquefaction, soft-clay sites and slope hazards. Soil liquefaction includes the analysis for lateral spread, loss of support, dynamic settlement, as well as possible means of mitigation (site improvements). Large site amplification effects are usually considered for soft-clay sites. Earthquakes have been recognized as major causes of slope hazards.

15.2 SEISMIC RESISTANT PRECAST CONCRETE BRIDGES

15.2.1 Spliced Precast Concrete Beam Bridges

Spliced precast concrete beam techniques have received interest in recent years as evidenced by the amount of research in this area and the number of spliced-beam bridges built. The impressive performance and the increased use of these techniques signify an emerging application, which is expected to expand in coming years. Spliced beams provide an effective alternative to steel and cast-in-place concrete bridges in the 150- to 300-ft span range, a range previously unattainable by precast concrete beams. As a result of continuity, spliced-beam bridges also provide increased redundancy and improved ductility and seismic behavior. The precast, prestressed concrete industry, in cooperation with Caltrans, has sponsored the development of a competitive precast concrete beam system that can be used in areas of high seismicity.

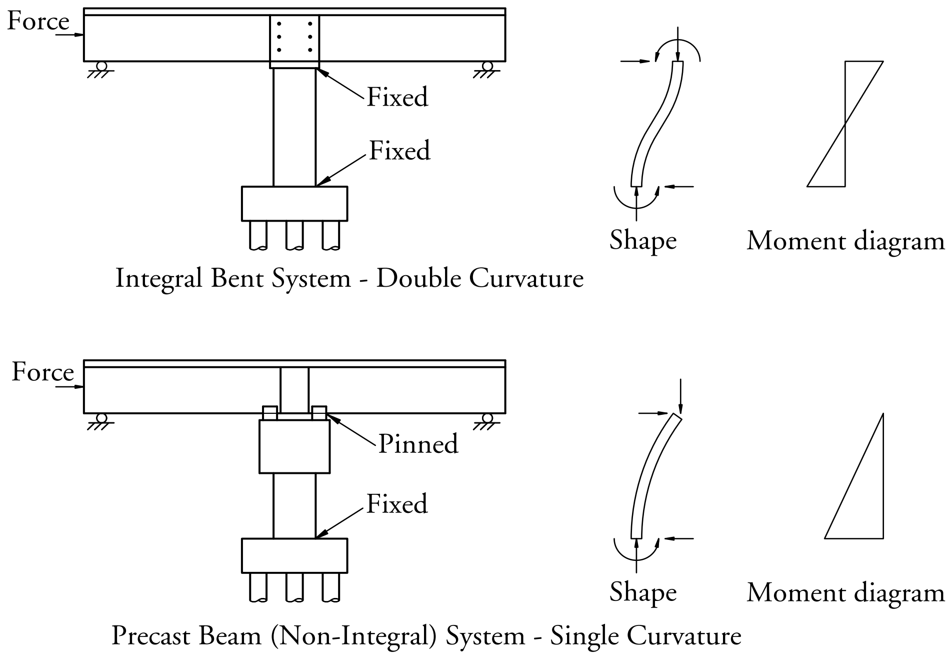
15.2.2 Current Practice

Seismic design practices and requirements vary from region to region, depending on the level of anticipated seismic activity. For example, integral superstructure-to-substructure connections may not be necessary to resist earthquake forces in areas of low to moderate seismicity. However, precast concrete bridge systems developed for some level of seismic resistance may offer certain desirable qualities which can result in better and more economical designs, even when earthquakes are not among the major design considerations.

The most common form of concrete bridge consists of a cast-in-place (CIP) concrete deck on precast, prestressed concrete beams. The beams are set on elastomeric bearing pads, which rest on the multi-column bents consisting of circular or rectangular columns with a rectangular bent cap or abutments. The columns, in turn, are supported on either isolated or combined footings.

In California, cast-in-place prestressed concrete box girders monolithically connected to the substructure are used to create longitudinal frames with multiple spans. The box girders are, in some cases, supported on single columns. Multi-column bents are usually provided on wider bridges. Unlike the precast concrete beam system of a drop-cap pier, the CIP box girder system with a monolithic connection to the substructure resists longitudinal forces in double curvature bending of the column as shown in **Figure 15.2.2-1**. This is a decided advantage in areas where large longitudinal forces are possible such as from a seismic event. However, CIP construction requires extensive falsework and formwork, which can result in lengthy periods for construction with possible traffic disruptions in roadways below the bridge.

Figure 15.2.2-1
Single- versus Double-Curvature Column



15.2.3 Seismic Response Characteristics of Precast Concrete Bridge Systems

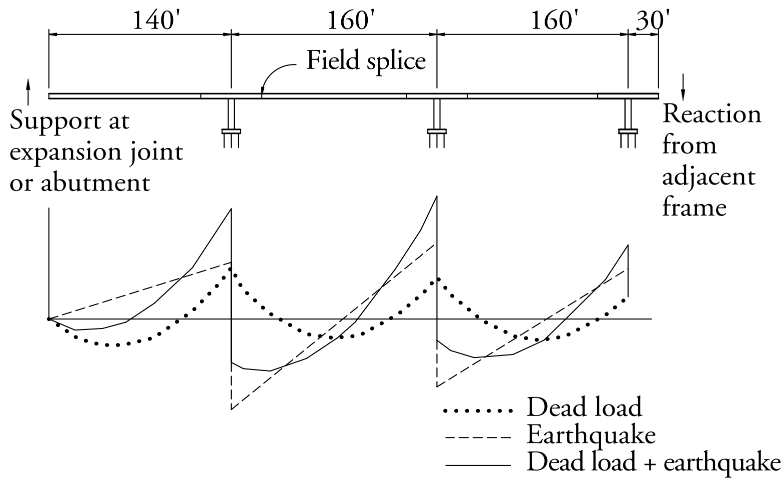
The lack of monolithic action between the superstructure and bent cap in precast, prestressed concrete beam systems causes the column tops to act as a pinned connection. Consequently, while the transverse stability of multi-column bents is ensured by frame action in that direction, stability in the longitudinal direction requires the column bases to be fixed to the foundation supports. This requirement places substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Developing a moment connection between the superstructure and substructure makes it possible to introduce a pinned connection at the column bases. This results in less expensive foundations.

Integral bent caps are also beneficial in precast, prestressed concrete beam systems with single-column bents. By introducing moment continuity at the connection between the superstructure and the cap, the columns are forced into double-curvature bending, which tends to substantially reduce their moment demands. As a result, the sizes and overall cost of the adjoining foundations are also reduced.

In a seismic event, it is essential to have plastic hinging occur in the column rather than the superstructure or footing. This is because plastic hinging is accompanied by a certain degree of damage in the form of inelastic displacements, cracked and spalled concrete and yielded reinforcement. Allowing such damage to occur in the superstructure near the ends of a span could reduce the load-carrying capacity of the superstructure, thereby increasing the likelihood of collapse. Damage to a footing or pile system is not easily detected and is extremely difficult to repair. Plastic hinging in the column can be quickly identified by inspection and sometimes repaired. More importantly, a properly confined column will continue to carry axial load and therefore structural collapse may be avoided.

The longitudinal moment in a typical beam near the pier consists of the sum of the dead load and a portion of the column seismic (plastic) moment on one side of the pier, and the difference between dead load and the remaining portion of the column seismic (plastic) moment on the other side. The result is a high, rapidly changing moment on the side where the moments are additive and a smaller, relatively constant positive moment on the

Figure 15.2.3-1
Moment Distribution along the Superstructure of a Longitudinal Frame Unit

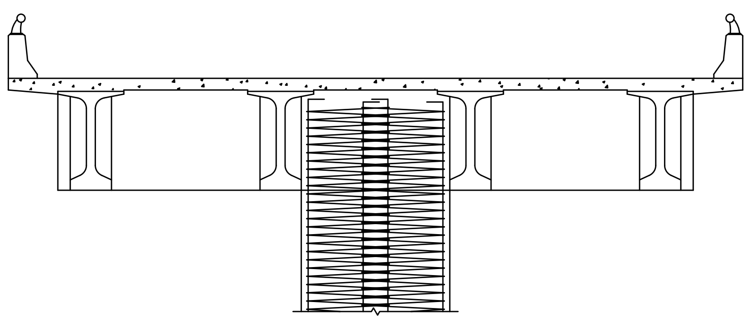


opposite side. This distribution is reversible depending on the direction of the earthquake force. Therefore, the beams must be designed to carry both a high negative moment near the pier, and a smaller positive moment for an extended length on each side of the pier (see **Figure 15.2.3-1**). The dead load moment considered should properly account for time-dependent and construction staging effects, which are not included in **Figure 15.2.3-1**.

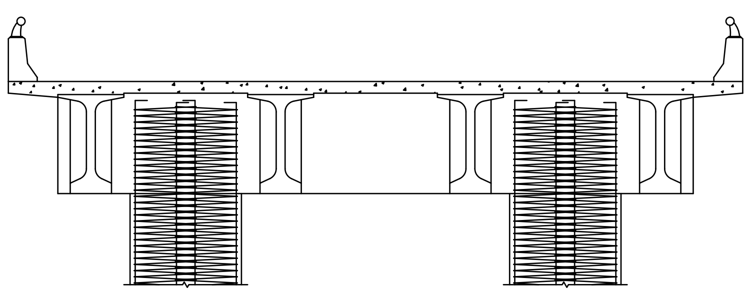
15.2.4 Integral Precast Concrete Beam System

Recently, a precast concrete girder system was developed, tested and introduced in California to address the requirements of superstructure and substructure continuity, aesthetics and minimized traffic impact during construction. Cross-sections for both single and two-column bents are shown in **Figure 15.2.4-1**. The superstructure of this bridge system consists of three basic components as described in the following sections.

Figure 15.2.4-1
Typical Bridge Cross-Sections with Single- and Two-Column Bents



Single-Column Bent

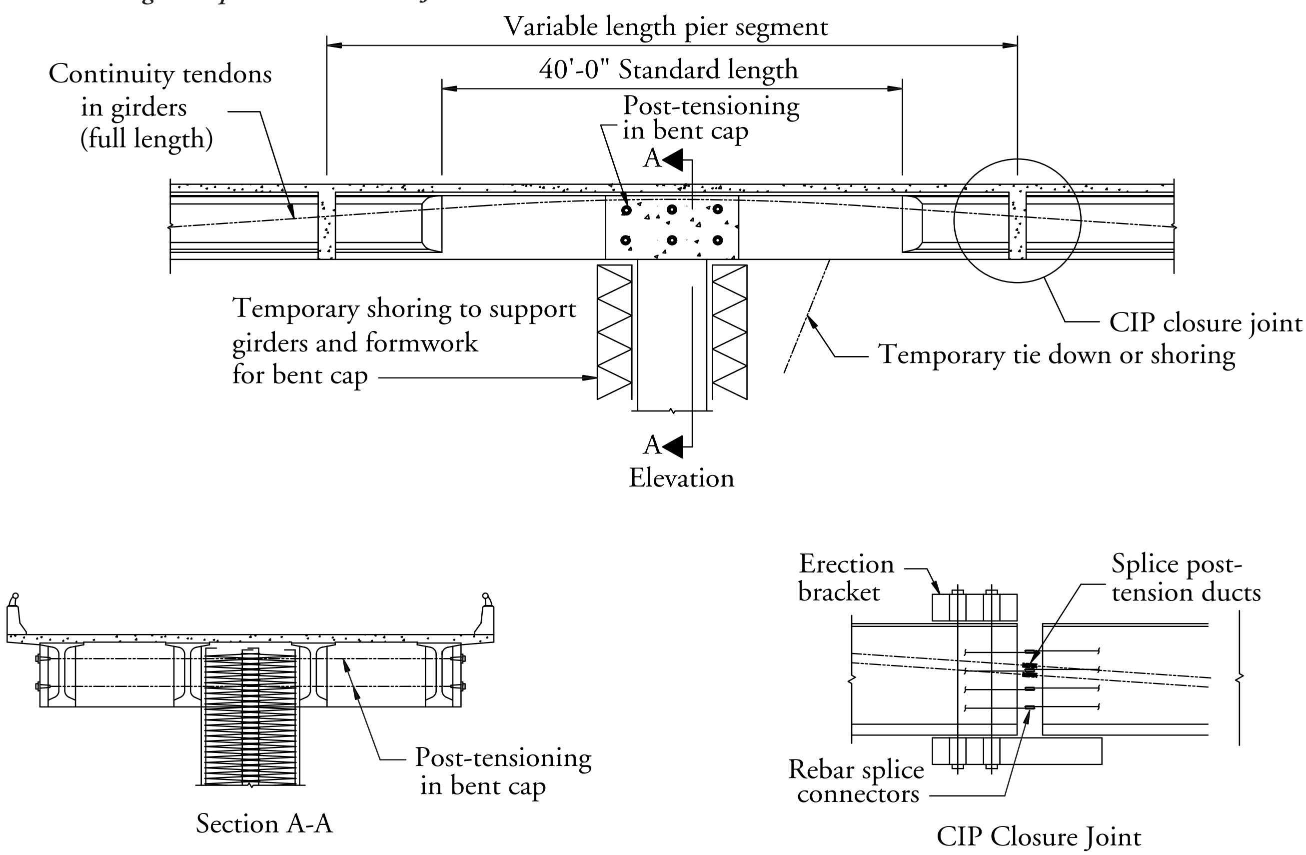


Two-Column Bent

15.2.4.1 Precast Concrete Pier Segment

The precast concrete pier segment as shown in **Figure 15.2.4.1-1**. It is a variable length section comprised of a prismatic bulb-tee beam continuous over the columns. The length is variable in order to locate the splice at the approximate point of inflection for a given span arrangement. Subject to weight limitations, the web in the central portion of the pier segment may be thickened to accommodate the large negative moments and shear forces at the pier. The section is pretensioned for shipping and handling stresses and for a portion of the service negative moment over the pier. The pier segment also contains ducts for two stages of longitudinal post-tensioning: one for the beam-only section and one for the beam-deck composite section.

Figure 15.2.4.1-1
Details at Integral Cap and CIP Closure Joint



15.2.4.2 Cast-in-Place Concrete Bent Cap

This portion of the system provides for the connection of the precast pier segment to the column as shown in **Figure 15.2.4.2-1**. The pier diaphragm is formed and poured around the precast pier segments and the entire system is connected by means of transverse post-tensioning through the complete length of the pier cap. Reinforcing steel in the top slab and in the cap improves the monolithic response of the superstructure-column interface. The principal mechanism for developing monolithic response is a combination of torsion and shear-friction through the bent cap, which then translates into longitudinal bending of the beams. The corresponding bent cap design procedure is presented in Example 15.5.

15.2.4.3 Drop-In Precast Concrete Segment

This drop-in section traverses the positive moment region of a span and utilizes a standard bulb-tee shape. It is pretensioned for lifting and handling stresses and contains ducts for the two-stage post-tensioning of the continuous beam and composite sections. See **Figure 15.2.4.3-1**. The drop-in segment is supported from the pier segments by erection brackets as shown in **Figure 15.2.4.1-1** and described in Chapter 11.

Figure 15.2.4.2-1 Longitudinal Section and Cross-Section of CIP Pier Cap

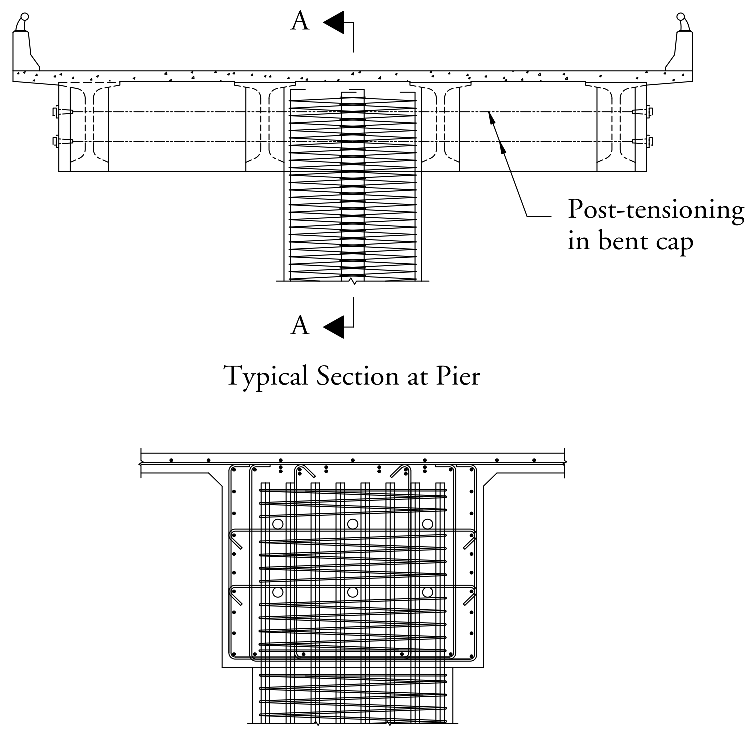
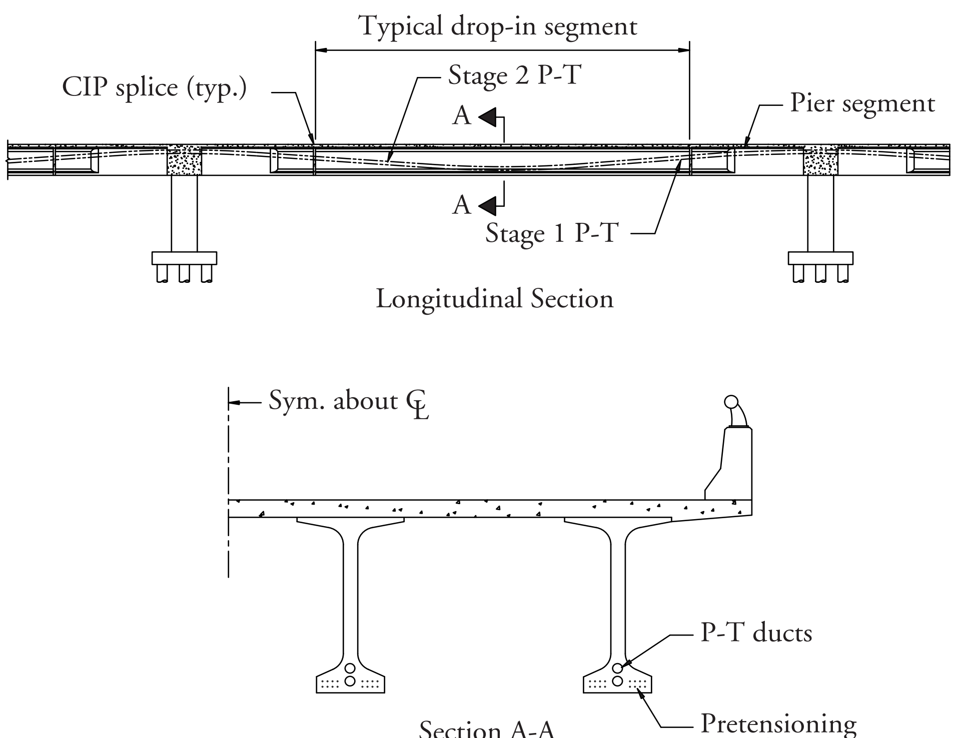


Figure 15.2.4.3-1 Longitudinal Section and Cross-Section of Drop-In Segment



One of the important features of the integral system is its minimal impact on traffic during the construction process, compared to CIP box-girder systems. This is of critical interest in regions where bridge construction occurs in urban areas with minimal vertical clearances. The proposed construction sequence using the system for a two-span bridge is illustrated in **Figure 15.2.4.3-2**. Additional details for spliced beams are found in Chapter 11.

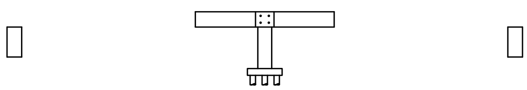
Figure 15.2.4.3-2
Construction Sequence for Bulb-Tee Bridge

Stage 1 (day 1): Pretension strands and cast girder concrete

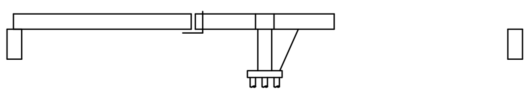
Stage 2 (day 2): Detension strands

Stage 3 (days 19 thru 28): Erect segments on temporary suppports

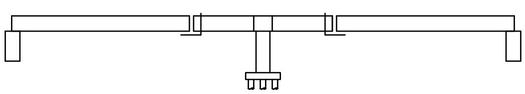
Stage 4 (days 29 thru 38): Form and cast bent cap



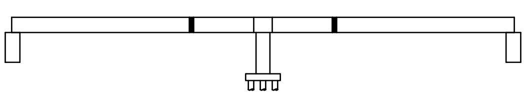
Stage 5 (days 39 thru 41): Post-tension bent cap



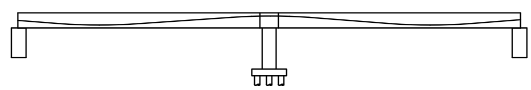
Stage 6A (days 43 thru 44): Erect left span segments and tie down



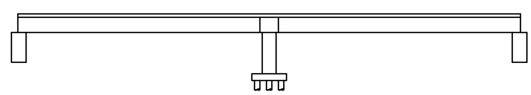
Stage 6B (days 45 thru 46): Erect right span segments



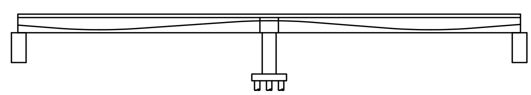
Stage 7 (days 49 thru 55): Cast closure joints



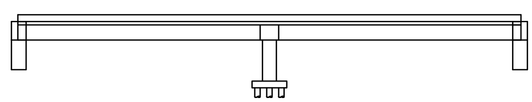
Stage 8 (days 56 thru 57): Tension first phase tendons



Stage 9 (days 58 thru 72): Construct cast-in-place deck



Stage 10 (day 73): Tension second phase tendons



Stage 11 (days 74 thru 83): Complete bridge

15.2.5 Seismic Details

15.2.5.1 Superstructure-to-Bent Cap Connection

The goal of a seismic connection at this location is to transfer the plastic moment demands at the top of the column into the superstructure without yielding either the connection itself or the beam ends. To achieve this, both the connection and the beam ends must be designed to provide a design strength exceeding the required strength from the forces transferred i.e., capacity must exceed demand. Additionally, the connection should be detailed to ensure adequate distribution of the longitudinal moment from the top of the column to the various beams.

The design procedure involves the following steps:

1. Determination of the plastic moment capacities at the top and bottom of the column.
2. Calculation of the principal stresses in the bent cap due to joint shear.
3. Design of joint reinforcement.
4. A torsion-shear friction analysis to verify the ability of the bent cap to transfer the column plastic moments to the bridge superstructure.
5. A check of the bridge superstructure capacity to ensure that the plastic hinges form in the column rather than the superstructure.

15.2.5.2 Ductility of Precast Concrete Piles

Piles in soft soils supporting bridge structures may be subjected to large horizontal displacements due to design earthquakes. These deformations produce significant curvatures in the piles. The pile-cap interface (end fixity of the pile in the pile cap) is a region of significant curvature. Another region of high curvature is within the soil. These regions of high curvature need to be designed to possess adequate ductility. Ductility is improved by confining the concrete with spiral or hoop reinforcement. In addition to confining the concrete, spiral or hoop reinforcement prevents the buckling of reinforcing bars and tendons at large deformations and ensures adequate shear resistance.

Gerwick (1982) and Sheppard (1983) reported on the results of lateral load tests on prestressed concrete piles conducted in California. They provide specific recommendations for the required transverse reinforcement in critical regions of the pile. Park and Falconer (1983), summarize the results of experimental tests conducted on precast, prestressed concrete piles at the University of Canterbury, New Zealand. The objective of these tests was to determine if the requirements for transverse spiral reinforcement in concrete columns and piers of the *Standard Code of Practice of New Zealand* (1982) would result in ductile behavior of precast, prestressed concrete piles. The spiral reinforcement in the test specimens was in accordance with the New Zealand Code requirements for potential plastic hinge regions of ductile reinforced concrete columns and piers. The tests showed that when there is adequate transverse reinforcement, piles subjected to cyclic lateral loading are capable of undergoing large post-elastic deformations without significant loss of load carrying capacity.

For pile bents in Seismic Performance Categories B, C and D, the *Standard Specifications* requires that the volumetric ratio of spiral reinforcement in potential plastic hinge regions be:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\left(\frac{f'_c}{f_{yh}}\right) \qquad \text{[STD Div. I-A, Eq. 6-4]}$$

or,

$$\rho_s = 0.12\left(\frac{f'_c}{f_{yh}}\right) \qquad \text{[STD Div. I-A, Eq. 6-5]}$$

whichever is greater

where

ρ_s = ratio of the volume of spiral reinforcement to total volume of concrete core (out-to-out of spiral)

A_g = gross area of the pile

A_c = area of pile core measured to the outside of the transverse spiral reinforcement

f'_c = specified compressive strength of concrete

f_{yh} = yield strength of hoop or spiral reinforcement

The *Standard Specifications* also requires that center-to-center spacing of the spirals not exceed the smaller of 0.25 times the pile diameter, or 4 in. for Categories C and D and 6 in. for Category B. At the top of piles in pile bents, the transverse reinforcement for confinement must be provided over a length equal to the maximum cross-sectional pile dimension or one-sixth of the clear height of the pile, whichever is the larger, but not less than 18 in. At the bottom of piles in pile bents, transverse reinforcement must be provided over a length extending from three pile diameters below the calculated point of moment fixity to one pile diameter, but not less than 18 in., above the mud line. Lapping of spiral reinforcement in the transverse confinement regions is prohibited; connections of spiral reinforcement in this critical region must be full strength lap welds.

In the *New Zealand Standard Code of Practice* satisfactory results have been obtained by multiplying the generally accepted AASHTO volumetric ratios, ρ_s, by the expression:

$$0.5 + 1.25\left(\frac{P_e}{f'_c A_g}\right)$$

where P_e = axial compression load on the pile

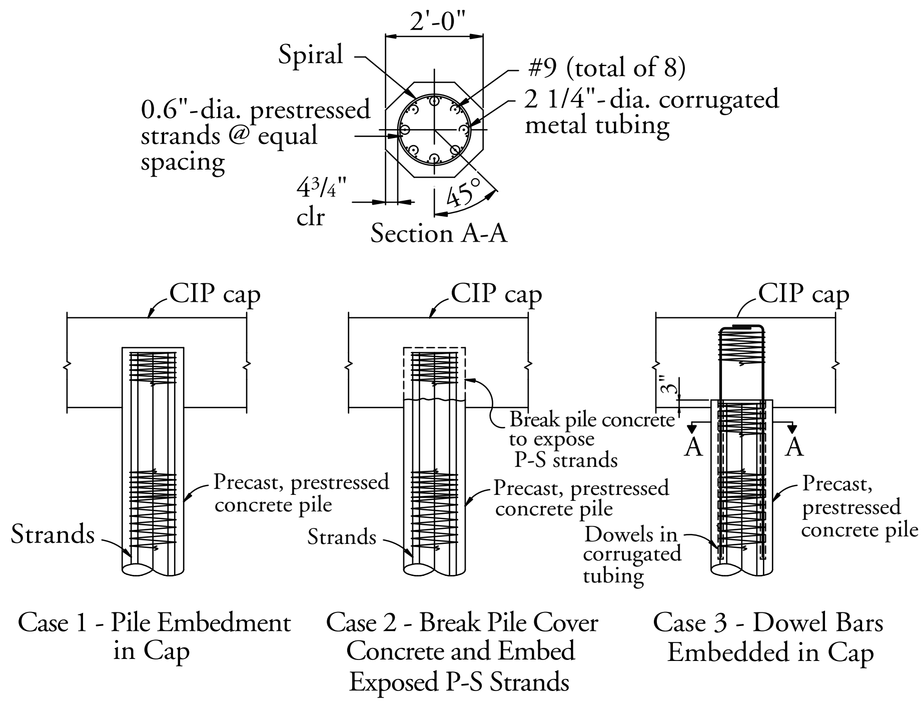
For a perspective on Caltrans state of the practice, the engineer should refer to Seismic Design Criteria SDC V1.3, 2004 and applicable references. In summary, piles with a cap placed in competent soil are not designed explicitly for lateral displacements; typical pile standard details (referred to as XS Sheets and downloaded from www.dot.ca.gov) are used. For bridges with flexible foundations (i.e. soft or marginal soil, liquefaction, scour), the piles are explicitly designed for both vertical and lateral load path.

15.2.5.3 Pile-to-Cap Connections

The strength and ductility of the connection between precast, prestressed concrete piles or pile extensions and reinforced concrete pile caps or bent caps is vital to the seismic performance of the piles. Gerwick (1993) describes three types of pile-to-cap connections that have been successfully employed. The connections are illustrated in **Figure 15.2.5.3-1**. They are described as follows:

Case 1—Pile embedment into the pile cap. The pile is designed to extend into the cap. Prior to concreting the cap, the surface of the pile is cleaned and roughened to provide shear transfer.

Figure 15.2.5.3-1
Alternative Pile-to-Pile Cap Connections



Case 2—Break away pile cover concrete and exposed strands. The concrete at the end of the pile is broken back to expose the strands, which are then embedded in the cast-in-place cap. The spirals are removed and the exposed strands are splayed to facilitate the development of the full strand strength in the cap.

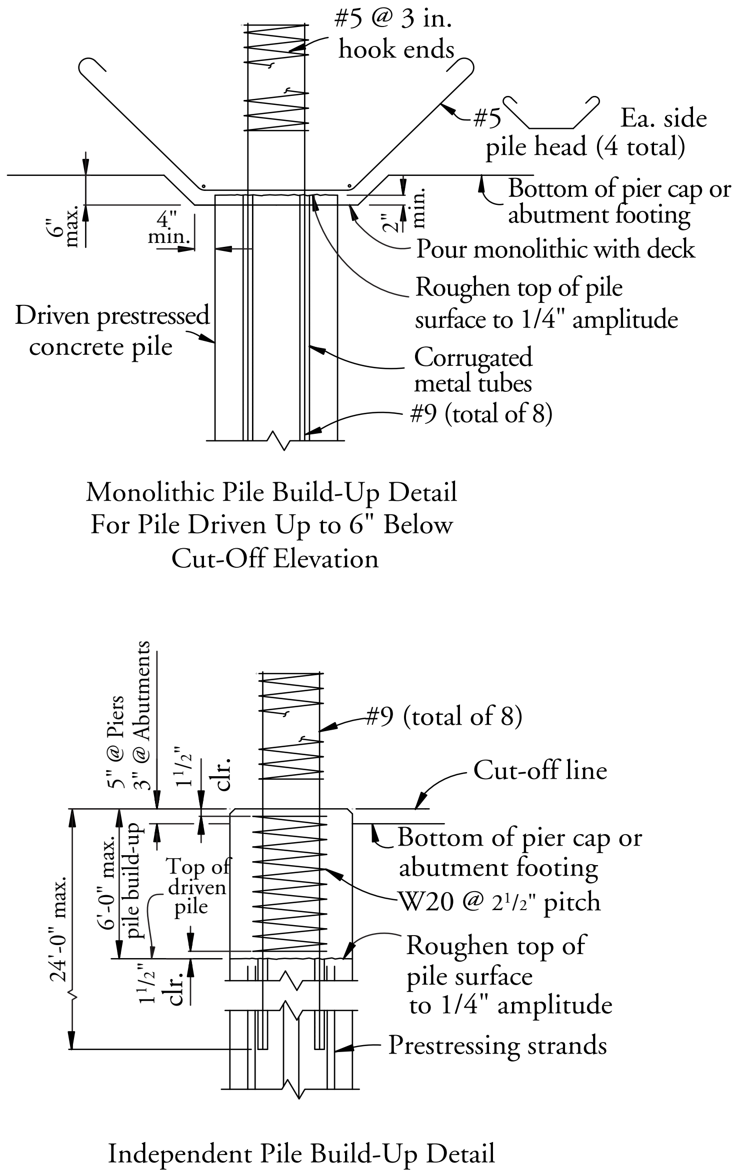
Case 3—Dowel bars embedded in the cap. Holes may be pre-formed in the pile with flexible metal ducts that are held in place during concreting by a mandrel. The holes may also be drilled in the pile after it is driven, provided the pile is not damaged during driving. The dowels should be embedded a distance sufficient to develop their full strength and the moment in the pile head. Dowel bars are typically grouted with a non-shrink grout.

Tests at the University of Canterbury, New Zealand, (Joen and Park, 1990) on prestressed concrete piles showed that well detailed prestressed concrete piles and pile-pile cap connections are capable of undergoing large post-elastic deformations without significant loss in strength when subjected to severe seismic loading. The three connection types mentioned above were investigated. All three permitted the pile to reach its full flexural strength and all three were found to have satisfactory ductile behavior.

The tests indicated that spiral steel, similar to that provided in the potential plastic hinge regions should be provided within the region of the pile that is embedded in the pile cap, especially in the broken-back pile head type connection (Case 2) described above. The spiral steel improves the bond of the strands and assists in the transfer of the lateral forces to the surrounding concrete in the cap. The tests showed that the non-prestressed reinforcement was not essential to the satisfactory ductile performance of the pile but did permit a greater dissipation of seismic energy by the pile.

Figure 15.2.5.3-2 details typical monolithic and independent pile extension or "build-up" details for applications where the pile cut-off elevation may be different from that specified on the drawings due to field conditions.

Figure 15.2.5.3-2
Typical Monolithic and Independent Pile Buildup Details



15.2.6 Isolation Methods

Seismic isolation is gaining increased acceptance in the United States both as a means of enhancing the seismic performance of existing structures and as a way of reducing the seismic force demand on substructures for new bridges. Seismic isolators decouple the superstructure from the substructure, which is the opposite strategy to the integral superstructure-substructure connection. The objective of seismic isolation of bridge superstructures is to protect the piers, abutments and their foundations by limiting the forces transferred through the beams. Besides reducing seismic loads, the isolation design helps distribute the seismic forces to the piers and abutments in relationship to their capacities.

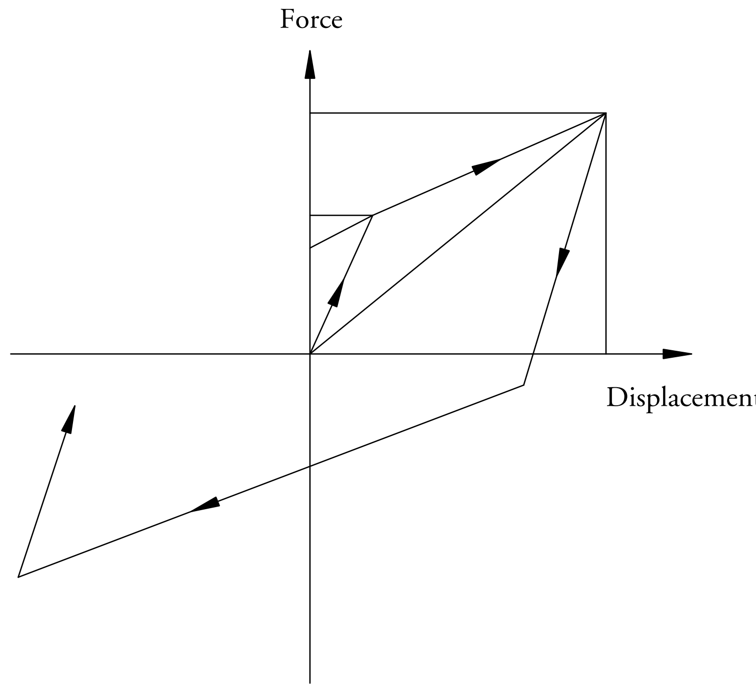
Decoupling the bridge superstructure and substructure from damaging horizontal components of earthquake ground motions reduces the seismic demand on the substructure. However, it is essential to limit the seismic displacements of isolated bridge structures to tolerable levels in order to reduce the problem of supporting traffic across excessive gaps (typically at the end abutments) and other structural problems arising from large displacements.

In addition to performing the function of regular bridge bearings, seismic isolation bearings should provide:

1. Additional flexibility in the bearings in order to lengthen the period of vibration of the structure
2. Additional damping and energy dissipation to control relative displacements across the isolator
3. Rigidity under service loads such as wind, braking and centrifugal forces

The required characteristics of the isolation bearing system result in the bilinear force-deformation characteristics such as shown in **Figure 15.2.6-1**. Several proprietary seismic isolation bearing systems are available in the United States.

Figure 15.2.6-1
Force-Displacement Characteristics of Bilinear Isolation Bearings



Two design philosophies are utilized in the AASHTO *Guide Specifications for Seismic Isolation Design* (1999). The first is to take advantage of the reduced forces and provide a more economical bridge design than conventional construction. This option uses the same modification factors as the *Standard Specifications* and hence provides the same level of safety. The second option intends to eliminate or significantly reduce damage to the substructure due to the design event. In this option, an R (ductility) factor ranging from 1.5 to 2.5 will ensure an essentially elastic response by eliminating the ductility demand on the substructure. The *Guide Specifications* also accounts for a reduction in the post-peak Acceleration Response Spectrum (ARS) curve due to the damping characteristics of the isolation bearings.

Further information on isolation methods is provided by Billings and Kirkcaldie, (1985) and Buckle and Mayes, (1990a and 1990b).

15.3 SEISMIC ANALYSIS AND DESIGN

15.3.1 Analysis Methods

There are two general approaches to evaluate the seismic response of a bridge. The first approach is the conventional force-based analysis while the second involves the use of a displacement criterion. Caltrans uses the displacement method as described in the Seismic Design Criteria V1.3, 2004.

15.3.1.1 Conventional Force Method

In this method, the bridge analysis is performed and the forces on its various components are determined. Next, the capacities of the components are evaluated. The component demand/capacity (D/C) ratios are then calculated. A particular component is said to have adequate capacity if its D/C ratio is less than a prescribed force reduction factor, R (or Z). This factor allows for limited inelastic behavior and depends on the type of component considered. The provisions contained in the *Standard Specifications*, Division I-A, are largely based on this approach. **Figure 15.3.1.1-1** is a flow chart of the basic steps of seismic design in the *Standard Specifications*. The corresponding AASHTO classifications and analysis requirements are given in **Table 15.3.1.1-1**.

Table 15.3.1.1-1 AASHTO Seismic Classifications and Analysis Requirements

<table>
<tr><td rowspan="2">Seismic Acceleration Coefficient</td><td colspan="6">Importance Classification (IC)</td></tr>
<tr><td colspan="3">IC = I
(Essential Bridges)</td><td colspan="3">IC = II
(Other Bridges)</td></tr>
<tr><td>A[a]</td><td>SPC[b]</td><td colspan="2">Minimum Analysis Requirements</td><td>SPC[b]</td><td colspan="2">Minimum Analysis Requirements</td></tr>
<tr><td>A ≤ 0.09</td><td>A</td><td colspan="2">Not Required</td><td>A</td><td colspan="2">Not Required</td></tr>
<tr><td>0.09 < A ≤ 0.19</td><td>B</td><td rowspan="3">Regular bridges with 2 through 6 spans:[c]
Use Procedure 1 or 2[d]</td><td rowspan="3">Not regular bridges with 2 or more spans:[c]
Use Procedure 3[d]</td><td>B</td><td rowspan="3">Regular bridges with 2 through 6 spans:[c]
Use Procedure 1 or 2[d]</td><td rowspan="3">Not regular bridges with 2 or more spans:[c]
Use Procedure 3[d]</td></tr>
<tr><td>0.19 < A ≤ 0.29</td><td>C</td><td>C</td></tr>
<tr><td>0.29 < A</td><td>D</td><td>C</td></tr>
</table>

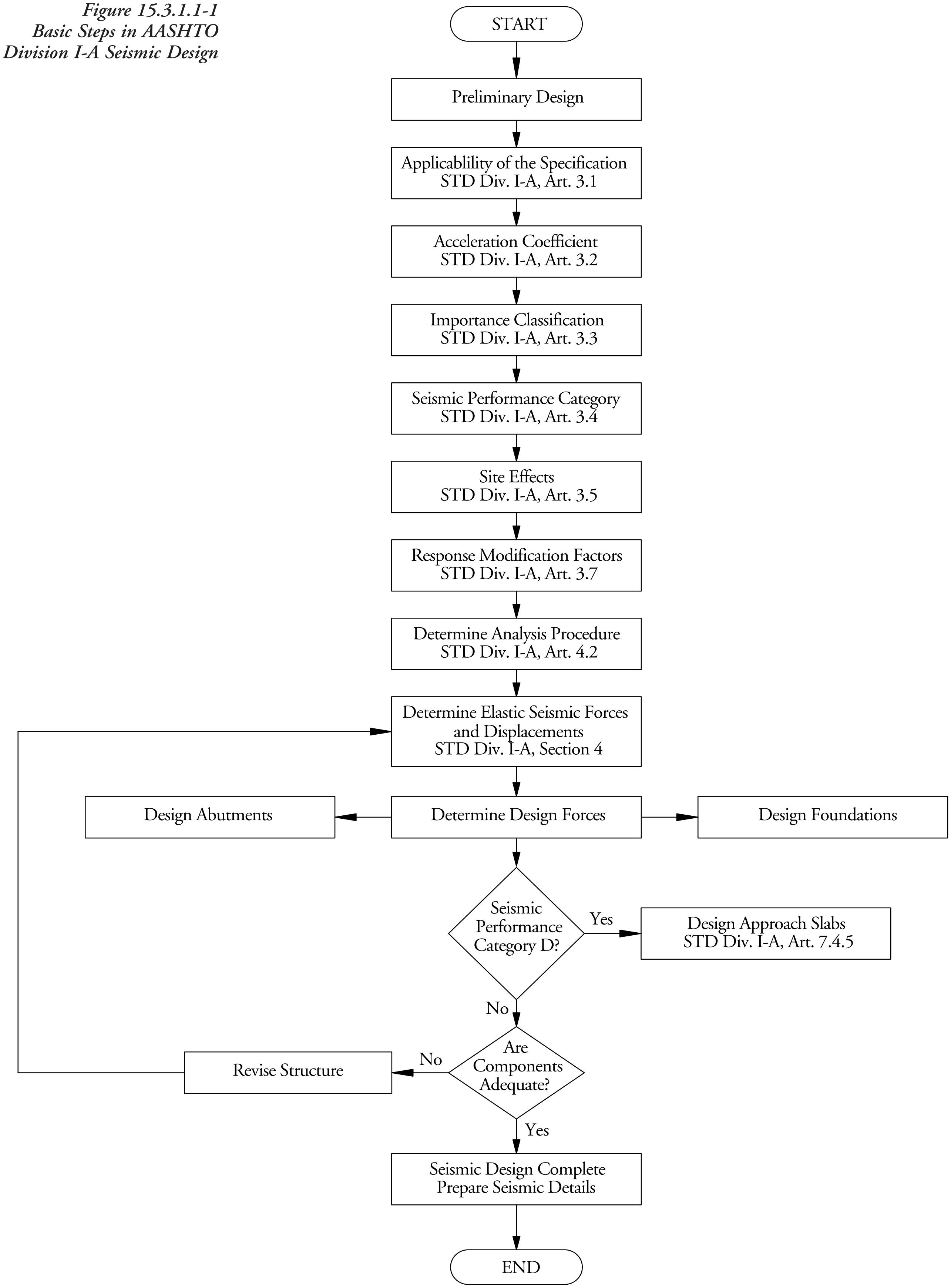
[a] Acceleration Coefficient, A, is defined in STD Div. I-A, Art 3.2

[b] Seismic Performance Category, SPC, is defined in STD Div. I-A, Art 3.4

[c] Regular bridge requirements are given in Table 4.2B of STD Div. I-A, Art 4.2

[d] Seismic Analysis Procedures are defined in STD Div. I-A, Art 4.3 and 4.4

Figure 15.3.1.1-1
Basic Steps in AASHTO Division I-A Seismic Design



15.3.1.2 Displacement Ductility Method

In the second approach, a more rational form of ductility assessment is sought by taking the effect of sequential yielding into account when evaluating capacity. Capacity thus takes on a more global meaning since it refers to the entire structure rather than to a given component, as in the force analysis. Displacement is taken as the measure of the capacity of the structure. Failure occurs when enough plastic hinges have formed to render the structure unstable or when a plastic hinge cannot sustain any further increase in rotation. Typically, displacement demand is obtained from a three-dimensional analysis using reduced flexural and torsional section properties. By relying on the reserve strength of the materials involved in constructing the bridge, this method results in considerable savings.

15.3.2 Computer Modeling

In most cases, the solutions to the equations of motion to determine demand forces and displacements are based on a linear elastic multi-mode Response Spectrum Analysis (RSA). This type of analysis is considered acceptable for basic regular structures. RSA offers the following advantages:

1. It is usually simple to use.
2. It eliminates the need for extensive testing. Representing non-linearities often requires additional data to describe the behavior of the material.
3. It provides acceptable limit-state solutions. In most cases, there are no real gains in resorting to a higher level of analysis. When discontinuities or other sources of non-linearity exist, an iterative procedure based on the equivalent linear solution may be used to satisfy force and displacement requirements. Limit states are often used in conjunction with an iterative process to envelop the behavior of the structure. Each limit state is a worst-case scenario corresponding to a set of boundary conditions or material properties. Examples of the commonly used limit states are the tension and compression models of a bridge with expansion hinges and abutment supports. The tension model corresponds to the opening of all expansion hinges and lack of abutment soil springs (stiffness), while the compression model corresponds to the closing of all gaps and the engaging of the soil at one or both abutments.
4. It uses predefined ARS curves, except when required by the size of the project and/or the geology of the site. The ARS curves take into account such factors as proximity to fault zone and site geology (primarily the depth to rock).

Typical sources of non-linearity include:

Material:

- Soil
- Concrete
- Soil-structure interaction
- Inelastic action (yielding of the reinforcement)

Geometric:

- P-Δ effects
- Gap elements
- Expansion hinges
- Abutments
- Support system such as bearings

Linear elastic solutions often provide adequate accuracy. The extra effort needed to produce additional accuracy is rarely justified in the majority of bridge applications. In fact, there are instances where the effort to obtain added accuracy may be counterproductive and create misleading results. This is particularly true in cases where an attempt is made to use non-linear time-history analysis without the proper model parameters.

15.3.3 Seismic Design Issues

As more is learned about earthquake mechanics and its effects on structures, the demand for improved seismic performance of bridges has been increasing. The general trend is toward an increase in seismic design requirements and an emphasis on the mechanics of resistance.

15.3.3.1 Causes of Failures

Based on experience learned from major earthquakes, bridge failures during an earthquake may be attributed to one or more of the following causes:

1. Unseating of the superstructure at abutments, hinges or expansion joints due to insufficient support width.
2. Inadequate or poor distribution of lap splices of vertical column steel.
3. Column failure due to longitudinal bar buckling from inadequate lateral reinforcement.
4. Column failure due to horizontal shear forces and inadequate lateral reinforcement.
5. Joint shear failure at critical superstructure-substructure connections.
6. Columns punching through the superstructure due to large vertical acceleration or inadequate connection details.
7. Footing failure due to lack of a top layer of reinforcement.

A systems approach to seismic design of bridges must be used because of the large movements usually associated with earthquakes. The ability of the bridge to withstand such movements depends not only on the primary system displacement capacity but also depends on the displacement compatibility of individual components. The movements of components must be assessed in relation to other components and to the overall bridge system. By providing the necessary displacement capacities, the potential for both local and global failures will be minimized.

15.3.3.2 Preliminary Design Recommendations

Several recommendations can be made regarding the preliminary design stages. These guidelines can help avoid problems during final design and enhance seismic performance.

1. Avoid span arrangements that induce large dead load moments in the columns, thereby reducing column capacity to resist seismic moments.
2. Use continuous frames.
3. Avoid highly irregular or suddenly changing stiffnesses of members to prevent concentration of load demands on a particular bent or frame. This will also minimize the tendency of the bridge to undergo in-plane rotation.
4. Do not allow plastic hinges to form in the superstructure.
5. Consider a depth of flexibility for piers below the actual ground level.
6. Assume the footings to be fixed except where soft soil conditions exist. In those cases, foundation flexibility should be considered when evaluating the demand.
7. Avoid skews at the abutments and hinges that are greater than 30° from the normal to the centerline of the bridge.

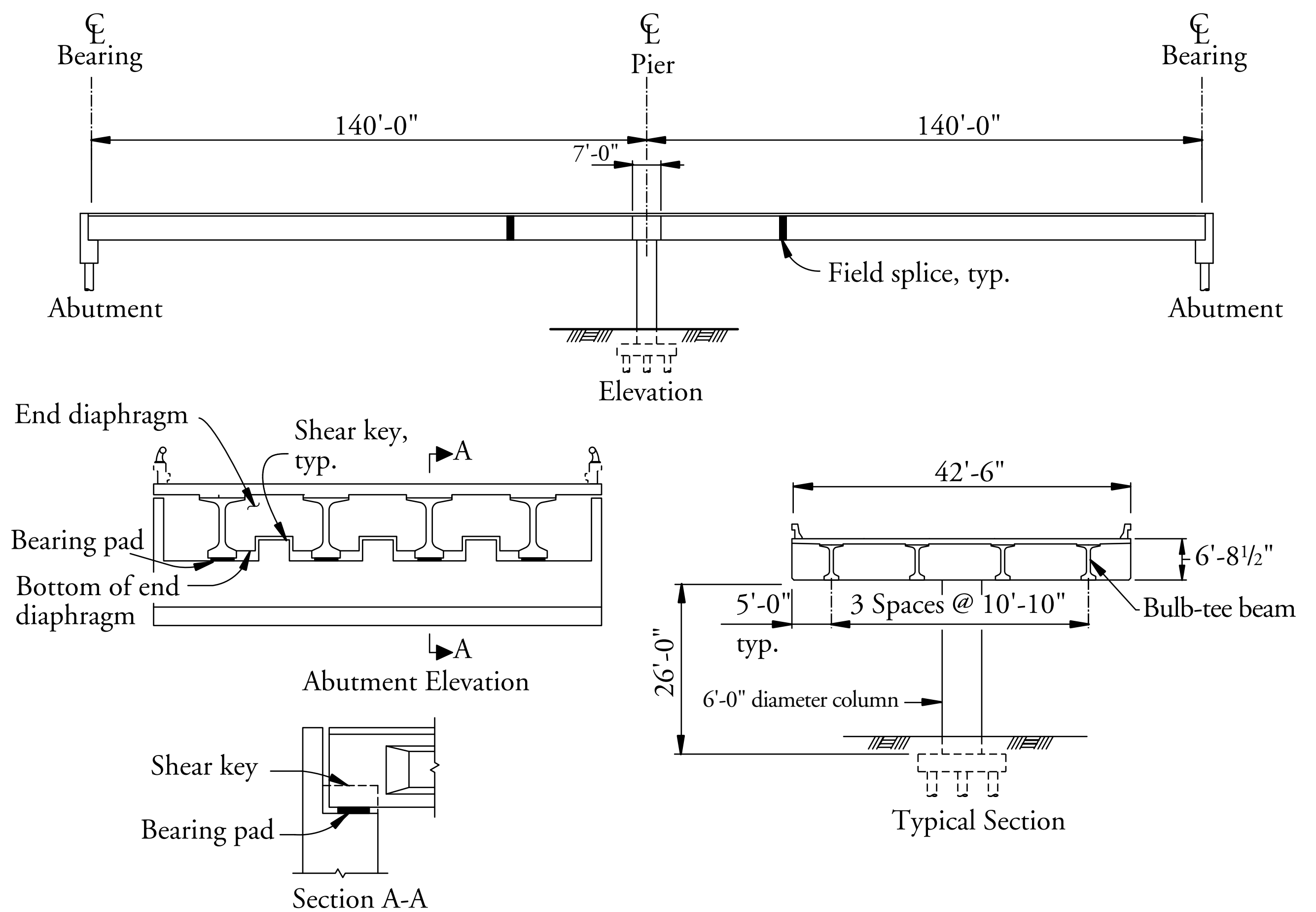
8. Make the superstructure depth at integral bent caps equal to or greater than the maximum column diameter.
9. Use isolation details at column architectural flares, or if the flares are to be relied upon structurally, use proper confinement.
10. Consider using integral abutments for shorter bridges.
11. Consider using isolation methods.

15.4 SEISMIC DESIGN EXAMPLE—BULB-TEE, TWO SPANS, DESIGNED IN ACCORDANCE WITH STANDARD SPECIFICATIONS DIVISION I-A

15.4.1 Introduction

This design example is of a bridge with two 140-ft-long spans supported by abutments at each end and a single column midway between abutments as shown in **Figure 15.4.1-1**. The superstructure consists of four precast, prestressed concrete bulb-tee beams made continuous over the column and bent cap through post-tensioning and a cast-in-place deck slab. The column is supported on a pile footing and is therefore considered fixed at its base. The superstructure is integrally connected to the column through a cast-in-place, post-tensioned bent cap.

Figure 15.4.1-1
Bridge Elevation and Typical Sections



It should be noted that superstructure-to-substructure continuity is not a requirement for seismic design. Introduction of continuity in this example provides a prototype structure for the integral bent design, presented in the following section. The seismic analysis procedure presented here is equally valid for other conventional precast bridge systems.

Because the bridge is a two-span concrete structure, the seismic loads and analysis procedures of Division I-A of the *Standard Specifications* are applicable [STD Div. I-A, Art. 3.1]. The bridge is assumed to be located in an area where the Seismic Acceleration Coefficient, A, is 0.15 [STD Div. I-A, Art. 3.2]. Since the bridge Acceleration Coefficient, A, is less than 0.29, the assignment of importance classification (IC) is not required [STD Div. I-A, Art. 3.3].

Since "A" falls between 0.09 and 0.19, the Seismic Performance Category (SPC) is B [STD Div. I-A, Art. 3.4].

The soil profile at the site is used to determine the Site Coefficient, S. In this example, soil profile Type II is assumed. This soil type corresponds to stable deposits of stiff clay and sand with a depth exceeding 200 ft. From STD Div. I-A, Table 3.5.1, the corresponding S is 1.2.

The Response Modification Factors, R values, for the various components are shown in **Table 15.4.1-1**

Table 15.4.1-1
Response Modification Factors
[STD Div. I-A, Table 3.7]

Component	R Value
For a single column	3.0
Superstructure-to-abutment connection (shear key)	0.8
Column-to-superstructure connection	1.0
Column-to-foundation connection	1.0

15.4.1.1
Bridge Geometry

For design, the bridge has the following dimensions:

Span length = 140.0 ft

Bent cap width = 7.00 ft

Bent cap depth = 6.71 ft for structural calculations

Column height = 26.00 ft

Column diameter = 6.00 ft

Beam spacing = 10.83 ft

Deck thickness = 8 in. for structural calculations

Abutment diaphragm thickness = 3.00 ft

15.4.1.2
Level of Precision

The calculations in the design example are made using a minimum of three significant figures.

15.4.2
Material Properties

Beam concrete strength = 6,000 psi

Bent cap concrete strength = 6,000 psi

Deck concrete strength = 4,000 psi

Column concrete strength = 4,000 psi

Unit weight of concrete = 144 pcf

Unit weight of beams and bent cap = 155 pcf

Unit weight of deck, columns, and abutment diaphragms = 150 pcf

Modulus of elasticity of concrete in the beam and bent cap, E_{cs}:

$$E_{cs} = 57{,}000\sqrt{f'_c}\ \text{psi} = 57\sqrt{f'_c}\ \text{ksi}$$

where f'_c = concrete strength, psi

$$E_{cs} = 57\sqrt{6{,}000}(144) = 635{,}800\ \text{ksf}$$

Modulus of elasticity of concrete in deck and column, E_{cc}:

$$E_{cc} = 57\sqrt{4{,}000}(144) = 519{,}100\ \text{ksf}$$

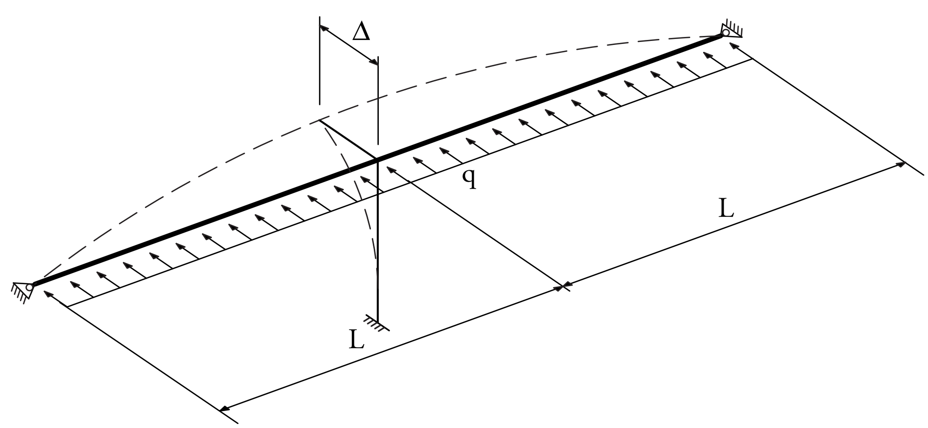
Modular ratio between the two concretes, n:

$$n = \frac{E_{cc}}{E_{cs}} = \frac{519{,}100}{635{,}800} = 0.816$$

15.4.3 Seismic Analysis in Transverse Direction

Procedure 1 (Uniform Load Method) may be used because the SPC of the bridge is B. According to this method, the seismic load is approximated as a uniform static load applied at the center of gravity of the superstructure, transverse to its axis, as shown in **Figure 15.4.3-1**. The total seismic load (uniform load times bridge length) is taken equal to the total dead weight of the superstructure plus the tributary weight of the columns multiplied by the seismic response coefficient. The superstructure is assumed to respond to the uniform seismic load as a continuous beam supported on a flexible column.

Figure 15.4.3-1 Assumed Transverse Response According to the Uniform Load Method



15.4.3.1 Section Properties

15.4.3.1.1 Beam Properties

Area of cross-section of precast beam = 7.39 ft^2

Overall depth of precast beam = 6.0 ft

Moment of inertia about the centroid of the major axis of the non-composite precast beam = 36.44 ft^3

Moment of inertia about the centroid of the minor axis of the non-composite precast beam = 3.33 ft^2

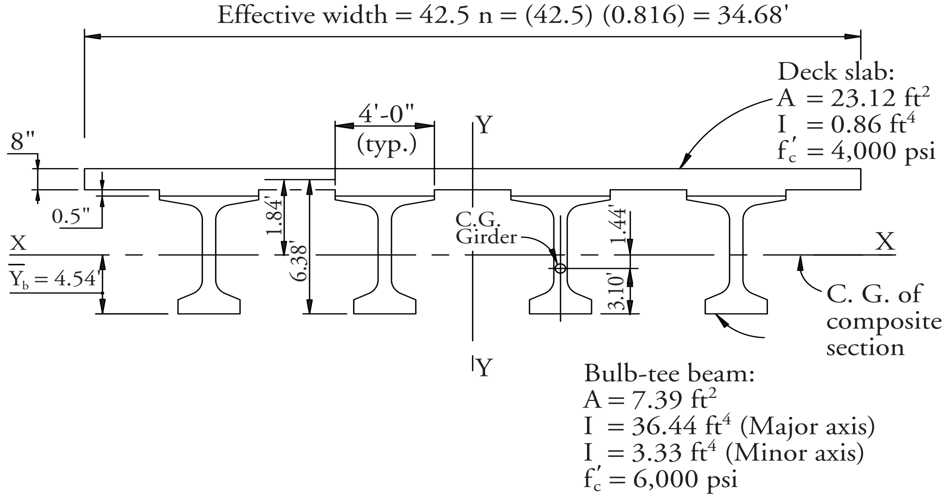
Distance from centroid to the extreme bottom fiber of the non-composite precast beam = 3.10 ft

Distance from centroid to the extreme top fiber of the non-composite precast beam = 2.90 ft

15.4.3.1.2
Composite Section Properties

As a first step, the moment of inertia, I_s, of the bridge cross-section about the vertical axis through the centroid is calculated. The cast-in-place haunch above the beam contributes very little to the moment of inertia of the section and may be ignored. However, the deck eccentricity including the haunch thickness is used to determine the location of the centroid.

Figure 15.4.3.1.2-1
Bridge Cross-Section Geometric Properties



Referring to **Figure 15.4.3.1.2-1**:

Transformed flange width = n(flange width) = 0.816 (42.5) = 34.68 ft

Transformed flange area = (transformed flange width)(deck thickness)

$$= \frac{(34.68)(8)}{12} = 23.12 \text{ ft}^2$$

$$I_s = \frac{(0.667)(34.68)^3}{12} + 4(3.33) + 2(7.39)[(5.42)2 + (16.25)2] = 6{,}669 \text{ ft}^4$$

The location of the centroid of the superstructure from the extreme bottom fiber is

$$\overline{Y}_b = \frac{(4)(7.39)(3.10) + (23.12)(6.38)}{4(7.39) + (23.12)} = 4.54 \text{ ft}$$

15.4.3.1.3
Column Properties

The moment of inertia of the uncracked circular column is:

$$I_c = \frac{\pi D^4}{64} = \frac{\pi(6)^4}{64} = 63.62 \text{ ft}^4$$

15.4.3.2
Tributary Dead Load

The total dead load to be included in the seismic analysis is equal to the sum of the weights of the deck slab, four beams with haunches, bent cap, two barriers, future wearing surface, end diaphragms, and one-half of the column weight. Refer to **Figures 15.4.1-1** and **15.4.3.1.2-1** for component dimensions and section properties.

Deck slab, haunch and beams:

[(42.5)(0.667) + (4)(4)(0.5/12)]0.150 + (4)(7.39)(0.155) = 8.93 kip/ft

Bent cap: (7.0)(6.71)(42.5)(0.155) = 309 kips

Barriers (2 barriers at 0.4 kip/ft): 2(0.4) = 0.8 kip/ft

Future wearing surface at 35 psf: (0.035)(42.5) = 1.49 kip/ft

Abutment diaphragm: (3)(6.71)(42.5)(0.150) = 128 kips per diaphragm

Column: $\pi\left(\frac{6}{2}\right)^2(0.150) = 4.24$ kip/ft

Total dead weight of superstructure:

(8.93)[(2)(140) – 7.00 – (2)(3.00)] + 309 + (2)(140)(0.80 + 1.49) + (2)(128) = 3,591 kips

Tributary dead load of column: (4.24)(26/2) = 55.1 kips

Total dead load: 3,591 + 55.1 = 3,646 kips – Use 3,700 kips.

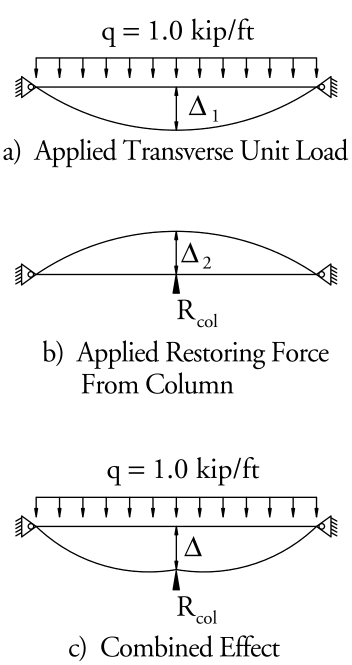
15.4.3.3 Equivalent Transverse Stiffness

In the transverse response mode, the superstructure is assumed to provide negligible restraint against column-top rotations. Hence, the column is assumed to undergo single-curvature bending (i.e., it acts like a cantilever). The column length, h_c, used for calculating shear stiffness is measured from the top of footing to the center of gravity of the superstructure.

In this example, $h_c = 26 + \overline{Y}_b = 26 + 4.54 = 30.54$ ft.

By referring to **Figure 15.4.3.3-1**, the following equations may be written:

Figure 15.4.3.3-1
Column Reaction Due to a Uniform Transverse Load



$$\Delta_1 = \frac{5q(2L)^4}{384E_{cs}I_s} \quad \text{(Eq. 15.4.3.3-1)}$$

where

Δ_1 = lateral displacement from a uniformly distributed load of q

q = uniformly distributed load

L = length of one span

$$\Delta_2 = \frac{R_{col}(2L)^3}{48E_{cs}I_s} \quad \text{(Eq. 15.4.3.3-2)}$$

where

Δ_2 = lateral displacement caused by a column force of R_{col}

R_{col} = column force

$$\Delta = \Delta_1 - \Delta_2 \quad \text{(Eq. 15.4.3.3-3)}$$

where Δ = net displacement

$$R_{col} = K_c\Delta = \frac{3E_{cc}I_c}{h_c^3}\Delta \quad \text{(Eq. 15.4.3.3-4)}$$

where K_c = column shear stiffness

Substituting Eq. (15.4.3.3-1 and Eq. (15.4.3.3-2) in Eq. (15.4.3.3-3):

$$\Delta = \frac{5q(16)L^4}{384E_{cs}I_s} - \frac{R_{col}(8)L^3}{48E_{cs}I_s} \quad \text{(Eq. 15.4.3.3-5)}$$

Substitute q = 1.0 k/ft and R_{col} from Eq. (15.4.3.3-4):

$$\Delta = \frac{(5)(1.0)(16)L^4}{384E_{cs}I_s} - \frac{24E_{cc}I_c(\Delta)L^3}{48h_c^3E_{cs}I_s} = \frac{80L^4}{384E_{cs}I_s} - \frac{24E_{cc}I_cL^3}{48h_c^3E_{cs}I_s}\Delta \qquad \text{(Eq. 15.4.3.3-6)}$$

$E_{cs}I_s$ (superstructure) = (635,800)(6,669) = $4.240\text{x}10^9$ kip-ft^2

$E_{cc}I_c$ (column) = (519,100)(63.62) = $3.303\text{x}10^7$ kip-ft^2

$$\Delta = \frac{(80)(140)^4}{(384)(4.240\,\text{x}10^9)} - \frac{(24)(3.303\,\text{x}10^7)(140)^3}{48(30.54)^3(4.240\,\text{x}10^9)}\Delta = 0.0189 - 0.3752\Delta$$

$$\Delta = \frac{0.0189}{1.3752} = 0.0137 \text{ ft}$$

Equivalent transverse stiffness, $K = \dfrac{(1.0)(2)(140)}{0.0137} = 20,440 \text{ kip/ft} \approx 20,500 \text{ kip/ft}$

15.4.3.4 Period of Structure in the Transverse Direction

$$T_{(tr)} = 2\pi\sqrt{\frac{M}{K}} = 2\pi\sqrt{\frac{W}{gK}} = 2\pi\sqrt{\frac{3,700}{32.2(20,500)}} = 0.470 \text{ seconds}$$

where

$T_{(tr)}$ = period of structure in the transverse direction
M = total contributory mass of superstructure and column
W = total contributory weight of superstructure and column
g = gravitational acceleration

15.4.3.5 Elastic Seismic Response Coefficient

$$C_{s(tr)} = \frac{1.2AS}{T_{(tr)}^{2/3}} \le 2.5A \qquad \text{[STD Div. I-A, Eq. 3-1]}$$

where $C_{s(tr)}$ = elastic seismic response coefficient in the transverse direction

Substituting:

$$C_{s(tr)} = \frac{(1.2)(0.15)(1.2)}{(0.470)^{2/3}} = 0.357 \le 2.5A = (2.5)(0.15) = 0.375$$

Therefore, $C_{s(tr)} = 0.357$

15.4.3.6 Column Forces in the Transverse Direction

Equivalent uniform static load = $C_{s(tr)}\left(\dfrac{W}{2L}\right) = 0.357\left(\dfrac{3,700}{2(140)}\right) = 4.72$ kip/ft

Δ = (4.72)(0.0137) = 0.0647 ft

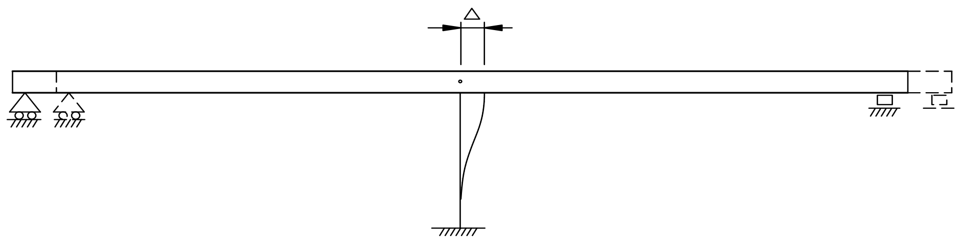
$$R_{col} = \frac{3E_{cc}I_c\Delta}{h_c^3} = \frac{(3)(3.303\,\text{x}\,10^7)(0.0647)}{(30.54)^3} = 225 \text{ kips}$$

Column seismic moment at bottom of column: (225)(30.54) = 6,872 ft-kips

15.4.4 Seismic Analysis in Longitudinal Direction

The Uniform Load Method may also be used to calculate the longitudinal seismic forces on the structure. The superstructure is assumed to displace as a rigid unit as the supporting column undergoes bending deformations as shown in **Figure 15.4.4-1**. Thus, the longitudinal stiffness is assumed equal to the shear stiffness of the column. The total dead load that contributes to the seismic load in the longitudinal direction, W, is equal to 3,700 kips (the same as the dead load used in the transverse direction).

Figure 15.4.4-1 Assumed Seismic Response in the Longitudinal Direction



Elevation

15.4.4.1 Equivalent Longitudinal Stiffness

The assumption of a rigid superstructure implies that the top of the column is restrained against rotation. Therefore, the column undergoes double-curvature bending, as opposed to single-curvature bending, which occurs in the transverse direction. The column length used for calculating shear stiffness, H, is measured from the top of footing to the bottom of the bent cap. In this example, H = 26 ft.

$$\text{Column shear stiffness} = \frac{12E_{cc}I_c}{H^3} = \frac{12(3.303\,x10^7)}{(26)^3} = 22{,}550 \quad \text{kip/ft}$$

In general, the abutments and soil behind them may contribute to the longitudinal stiffness. Their contribution depends on the abutment type (i.e., integral vs. seat abutment) and the longitudinal displacement of the structure. Several iterations may be needed to evaluate the abutment effect on the stiffness. Additionally, a minimum displacement in the range of 2 to 4 in. is typically required to mobilize the soil stiffness. In this example, the total longitudinal displacement is small (0.75 in.), and thus the abutment contribution to stiffness is ignored.

15.4.4.2 Period of Structure in the Longitudinal Direction

$$T_{(long)} = 2\pi\sqrt{\frac{M}{K}} = 2\pi\sqrt{\frac{W}{gK}} = 2\pi\sqrt{\frac{3{,}700}{32.2(22{,}550)}} = 0.449 \text{ seconds}$$

where $T_{(long)}$ = period of structure in the longitudinal direction

15.4.4.3 Elastic Seismic Response Coefficient

$$C_{s(long)} = \frac{1.2AS}{T_{(long)}^{2/3}} \leq 2.5A \qquad \text{[STD Div. I-A, Eq. 3.1]}$$

where $C_{s(long)}$ = elastic seismic response coefficient in the longitudinal direction

$$C_{s(long)} = \frac{(1.2)(0.15)(1.2)}{(0.449)^{2/3}} = 0.368 < 2.5A = 0.375$$

Therefore, $C_{s(long)} = 0.368$

15.4.4.4 Column Forces in the Longitudinal Direction

Column shear = (0.368)(3,700) = 1,362 kips

Column moment = (1,362)(26/2) = 17,706 ft-kips

15.4.5 Combination of Orthogonal Forces

A summary of the moments and shear forces are shown in **Table 15.4.5-1**.

Table 15.4.5-1 Summary of Column Forces

Earthquake Direction	Transverse		Longitudinal	
	Moment	Shear	Moment	Shear
	ft-kips	kips	ft-kips	kips
Transverse	6,872	225	0	0
Longitudinal	0	0	17,706	1,362

Seismic combination 1:

100% longitudinal force + 30% transverse force [STD Div. I-A, Art 3.9]

Transverse moment, M_T:

$M_T = (1.0)(0) + (0.30)(6,872) = 2,062$ ft-kips

Longitudinal moment, M_L:

$M_L = (1.0)(17,706) + (0.30)(0) = 17,706$ ft-kips

Transverse shear force, V_T:

$V_T = (1.0)(0) + (0.30)\ (225) = 68$ kips

Longitudinal shear force, V_L:

$V_L = (1.0)(1,362) + (0.30)(0) = 1,362$ kips

Seismic combination 2:

100% transverse force + 30% longitudinal force [STD Div. I-A, Art. 3.9]

$M_T = (1.0)(6,872) + (0.30)(0) = 6,872$ ft-kips

$M_L = (1.0)(0) + (0.30)(17,706) = 5,312$ ft-kips

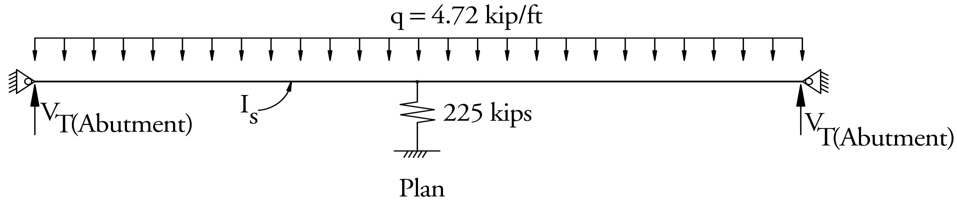
$V_T = (1.0)(225) + (0.30)(0) = 225$ kips

$V_L = (1.0)(0) + (0.30)(1,362) = 409$ kips

15.4.6 Abutment Design Forces

Reinforced concrete shear keys, such as those shown in **Figure 15.4.1-1**, will resist seismic transverse forces at the abutments. From statics, the total abutment reactions are equal to the equivalent uniform static load minus the column reaction (See **Figure 15.4.6-1**):

Figure 15.4.6-1
Transverse Shear at the Abutments



Abutment transverse shear force, $V_{T(Abutment)}$:

$$V_{T(Abutment)} = \frac{(4.72)(2)(140) - 225}{2} = 548 \text{ kips}$$

The elastic seismic force per shear key = 548/3 = 183 kips

Design shear per key = 183/R = 183/0.8 = 229 kips

The shear key design is typically based on the shear friction method. [STD Art. 8.15.5.4]

15.4.7 Minimum Abutment Seat Width

[STD Div. I-A, Art. 6.3.1]

The minimum support length N (in.), shown in **Figure 15.4.7-1**, is calculated by the following equation:

$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2)$ [STD Div. I-A, Eq. 6-3A]

where

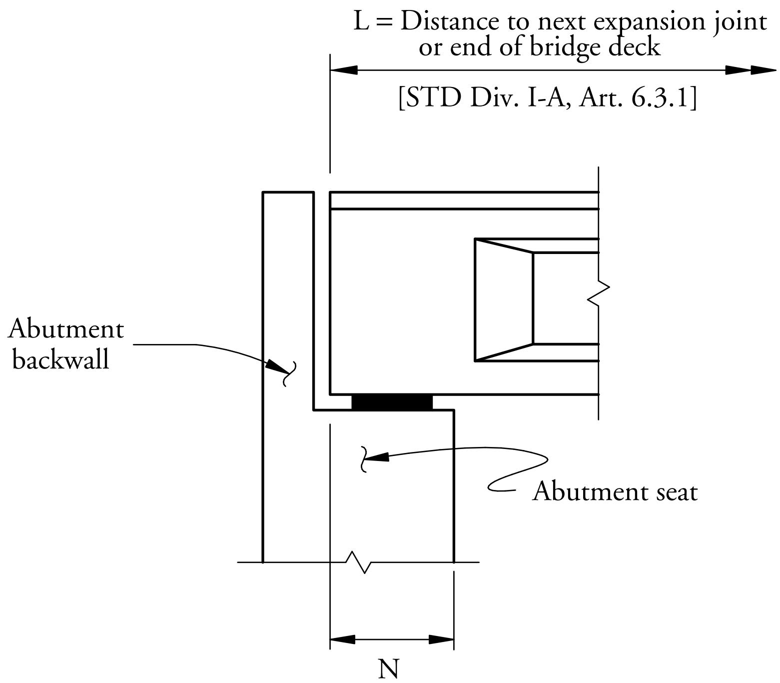
L = length (ft) of the longitudinal frame between expansion joints (2 x 140 = 280 ft)

S = skew angle (degrees) measured from a line normal to the span (0 degrees)

$N = [8 + (0.02)(280) + (0.08)(26)][1 + (0.000125)(0)^2] = 15.7$ in. Use 16 in.

The seat width provided should be the larger of N and the elastic seismic displacement in the longitudinal direction = column shear/longitudinal stiffness = (1,362/22,550)(12) = 0.72 in. While in this example, N clearly controls, additional factors such as the bearing size may control the final seat width.

Figure 15.4.7-1
Minimum Abutment Seat Width



15.5 SEISMIC DESIGN EXAMPLE—INTEGRAL BENT CAP

15.5.1 Introduction

This design example illustrates the procedure for integral bent cap design in spliced I-beam bridges. The design procedure evolved from successful experimental testing of a scale model of the Florida-type bulb-tee beam at the University of California at San Diego. The results of this testing, which verified the longitudinal seismic response of precast spliced-beam bridges, are reported in Holombo, et al (2000).

The integral bent cap is designed to provide force transfer from the spliced I-beam bridge superstructure to the foundation through the development of column plastic moments in a ductile manner.

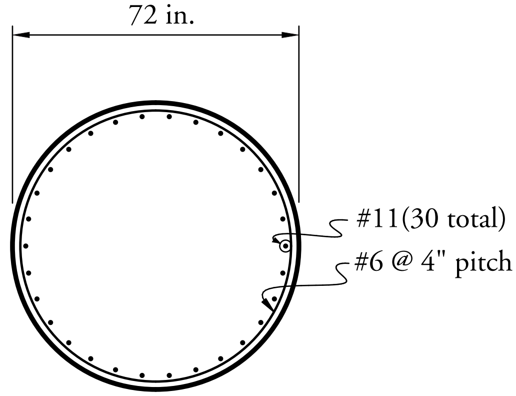
15.5.1.1 Bent Cap Geometry

Bent cap width = 84 in.

Bent cap depth, h_b = 87 in.

Column cross-section: Circular with a 6.00-ft diameter, see **Figure 15.5.1.1-1**

Figure 15.5.1.1-1
Column Cross-Section



15.5.1.2 Reinforcement

Column reinforcement: Longitudinal: 30 #11 bars
Transverse: #6 spirals @ 4-in. pitch

Bent cap post-tensioning: Six 19x0.6-in.-dia strand tendons

15.5.1.3 Material Properties

Cast-in-place concrete strength, f'_c = 4 ksi

Reinforcing steel yield strength, f_y = 60 ksi

Modulus of elasticity of tendons, E_s = 29,000 ksi

15.5.1.4 Forces

Axial load: Top of column, $P_{DL,TOP}$ = 2,225 kips
Bottom of column, $P_{DL,BOT.}$ = 2,350 kips

Column over-strength moment capacity at the top, M^o_{top} = 14,115 ft-kips
Column over-strength moment capacity at the bottom, M^o_{bot}= 14,340 ft-kips

The over-strength moment capacities are taken as 20% greater than the plastic moment capacities.

15.5.1.5 Precision

The calculations in the design example are made using a minimum of three significant figures.

15.5.2 Design Procedure

The design procedure involves the following steps:

1. Calculate principal stresses in the bent cap.
2. Design joint shear reinforcement and ensure minimum embedment length for column bars with the reinforcement being extended as close as possible to the top reinforcement in the bent cap.
3. Perform a torsional shear-friction analysis to verify the ability of the bent cap to transfer column plastic moments to the bridge superstructure.

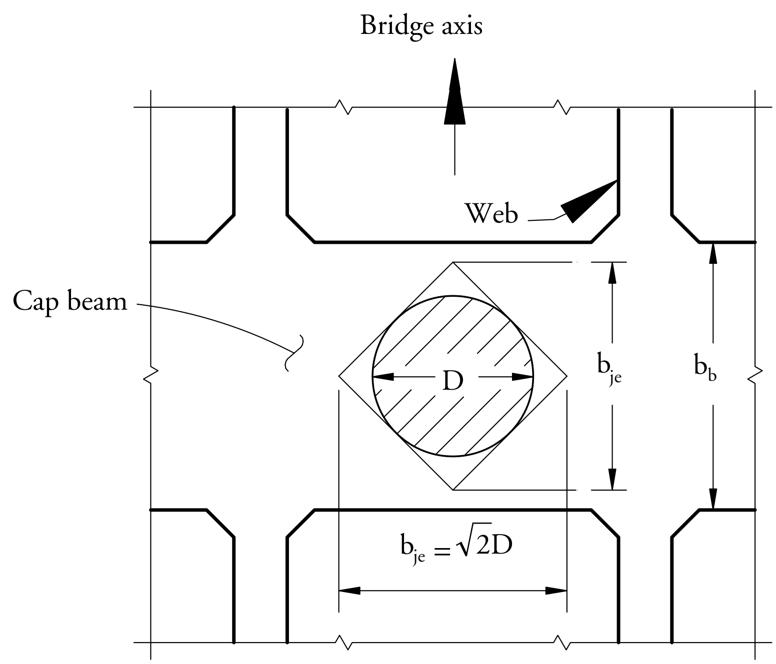
15.5.3 Principal Stresses in the Bent Cap

Horizontal joint shear force, V_{jh}, is given by:

$$V_{jh} = \frac{M^o_{top}}{h_b} = \frac{14{,}115 \text{ x } 12}{87} = 1{,}947 \text{ kips}$$

Effective width of bent cap, b_{je}, by the geometry shown in **Figure 15.5.3-1** is

Figure 15.5.3-1 Effective Joint Width for Joint Shear Stress Calculations



$b_{je} = \sqrt{2}\,D$ (for circular sections) $\leq b_b$

where

D = column diameter

b_b = bent cap width parallel to the longitudinal axis of the bridge

b_{je} = 1.414(72) = 101.8 in. > b_b = 84 in. Therefore, use b_b = 84 in.

Nominal horizontal shear stress in the joint, v_{jh}:

$$v_{jh} = \frac{V_{jh}}{b_b D} = \frac{1{,}947}{(84)(72)} = 0.322 \text{ ksi}$$

Calculate average joint axial stress, f_v, in the vertical direction at middepth of the bent cap:

$$f_v = \frac{P_{DL,TOP}}{b_b\left(D + h_b\right)} = \frac{2{,}225}{(84)(72+87)} = 0.167 \text{ ksi}$$

The average joint axial stress in the horizontal direction, $f_h = 0$ (I-beam superstructure, no significant axial stress transferred to bent cap at middepth of bent cap)

The principal tensile stress, p_t, in the bent cap/column connection is given by:

$$p_t = \frac{f_y + f_h}{2} - \sqrt{\left(\frac{f_v - f_h}{2}\right)^2 + v_{jh}^2}$$

$$= \frac{(0.167) + (0)}{2} - \sqrt{\left(\frac{0.167 - 0}{2}\right)^2 + (0.322)^2} = -0.249 \text{ ksi}$$

$$0.249 \text{ ksi} = 249 \text{ psi} = 3.94\sqrt{f_c'} = 3.94\sqrt{4{,}000} > 3.5\sqrt{f_c'} \text{ psi}$$

According to Priestley, et al (1996):

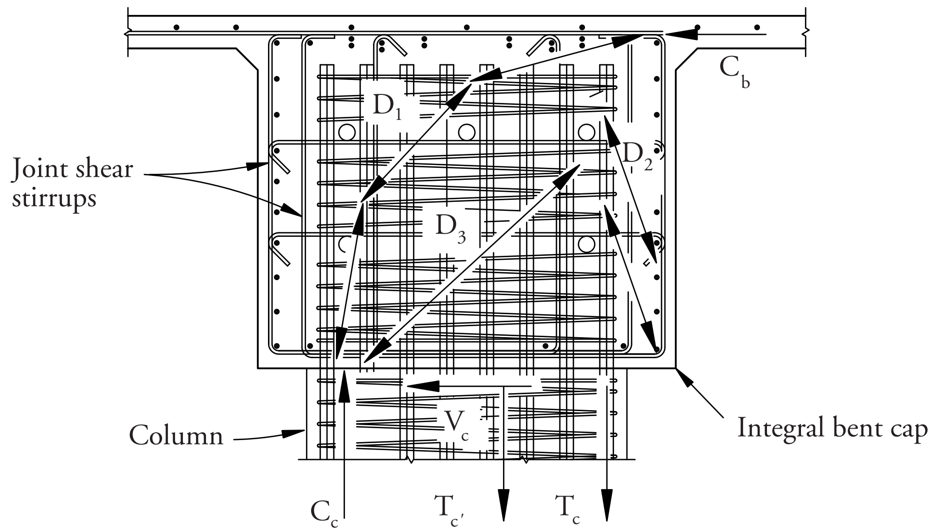
- If the principal tension stress $\leq 3.5\sqrt{f_c'}$ psi, only nominal joint reinforcement is required.
- If the principal tension stress $> 5\sqrt{f_c'}$ psi, all requirements for joint reinforcement must be met in accordance with a force-transfer mechanism.
- If the principal tension stress is $> 3.5\sqrt{f_c'}$ psi and $\leq 5\sqrt{f_c'}$ psi, linear interpolation between full and nominal requirements for joint reinforcement must be met.

The principal tension stress is between $3.5\sqrt{f_c'}$ psi and $5\sqrt{f_c'}$ psi, so a linear interpolation between full and nominal joint reinforcement requirement would need to be provided. However, for the purpose of this design example, the cap will be designed for the full joint shear requirement.

15.5.4 Joint Reinforcement Design

The joint design is in accordance with the procedure described in Priestley, et al (1996) and verified by experimental results reported by Holombo, et al (2000). The assumed joint force transfer mechanism is shown in **Figure 15.5.4-1**. The assumed mechanism reduces congestion by placing the joint reinforcing steel outside the column core region. The joint reinforcing steel facilitates the transfer of the column tension force to the top of the joint.

Figure 15.5.4-1
Assumed Mechanism for Joint Force Transfer in Pier Cap



Assumptions:

1. 75% of all column reinforcement providing $T_{c'}$ is clamped by the main diagonal compression strut D_1 (see **Figure 15.5.4-1**).
2. The remaining 25% of the total longitudinal column reinforcement at appropriate strain hardening stress, T_c, is clamped by the diagonal compression struts, D_2 and D_3. The vertical components of D_2 and D_3 are assumed equal. External joint stirrups allow the development of strut, D_2, which helps redirect the compression force, C_b, into the middle of the joint.

The external vertical reinforcement, A_{jv}, should be placed over a distance of $h_b/2$ from the column face on each side of the column in accordance with the following equation:

$$A_{jv} = 0.125\, A_{sc} \frac{f^o_{yc}}{f_{yv}}$$

where

A_{sc} = total area of longitudinal reinforcement in column section = (30)(1.56) = 46.8 in.^{2}

f^o_{yc} = material over-strength stress of column reinforcement allowing for strain hardening

f_{yv} = yield strength of joint vertical reinforcement

Taking $f^o_{yc} = 1.4 f_{yv}$ for Grade 60 reinforcement:

$$A_{jv} = (0.125)(46.8)\frac{1.4f_{yv}}{f_{yv}} = 8.19 \text{ in.}^2$$

The number of #6 stirrup legs required = A_{jv}/0.44 = 8.19/0.44 = 18.6, say 20 legs. Provide 10 #6 two-legged stirrups on each side of the column face over a distance of h_b/2 = 87/2 = 43.5 in. from the column face.

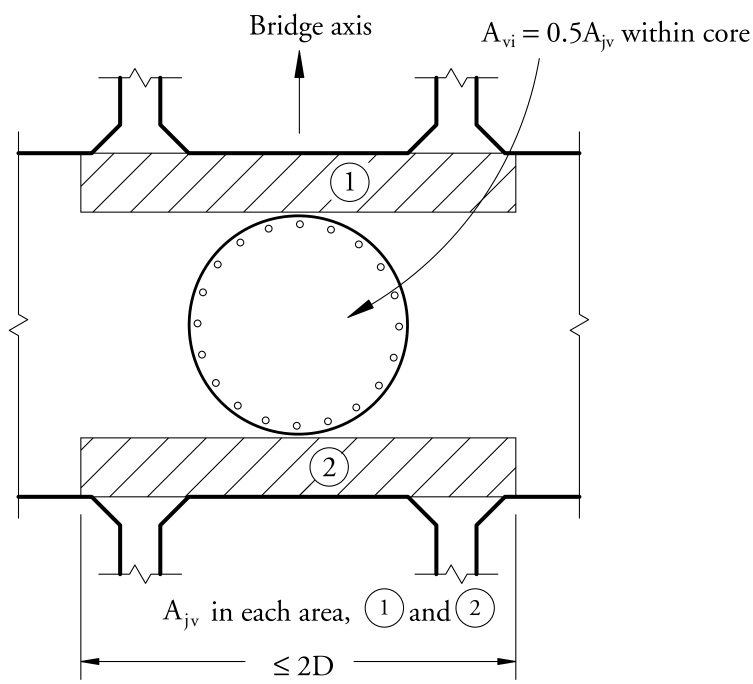
An additional amount of vertical reinforcement equal to half of this amount should be placed within the joint confines to help stabilize top beam reinforcement and assist in the transfer of column tension force by bond.

Interior vertical joint stirrup area, A_{vi}, is determined by:

$$A_{vi} = 0.0625 A_{sc}\frac{f^{o}_{yc}}{f_{yv}} = (0.0625)\ (46.8)\frac{1.4\ f_{yv}}{f_{yv}} = 4.10 \text{ in.}^2$$

The number of #6 stirrups required = A_{vi}/0.44 = 4.10/0.44 = 9.3, say 10 legs. Provide (10) #6 single leg stirrups within the column core. As the clamping action occurs at the top of the joint, these stirrups need not extend to the base of the joint. They are extended at least two-thirds of the bent cap depth or 2/3(87) = 58 in. **Figure 15.5.4-2** indicates the locations for the placement of vertical joint reinforcement.

Figure 15.5.4-2
Locations for Vertical Joint Reinforcement



Note: The reinforcement placed outside the column core over a length of h_b/2 is in addition to the shear reinforcement required for conventional shear transfer in the beam.

Horizontal hoop reinforcement must be provided to resist a force of one-quarter of the tension force in the column steel due to the plastic moment ($0.25T_c$) in accordance with the following equations:

$$\rho_s = \frac{3.3}{Df_{yh}l_a}\left(\frac{0.09A_{sc}f^o_{yc}D}{l_a}\right) - F$$

where

l_a = assumed length of column anchorage reinforcement in joint = 80 in.

F = bent cap prestressing force after all losses

Assuming F = 0, the simplified equation is:

$$\rho_s = \frac{0.3A_{sc}f^o_{yc}}{l_a^2 f_{yh}}$$

$$= \frac{(0.3)(46.8)(1.4f_{yh})}{(80)^2 f_{yh}} = 0.00307$$

However, minimum horizontal hoop reinforcement, $\rho_{s,min}$, must be provided according to the following:

$$\rho_{s,min} = \frac{3.5\sqrt{f'_c}}{f_{yh}} = \frac{(3.5)\sqrt{4000}}{60{,}000} = 0.00369 > 0.00307$$

Use $\rho_s = 0.00369$

$$s_{reqd} = \frac{4A_h}{D'\rho_s} = \frac{(4)(0.44)}{(68.25)(0.00369)} = 6.99 \text{ in.}$$

where

s_{reqd} = required spacing of hoop reinforcement

A_h = area of hoop reinforcement

D' = core diameter of spirally confined column = 68.25 in.

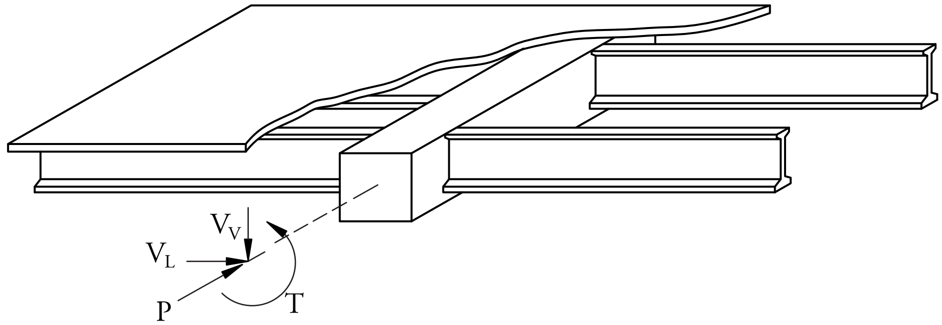
Provide #6 stirrups @ 6 in. spacing. If the hoop reinforcement ratio provided is less than the required ratio, the difference could be made up with split hairpins as described in Holombo, et al (2000).

Note: The hoop spacing could be decreased if the cap beam prestress force, F, is considered.

15.5.5 Shear-Friction Analysis

In the absence of the bottom slab in spliced I-beam bridges, column moments and shears are transferred into the beams through the cap completely through torsional mechanisms. Due to the limited length available between the face of the column and the beam, spiral cracks typically associated with torsion cannot fully develop. Therefore, conventional torsion design methodologies that are primarily based on this cracking pattern are not applicable. Instead, the torsional capacity is calculated using a plastic friction model as illustrated in **Figure 15.5.5-1**.

Figure 15.5.5-1
Torsional Shear-Friction Mechanism

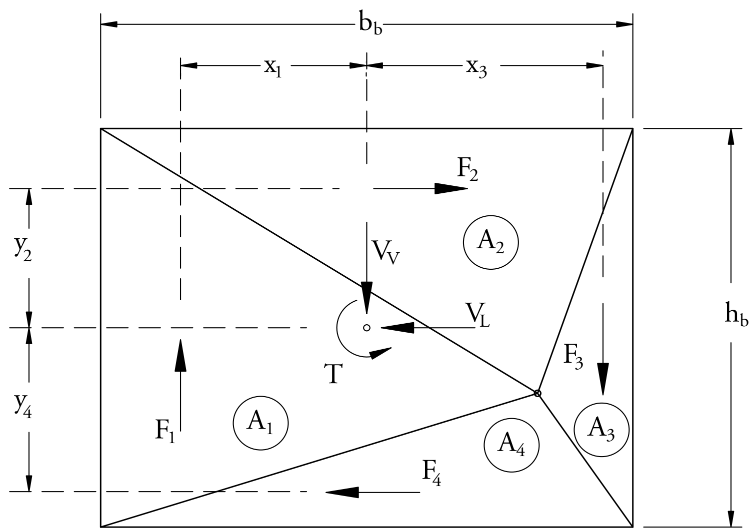


Assumptions:

1. Shearing stress is assumed constant over the cross-section and proportional to the normal force, P.
2. Shear-friction contribution of each segment is assumed proportional to the area of each segment.

The bent cap section is subjected to a vertical shear force, V_V, a horizontal shear force, V_L, a torsion, T, and an axial clamping force, P. The cap is divided, conceptually, into four unequal segments of areas, A_1 to A_4, as shown in **Figure 15.5.5-2**.

Figure 15.5.5-2
Conceptual Force Diagram For Resisting Torque In Bent Cap



The direction of shear-friction resistance within each of the four segments is taken as parallel to the outer edge, and the shear-friction stress, τ, is taken as:

$$\tau = \mu\ P/A$$

where

A = total section area

μ = coefficient of friction over the interface

The force, F_i, in each segment, is then given by:

$F_i = \tau A_i$

where A_i = area of the segment.

Equilibrium under external torsion, T, longitudinal shear force, V_L, and vertical shear force, V_V, requires that:

$V_V = F_1 - F_3 = \tau(A_1 - A_3)$

$V_L = F_2 - F_4 = \tau(A_2 - A_4)$

$T = F_1x_1 + F_2y_2 + F_3x_3 + F_4y_4 = \tau(A_1x_1 + A_2y_2 + A_3x_3 + A_4y_4)$

where

T = external torsion

x_1, x_3, y_2 and y_4 are distances defined in **Figure 15.5.5-2**

The equations can be solved through trial and error by dividing the section into segments and trying different values until all three equations are satisfied, then checking that the implied value of the friction coefficient, μ, is reasonable. Alternatively, a limit design value of μ = 1.4 can be used with F_1 to F_4 selected to satisfy the first two equations shown above. The torque predicted by the third equation must then be checked to ensure that it exceeds the applied torque. The latter alternative is utilized in this design solution.

The normal force, P, is calculated assuming a dilation strain of 0.0005 developed on the shear plane.

$P = F + A_s(0.0005)E_s$

where

F = prestressing force after all losses

A_s = area of reinforcing steel passing through shear plane including the prestressing steel

= three layers of ten #10 bars

= (3)(10)(1.27) = 38.1 in.^{2} (ignore prestressing steel to be conservative)

A_{ps} = area of prestressing steel

= (6)(19)(0.217) = 24.7 in.^{2}

F = (202.5)(0.8)(24.7) = 4,001 kips (A 20% loss is assumed)

P = 4,001 + (38.1)(0.0005)(29,000) = 4,553 kips

Assume μ = 1.4 because surface of beam in joint region is roughened (Caltrans, 2000)

$$\tau = \frac{\mu P}{A} = \frac{(1.4)(4{,}553)}{(7.25)(7.00)} = 125.6 \text{ kips/ft}^2$$

Horizontal shear force at top of column, $V^o_{i,column} = \dfrac{M^o_{bot} + M^o_{top}}{H}$

$$V^o_{i,column} = \left(\frac{14,340 + 14,115}{25.75}\right) = 1,105 \text{ kips}$$

Moment at middepth of bent cap, $M^o_{i,bentcap} = M^o_{top} + V^o_{i,column}\left(\frac{h_b}{2}\right)$

$$M^o_{i,bentcap} = 14,115 + 1,105\left(\frac{87}{(2)(12)}\right) = 18,120 \text{ ft-kips}$$

$P_{DL,TOP} = 2,225$ kips

Using a factor of safety of 1.1 for shear-friction analysis, the required resistances are as follows:

Torsion: $\frac{M^o_{i,bentcap}(1.1)}{2} = \frac{(18,120)(1.1)}{2} = 9,966$ ft-kips

Longitudinal shear: $\frac{V^o_{i,column}(1.1)}{2} = \frac{(1,105)(1.1)}{2} = 608$ kips

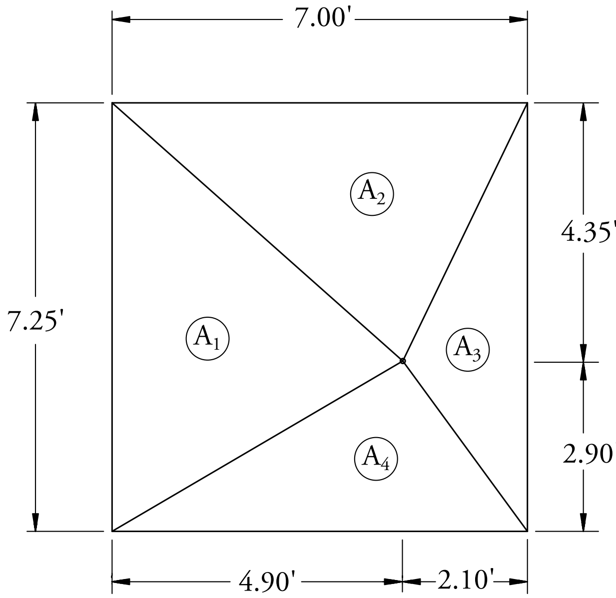
Vertical Shear: $\frac{P_{DL,TOP}}{2} = \frac{(2,225)(1.1)}{2} = 1,224$ kips

Table 15.5.5-1 summarizes the torsional shear-friction computations to ascertain the ability of the bent cap to transfer the column plastic moment capacity to the bridge superstructure. The assumed dimensions of each segment are shown in **Figure 15.5.5-3**.

Table 15.5.5-1
Torsional Shear-Friction Computations

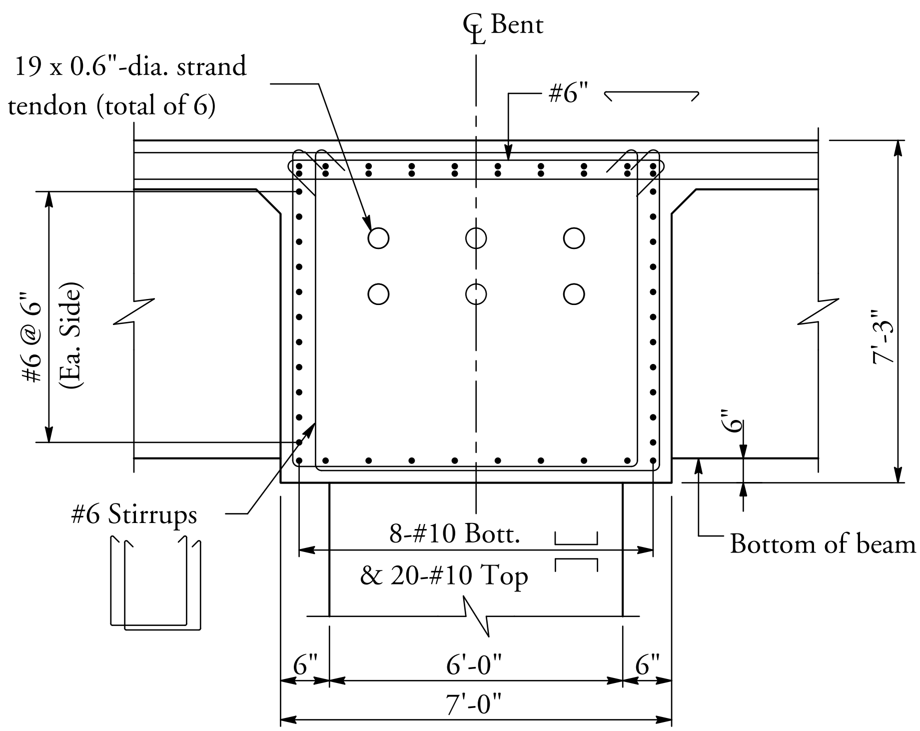
Given: Bent Cap Depth = 7.25 ft; Bent Cap Width = 7.00 ft; Axial Force = 4,553 kips; Friction Coefficient = 1.4; τ = 125.6 kips/ft^2			Assumed: X-Coordinate = 4.90 ft; Y-Coordinate = 2.90 ft	
Segment No.	Segment Size, ft	Area, ft^2	Distance from Centroid, ft	First Moment about Centroid, ft^3
1	7.25 x 4.90	A_1 = 17.763	x_1 = 1.867	33.16
2	7.00 x 4.35	A_2 = 15.225	y_2 = 2.175	33.11
3	7.25 x 2.10	A_3 = 7.613	x_3 = 2.800	21.32
4	7.00 x 2.90	A_4 = 10.150	y_4 = 2.658	26.98
Total		50.750		114.57
			Capacity	Required
$T = \tau(A_1x_1 + A_2y_2 + A_3x_3 + A_4y_4)$, ft-kips			14,390	9,966
$V_V = \tau\ (A_1 - A_3)$, kips			1,275	1,224
$V_L = \tau\ (A_2 - A_4)$, kips			637	608

Figure 15.5.5-3
Assumed Dimensions for Shear-Friction Computations



The bridge superstructure moment capacity must also be checked to ensure that the plastic hinges form in the column and not in the superstructure. **Figure 15.5.5-4** depicts the reinforcement details for the integral bent cap.

Figure 15.5.5-4
Integral Bent Cap Reinforcement Details



Bent Cap Detail at Column

15.6 CALTRANS RESEARCH

Bridges in California have been predominantly CIP box girder systems due to the requirements for high seismic resistance. The examples presented in **Sections 15.4** and **15.5**, show that spliced beams with integral bent caps can provide a viable solution for highway bridges in moderate to high seismic areas. The precast beam, integral-cap system was tested and has proven to provide levels of seismic resistance comparable to CIP box girders. With minimal shoring and forming requirements, the new system will shorten construction time, reduce interruption to traffic, and lower the environmental impact. Other benefits of precast beams are reduced cracking due to better quality control and efficient utilization of higher concrete strengths. As a result, significant initial and long term cost savings are possible with the new system.

Research conducted at the University of California at San Diego (UCSD) included constructing and testing two 40% scale models under fully-reversed longitudinal seismic loading. The first model utilized a modified version of the Florida bulb-tee beam. The second model, of similar scale, incorporated trapezoidal “U-shaped” beams. The objective of the testing program was to verify the structural adequacy of newly developed integral column-superstructure details under simulated seismic loads, and to allow Caltrans engineers to evaluate the constructibility of these details via large-scale models. The following sections describe the tests and results.

15.6.1 Test Model Set-Up

The focus of this research was to study the effects of longitudinal seismic forces on the column-superstructure continuity. The prototype structure for the bulb-tee beam system is shown in **Figure 15.6.1-1**. The dimensions and forces of the model test unit were scaled directly from the prototype structure. The region selected for study included the column, bent cap and full-width superstructure extending from midspan to midspan. Two horizontal actuators placed on either side of the unit applied load to model the seismic inertia forces acting along the bridge. Four vertical actuators located at the corners of the test unit applied seismic shear into the beams. The test setup is shown schematically in **Figure 15.6.1-2** (Holombo, et al, 2000).

Figure 15.6.1-1
Prototype Structure for Bulb-Tee System Testing Program

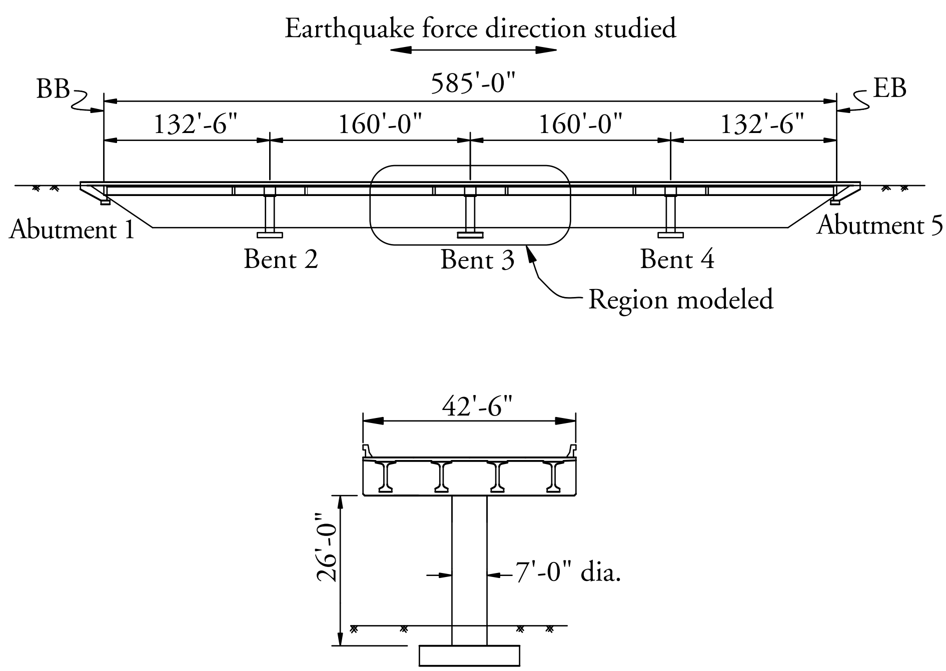
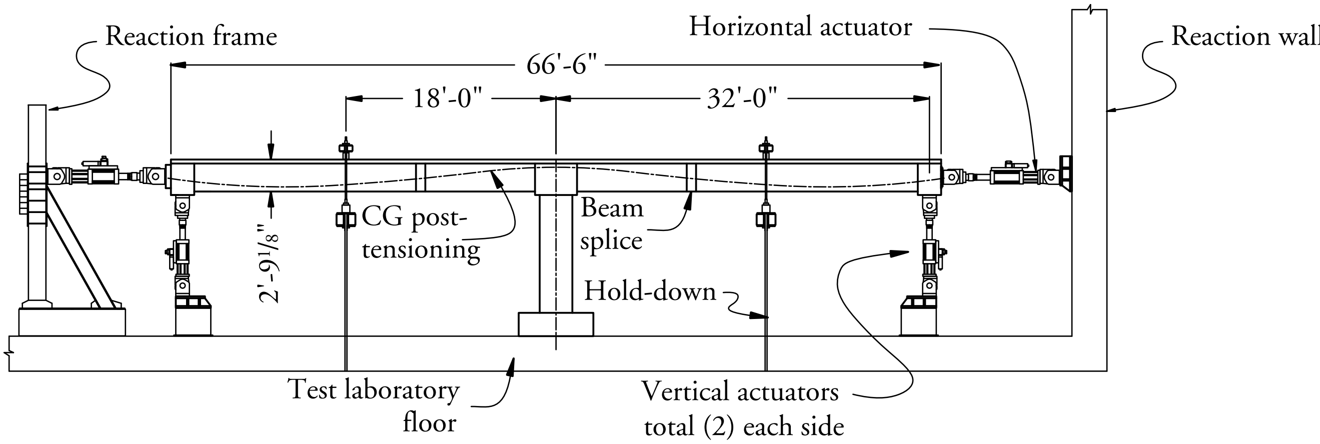


Figure 15.6.1-2
Test Setup



The prototype structure for the second test used precast U-beams. The U-beam segments were spliced at the bent cap and at the midpoint of each span. A single-pour, monolithic bent cap is possible with this system because it is not feasible to make the U-beams continuous over the bent. A setup similar to the one shown in **Figure 15.6.1-2** was used for the second test.

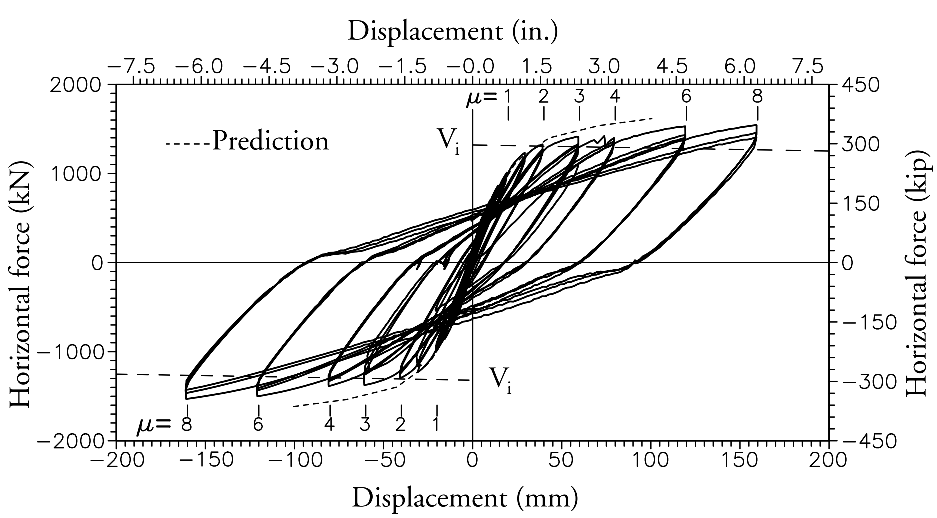
15.6.2 Test Results

Performance of the model bridge structures exceeded the design requirements during the tests (Holombo, et at, 2000).

15.6.2.1 Columns

Ductile plastic hinges formed at the top and bottom of the column with little strength degradation up to a displacement ductility of eight and six for bulb-tee and U-beam models, respectively. Both models exceeded the design ductility capacity of four. The force-displacement loop for the bulb-tee model is shown in **Figure 15.6.2.1-1**.

Figure 15.6.2.1-1
Hysteresis Loop from Testing of the Bulb-Tee System



15.6.2.2 Superstructure

The response of the superstructure to the simulated longitudinal seismic loading was essentially elastic; only minor cracking was observed. Due to prestressing, the cracking in the bent cap and the beams closed upon removal of the seismic loads, making potential repair of the superstructure after a design level earthquake, essentially cosmetic.

These tests demonstrate the versatility and flexibility of precast spliced-beam systems. Specifically, the tests proved that an integral connection between the superstructure and substructure can be achieved with or without beam continuity through the bent cap. They also proved that, with proper design and detailing, the beam splice points could be placed anywhere in the span or over the supports without any measurable reduction in performance of the system.

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TABLE OF CONTENTS
ADDITIONAL BRIDGE PRODUCTS

Additional Bridge Products

16.1 INTRODUCTION

Precast concrete products that are manufactured for bridges are not limited to superstructure members such as I-beams, box beams and voided slabs. Many additional precast components are widely used. These products have been used successfully, some for more than 50 years, and have been proven economical. The advantages cited for precast beams are characteristic of a large number of other bridge components. Speed of construction and the consistent high quality of precast products are equally valuable to the owner and contractor for these other parts of the bridge. Some of these precast components for bridges include stay-in-place (SIP) deck panels, full-depth deck panels, piles, pile bent caps, railings, piers, pier columns and abutments. Precast systems are also used for earth retaining systems and box culverts. Each of these applications of precast products is discussed in this chapter.

16.2 STAY-IN-PLACE DECK PANELS

Precast, stay-in-place (SIP) deck panels are precisely-designed and fabricated precast concrete slabs spanning between concrete or steel beams. They serve as a form for the cast-in-place (CIP) deck concrete and provide the principal positive moment reinforcement in the composite deck. That is, the prestressing or conventional reinforcement in the panel is designed to resist the self-weight of the panel and the weight of the field-placed concrete topping as a non-composite section. Then, after the field-placed concrete topping cures, the SIP panel becomes an integral part of the composite deck that resists all subsequent dead and live loads applied to the deck. See Section 3.8 for more information on the fabrication of SIP panels.

16.2.1 Advantages

SIP panels provide a permanent structural form for the bridge deck. The contractor is able to quickly use a broader and therefore safer working area provided by the panels. In addition, there is no need for the contractor to remove formwork after the CIP portion of the deck has cured, thereby saving a considerable amount of labor, time and exposure. The contractor saves the time, labor and material of having to install a bottom mat of reinforcing steel. The amount of field-placed and field-cured concrete is substantially reduced compared to a concrete deck cast on removable or steel forms. **Figure 16.2.1-1** illustrates the working surface provided by the installation of SIP panels on the Louetta Road Bridge (Ralls, et al, 1993).

Figure 16.2.1-1
SIP Panel Erection between Precast Concete U-Beams

SIP panels are fabricated using dense, high performance concrete that resists chloride intrusion. The prestressed panel is designed to be crack-free throughout its depth for all loading conditions. Research and practice have shown that panels placed with tight joints result in excellent local continuity even without dowels projecting across joints. Although some cracks have appeared in the CIP portion of the deck, these cracks are generally smaller and of less consequence than cracks in fully-cast-in-place decks. Research by Tsui, et al, 1986, has shown that decks properly constructed with precast, prestressed SIP panels are stronger, stiffer and more crack-resistant than CIP decks.

16.2.2
Size Selection

The depth of SIP panels that are commonly used for composite decks, ranges from 3 in. to 4.5 in., based largely on the design span of the deck. The depth of the SIP panel should be as deep as necessary to provide the maximum benefits from the use of precast concrete while allowing for a depth of CIP concrete adequate to provide cover over the top mat of mild reinforcing steel.

The width of SIP panels (perpendicular to the panel span) is typically standardized on an even dimension of 4 or 8 ft. Narrower panels can be cast or cut to fill the spaces remaining at the ends of the bridge span. Panels cast or cut to skews may also be used at the ends of each bridge span.

SIP panel span lengths vary based on beam spacing and are feasible whenever a CIP deck may be used. Panel lengths can be as short as needed. Panels shorter than two times the development length of the prestressing strands must be designed to account for a reduced ultimate moment capacity or be provided with additional mild steel reinforcement. Sometimes only conventional reinforcement is used. The details should accommodate a minimum of 3 in. lap over each beam.

Panel thickness is important when choosing the diameter of the prestressing strands to be used. As illustrated in **Table 16.2.2-1**, the recommended ratio of panel thickness to strand diameter is 8:1.

Table 16.2.2-1
Recommended Strand Diameter for Panel Thickness Shown

Panel Thickness	Recommended Strand Diameter
3"	3/8"
3 1/2"	3/8" or 7/16"
4" or more	1/2"

16.2.3
Design Criteria

Stress limits for prestressed members are shown in the *AASHTO Specifications* (STD Art. 9.15.2; LRFD Art. 5.9.4.2). Tension limits depend on the environmental conditions at the project site. Typically, tension is limited to $6\sqrt{f'_c}$ for non-corrosive locations and $3\sqrt{f'_c}$ for corrosive environments. The stress history for both tension and compression must be determined for the SIP panel for non-composite and composite loads. As with other prestressed concrete design procedures, the design is based on the service condition (allowable stress) and then evaluated for ultimate capacity or strength. Design live load plus impact is identical to that used for a CIP deck. Considerable additional information on design is found in Section 8.8 and Design Examples 9.7 and 9.8.

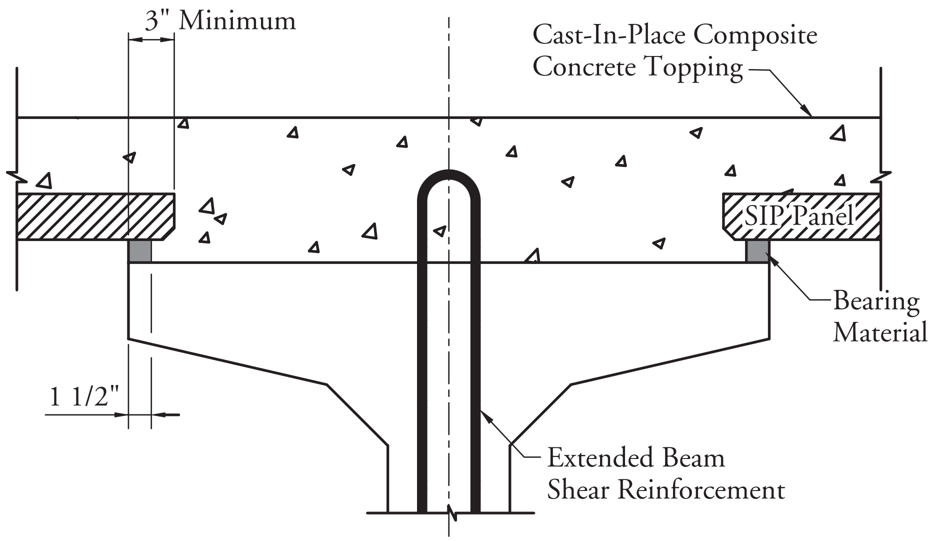
16.2.4
Design, Fabrication and Construction

The successful and economical use of SIP deck panels depends entirely on the selection of proper design, fabrication and construction details. These details include bearing on the girder, strand placement, strand extension, horizontal shear, provisions for handling and shipping, and even the age of the panels at installation. A discussion of each of these topics follows.

16.2.4.1
Bearing Details

The SIP panel must have positive bearing on the supporting beams. Typically, the panel is temporarily supported on a strip of slightly compressible material near the edge of the beam. The panel should overhang this bearing material by a minimum of 1.5 in. The bearing material must be tall enough to allow the CIP deck concrete to flow under the panel and completely fill the space between the top of the beam and the bottom of the SIP panel. Care must be used to ensure proper consolidation of concrete into this space. When the CIP concrete cures, it provides positive bearing for the panel and allows for transfer of superimposed loads from the deck to the beam **Figure 16.2.4.1-1**. Some panel systems use adjustable screw jacks to set the elevation of the panel and temporarily support the panel on the beam. Fillers are used between the panel and the girder. When the concrete cures, the screw jacks are removed and a solid, positive and permanent bearing remains. One system is shown in Chapter 3, Fig. 3.8.3-1. Some projects such as Fancher Road Bridge, 1984, have used side forms mounted on the beams to support panels together with non-shrink grout beneath panels. This method has proven less effective than others described.

Figure 16.2.4.1-1
SIP Panel Bearing Detail



16.2.4.2
Reinforcement

Proper strand placement during fabrication is very important particularly in a thin section like a SIP panel. The strands may be designed to be eccentric about the horizontal center of gravity of the panel. However, most designers place the strands concentric at mid-depth of the panel. Eccentricity increases the efficiency of the pre-stress force. It provides initial camber prior to placement of the CIP concrete. As always, specified concrete cover over the prestress strands must be maintained.

Welded wire reinforcement or reinforcing bars are provided as nominal shrinkage and temperature reinforcement to control potential cracking. Small-diameter bars are often used at the ends, both above and below strands, to control potential cracks due to transfer of force upon detensioning.

16.2.4.3
Strand Extensions

Research performed at the University of Texas at Austin (Bieschke and Klingner, 1982) indicates that there is no local or global difference in the performance of decks constructed using SIP panels with or without strand extensions. There is a significant implication to strand extensions. Strand extensions require the fabricator to install forms in the bed between each panel thereby increasing fabrication cost. If there is no strand extension, the panels may be cast in a single, long slab and subsequently cut to length when the concrete reaches transfer strength.

Strand extensions are generally not recommended. However, some designers prefer to use them citing these benefits:

- The use of a form between each panel in the prestressing bed helps assure proper strand position
- The extension acts as a dowel in the CIP deck
- The embedment of the strand into the CIP concrete may reduce the potential separation of the panel end from the CIP concrete that may be caused by creep, shrinkage and thermal effects.

16.2.4.4
Composite Behavior

Mechanical shear connectors are not required to achieve composite action between the SIP panel and the CIP slab. Full composite action is achieved if the deck panel is intentionally roughened during fabrication and is free of contaminants. Research by Kumar and Ramirez (1996) has shown that prestressed SIP deck panels with a 0.05 to 0.075-in. amplitude broom-finished surface do not require horizontal shear connectors to achieve full composite action with the CIP topping.

Thorough wetting of the SIP panel without ponding prior to placement of the CIP slab will provide adequate bond at the interface. Tests conducted by the University of Texas at Austin for the Texas Department of Transportation confirm excellent structural performance (Burns, et al, 1990-A and 1990-B).

16.2.4.5
Handling

Generally, the larger the deck panel, the more susceptible it is to damage during shipping and handling. Handling stresses should be investigated during design. Plans should provide details and locations for lifting and storing the panels. Storing and stacking should be done in a manner that does not induce undesirable stresses in the panels. See Section 3.3.8.3. **Figure 16.2.4.5-1** shows panels being delivered to a project site. If panels are handled in stacks, special support slings or lift devices may be required to prevent bending or torsion in the panels and overloading of panels near the bottom of the stack.

Figure 16.2.4.5-1
SIP Deck Panels being Delivered to Bridge Site

16.2.4.6
Age at Installation

SIP panels often contain relatively high levels of prestress. Therefore, creep can be an important consideration. It is desirable that panels be from one- to two-months old when the CIP portion of the deck is placed. This substantially reduces the potential for distress in the CIP deck associated with creep and shrinkage of the panel. Klingner (1989) makes the conservative recommendation that planks be at least two months old. He notes that satisfactory results have been achieved with less time. He provides procedures for calculation of more precise minimum time estimates.

16.2.5
Applications

Precast, prestressed concrete SIP deck panels are used extensively in several parts of the country. In most of these areas, state highway agencies have prepared standard designs and details that are incorporated in the plans for most bridges. Many major projects, as well as hundreds of smaller bridges have used SIP panels since the mid-1950s. When offered on plans as an alternate, experienced general contractors usually select SIP panels because of their inherent cost savings due to speed, safety and simplicity. These benefits are apparent in **Figure 16.2.5-1** where SIP panels were used to both cover and span between open-topped trapezoidal box beams.

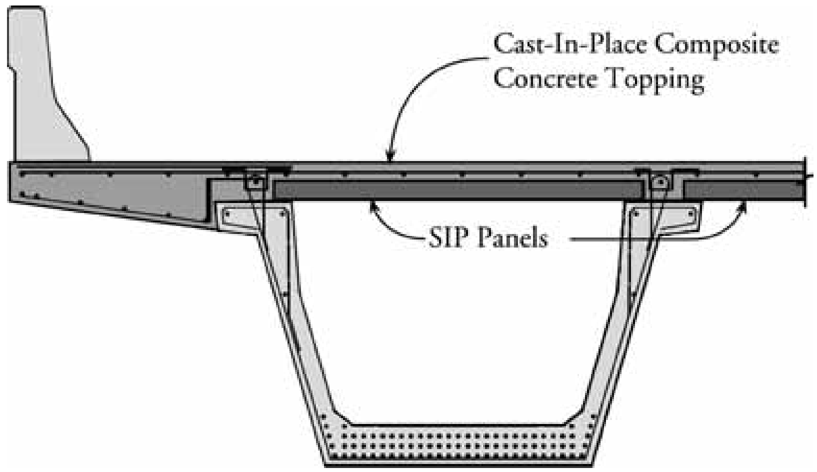


Figure 16.2.5-1
SIP Panels used with U-Beams
(Texas DOT)

16.2.6
Additional Information

The March/April 1988 issue of the PCI JOURNAL contains a Recommended Practice for Precast Concrete Composite Bridge Deck Panels. This report presents detailed recommendations for the design, manufacture and erection of SIP panels. The report also includes a thorough design example in accordance with the *AASHTO Standard Specification* in effect at that time.

Section 8.8 provides additional detailed design information. Design Examples 9.7 and 9.8 provide both *Standard* and *LRFD Specifications* examples, respectively, of deck panel design.

A unique precast composite deck system, NUDEK, is described in Section 8.8.3.1 and in Badie, et al, 1998. It provides a method to span multiple beams and eliminate the conventional forming of overhangs. The system is estimated to reduce construction time by 60 percent.

The following are references where additional detailed information on panel research and installations may be found: Buth, et al, 1972; Barker, 1975; Kluge and Sawyer, 1975; Texas Highway Department, 1975; Barnoff, et al, 1977; PCI Bridge Committee, 1978; Kelly, 1979; Kao and Ballinger, 1982; Slavis, 1983; and PCI Committee on Bridges, 1987.

16.3 FULL-DEPTH DECK PANELS

In addition to using precast concrete panels as stay-in-place forms for bridge decks, more and more projects are using precast concrete to provide the full depth of the bridge deck (Biswas, 1986). This product is particularly useful on projects where:

- The installation and stripping of forms proves difficult;
- The placement of large quantities of CIP concrete is difficult;
- Disruption of traffic is an overriding concern due to safety or user costs.

In these situations, the deck slab may be fully precast and then erected on the supporting members.

16.3.1 Applications

Full-depth deck panels are suited for bridge rehabilitation projects requiring new decks and especially in areas with high traffic volumes. The speed of installation is extremely important under these conditions. Culmo (1991) reports that in Connecticut, a six-span, 700-ft-long, one-lane ramp with compound curvature on a seven-percent grade was completely replaced in 48 days. On another project, 34 spans were replaced over two construction seasons using only 60-hour weekend closures. Entire spans were replaced within a 60-hour window (Culmo, 2000). The Pimmit Run Bridge on the George Washington Memorial Parkway in Fairfield, VA, has a length of 345 ft and a width of 68 ft. It was redecked inside of six weekend closures (Miller, et al, 1991).

Large sections of deck can be removed and replaced quickly, minimizing the impact to traffic. In many projects, traffic on a bridge has been maintained without significant interruption during construction. This was the case in the redecking of the Woodrow Wilson Memorial Bridge in Washington, DC (Lutz and Scalia, 1984).

Full-depth panels benefit new construction as well. In addition to speed of installation, all of the other unique advantages of precast concrete come into play, such as dense, durable concrete, reduced reliance on ready-mixed concrete and field labor, elimination of field curing systems and time-consuming field forming. Full-depth deck panel systems have proven cost competitive when compared to alternate construction methods. Enthusiasm over a redecking project in 1982 led to a proposed new deck system for bridges. It included a cost estimate that showed a system savings of more than eight percent (Kempf, 1983).

The report of a survey on full-depth panels conducted by Issa, et al (1995) revealed experiences and methods used by 13 states and one providence. The survey indicated that 43 states were interested in using this system of construction.

The growing interest by the owner agencies in methods to speed construction have resulted in the development of new full-depth deck panel systems. One unique panel system is described briefly in Section 8.8.3.2. It resulted from a federally-funded study by the Center for Infrastructure Research at the University of Nebraska (Tadros and Baishya, 1998). Even though it is a full-deck system, it is reported to be thinner and lighter then other concrete deck systems available. The panels use high performance concrete and are prestressed in both directions. A portion of this study with a description of load tests is reported in Yamane, et al (1998).

16.3.2 Reinforcement

Bridge decks have been successfully installed using panels incorporating either mild steel reinforcement or prestressing. Prestressed concrete panels may be pretensioned in the plant or post-tensioned in the plant or in the field. In most installations, some amount of field-applied post-tensioning has been used. The need for prestressing is dependent on specific project objectives and panel handling stresses.

16.3.3 Connections

Proper connection of full-depth deck panels to the supporting superstructure and to adjacent panels is important to ensure satisfactory performance of the deck. Various means have been used to connect full-depth panels to beams and panels to panels.

16.3.3.1 Panel-to-Beam Connections

Early installations of full-depth precast concrete deck panels used connections to beams that did not provide composite action. These simple connections, such as used at the Pintala Creek Bridge in Montgomery, AL, (Biswas, 1986) clamped the panels to the beams with a bolt and plate system. This ensured only that the panels would not be dislodged from the beams during subsequent construction operations (**Figure 16.3.3.1-1**). More recently, connections have been designed to transfer horizontal shear between the beams and slabs to make use of the efficiency of composite action. In most cases, a pocket is cast in the precast slab during fabrication. In some

Figure 16.3.3.1-1
Slab-to-Beam Connection, Pintala Creek Bridge

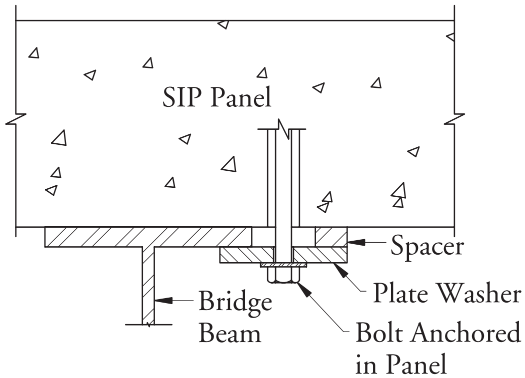
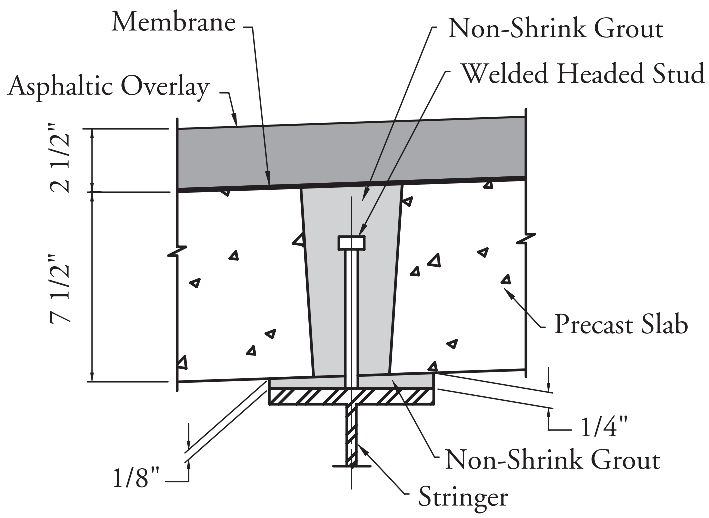


Figure 16.3.3.1-2
Slab-to-Beam Connection, Bridge No. 6, NYSDOT

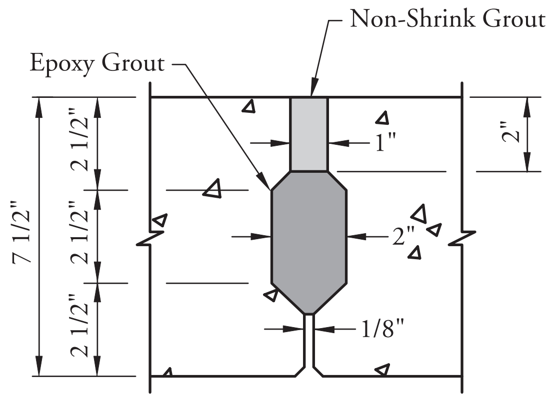


instances, the locations of these pockets are coordinated with those of shear connectors attached to the beams (**Figure 16.3.3.1-2**). In most cases, shear connectors are attached to beams through pockets in the panels following erection of the panels. This eliminates problems with coordinating the locations of pockets and connectors. There is, of course, a slight increase in field labor. There were a few bridge decks built by the New York State Thruway Authority that had panels bolted directly to steel girders. Difficulty was reported in achieving proper bolt tension. In addition, slab-cracking caused by deflection due to bolt tensioning resulted in alternate methods of connecting decks to beams in subsequent installations (Biswas, 1986).

16.3.3.2
Panel-to-Panel Connections

Several panel-to-panel connection techniques have been developed in response to specific project requirements. Open gaps between panels have been provided that are sufficiently large to provide for lapped reinforcing bars that project from panel edges. These joints are then filled with field-placed concrete. This type of joint diminishes a primary advantage of speed of construction by requiring installation of forms and the placement and curing of concrete. Most installations have used keyways that are filled with non-shrink grout or epoxy mortar (**Figure 16.3.3.2-1**). In several projects, post-tensioning perpendicular to the panel joints was used to enhance performance of the keyway (Biswas, 1986). A significant analytical study was conducted by Issa, et al, 1998, that evaluates the performance of joints with different levels of post-tensioning.

Figure 16.3.3.2-1
Panel-to-Panel Connection, Bridge No. 6, NYSDOT (Biswas, 1986)



16.4 PILES

One of the most versatile precast concrete components is prestressed concrete piling. Piles are not only used in bridge construction, but also in buildings and marine facilities. Prestressed piles have been in constant use since the inception of prestressed concrete. Thousands of installations throughout the country date back to the early-to-mid-nineteen fifties. Prestressed piles have generally demonstrated outstanding performance. Chapter 20 is devoted to an in-depth discussion of prestressed concrete piling applications including design, fabrication and installation.

16.4.1 Applications

The uses of prestressed concrete piling range from pier foundations for minor grade crossings to the foundations of major bridges over navigable waterways, such as the Sunshine Skyway Bridge in Tampa Bay, FL (Chandra and Szecsei, 1988). Concrete piles are especially effective for supporting bent caps. In this application, they extend from the ground into a cap that supports the superstructure. Two examples appeared in the PCI JOURNAL in 1989. These are the Bohemia River Bridge and the Richmond, Fredericksburg and Potomac Railroad Bridge.

Prestressed piles are used extensively to support abutments. Moreover, where soil conditions will not adequately support the spread footing of a retaining wall, piles have been successfully used to withstand both axial and horizontal loads from the soil. Prestressed concrete piles are also used as soldier piles for other earth retaining systems. Due to their ductility and large moment capacity, prestressed concrete piles are used as fender piles and to support piers that must be designed for large ship impact forces.

16.4.2 Pile Shapes

Prestressed piles normally are one of three cross-sections: square, octagonal or cylindrical. Square piles as small as 12 in. by 12 in. are commonly used in some states for bridge construction. Pile sizes generally increase in 2-in. increments from 10 in. to 20 in. and starting at 24 in., in 6-in. increments to 30 in. and 36 in. In some states, the three larger sizes are cast with a circular void through the cross-section. This reduces the volume of concrete, lowers cost, shipping and handling weights, and facilitates field handling. Using a voided section also reduces the required prestress force. Octagonal piles typically range in width from 10 in. to 24 in., measured from flat-to-flat, in 2-in. increments. The three largest sizes (20, 22 and 24 in.) may be solid or cast with a central circular void.

Round piles are usually made with a circular void and are referred to as cylinder piles. These piles are generally larger than square or octagonal piles. Typical outside dimensions for cylinder piles are 36 in., 42 in., 54 in. and 66 in. The wall thickness is usually

5 in. but may be as much as 8 in. Cylinder piles are very effective when large vertical and/or horizontal forces are expected, such as for crossings over navigable waterways.

Square and octagonal piles are usually cast in long-line forms. Cylinder piles are usually fabricated in shorter sections and then post-tensioned together in the casting plant to provide the required length of pile. They may also be cast as one piece in long-line forms. Cylinder pile sections can be spun cast (compacted centrifugally) using closed forms. This increases the density of the concrete, reduces the water/cementitious materials ratio and even further improves the quality of these precast, prestressed concrete products.

16.4.3 Advantages

Prestressed concrete piles provide a very economical method of constructing deep foundations. Readily available local materials are used in the concrete mixture. Being prestressed, the pile can easily be handled, shipped to the project and driven to the required capacity. **Figure 16.4.3-1**, taken from Table 2.7.1 in the *PCI Design Handbook*, provides guidance for the designer concerning allowable concentric loads on prestressed piles. These values must be evaluated based on soil conditions and the ability of the soil to support loads.

Prestressed concrete piles exhibit substantial corrosion resistance. There is no steel exposed to the elements. The concrete is uncracked, dense and of high quality, typical of the concrete associated with plant-cast products. In some areas, engineered concrete mixtures are used to enhance the durability of the piles.

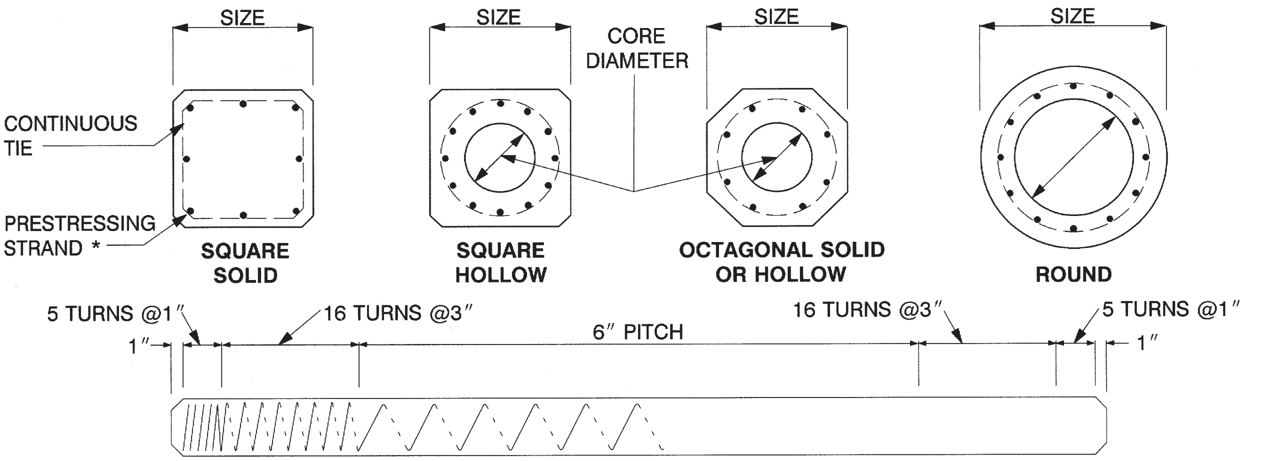
For most applications, the ability of the pile to transfer loads to soil is based on a combination of skin friction and end bearing. The shapes of prestressed concrete piles provide both a large surface area for friction, and a larger bearing area than most other pile shapes. This likely will reduce the length of pile necessary to obtain specified capacities.

16.4.4 Installation

The successful installation of prestressed concrete piles begins with proper handling and transportation. Damage to the piles may occur if not lifted, stored and transported according to proper industry practices. Once properly placed in the driving leads, a cushion, usually made from layers of plywood, is placed on the top of the pile to protect it from hammer damage during driving. The size of the hammer, the energy imparted during driving and the thickness of the cushioning material are all important considerations. A Wave Equation Analysis of Piles (WEAP) is often used to model the hammer-pile-soil interaction and to identify potential installation problems. These problems may include overstressing due to the use of an oversized hammer or the pile may reach premature refusal above the required pile tip elevation due to the use of an undersized hammer. Use of WEAP can supplement dynamic formulas in estimating the ultimate capacity of the piles and the required blow count to achieve that capacity. **Figure 16.4.4-1** shows the installation of cylinder piles. Note the joints where the pile segments have been grouted and post-tensioned together during production.

Figure 16.4.4-1
Driving Cylinder Piles

Figure 16.4.3-1 Section Properties and Allowable Service Loads of Prestressed Concrete Piles



TYPICAL ELEVATION

* Strand pattern may be circular or square.

SIZE (in.)	CORE DIA. (in.)	SECTION PROPERTIES [a] AREA (in^2)	WEIGHT (plf)	MOMENT OF INERTIA (in^4)	SECTION MODULUS (in^3)	RADIUS OF GYRATION (in.)	PERI-METER (in.)	ALLOWABLE CONCENTRIC SERVICE LOADS, tons [b] f'_c 5000	6000	8000	10,000
SQUARE PILES											
10	SOLID	100	104	833	167	2.89	3.33	73	89	122	156
12	SOLID	144	150	1,728	288	3.46	4.00	105	129	176	224
14	SOLID	196	204	3,201	457	4.04	4.67	143	175	240	305
16	SOLID	256	267	5,461	683	4.62	5.33	187	229	314	398
18	SOLID	324	338	8,748	972	5.20	6.00	236	290	397	504
20	SOLID	400	417	13,333	1,333	5.77	6.67	292	358	490	622
20	11	305	318	12,615	1,262	6.43	6.67	222	273	373	474
24	SOLID	576	600	27,648	2,304	6.93	8.00	420	515	705	896
24	12	463	482	26,630	2,219	7.58	8.00	338	414	567	720
24	14	422	439	25,762	2,147	7.81	8.00	308	377	517	656
24	15	399	415	25,163	2,097	7.94	8.00	291	357	488	621
30	18	646	672	62,347	4,157	9.82	10.00	471	578	791	1005
36	18	1,042	1,085	134,815	7,490	11.38	12.00	761	933	1,276	1,621
OCTAGONAL PILES											
10	SOLID	83	85	555	111	2.59	2.76	60	74	101	129
12	SOLID	119	125	1,134	189	3.09	3.31	86	106	145	185
14	SOLID	162	169	2,105	301	3.60	3.87	118	145	198	252
16	SOLID	212	220	3,592	449	4.12	4.42	154	189	259	330
18	SOLID	268	280	5,705	639	4.61	4.97	195	240	328	417
20	SOLID	331	345	8,770	877	5.15	5.52	241	296	405	515
20	11	236	245	8,050	805	5.84	5.52	172	211	289	367
22	SOLID	401	420	12,837	1,167	5.66	6.08	292	359	491	624
22	13	268	280	11,440	1,040	6.53	6.08	195	240	328	417
24	SOLID	477	495	18,180	1,515	6.17	6.63	348	427	584	742
24	15	300	315	15,696	1,308	7.23	6.63	219	268	368	467
ROUND PILES											
36	26	487	507	60,007	3,334	11.10	9.43	355	436	596	758
42	32	581	605	101,273	4,823	13.20	11.00	424	520	712	904
48	38	675	703	158,222	6,592	15.31	12.57	493	604	827	1,050
54	44	770	802	233,373	8,643	17.41	14.14	562	689	943	1,198
66	54	1,131	1,178	514,027	15,577	21.32	17.28	826	1,013	1,386	1,759

a. Form dimensions may vary with producers with corresponding variations in section properties.

b. Allowable loads based on $N = A_g(0.33f'_c - 0.27f_{pc})$; f_{pc} = 700 psi. Check local producer for available concrete strengths.

16.4.5 Additional Information

The PCI Committee on Prestressed Concrete Piling has published a "Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling" (1993). This report discusses design of prestressed concrete piling, materials, manufacturing, handling, transportation and installation.

Other references on piles include: Falconer and Park, 1983; Sheppard, 1983; Lincoln, 1988; Joen and Park, 1990A & 1990B; Shahawy and Issa, 1992; Kamel, et al, 1996; and Nigels, 1998.

16.5 PILE BENT CAPS

The pile bent is a common form of bridge substructure, particularly for stream crossings or long trestles over larger bodies of water. The pile bent consists of a cap supported by piles driven in the soil. The bent cap is usually a rectangular, cast-in-place, mildly reinforced concrete beam. Some designers have taken advantage of the positive characteristics of precast concrete by using precast bent caps. Precast bent caps have been used in combination with steel H-piles, pipe piles, prestressed concrete piles (including cylinder piles), and even timber piles.

16.5.1 Advantages

There are several advantages to using precast bent caps. The quality of the concrete is superior to field-placed concrete and problems associated with hauling ready-mixed concrete to the site are substantially reduced. Erection of forms, placement of reinforcing steel, and placement and curing of the concrete are replaced with simple erection of the precast cap onto the piles and the connection to the piles. Reduced field labor translates into a substantial reduction in construction time. This was the case for the Sandpoint Bridge (1984), Bonner County, ID, where 177 precast caps were used after the contractor started with, but later rejected, a cast-in-place alternate.

16.5.2 Pile-to-Cap Connections

There are several ways to connect precast bent caps to piles. If steel piles are used, steel plates can be cast into the bottom of the precast bent cap and welded to the tops of the piles. Even though this is an effective connection, it requires field welding in a difficult location. A better method of connecting piles to precast caps involves forming a simple void in the bottom of the cap that fits over and around the piles. The void is then filled with concrete or grout to make the connection. This method has been used successfully with all types of piles. The Bayside Bridge in Pinellas County, FL, used precast bent caps supported on prestressed concrete piles for the low-level approach spans.

For those situations when a moment connection is required between the pile and cap, several methods can be used. For steel piles, studs or reinforcing steel can be welded to the pile and anchored into the grout. For solid concrete piles, reinforcing steel can be anchored into the tops of the piles with epoxy grout and extended into the void.

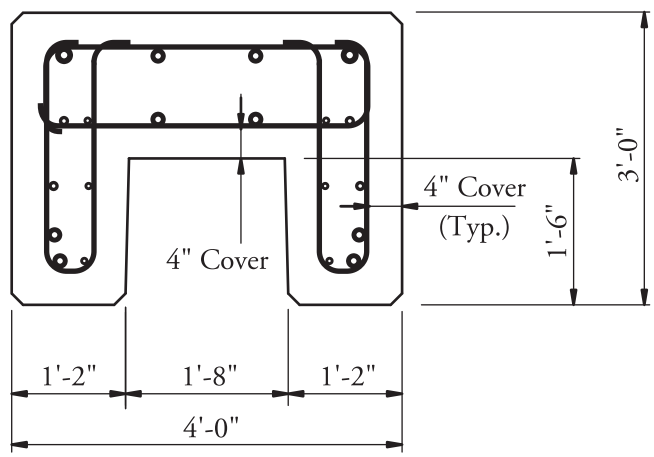
The final installed location of a pile is seldom the exact location called for on the plans, so provisions must be made for pile driving tolerances.

The void in the bent cap into which the pile will fit must be large enough to accommodate the pile placement tolerance and, conservatively, a little more. Adequate clearance must be maintained between the pile and the inside face of the void. The space between the pile and the face of the void must be large enough to adequately distribute the grout.

16.5.3 Size and Weight Limitations

Designers must be aware of limitations related to length and weight of products. These limitations will vary from project to project. The weight per unit length is a function of width. The width, in turn, is typically a function of the size of the piles supporting the cap. The overall weight of the cap must be kept within limits established by the project scope and location. Shipping and erection of a cap at a small stream crossing will probably necessitate a smaller, lighter section than at a large water crossing where shipping and erection can take place with barges. As a rule, smaller projects where product will be shipped overland should be limited to a maximum weight of 50 tons and sometimes, even 40 tons. Projects where larger cranes will be available and shipping weights are less restrictive may have a limiting weight of 70 to 80 tons. One means of reducing the weight of the cap is to fabricate and ship caps in several pieces. Connections can be made by means of a field-cast section, by welding or by post-tensioning the pieces or segments together. By adjusting pile spacing to provide reasonable cap cantilevers in the interior portion of the bridge, the need to make a connection between separate cap segments at a given bent may be eliminated. Another method to reduce weight is to create a void in the cap between the piles. This can be accomplished by using an inverted-U shape, or by casting a hollow void-former in the interior of the cap. **Figure 16.5.4-1**

Figure 16.5.4-1
Typical Inverted-U Pile Bent Cap



16.5.4 Handling

A bent cap is typically considered a continuous beam supported by the piles. Therefore, the span length for a bent cap in its service condition may be relatively short compared to its overall length. However, prior to placement on the piles, the span length is the distance between the lifting locations during handling or the support points during storage and shipping. The cap may be subjected to greater moments and shears during handling and shipping than when in service. These forces must be considered when designing the reinforcement in the cap. Lifting and storage locations should be clearly shown on the shop drawings.

16.6 PIERS

The use of precast concrete used for bridge piers continues to increase steadily. A few recent projects may be used to illustrate this trend.

- The Edison Bridge in Fort Myers, FL, was cited in Chapter 4 (Fig. 4.5.3.2-1). This large water crossing consists of two bridges, one supporting two lanes of traffic plus shoulders and the other supporting two lanes of traffic plus a sidewalk and shoulders. The maximum clearance over the water is 55 ft. The shorter piers were erected in a single day. It took only a maximum of three days each to erect the tallest piers.

- The four precast concrete bridges of the Baldorioty de Castro Avenue in San Juan, Puerto Rico, range in length from 700 to 900 ft (Endicott, 1993). All piers and caps are precast concrete. The four bridges were each constructed and opened to traffic in less than 36 continuous hours from the start of precast erection.
- The Louetta Road Overpass in Houston, TX, (Ralls, et al, 1993) uses precast piers. Each of the superstructure's spread trapezoidal U-beams is supported by an individual precast pier column. These cantilevered columns consist of hollow, multi-sided, match-cast segments. The segments were plant-cast and transported to the site where they were erected and post-tensioned together. See **Figures 16.6-1** and **16.6-2**.

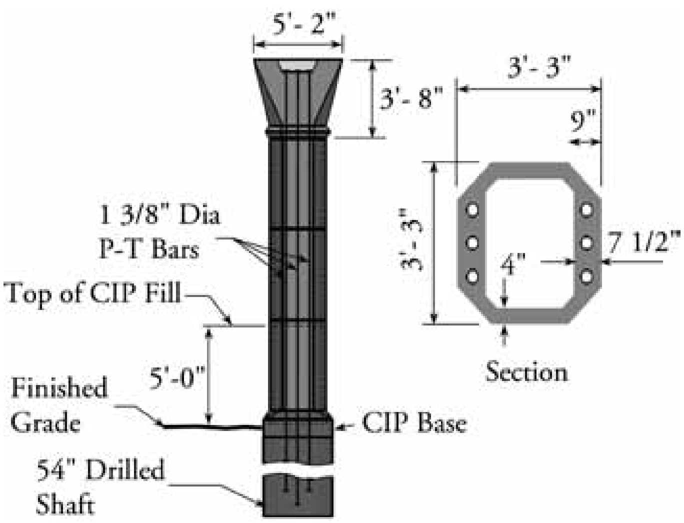


Figure 16.6-1
Louetta Road Overpass Precast Segmental Pier Column

Figure 16.6-2
Columns and U-Beams at Louetta Road Overpass

- The Chesapeake and Delaware Canal Bridge, SR-1, DE, uses 463 box pier segments for 48 columns ranging from 50- to 130-ft tall. The typical box segment measures 18 ft by 8 ft and varies from 4- to 10-ft tall. The segments were stacked and post-tensioned vertically. Joints were sealed with epoxy grout. **Figure 16.6-3** shows a typical pier segment being erected. **Figure 16.6-4** shows several finished piers supporting the cable-stayed bridge under construction. Typically, 100 ft of box pier column was constructed in a day (Pate, 1995).

Figure 16.6-3
Erection of Pier Column Segment

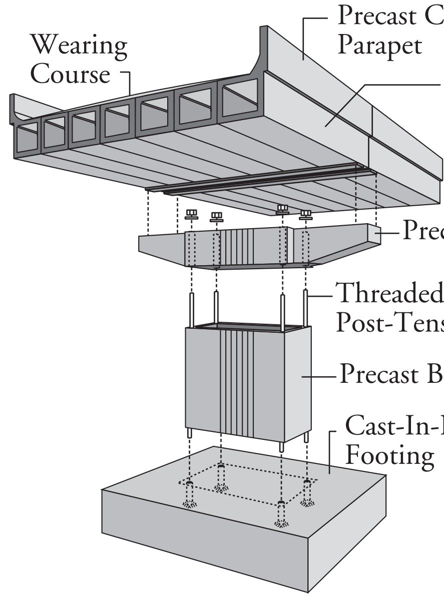
Figure 16.6-4
Finished Pier Columns, C & D Canal Bridge

16.6.1 Advantages

The use of precast concrete bridge piers offers unique opportunities (Billington, et al, 1999):

- Reduced construction time
- Reduced inconvenience to the public during construction
- Lower cost
- Plant-cast quality concrete and enhanced architectural surface treatments.

Figure 16.6.1-1 Pier Schematic, Baldorioty de Castro Avenue Bridges



Significant savings in construction time can be achieved by fabricating precast substructures off site while the foundations are installed. As discussed in Section 16.6, the Baldorioty de Castro Avenue bridges were each constructed exceptionally fast – in just a matter of hours. This unique systems approach to bridge construction is depicted in **Figure 16.6.1-1**. For long water crossings, reduced labor represents not only a direct reduction in the salary paid to the construction workers, but also reduced costs associated with workman's compensation taxes, insurance and exposure. The use of precast caps represents a maximum reduction in the contractor's construction effort because it eliminates the need to erect and strip cap forms. In addition to the time and cost savings, the quality of the concrete is nearly always better than when cast in the field.

16.6.2 Pier Caps

Caps for single and multicolumn piers can be precast in a variety of shapes. For typical grade crossings, simple rectangular sections can be fabricated, shipped to the site and erected on the columns. Inverted-T sections can also be easily fabricated. These simple shapes do not require special forming systems. The reinforcement required is essentially the same as for cast-in-place concrete caps.

For larger caps, such as those used on the Edison Bridge, an inverted-U shape may be more desirable. The inverted-U shape reduces the volume of concrete and therefore the weight for shipping and handling. However, the forming system for U shapes is more complex, as is the reinforcement.

Hammerhead piers can also use precast caps. They consist of a single column and a cap with equal or near equal cantilever lengths. However, these caps are normally fairly large, and special considerations are usually necessary to reduce the weight of these components.

One method to reduce the weight of the cap is to produce the cap in sections and make appropriate connections in the field. Since this can sometimes be complicated, another method is to use a precast concrete shell as a form for the cap. The shell is designed to

Figure 16.6.2-1
Erection of Precast Cap over Column Dowels

support its own weight plus that of the CIP concrete that fills it. While not completely a precast system, this process eliminates the need to erect and later strip forms for the cap. Relatively large caps can be constructed in this manner while achieving manageable weights. For hammerhead piers with smaller caps, a solid cap can be fabricated in the plant, shipped to the site and erected with routine effort. The caps for the piers supporting the guideway for the J. Paul Getty Museum Tram near Los Angeles, CA, were constructed in this manner as shown in **Figure 16.6.2-1** (Josten, et al, 1995).

16.6.3 Columns

Using precast columns in a multicolumn pier usually does not offer the same level of timesavings to the contractor as do precast pier caps, particularly for grade crossings. However, use of precast columns does provide the advantage of high performance concrete associated with precast products. Another advantage of precast concrete columns is the manufacturer's ability to produce uniform architectural finishes. Precast concrete fabricators can typically incorporate form liners into their forming systems more easily and with more uniform results than can be achieved with cast-in-place concrete. The most economical precast column shapes for multicolumn grade crossings are square, rectangular and octagonal.

Precast columns for multicolumn piers at water crossings generally provide a greater potential for time and cost savings. The heavier weights typical of the longer columns required for navigable water crossings are usually not prohibitive because pieces can be shipped by barge and because larger cranes are generally available on marine sites. In addition to the shapes previously discussed, I-shaped columns such as those used for the Edison Bridge can be used to reduce concrete volume and column weight (see Section 4.5.3.2).

Hammerhead piers provide another opportunity for the use of precast columns. These columns are usually fairly massive, and the weight of each component can present a special challenge. Solid rectangular or oval shapes can easily be fabricated, but will be very heavy. By using hollow sections, weights can be reduced significantly. If a solid section is necessary to resist horizontal forces, such as ship impact, the hollow precast column can serve as a stay-in-place form for field-placed concrete. Both solid and hollow columns can be cast in shorter segments and joined in the field. The large flat surfaces of this type of column provide an excellent location for architectural finishes including striations, rustications, reveals and ribs.

The use of precast columns for hammerhead piers presents an opportunity to significantly reduce the volume of field-cast concrete. This may be especially beneficial when existing interchanges are rebuilt. At these locations, the motoring public will be better served by the reduced time of construction. Over water, precast columns for hammerhead piers provide the same time and monetary incentives as the other previously discussed precast pier systems.

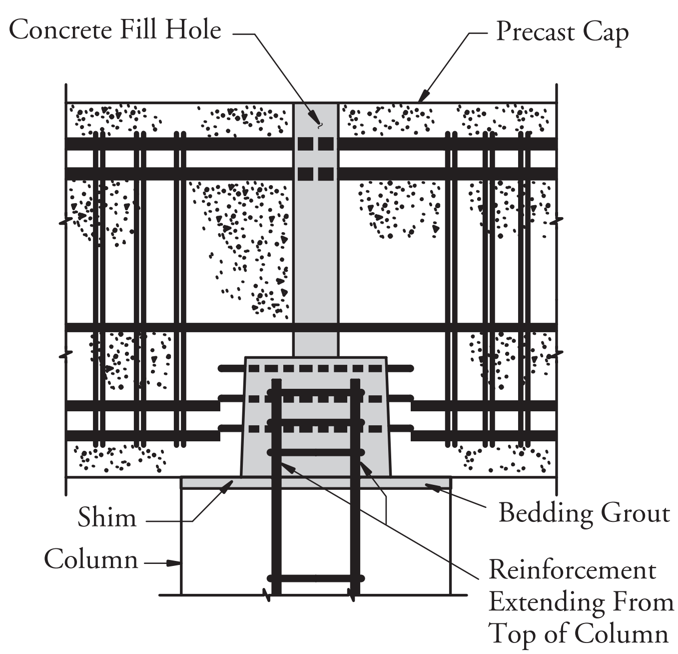
16.6.4 Connections

Connection design, construction details and installation are all important issues. Several methods to achieve pinned or rigid connections can be effectively applied to bridge substructure components. Simplicity and reasonable construction tolerances are primary concerns of both the contractor and precast concrete fabricator.

16.6.4.1 Multicolumn Piers

Most multicolumn piers are designed as rigid frames, with moment transfer between the cap and the columns. However, at typical grade crossings, especially single-level crossings, it is not always necessary to provide a full moment connection between the cap and the columns in multicolumn systems. Often, the design loads can be resisted by the columns acting as cantilevers fixed at the foundations. In these instances, a very simple method can be used to provide a pinned connection at the top. This connection uses a small cage of reinforcing steel that projects from the column into a pocket or void in the bottom of the cap (**Figure 16.6.4.1-1**). This steel can easily transfer the very small moment necessary to keep the cap from rolling off the column. The cap is designed as a continuous beam, simply supported at the columns. The columns are designed as cantilevers about both axes. Although the moment at the bottom of the column is larger than for a comparable rigid frame, the use of minimum steel in the column usually provides the required capacity. In most instances, connections from a precast column to a footing must be capable of transferring moment. This can most easily be accomplished with the use of grouted bar couplers that are described below. Although post-tensioning may be used, the need to anchor the strand or bar in the foundation can present a challenge. Post-tensioning will likely not be cost effective except on very large structures.

Figure 16.6.4.1-1 Precast Cap-to-Column Pinned Connection

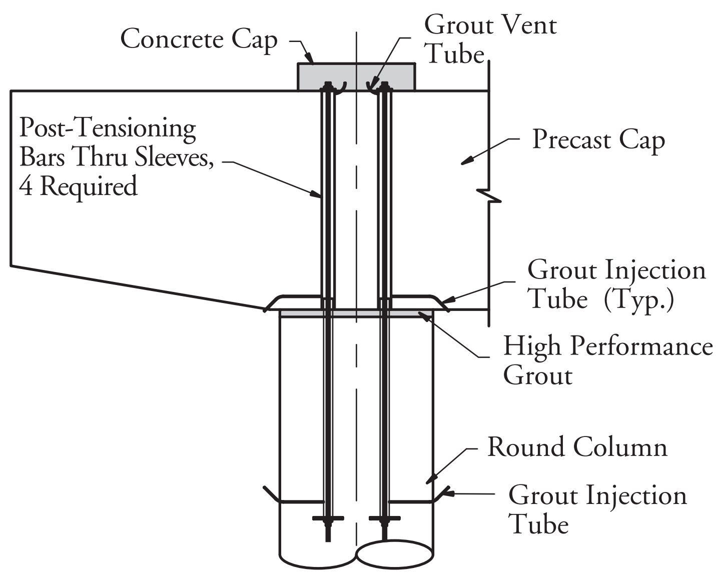


For piers designed as rigid frames, there are two basic methods to make full moment-resisting connections between the columns and pier caps.

16.6.4.1.1 Post-Tensioning

Post-tensioning the cap vertically to the tops of the columns provides the necessary mechanism to transfer moments. This system has been used effectively in a contractor-proposed alternate for a series of two-column piers with solid rectangular precast caps at the SR 10 over Western Flood Plain (Victory Bridge) in Jackson and Gadsden

Figure 16.6.4.1.1-1
Precast Pier Cap-to-Column Connection, Victory Bridge

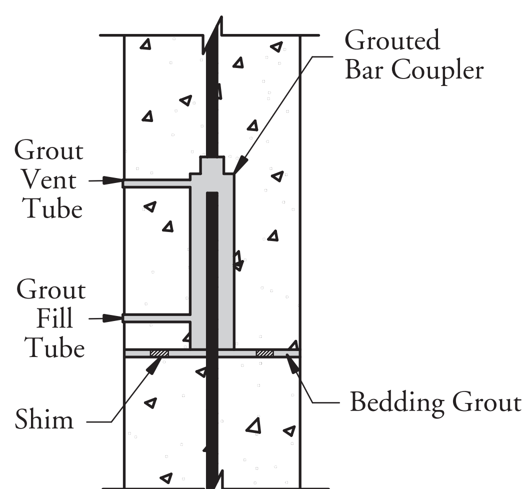


Counties, FL. Post-tensioning bars were anchored in the tops of the CIP columns. The cap was then lowered into place, the bars tensioned, and the ducts grouted (**Figure 16.6.4.1.1-1**).

16.6.4.1.2
Spliced Reinforcement

Another means of providing a moment connection is the use of mechanical splices for reinforcing steel known as splicing sleeves or grouted bar couplers. These devices emulate continuous reinforcement across joints between concrete components. This scheme was used effectively on the Edison Bridge in Florida. The connection used a grouted bar coupler cast into the precast component for each main reinforcing bar. A dowel protruding from the footing or adjoining precast component is inserted into each coupler. When the precast unit is set, the coupler is filled with grout to provide a full-strength splice of the reinforcing steel. See **Figure 16.6.4.1.2-1**. The use of a grouted coupler requires coordination and close tolerances to ensure that the dowels and couplers are in the proper location and orientation. This is easily accomplished with the use of jigs and templates both in the field and in the precast plant. All of the connections for the Edison Bridge were successfully installed.

Figure 16.6.4.1.2-1
Typical Grouted Coupler Detail



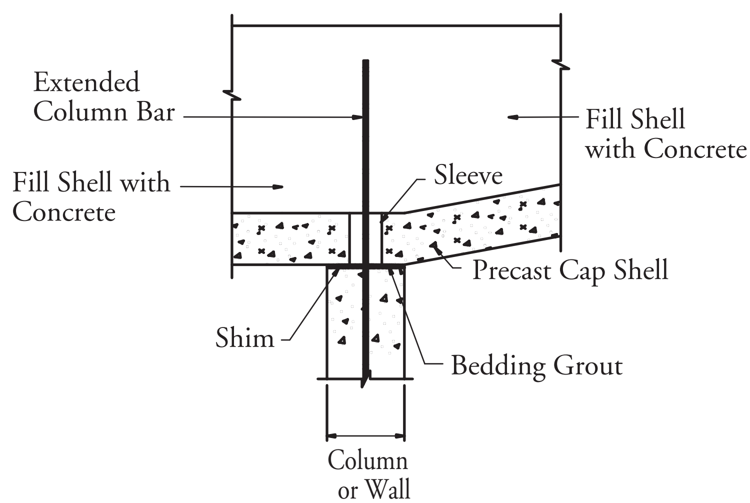
16.6.4.2 Hammerhead Piers

The connection of cap to column in a hammerhead pier must be capable of transferring moments in two directions. As with multicolumn piers, a moment connection can be made using post-tensioning or grouted bar couplers. Since the moments at the column-cap interface are substantially less than at the bottom of the column, the full amount of column steel is not required to secure the cap to the column. For post-tensioned connections, it may be possible to reduce the amount of post-tensioning at the top of the column. When bar couplers are used, a reduction in the steel will reduce the number of couplers required.

If the precast cap shell described in Section 16.6.2 is used, there is a simple means of connecting the column to the cap that does not require post-tensioning or grouted couplers. In this method, a percentage of longitudinal column reinforcing bars is extended through sleeves in the bottom of the shell. These bars are developed when field-placed concrete in the shell cures, thereby providing moment capacity. Refer to **Figure 16.6.4.2-1**.

As previously described, precast columns for hammerhead piers must often be erected in more than one piece. Connections between these segments, as well as the connection to the foundation, must be capable of transferring the full column forces and moments. Post-tensioning can be an effective means of providing these connections, particularly for larger columns. Grouted bar couplers can also be used. With couplers, it is usually more cost effective to use fewer, larger reinforcing bars (#11 or #14) to reduce both the costs of the couplers and the potential for misaligned connections in the field.

Figure 16.6.4.2-1 Precast Hammerhead Cap-to-Column Connection



16.7 ABUTMENTS

Abutments are the end supports of a bridge. There are many variations of abutments in use such as bench or spill-through, stub, deep, etc. A bench abutment is a “resting pad” for the superstructure components with a vertical back wall that retains the soil and supports the approach slab. The abutment may be supported on a spread footing or on piles. A spill-through abutment usually refers to a pile bent where the fill behind the bent slopes down from the bottom of the cap to the channel or roadway shoulder below. The precast version of this system is essentially a precast pile bent cap. Stub abutments and deep abutments refer to those end supports that include a load-bearing wall from the superstructure support to the ground beneath the bridge. The basic difference between stub and deep abutments is the height of the wall.

16.7.1 Applications

Precast abutments can be used effectively almost anywhere that a CIP abutment is used. A precast cap can be used for the spill-through type of abutment as easily as for a typical pile bent. In states where the back wall is made part of the substructure, the back wall could be cast in the plant with the cap, or it could be cast in the field. Stub and deep abutments are essentially walls, for which precast concrete is very effective. The walls can be cast and shipped to the project horizontally, then erected into their vertical position on a foundation, which may be either precast or cast-in-place.

16.7.2 Advantages

In addition to the advantages involving the quality of precast concrete components, the speed of construction is very important. For stub and deep abutments, the use of precast elements eliminates costly and time-consuming forming and curing of walls at the project site. The reduced volume of field-placed concrete may be especially advantageous in remote locations, where there are long distances to ready-mix plants or where concrete delivery is otherwise difficult.

16.7.3 Connections

For spill-through abutments, connection options are the same as those outlined previously for precast pile bents. For wall-type abutments, the connections will be similar to those used for precast wall systems for building structures. Grouted reinforcing bar couplers are an excellent method to make a moment connection at the base of an abutment. A moment-resisting connection between the abutment and the ends of the superstructure can also be achieved to create an integral abutment. This eliminates costly, high maintenance expansion joints. See **Figures 16.7.3-1** and **16.7.3-2**. Other mechanical bar splicing systems require patching of the concrete where the splice is made and may be less desirable because the area will be exposed to earth backfill. Although post-tensioning is an option for stub abutments, it is generally cost effective only for taller systems.

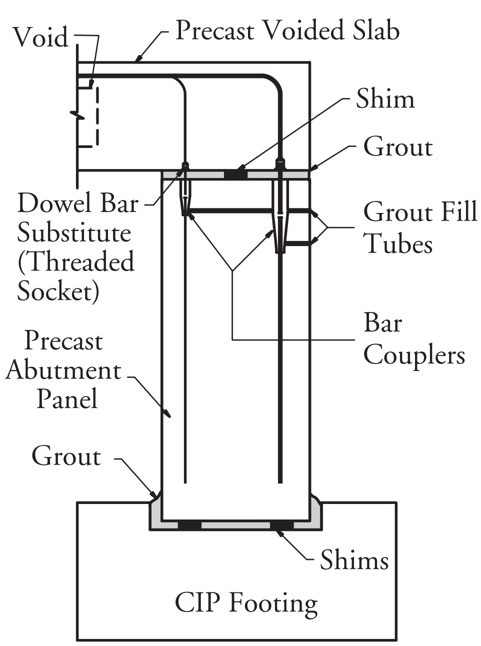


Figure 16.7.3-1
Precast Integral Abutment, Route 9N over Sucker Creek, Hague, NY

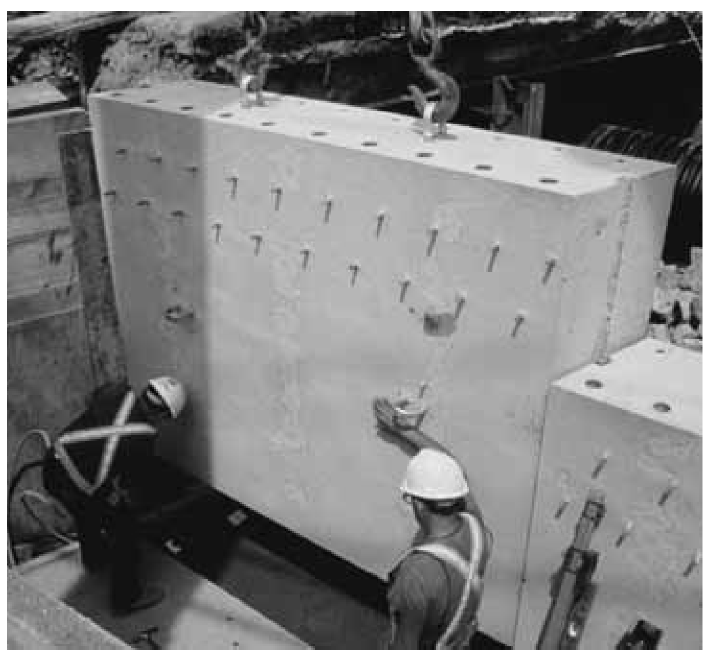


Figure 16.7.3-2
Erection of Abutment Wall , Route 9N over Sucker Creek

16.8 EARTH RETAINING SYSTEMS

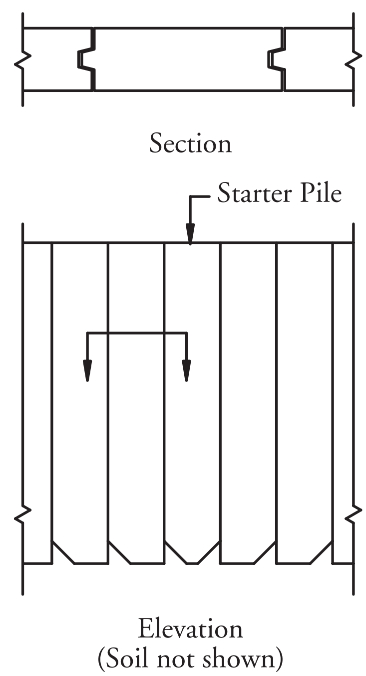
The use of precast concrete components has become popular for earth retaining systems. Concrete sheet piles, gravity retaining walls and mechanically-stabilized earth walls have been successfully and economically installed with an excellent service record.

16.8.1 Sheet Piles

Precast concrete sheet piles, both prestressed and non-prestressed, have been in use for several decades. Applications of sheet piles include both cantilever and anchored bulkheads. Corrosive environments, such as marine installations, exploit the advantage of prestressing to the fullest, since the uncracked section substantially reduces the potential for chloride penetration. The durability of precast concrete in corrosive environments has been proven repeatedly.

Sheet piles are usually fabricated with a concentric prestress force, or a reinforcing pattern symmetric about the center of the sheet. Pile widths vary, but the most economical installations use a single sheet width. Typical pile widths range from 2 ft-6 in. to 5 ft-0 in. Where changes in the alignment of the wall occur, non-prestressed corner pieces are often used. A simple tongue and groove connection between sheet piles assists in aligning the panels during installation and distributes horizontal earth loads between the sheets. It is advantageous to place the tongue of the panel on the leading edge of the sheet, so that the groove of the subsequently placed sheet pile is free of soil. Concrete sheet piles are most easily installed by jetting into place with a water stream. As shown in **Figure 16.8.1-1**, the beveled portion at the leading edge of the sheet pile helps to snug the sheet being installed to the previously placed panel.

Figure 16.8.1-1
Typical Layout, Precast Concrete Sheet Piles



16.8.2 Gravity Retaining Walls

Several precast concrete retaining wall systems have been developed that depend on the weight of the soil being retained to resist overturning. These gravity retaining walls are similar to a traditional CIP wall and footing system. These systems must resist both the horizontal earth pressure as well as an overturning moment. Precast counterfort walls have also been successfully employed.

16.8.3 Mechanically-Stabilized Earth Walls

Using the weight of the soil to resist the horizontal earth pressure, mechanically-stabilized earth (MSE) walls have proven to be very economical systems, especially for large wall areas. MSE walls usually consist of interlocking precast concrete panels that are stacked. A leveling pad of field-placed concrete is used to insure the proper elevation at the bottom of the wall. Horizontal straps connected to the precast concrete panels resist the horizontal earth pressure. The straps extend into the earth fill behind the wall. Strap lengths are determined by the supplier, and are governed by the height of the wall, surcharge loadings and the soil properties of the backfill. Most MSE wall systems are proprietary.

16.9 RAILINGS

Precast bridge railings, also known as parapets or barriers, have been used very successfully. The reduced time used to install the precast product increases the cost effectiveness of the system. The concrete is more durable and can be made more attractive than most CIP systems.

16.9.1 Connections

There are two typical connections used between precast railings and bridge decks. In one system, threaded reinforcing bars project from the bottom of the railing through the deck and are secured with washers and bolts. This system requires careful coordination between the contractor and the precast railing producer to ensure proper location of the holes in the deck. A second type of connection requiring additional field labor uses shorter dowels projecting from the bottom of the railing. These dowels are set with adhesive anchors into holes drilled in the top of the deck. Crash testing of either of these connection systems may be required.

16.10 CULVERTS

When small stream crossings require greater hydraulic capacity than can be provided by circular conduits, concrete culverts provide an excellent alternative to small bridge structures. Precast concrete culverts provide an option that includes all the advantages associated with quality concrete and speed of construction. Precast culverts are especially attractive if stream flow must be maintained during construction. Precast concrete culvert installations include boxes and three-sided systems, placed with single cells or multiple cells to gain greater hydraulic capacity.

16.10.1 Sizes

Precast culverts range in size from small boxes to three-sided systems that span more than 60 feet. As with most precast components, the maximum size is limited by weight and vertical and horizontal clearances when being transported. By placing multiple cells side by side, the hydraulic capacity is virtually limitless. Precast systems can be used anywhere CIP box culverts are feasible and as an alternative to a low-level bridge.

16.10.2 Standards for Design

In the *Standard Specifications*, loadings on culverts and requirements for footings are contained in Section 6. Article. 17.7, contains design requirements for precast reinforced box culverts. Article 17.8 provides design requirements for precast, reinforced concrete three-sided structures.

Concrete design in the *LRFD Specifications* is contained in Section 5. For culverts that are under two or more feet of fill, special considerations for the shear design of culvert slabs are presented in Article 5.15.5.1. The soil-structure aspects of culvert design are contained in Section 12.

AASHTO and ASTM Standards contain designs for box culverts with heights and spans to 12 ft. The standard designs are based on specific concrete strengths and use welded wire reinforcement with small concrete cover. Some agencies have referred to these standard sections for use on projects, but specify increased cover for the reinforcing steel. These standards also address the fabrication and installation of precast box culverts.

AASHTO M273 and ASTM C850 address the use of precast box culverts with less than two feet of earth between the driving surface and the top of the culvert. AASHTO M259 and ASTM C789 govern where the earth cover is greater than two feet. The ASTM Standard C1433 covers both conditions.

16.10.3 Three-Sided Culverts

When the span of a box culvert becomes large, the use of a three-sided culvert is an attractive alternative. These systems consist of a top slab and vertical walls with an open bottom. Three-sided culverts are generally supported on spread footings, which can be either precast or CIP. Infrequently, this system has been installed on pile-supported foundations. Some three-sided systems rely on the horizontal earth pressures to help support the vertical load on the structure. Other systems are rigid frames, supporting loads with only the structure. These systems are generally proprietary and are manufactured regionally by precasters throughout the country. All have standard designs for various applications and loadings.

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RAILROAD BRIDGES

A = area of cross-section of precast beam
A_c = total transformed area of composite section
A_{ps} = area of one pretensioning strand or post-tensioning bar
A_{ps} = area of strands in the tensile zone
A_v = area of shear reinforcement
A_{vh} = area of web reinforcement per unit length required for horizontal shear
a = compression block depth
B = buoyancy
b = width of compression face of member
b_e = top flange width of precast beam
b_v = width of cross section at the contact surface being investigated for horizontal shear
b_w = width of web of a flanged member
CF = centrifugal force
D = dead load
DF = live load distribution factor
d = distance from extreme compressive fiber to centroid of the prestressing force
E = earth pressure
E_c = modulus of elasticity of concrete at 28 days
E_{ci} = modulus of elasticity of concrete at transfer
E_s = modulus of elasticity of pretensioning steel
E_s = modulus of elasticity of non-pretensioned reinforcement
EQ = earthquake (seismic)
e = eccentricity of strands at transfer length
e_c = eccentricity of strands at midspan
F = longitudinal force due to friction or shear resistance at expansion bearings
f_b = concrete stress at the bottom fiber of the beam
f'_c = specified concrete strength at 28 days
f_{cds} = concrete stress at the centroid of the pretensioning steel due to all dead loads except the dead load present at the time the pretensioning force is applied
f'_{ci} = specified concrete strength at transfer
f_{cr} = stress in the concrete at the centroid of the pretensioning steel
f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads
$f_{\ell c}$ = loss of prestress due to creep of concrete
$f_{\ell e}$ = loss of prestress due to elastic shortening
$f_{\ell r}$ = loss of prestress due to relaxation of pretensioning steel
$f_{\ell s}$ = loss of prestress due to concrete shrinkage
f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at the centroid of cross section resisting externally applied loads
f_{pe} = compressive stress in concrete due to effective pretensioning forces only (after allowance for all pretension losses) at the extreme fiber of section where tensile stress is caused by externally applied loads
f_{ps} = stress in the pretensioning steel at nominal strength

f_{pu} = ultimate tensile strength of pretensioning steel
f_{se} = effective stress in pretensioning steel after losses
f_t = concrete stress at the top fiber of the precast beam
f_{tc} = concrete stress at top of fiber of the slab for the composite section
f_y = specified yield strength of non-prestressed reinforcement
h = overall depth of precast beam
h_c = total height of composite section
I = moment of inertia about the centroid of the non-composite precast beam
I = the percentage of the live load for impact
ICE = ice pressure
I_c = moment of inertia of composite section
L = span length
L = live load
LF = longitudinal force from live load
M = maximum service load design moment
M_{cr} = moment causing flexural cracking at section due to externally applied loads
M_D = unfactored bending moment due to total dead load
M_d = unfactored bending moment due to composite beam dead load
M_g = unfactored bending moment due to precast beam self-weight
M_{LL+I} = unfactored bending moment due to live load + impact
M_{max} = maximum factored moment at the section due to externally applied loads
M_n = nominal moment strength of a section
M_{SDL} = unfactored bending moment due to superimposed dead load
M_u = factored bending moment at the section
M_x = bending moment at a distance x from the support
n = modular ratio of elasticity between slab and beam materials
OF = other forces (rib shortening, shrinkage, temperature and/or settlement of supports)
P_{eff} = effective post-tensioning force
P_{se} = effective pretension force after allowing for all losses
P_{si} = total pretensioning force immediately after transfer
R = relative humidity
S_b = non-composite section modulus of the extreme bottom fiber of the precast beam
S_{bc} = composite section modulus for extreme bottom fiber of the precast beam
S_t = non-composite section modulus of the extreme top fiber of the precast beam
S_{tc} = composite section modulus for extreme top fiber of the slab
S_{tg} = composite section modulus for top fiber of the precast beam
SF = stream flow
s = spacing of the shear reinforcement in direction parallel to the longitudinal reinforcement
V = service load shear force
V_c = nominal shear strength provided by concrete

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

V_D = unfactored shear force at section due to total dead load

V_d = unfactored shear force due to composite beam dead load

V_g = unfactored shear force due to precast beam self-weight

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

V_{LL+I} = unfactored shear force at section due to live load plus impact

V_p = component of pretensioning force in the direction of the applied shear

V_s = nominal shear strength provided by shear reinforcement

V_{SDL} = unfactored shear force due to superimposed dead loads

V_u = factored shear force at the section

V_x = shear force at a distance x from the support

v_{dh} = horizontal shear stress

W = wind load on structure

WL = wind load on live load

w = weight per foot

w_c = unit weight of concrete

w_{equ} = equivalent uniform load

x = distance from the support

y_b = distance from centroid to extreme bottom fiber of the non-composite precast beam

y_{bc} = distance from the centroid of the composite section to extreme bottom fiber of the precast beam

y_{bs} = distance from the center of gravity of strands to the bottom fiber of the beam

y_t = distance from centroid to extreme top fiber of the non-composite precast beam

y_{tc} = distance from the centroid of the composite section to the extreme top fiber of the slab

y_{tg} = distance from the centroid of the composite section to extreme top fiber of the precast beam

Δ = deflection

ϕ = strength reduction factor

ρ_p = ratio of pretensioning reinforcement

ψ = angle of harped pretensioned reinforcement

Railroad Bridges

17.0 INTRODUCTION

Precast concrete is playing an increasingly important role in railroad bridge structures. The economy, durability and speed of construction make precast concrete the material of choice for new and replacement railroad bridges. The focus of this chapter is on the specific requirements and guidelines for railroad bridges. Typical products and details, construction considerations, and identification of applicable AREMA (American Railway Engineering and Maintenance-of-Way Association, formerly AREA) provisions are also discussed. Two case studies and a railroad superstructure design example are presented.

17.1 TYPICAL PRODUCTS AND DETAILS

A wide variety of precast products is used for railroad bridge construction. From the ground up, these include concrete piles, pile caps, abutments and superstructure beams. Over the years, many railroads have developed standards for precast concrete, including concrete mixes, member design, member detailing and quality control.

17.1.1 Piles

Several railroads use precast, prestressed concrete piles, but their use may be limited by the capacity of track-mounted pile drivers. Concrete piles are preferred for use in marine environments. In highly corrosive locations, precast concrete pile extensions are spliced to steel pipe piles. This permits the embedment of the steel into the anaerobic soil zone and provides a more durable prestressed concrete pile in the more corrosive environment.

17.1.2 Pile Caps and Abutments

Precast concrete pile caps are widely used throughout the country. Typically, these are fabricated with an embedded plate running along the bottom of the cap. This allows welding of steel piles to the bottom of the cap. Concrete pile caps are sometimes used to support steel or timber beams, as well as concrete beams. Some railroads are now beginning to use precast concrete caps with precast concrete piles. The caps are cast with a socket for the pile to fit into. Grouting is used to tie the components together after installation. Bridge abutments can also be prefabricated. The bases of these abutments are similar to the pile caps and serve the same function of supporting the superstructure. Abutment backwalls and wingwalls can be precast in sections and bolted or welded together in the field.

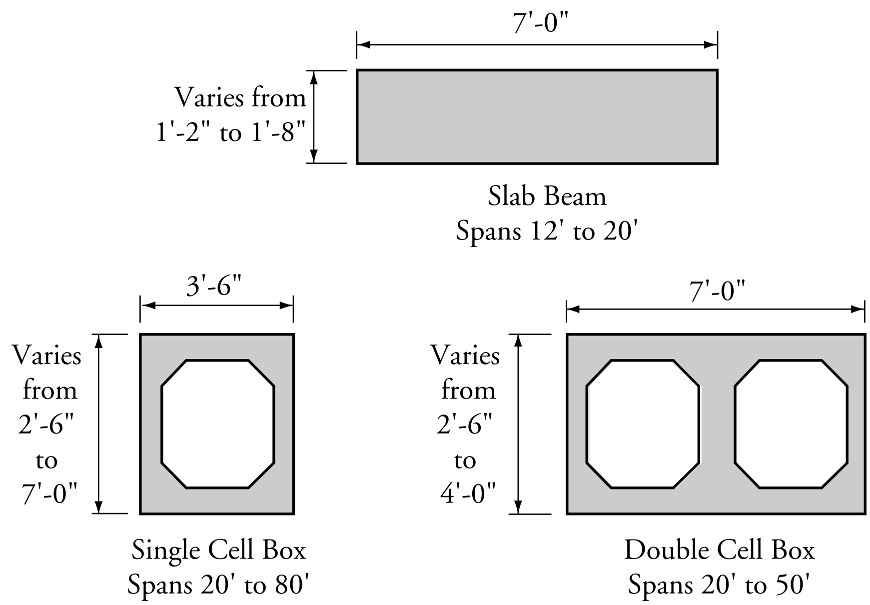
17.1.3 Superstructures

Railroads use a wide variety of superstructure elements. Spans typically range from 12 ft to over 80 ft. Since many precast concrete spans are installed to replace timber trestles, standard span lengths for a given railroad are frequently multiples of their standard timber stringer span lengths (typically 14 to 16 ft). For spans of 12 ft to 20 ft, precast slab beams are frequently used. For spans in the 20- to 30-ft range, precast, prestressed concrete box beams are the most common although tee-beams and I-beams are occasionally used. For spans over 30 ft, box beams are dominant. Spans up to 50 ft typically use two box beams per track. Generally, these are double celled with through-voids. Through-voids allow fabricators to use removable and reusable void forms in casting the beams. This helps reduce costs. Spans over 50 ft generally use four single-void box beams per track. The shift from two beams per track on shorter spans to four on longer spans is dictated by the lifting restrictions associated with the heavier weight of the longer beams. Shear keys and transverse post-tensioned steel tie rods are frequently used to tie the box beams together with diaphragms provided at the location of the tie rods. For spans greater than 70 to 80 ft, beams with composite cast-in-place concrete decks are frequently used.

17.1.3.1
Slab Beams and Box Beams

A variety of shapes with depth and width variations are available throughout the country. Designers should contact the manufacturers and the specific railroad to determine the properties and dimensions of products available for a proposed project. Typical superstructure shapes and span ranges applicable to railroad bridges are shown in **Figure 17.1.3.1-1**.

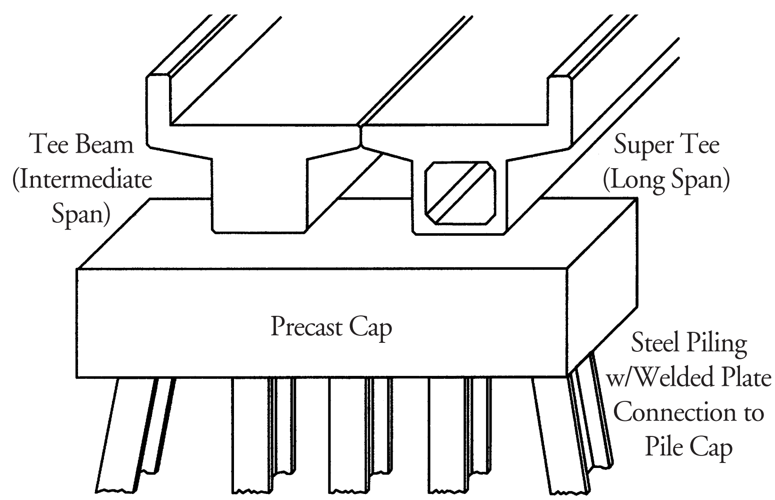
Figure 17.1.3.1-1
Typical Precast Concrete Superstructure Shapes



17.1.3.2
Other Products

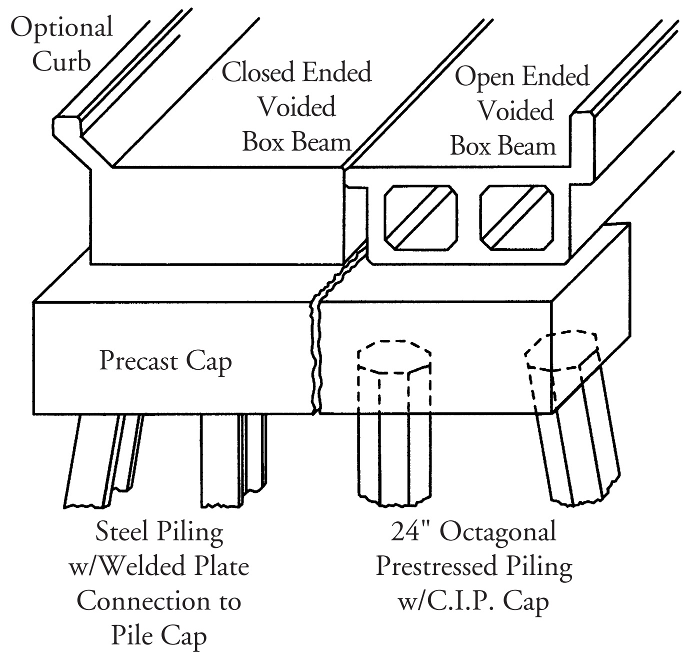
There are a few other precast products used for different span ranges. Brief descriptions of these products are given in **Figures 17.1.3.2-1 through 17.1.3.2-4**.

Figure 17.1.3.2-1
Tee Beam
(Intermediate and Long Spans)



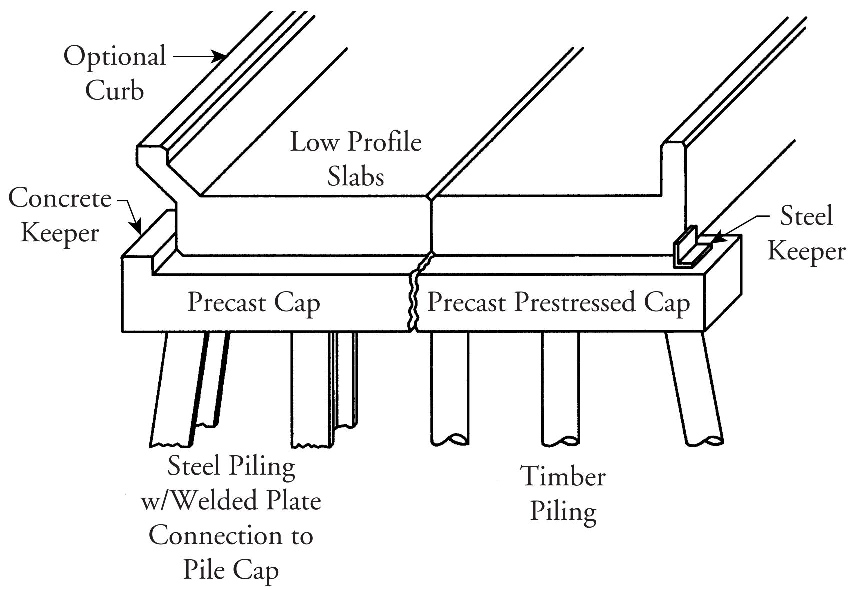
The solid single tee beam is used for spans of 20 to 34 ft, and the voided super tee for spans up to 55 ft in length. Both beams are set on a precast concrete cap that has a welded plate connection to the piles as needed.

Figure 17.1.3.2-2
Box Beam
(Intermediate Spans)



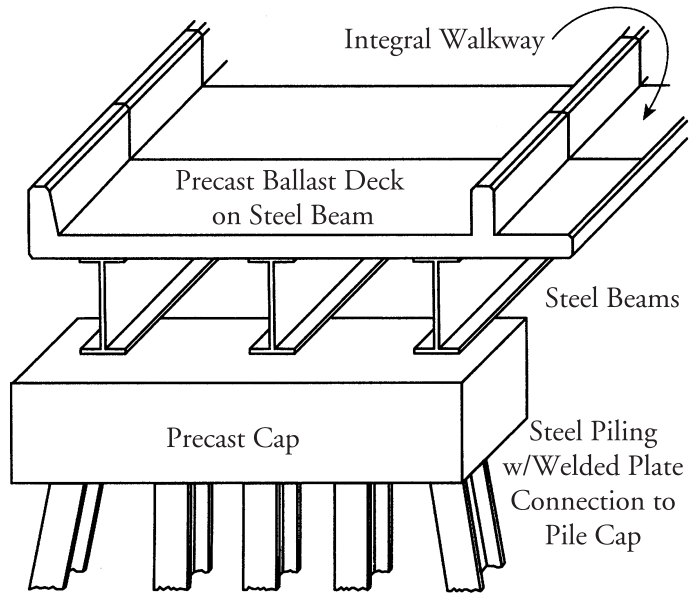
Voided box beams are used on 20- to 50-ft long spans, with optional diaphragms and curbs. Boxes may be set on precast or cast-in-place concrete caps with piling.

Figure 17.1.3.2-3
Low Profile Slab
(Short Spans)



Short span bridges up to 24 ft in length with limited headroom require the use of low profile slabs. These slabs may be set on precast caps that are either prestressed or non-prestressed.

Figure 17.1.3.2-4
Ballast Deck
(With Steel Beams)



Precast, prestressed concrete deck slabs are used in a variety of lengths and widths; with new or existing steel beams. These slabs can be cast with single and double ballast curbs and with integral walkways to further speed up construction of the bridge.

17.1.3.3 Connection Details

Structural steel tees or plates are frequently used to cover the longitudinal joint in slab beams and double-cell box beams as shown in **Figure 17.1.3.3-1**. Transverse post-tensioned steel tie rods, as shown in **Figure 17.1.3.3-2**, are generally provided in multiple single-cell box beam superstructures to help the group act as a unit. Concrete or structural steel “keepers” or retainers are usually provided at the ends of the caps to limit lateral movement, as shown in **Figure 17.1.3.3-3**. Designers should contact the specific railroad to determine their standards and preferred connection details.

Figure 17.1.3.3-1
Steel Tee between Box Beams

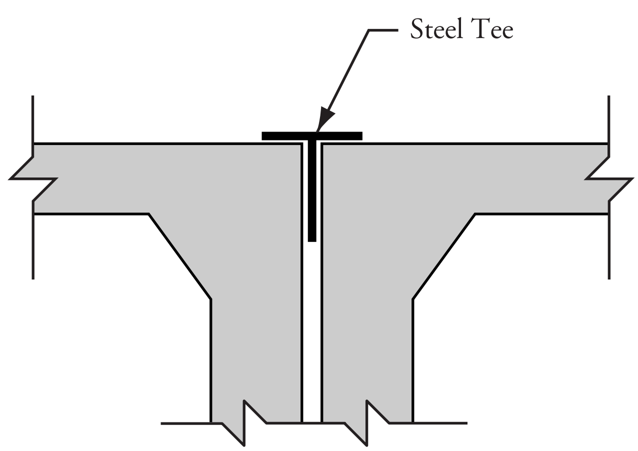


Figure 17.1.3.3-2
Post-Tensioned Steel Tie Rod

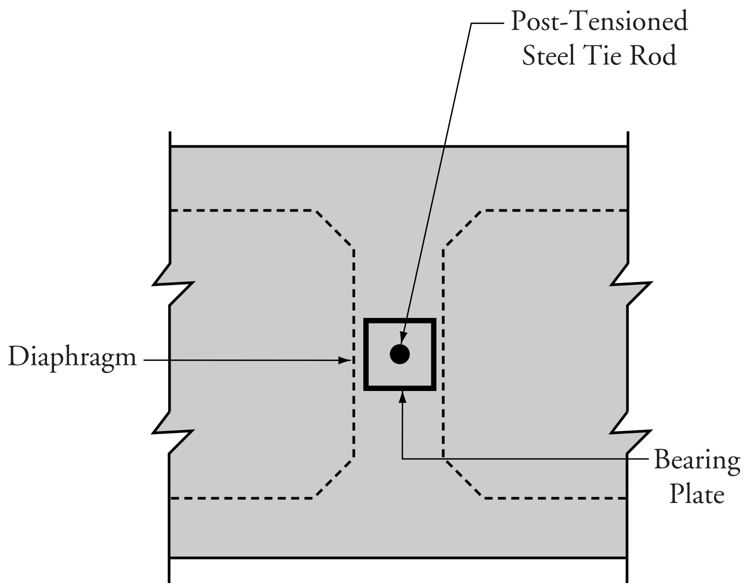
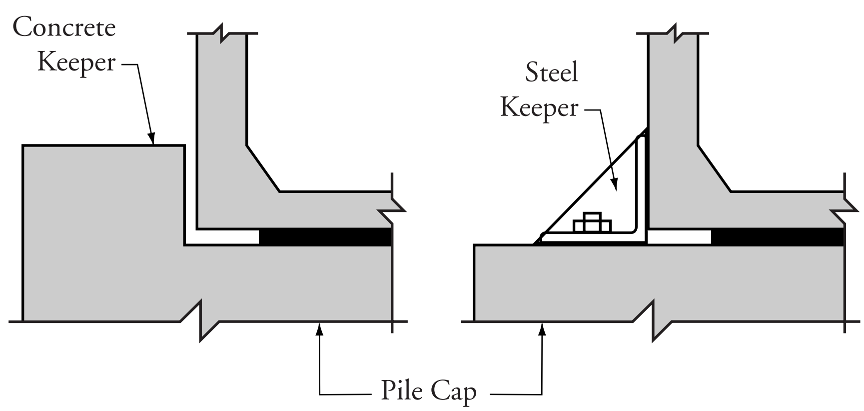


Figure 17.1.3.3-3
Concrete and Steel Keeper Details



17.2 CONSTRUCTION CONSIDERATIONS

17.2.1 Advantages

Precast concrete offers many advantages in the construction of railroad bridges. These include:

- Speed of construction—Precast concrete structures can usually be constructed faster than bridges comprised of alternative materials.
- Fabrication time—In addition to saving construction time, the lead time for fabricating elements is shorter than for competing materials such as steel.

- Durability—Compared with many older structures that require frequent inspections and maintenance, railroad engineers find the low maintenance requirements of precast concrete attractive. Use of concrete with low permeability and strict quality control in the casting plant help assure durable bridge components.
- Quality—The higher quality control of workmanship and materials available in casting plants compared to cast-in-place construction is another plus. Railroads can work with precast suppliers to ensure that members are cast to their satisfaction.
- Site constraints—The remote locations of many railroad bridges make the "precast" aspect of precast construction very useful. When the nearest ready-mix plant is many miles away from the site, cast-in-place construction within a railroad's time constraints is virtually impossible.
- Emergency response—Precast concrete bridge elements provide components for rapid repair of bridges as a result of damage caused by derailments or timber trestle fires. Several railroads keep entire precast bridges stockpiled for rapid emergency replacement. Concrete bridges are less vulnerable to damage from fire compared to steel or timber bridges.

17.2.2 Standard Designs

Most railroads have standard precast concrete trestle bridge designs that incorporate repetition of modular precast units. These standard designs are used for replacement of existing bridges, as well as construction of new bridges. Railroads and contractors familiar with railroad bridge construction have developed low-cost methods of trestle bridge construction. These methods minimize the time that railroad operations must be suspended. In addition, precast concrete bridge components are often shipped by rail, which, in many cases, is the only way to deliver components to remote locations.

17.2.3 Train Operations

For construction of bridges, railroads normally only permit train operations to be suspended from two to eight hours at any one time depending on the day and time. If an alternate route is available, 12 to 72 hours are the normal acceptable range. Additional costs of rerouting include obtaining operating rights on another railroad and using the other railroad's personnel. Use of either option is dependent upon the type and density of train traffic and the availability of alternate routes.

17.2.4 Construction Methods

The various methods used to construct railroad bridges to support existing trackage while minimizing disruptions to train operations include the following:

- rolling spans on runways
- floating spans on barges
- pick and set
- temporary rail line change
- permanent rail line change
- trestle bridge construction

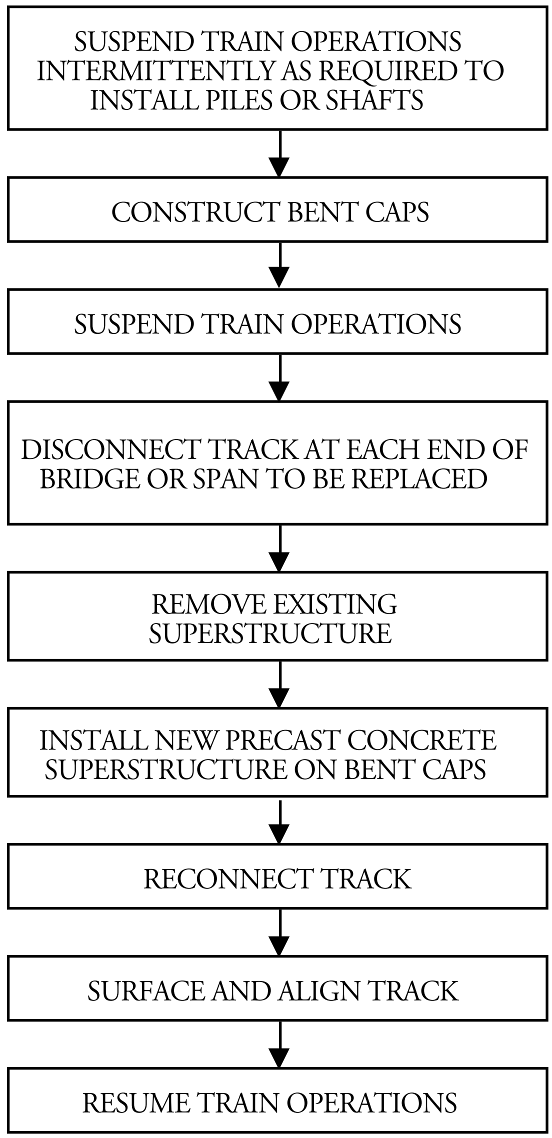
These methods are utilized because train operations cannot be suspended for the amount of time that would be required to construct the new bridge piece by piece in its permanent location.

17.2.5 Substructures

In many bridges, the existing substructure is reused and, if necessary, modified for replacement of the superstructure. Sometimes, the bridge may require new substructure elements. In both cases, the substructure work is performed beneath the existing track and superstructure so that the track is out of service for only very limited periods while driving piles or placing temporary supports. For replacement of existing bridges utilizing this method, ballast removal, as well as relocating the decks and beams of the existing bridge, may be required to allow pile driving for the new bridge. It is often necessary to reduce the speed of traffic over existing bridges during construction due to reduced load carrying capacity resulting from relocating the decks and beams.

Precast concrete beams are usually installed using pick and set methods. This method requires access to the bridge construction site for cranes that have adequate capacity to lift the beams. A typical bridge replacement procedure is illustrated in **Figure 17.2.5-1**.

Figure 17.2.5-1
Typical Bridge Replacement Construction Sequence



17.3 THE AMERICAN RAILWAY ENGINEERING AND MAINTENANCE-OF-WAY ASSOCIATION LOAD PROVISIONS

This section briefly discusses the types of loads on railroad bridges. The emphasis is on those loads that are different from highway bridge loads covered in Chapter 7. Provisions of the American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering* are introduced relative to design loads and load combinations. In addition, applicable portions of the manual are referenced.

17.3.1 AREMA Manual

The *AREMA Manual* provides the recommended practice for railroads and others concerned with the engineering, design and construction of railroad fixed properties, allied services and facilities. Prior to starting the design of a project, design engineers should discuss specific loadings, forces, standards and procedures with the appropriate railroad.

17.3.2 AREMA Loads

The *AREMA Manual* Chapter 8, Concrete Structures and Foundations, specifically addresses reinforced concrete and prestressed concrete structures. Article 2.2.3 covers the design loads and forces to be considered in the design of railroad structures supporting tracks, including bridges. Briefly, design loads include:

D = Dead Load
L = Live Load
I = Impact
CF = Centrifugal Force
E = Earth Pressure
B = Buoyancy
W = Wind Load on Structure
WL = Wind Load on Live Load
LF = Longitudinal Force from Live Load
F = Longitudinal Force due to Friction or Shear Resistance at Expansion Bearings
EQ = Earthquake (Seismic)
SF = Stream Flow Pressure
ICE = Ice Pressure
OF = Other Forces (Rib Shortening, Shrinkage, Temperature and/or Settlement of Supports)

17.3.2.1 Live Load

Design engineers familiar with highway bridge design will recognize the loads and forces listed above. The magnitude of the loads and forces are explained in detail in the *AREMA Manual*. Loads that are different from highway bridges are described in the following sections.

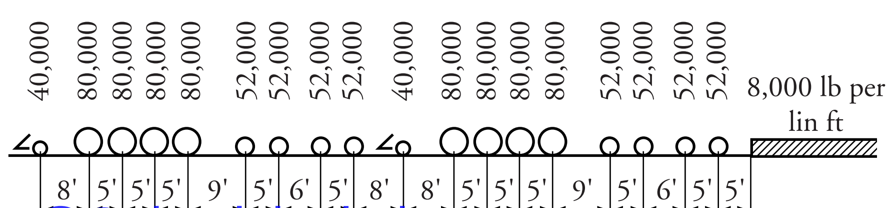
The following description of live load is based on the *AREMA Manual*:

(1) The recommended live load in pounds per axle and uniform trailing load for each track is the Cooper E 80 load, which is shown in **Figure 17.3.2.1-1**. **Table 17.3.2.1-1** provides a table for live load bending moments, shear forces and reactions for simple span bridges. Values for span lengths not shown are generally computed by interpolation.

(2) The Engineer (the Railroad's Chief Engineer) shall specify the Cooper live load to be used, and such load shall be proportional to the recommended load, with the same axle spacing.

(3) For bridges on curves, provisions shall be made for the increased proportion carried by any truss, beam or stringer due to the eccentricity of the load and centrifugal force.

Figure 17.3.2.1-1 Cooper E 80 Load



(4) For members receiving load from more than one track, the design live load on the tracks shall be as follows:
- For two tracks, full live load on two tracks
- For three tracks, full live load on two tracks and one-half on the other track
- For four tracks, full live load on two tracks, one-half on one track, and one-quarter on the remaining track
- For more than four tracks, as specified by the Engineer

The selection of the tracks for these loads shall be that which produces the most critical design condition in the member being designed.

Table 17.3.2.1-1
Maximum Bending Moments, Shear Forces and Pier Reactions for Cooper E 80 Live Load (Based on AREMA Manual Table 1-17)

All values are for one rail (one-half track load)

Span Length ft	Maximum Bending Moment ft-kips	Maximum Bending Moment at Quarter Point ft-kips	Maximum Shear Forces kips			Maximum Pier Reaction kips
			End	Quarter Point	Midspan	
5	50.00	37.50	40.00	30.00	20.00	40.00
6	60.00	45.00	46.67	30.00	20.00	53.33
7	70.00	55.00	51.43	31.43	20.00	62.86
8	80.00	70.00	55.00	35.00	20.00	70.00
9	93.89	85.00	57.58	37.78	20.00	75.76
10	112.50	100.00	60.00	40.00	20.00	80.00
11	131.36	115.00	65.45	41.82	21.82	87.28
12	160.00	130.00	70.00	43.33	23.33	93.33
13	190.00	145.00	73.84	44.61	24.61	98.46
14	220.00	165.00	77.14	47.14	25.71	104.29
16	280.00	210.00	85.00	52.50	27.50	113.74
18	340.00	255.00	93.33	56.67	28.89	121.33
20	412.50	300.00	100.00	60.00	28.70[1]	131.10
24	570.42	420.00	110.83	70.00	31.75	147.92
28	730.98	555.00	120.86	77.14	34.29	164.58
32	910.85	692.50	131.44	83.12	37.50	181.94
36	1,097.30	851.50	141.12	88.90	41.10	199.06
40	1,311.30	1,010.50	150.80	93.55	44.00	215.90
45	1,601.20	1,233.60	163.38	100.27	45.90	237.25
50	1,901.80	1,473.00	174.40	106.94	49.73	257.52
55	2,233.10	1,732.30	185.31	113.58	52.74	280.67
60	2,597.80	2,010.00	196.00	120.21	55.69	306.42
70	3,415.00	2,608.20	221.04	131.89	61.45	354.08
80	4,318.90	3,298.00	248.40	143.41	67.41	397.70
90	5,339.10	4,158.00	274.46	157.47	73.48	437.15
100	6,446.30	5,060.50	300.00	173.12	78.72	474.24

(1) AREMA table does not include a value for Cooper E 80 live load. A value of 28.70 kips is provided for alternative live load.

17.3.2.2 Impact Load

For reinforced concrete (precast and cast-in-place), the impact load is a percentage of the live load based on the ratio of live load to live load plus dead load:

$$I = \frac{100L}{L+D} \qquad \text{[AREMA Eq. 2-1]}$$

The impact load shall not exceed 60% for diesel engines and 80% for steam engines.

For prestressed concrete, the impact load is a percentage of the live load based on span length in ft:

$L \leq 60$ ft, $I = 35 - L^2/500$ [AREMA Eq. 17-1]

$60 < L \leq 135$ ft, $I = 14 + 800/(L - 2)$

$L > 135$ ft, $I = 20\%$

where L = span length of member in ft

17.3.2.3 Other Loads

All other loads and forces are defined similarly to highway bridges although the magnitudes are different. The design engineer should refer to the *AREMA Manual* for additional information.

17.3.2.4 Load Combinations

The various combinations of loads and forces to which a structure may be subjected are grouped in a similar manner as highway bridges. Each component of the structure or foundation upon which it rests, shall be proportioned for the group of loads that produces the most critical design condition. The group loading combinations for service load design and load factor design are as shown in **Table 17.3.2.4-1** and **Table 17.3.2.4-2**, respectively, and are reproduced from AREMA Article 2.2.4.

Table 17.3.2.4-1 Group Loading Combinations— Service Load Design

Group	Item	Allowable Percentage of Basic Unit Stress
I	D + L + I + CF + E + B + SF	100
II	D + E + B + SF + W	125
III	Group I + 0.5W + WL + LF + F	125
IV	Group I + OF	125
V	Group II + OF	140
VI	Group III + OF	140
VII	D + E + B + SF + EQ	133
VIII	Group I + ICE	140
IX	Group II + ICE	150

Table 17.3.2.4-2 Group Loading Combinations— Load Factor Design

Group	Item
I	1.4 (D + 5/3(L + I) + CF + E + B + SF)
IA	1.8 (D + L + I + CF + E + B + SF)
II	1.4 (D + E + B + SF + W)
III	1.4 (D + L + I + CF + E + B + SF + 0.5W + WL + LF + F)
IV	1.4 (D + L + I + CF + E + B + SF + OF)
V	Group II + 1.4 (OF)
VI	Group III + 1.4 (OF)
VII	1.4 (D + E + B + SF + EQ)
VIII	1.4 (D + L + I + E + B + SF + ICE)
IX	1.2 (D + E + B + SF + W + ICE)

17.4 CURRENT DESIGN PRACTICE

As with all engineering design practices, railroad industry practice continues to change as experience and research is incorporated into the *AREMA Manual* and individual railroad company standards and procedures. This section will discuss current railroad industry practice relative to overall railroad bridge design philosophy, skew limitations and superstructure continuity. Designers should discuss philosophies, standards and procedures with the specific railroad as applicable to the project.

17.4.1 New Bridges

New railroad bridges are constructed to support railroad tracks over existing waterways, roadways, and other railroads. In addition, new railroad bridges are built to replace existing bridges due to:

- unsatisfactory capacity to support current or future loadings
- unsafe condition resulting from deterioration and/or poor maintenance
- damage as a result of an accident or natural disaster
- inadequate waterway opening
- highway or railroad grade separation projects
- navigation, drainage and flood control projects

17.4.2 Replacement Bridges

The large majority of railroad bridge projects usually involve existing trackage. Consequently, one of the most important considerations for the railroad bridge designer is to design the bridge such that construction will have minimal disruption to train operations. This affects design details, construction methods and project costs. Much of today's rail traffic is under contract with the customer and the contract often includes a guarantee of service between origin and destination. Penalties and possible loss of a contract can result if unreasonable delays in the agreed upon schedule are experienced. Taking a track out of service or reducing the speed of rail traffic for an extended period of time for bridge construction can have a detrimental economic effect on the railroad. The project must be properly planned and coordinated with the operating and marketing departments of the railroad during the design and construction phases.

The use of standardized precast components speeds both the design and construction of bridges. Replacement spans can be specified by length alone, and railroad bridge workers are familiar with the sections and construction procedures. Since the vast majority of precast concrete bridges have all the superstructure below track level, vertical and horizontal clearance is not limited by these structures. This allows wide cargo or double stack containers to be shipped without clearance concerns and reduces the threat of bridge damage caused by shifted loads.

17.4.3 Simple Span Bridges

Many railroads prefer simple span bridges to continuous structures, finding them easier to install and maintain. Since they are structurally determinate, simple spans are better able to handle problems such as support settlement and thermal effects than some continuous bridges. Precast concrete elements are particularly suited to simple-span construction. Additional reasons many railroads prefer simply supported bridges to continuous span bridges include the following:

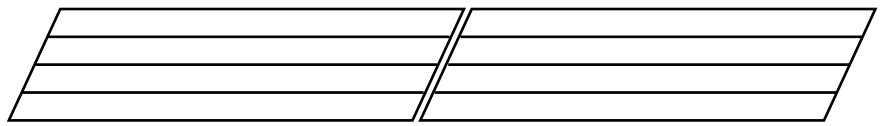
- If repair or replacement of superstructure elements is necessary, less interruption to train traffic is incurred with a simple span bridge than with a continuous span bridge.
- Installation of simple spans can be accomplished more quickly than continuous spans.

- If a bridge experiences substructure problems such as settlement, a continuous span bridge may require immediate and more extensive work, thereby resulting in greater interruptions to train traffic.
- Simple span bridges have a proven history of performing well.

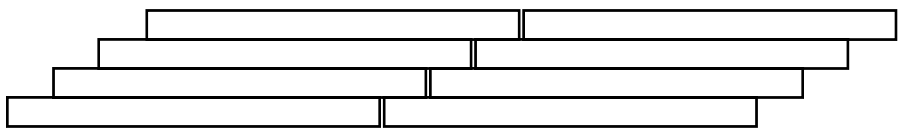
17.4.4
Skew Bridges

It is desirable to limit the end skew of railroad bridge precast beams to less than 30 degrees for constructibility and placement of reinforcing steel in the beam. When the bridge skew relative to the substructure exceeds 30 degrees, staggered precast elements as shown in **Figure 17.4.4-1** should be considered.

Figure 17.4.4-1
Layouts for Skewed Bridges



Skew ≤ 30°



Skew > 30°

17.5 CASE STUDY NO. 1— TRUSS BRIDGE REPLACEMENT

17.5.1
Existing Bridge

This case study describes a Southern Pacific railroad truss bridge replacement (Marianos, 1991). This project illustrates the use of precast concrete elements to replace a structure without serious interruption to rail traffic. The existing structure consisted of a 90-ft long timber trestle approach, two 154-ft long through-truss spans and a 30-ft long plate-girder approach span.

The truss spans were nearly 90 years old and were at the end of their useful service lives due to joint wear. Since the truss spans required replacement, the railroad decided to replace the entire bridge with precast concrete.

17.5.2
New Piles

Using a track-mounted pile driver, steel H-piles were driven through the track on the timber trestle. The pile bents were spaced to give 30-ft replacement span lengths in the trestle area. After the piles were cut off at the required elevation, precast concrete bent caps were placed and the piles welded to steel plates embedded in the bottom of the caps.

17.5.3
New Intermediate Piers

Since the truss spans crossed a creek subject to high flood flows, it was essential to minimize obstruction of the waterway. For this reason, new intermediate piers with four 79-ft long precast, prestressed box beams replaced the two 154-ft long truss spans. The 79-ft long beams were beyond the span range of the railroad standards and required a new design.

Railroad crews built intermediate piers at midspan of each truss by driving piles through the existing truss floor systems, and the 79-ft long box beams were ordered and fabricated.

17.5.4 New Superstructure for Approach Spans

When the substructure was completed, superstructure replacement began. The 90-ft long timber trestle was replaced by 30-ft long spans of precast, prestressed box beams, as shown in **Figure 17.5.4-1**. Two box beams placed side by side were used for each span. Each box beam has two through-voids and an integral ballast retaining sidewall and walkway cast on the outside edge. A shear key between the box beams helped ensure load distribution between the two beams. The box beams were placed using a track-mounted crane.

A similar 30-ft long box beam span was used to replace the steel plate-girder span on the approach opposite the timber trestle. Precast concrete bolster blocks were used on top of the existing masonry piers to obtain the proper elevation because the new structure was shallower than the existing one.

Figure 17.5.4-1
Precast 30-ft Approach Span on Precast Bolster Blocks

17.5.5 Truss Removal

After the approach spans were completed, preparation began for replacing the truss spans. An area under the truss spans was filled with ballast and leveled. Railroad track panels were laid perpendicular to the bridge on the fill below the structure. Steel frames mounted on rail trucks were placed on these tracks and used to support the trusses for removal. With these preparations for truss replacement complete, a carefully orchestrated construction effort began.

First, the truss ends were jacked up to lift them off the pier. The truss was then secured to the steel frames and rolled laterally clear of the work area, as shown in **Figure 17.5.5-1**. The construction crew then finished preparations on the pier top for placing the precast, prestressed concrete box beams. This work included removing the remaining truss attachments and placing elastomeric bearing pads.

Figure 17.5.5-1
Roll-Out of Truss Span to be Replaced

17.5.6 New Superstructure for Truss Spans

Each 154-ft long steel truss was replaced by two spans of precast box beams. When the pier preparation was completed, the four box beams of the first span were lifted into position using truck cranes. While workmen epoxied the longitudinal joints and shear keys between these beams, the box beams for the second span were being placed. After the joints of both spans were epoxied and handrail cables strung along the walkways, prefabricated panels of railroad track were placed on the spans. This allowed a hopper car to be moved out on the track to dump ballast on the new spans.

After the ballast was tamped and the track reconnected, the new spans were ready for rail traffic. Replacing a 154-ft long truss span was completed in a 12-hour track closure. Several weeks later, the second truss span was replaced, completing the reconstruction.

The use of precast elements, as shown in **Figure 17.5.6-1**, allowed the speedy and economical replacement of the structure, using the railroad's own work force.

Figure 17.5.6-1
Completed Structure

17.6 CASE STUDY NO. 2— TIMBER TRESTLE REPLACEMENT

17.6.1 Existing Bridge

This case study discusses a timber trestle bridge replacement on the Union Pacific Railroad system. Bridge 177.81 is located approximately 1.59 miles west of Marysville, CA on Union Pacific Railroad's Canyon Subdivision. The existing bridge, shown in **Figure 17.6.1-1**, consisted of numerous timber trestle spans and a steel plate-girder span over the Yuba River. The plate-girder was to remain in place and the timber trestle portion of the bridge was to be replaced.

Figure 17.6.1-1
Existing Plate-Girder and Timber Trestle Spans.

17.6.2 New Superstructure

Due to the volume of rail traffic and importance of on-time delivery by the Union Pacific Railroad, minimal disruption to train operations was mandatory. Substructure construction was to be performed without interference or downtime to the railroad. Superstructure change-out would be performed during “windows” approved by the railroad. A precast, prestressed concrete superstructure system was selected based on economics, speed of erection and the ability to meet the construction constraints associated with the need for minimal disruption to train operations.

The existing timber trestle spans varied in length with an average span of slightly less than 15 ft. Based on a field survey of the timber bent locations, new bent locations were selected to minimize interference with existing timber pile bents and optimize beam spans. A span length of 44 ft was selected for the new superstructure. For this span length, 45-in. deep double-cell, prestressed concrete box beams were determined to be the most economical structural system.

17.6.3 Substructure Construction

Based on field conditions, prevalent construction practice in the area and construction constraints governed by railroad operations, cast-in-place reinforced concrete bents were selected for the substructure. The bents consisted of 100-ft long, 4-ft diameter drilled shafts, 4-ft diameter cast-in-place reinforced concrete column extensions and cap beams. All structural components were designed in accordance with the *AREMA Manual* and Union Pacific Railroad standards and procedures.

The sequence of construction was as follows:

The existing bridge footwalk and handrail were removed as required to facilitate drilled shaft installation. The drilled shafts were spaced at 15-ft centers perpendicular to the track to allow installation of the drilled shafts without interference to railroad operations. Continuous train operations were maintained throughout the entire construction of the substructure. Due to foundation conditions, steel pipe casing was necessary for drilled shaft installation. The pipe casing was installed using a vibratory hammer. Reinforcing steel cages were set and the holes were filled with 4,000 psi compressive strength concrete. Drilled shaft column extensions, bent cap beams and the abutment were constructed under the existing timber superstructure. Due to the depth of the new concrete beams, the bent and abutment construction were completed without interfering with the existing timber superstructure, as shown in **Figure 17.6.3-1**.

Figure 17.6.3-1
Completed Concrete Bents under Existing Timber Trestle

17.6.4 Superstructure Construction

Working within railroad approved construction "windows," the timber structure was removed and precast beams were set. In a continuous, well-planned procedure, the ballast, ties and rail were placed and train operations were resumed. The use of precast concrete allowed the Union Pacific Railroad to replace a timber trestle with a stronger, more durable structural system with minimal disruption to railroad service. The completed bridge is shown in **Figure 17.6.4-1**.

Figure 17.6.4-1
Completed Bridge Structure

17.7 DESIGN EXAMPLE—DOUBLE-CELL BOX BEAM, SINGLE SPAN, NON-COMPOSITE, DESIGNED IN ACCORDANCE WITH AREMA SPECIFICATIONS

17.7.1 Background

Prestressed concrete double-cell box beams and solid slab beams are commonly used in the railroad industry. Solid slab beams are used for spans up to 20 ft, especially when superstructure depth has to be minimized. Prestressed concrete double-cell box beams are used for spans up to 50 ft in length. Prestressed concrete single-cell box beams are more economical for spans longer than 40 ft and are used for span lengths up to 80 ft. When span lengths exceed 80 ft, prestressed concrete I-beams with a composite deck become more feasible from a design, economic and construction point of view. This example illustrates the design of a non-composite, prestressed concrete, double-cell box beam.

17.7.2 Introduction

In non-composite design, the beam acts as the main structural element. Therefore, the beam has to carry all the dead loads, superimposed dead loads and live load. The beams are assumed to be fully prestressed under service load conditions. The dead load consists of the self-weight of the beam including diaphragms. The superimposed dead loads consist of ballast, ties, rails, concrete curbs and handrails, as shown in **Figures 17.7.2-1** and **17.7.2-2**. The live load used for this bridge is Cooper E 80, which is described in the *AREMA Manual*, Chapter 8, Part 2, Reinforced Concrete Design, Article 2.2.3. The prestressed concrete beams are designed using the *AREMA Manual*, Chapter 8, Part 17, Prestressed Concrete Design Specifications for Design of Prestressed Concrete Members. The beams in this example are checked for both serviceability and strength requirements.

Figure 17.7.2-1
Bridge Cross-Section

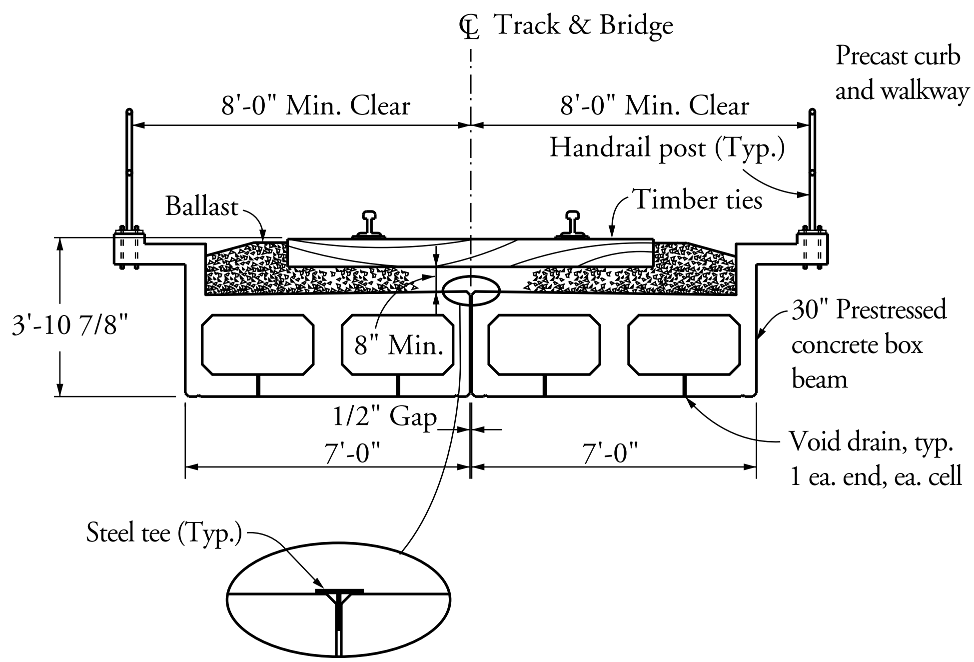
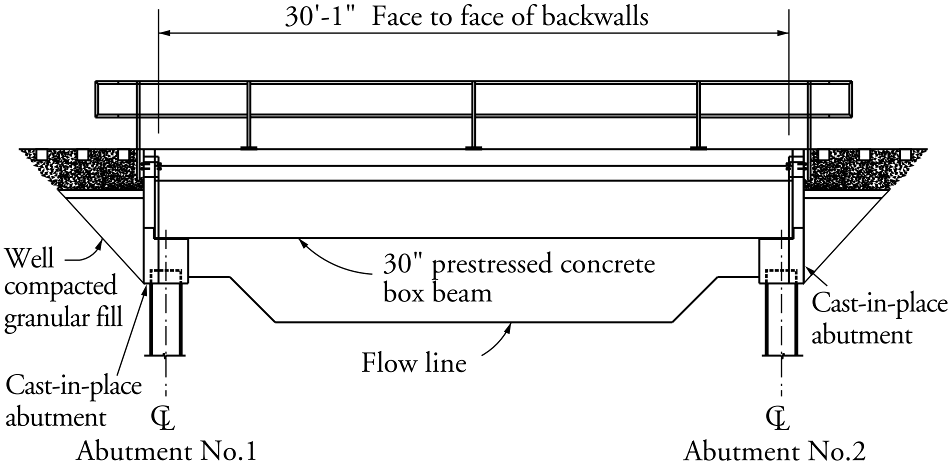


Figure 17.7.2-2
Bridge Elevation



17.7.2.1
Geometrics

For design, the bridge has the following dimensions:

Beam length = 30.0 ft

Beam width = 7.0 ft

Center-to-center distance between bearings = 29.0 ft

Bearing width (measured longitudinally) = 8 in.

Bearing length (measured transversely) = 6.67 ft

Depth of ballast = 15 in.

Timber ties: length = 9 ft; width = 9 in.; depth = 7 in.

Rail section = 132 RE (Bethlehem Steel Co.)

No. of tracks = one

17.7.2.2
Sign Convention

For concrete:

Compression positive (+ve)

Tension negative (–ve)

For steel:

Compression negative (–ve)

Tension positive (+ve)

Distance from center of gravity:

Downward positive (+ve)

Upward negative (–ve)

17.7.2.3
Level of Precision

Item	Units	Precision
Concrete Stress	ksi	1/1000
Steel Stress	ksi	1/10
Prestress Force	kips	1/10
Moments	ft-kips	1/10
Shears	kips	1/10
For the Beam:		
Cross-Section Dimensions	in.	1/100
Section Properties	in.	1
Length	ft	1/100
Area of Prestressing Steel	in.^{2}	1/1000
Area of Mild Reinforcement	in.^{2}	1/100

Some calculations are carried out to a higher number of significant figures than common practice with hand calculation. Depending on available computation resources and designer preferences, other levels of precision may be used.

17.7.3
Material Properties

17.7.3.1
Concrete

Concrete strength at transfer, f'_{ci} = 4,000 psi

Concrete strength at 28 days, f'_c = 7,000 psi

Concrete unit weight, w_c = 150 pcf

Modulus of elasticity of prestressed concrete, E_c

$$E_c = w_c^{1.5} 33\sqrt{f'_c}\text{, psi} \qquad \text{[AREMA Art. 2.23.4]}$$

where

w_c = unit weight of concrete, pcf

f'_c = specified strength of concrete, psi

Modulus of elasticity of concrete at transfer, using f'_{ci} = 4,000 psi, is:

$$E_{ci} = (150)^{1.5}(33)\sqrt{4,000} / 1,000 = 3,834 \text{ ksi}$$

Modulus of elasticity of concrete at 28 days, using f'_c = 7,000 psi, is:

$$E_c = (150)^{1.5}(33)\sqrt{7,000} / 1,000 = 5,072 \text{ ksi}$$

17.7.3.2
Pretensioning Strands

1/2-in. diameter, seven wire, low-relaxation strands

Area of one strand, A_{ps} = 0.153 in.^{2}

Ultimate tensile strength, f_{pu} = 270.0 ksi

Modulus of elasticity, E_s = 28,000 ksi

17.7.3.3 Reinforcing Bars

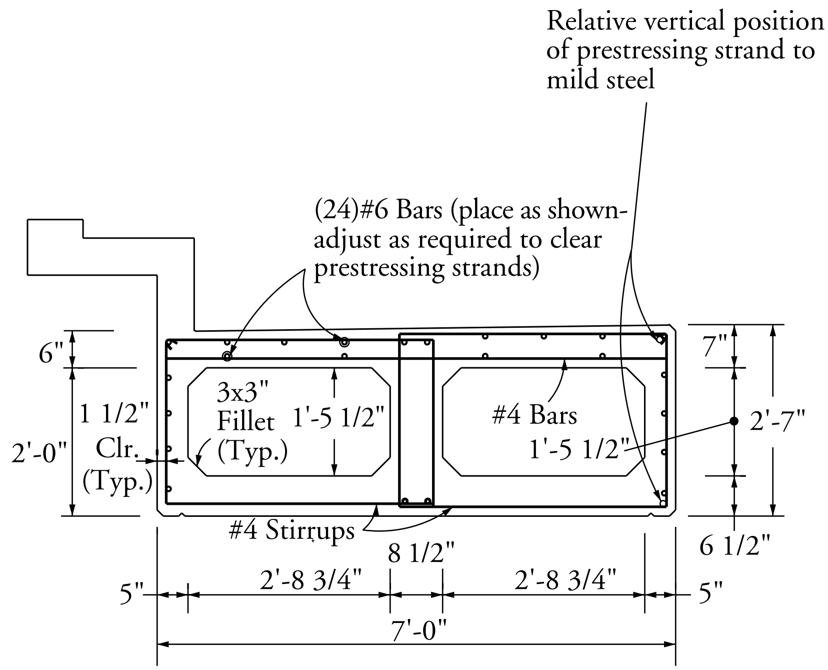
Yield strength, f_y = 60,000 psi

Modulus of elasticity, E_s = 29,000 ksi

17.7.4 Cross-Section Properties for a Single Beam

For cross-sectional dimensions of a single box beam, see **Figure 17.7.4-1**. Note that the depth varies from 30 in. to 31 in. to provide drainage

Figure 17.7.4-1 Box Beam Cross-Section



A = area of cross-section of precast beam = 1,452 in.^{2}

h = average depth of the precast beam = (0.5)(31 + 30) = 30.5 in.

I = moment of inertia about the centroid of the precast beam = 171,535 in.^{4}

y_b = distance from centroid to extreme bottom fiber of the precast beam = 15.25 in.

y_t = distance from centroid to extreme top fiber of the precast beam = 15.25 in.

S_b = section modulus for the extreme bottom fiber of the precast beam = 11,248 in.^{2}

S_t = section modulus for the extreme top fiber of the precast beam = 11,248 in.^{3}

NOTE: Section properties do not include precast curbs and walkway. Reinforcement in curbs and walkway not shown for clarity

17.7.5 Shear Forces and Bending Moments

17.7.5.1 Shear Forces and Bending Moments Due to Dead Load

$$\text{Self-weight of beam} = \frac{1{,}452(150)}{1{,}000(144)} = 1.513 \text{ kip/ft}$$

Weight of end diaphragm = 1.7 kips

The equations for shear force (V_x) and moment (M_x) for uniform loads on a simple span (L) are given by:

$$V_x = w\left(\frac{L}{2} - x\right) \qquad \text{(Eq. 17.7.5.1-1)}$$

$$M_x = \frac{wx}{2} - (L - x) \qquad \text{(Eq. 17.7.5.1-2)}$$

where

w = weight/ft = 1.513 kip/ft

L = span length, ft

x = distance from the support, ft

Using the above equations, values of shear forces (V_g) and bending moments (M_g) are computed and given in **Table 17.7.5.1-1**.

Table 17.7.5.1-1 Shear Forces and Bending Moments

x, ft	0.0*	1.27**	4.0	6.0	7.25	10.0	14.5
V_g, kip	21.9	20.0	15.9	12.8	10.9	6.8	0.0
M_g, ft-kips	0.0	26.6	75.7	104.4	119.3	143.7	159.1
V_{SDL}, kip	20.1	18.4	14.6	11.8	10.1	6.3	0.0
M_{SDL}, ft-kip	0.0	24.5	69.5	95.8	109.5	132.0	146.0
V_{LL+I}, kip	164.6	150.7	—	—	104.8	86.8	46.8
M_{LL+I}, kip	0.0	194.5	—	—	785.7	892.0	1,033.0

* At the support
** At the critical section for shear (See Section 17.7.11)

Diaphragm Load: Since distance between the centerline of the bearing and center of gravity of the diaphragm is less than the effective depth, ignore the effect of the diaphragm load in this example.

17.7.5.2 Shear Forces and Bending Moments Due to Superimposed Dead Load

Superimposed dead loads consist of ballast, ties, rails, curbs and handrails.

Ballast, including track ties at 120 pcf
= 15/12(7 + 0.04/2 gap)(0.120) = 1.053 kip/ft [AREMA Art. 2.2.3]

Track rails, inside guardrails and fastenings at 200 plf /track = $\frac{0.200}{2}$ = 0.100 kip/ft

For this example, assume concrete curb at 1.5 ft^2 + handrail post at 5% = (1.5)(0.150)(1.05) = 0.236 kip/ft

Total superimposed dead load per beam per linear ft = 1.389 kip/ft

Using a uniform load of 1.389 kip/ft and **Equations 17.7.5.1-1** and **17.7.5.1-2**, values of shear forces (V_{SDL}) and bending moments (M_{SDL}) are computed and given in **Table 17.7.5.1-1**.

17.7.5.3 Shear Forces and Bending Moments Due to Live Load

The actions caused by the Cooper E 80 live load can be determined by using the tables in the *AREMA Manual*, Chapter 15, Art. 1.15 Appendix or by using any commercially available computer program. A distribution factor (DF) equal to 0.5 is used, since there are two beams supporting one track.

For span lengths less than 60 ft, the impact factor is:

$$I = \left(35 - \frac{L^2}{500}\right) = \left(35 - \frac{(29)^2}{500}\right) = 33.32\% \text{ of live load} \qquad \text{[AREMA Eq. 17-1]}$$

The values of shear forces (V_{LL+I}) and bending moments (M_{LL+I}) for live load plus impact for one beam were determined using a computer program and are given in **Table 17.7.5.1-1**.

17.7.5.4 Load Combinations

For Group I loading:

Service Load Design = D + (L + I)(DF) [AREMA Table 2-2]

Load Factor Design = 1.4(D + 5/3(L + I)(DF)) [AREMA Table 2-3]

Values of shear forces and bending moments for service load design and factored load design are determined from **Table 17.7.5.1-1** and given in **Table 17.7.5.4-1**.

Table 17.7.5.4-1
Shear Forces and Bending Moments for Design

	Self Wt (g)	Dead (SDL)	Live + Impact (L+I)	Total Service Load	Total Factored Load
Max. Shear Force at 1.27 ft, kips	20.0	18.4	150.7	189.1	405.4
Max. Bending Moment at Midspan, ft-kips	159.1	1,46.0	1,033.0	1,338.1	2,837.5

The maximum value of shear occurs near the supports while the maximum value of bending moment occurs near midspan for a simply supported span.

17.7.6 Permissible Stresses in Concrete at Service Loads

At transfer (before time-dependent prestress losses): [AREMA Art. 17.6.4]

Compression: $0.60\ f'_{ci} = 0.60(4{,}000) = 2.400$ ksi

Tension: $3\sqrt{f'_{ci}}$ without bonded reinforcement = $3\sqrt{4{,}000} = 0.190$ ksi

At service loads (after allowance for all prestress losses): [AREMA Art. 17.6.4]

Compression: $0.40\ f'_c = 0.40(7{,}000) = 2.800$ ksi

Tension in precompressed tensile zone: 0 ksi

17.7.7 Estimate Required Prestressing Force

Try eccentricity of strands at midspan, $e_c = y_b - 2.5 = 12.75$ in.

Bottom tensile stress due to applied loads:

$$f_b = \frac{M_g + M_{SDL} + M_{LL+I}}{S_b}$$

where

f_b = concrete stress at the bottom fiber of the beam

M_g = unfactored bending moment due to precast beam self-weight, ft-kips

M_{SDL} = unfactored bending moment due to superimposed dead load, ft-kips

M_{LL+I} = unfactored bending moment due to live load plus impact, ft-kips

$$f_b = \frac{12(159.1 + 146.0 + 1,033.0)}{11,248} = 1.428 \text{ ksi}$$

Since allowable tensile stress in bottom fiber at service load is zero, required precompression is 1.428 ksi.

Bottom fiber stress due to prestress after all losses:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

where P_{se} = effective pretension force after allowing for all losses

$$\text{Then } 1.428 = \frac{P_{se}}{1,452} + \frac{P_{se}(12.75)}{11,248}$$

and P_{se} = 783.7 kips

Since losses are generally between 15 and 20%, assume 18% final prestress losses.

Allowable tensile stress in prestressing tendons immediately after prestress transfer is the larger of $0.82f_{py} = (0.82)(0.9f_{pu}) = 0.738\ f_{pu}$ or $0.75f_{pu}$

$0.75\ f_{pu} = 0.75(270) = 202.5$ ksi [AREMA Art. 17.6.5]

$$\text{Number of strands required} = \frac{783.7}{(1-0.18)(0.75)(270)(0.153)} = 30.8 \text{ strands}$$

Try 32 strands at bottom, y_{bs} = 2.5 in.

Plus 4 strands at mid-height, y_{bs} = 15.25 in.

Plus 6 strands at top, y_{bs} = 27.50 in.

Total No. of strands = 32 + 4 + 6 = 42 strands

$$\text{Center of gravity of strands, } y_{bs} = \frac{32(2.5) + 4(15.25) + 6(27.50)}{42} = 7.29 \text{ in.}$$

Eccentricity of strands, $e_c = y_b - y_{bs} = 15.25 - 7.29 = 7.96$ in.

Total initial prestressing force before loss = 202.5(0.153)(42) = 1,301.3 kips

17.7.8 Determine Prestress Losses

To determine effective prestress, f_{se}, allowance for losses of prestress due to elastic shortening of concrete, $f_{\ell e}$, creep of concrete, $f_{\ell c}$, shrinkage of concrete, $f_{\ell s}$, and relaxation of prestressing steel, $f_{\ell r}$, will be calculated.

17.7.8.1 Prestress Losses at Service Loads

17.7.8.1.1 Elastic Shortening of Concrete

$$f_{\ell e} = \frac{E_s}{E_{ci}} f_{cr} \qquad \text{[AREMA Eq. 17-16]}$$

where

f_{cr} = stress in concrete at centroid of prestressing reinforcement immediately after transfer, due to total prestress force and dead load acting at time of transfer, and is calculated as follows:

$$= \frac{P_{si}}{A} + \frac{P_{si}e_c^2}{I} - \frac{M_g e_c}{I}$$

where

P_{si} = pretension force after allowing for initial losses. Taken as 0.69 f_{pu}

$$f_{cr} = \frac{42(0.69)(0.153)(270)}{1,452} + \frac{42(0.69)(0.153)(270)(7.96)^2}{171,535}$$

$$- \frac{159.1(12)(7.96)}{171,535} = 0.824 + 0.442 - 0.089 = 1.177 \text{ ksi}$$

$$f_{\ell e} = \frac{28,000}{3,834}(1.177) = 8.6 \text{ ksi}$$

17.7.8.1.2 Creep of Concrete

$$f_{\ell c} = 12f_{cr} - 7f_{cds} \qquad \text{[AREMA Eq. 17-18]}$$

where

f_{cds} = concrete stress at centroid of prestressing reinforcement, due to all dead loads not included in calculation of f_{cr}

$$= \frac{M_{SDL}e_c}{I} = \frac{146.0(12)(7.96)}{171,535} = 0.081 \text{ ksi}$$

$$f_{\ell s} = 12(1.177) - 7(0.081) = 13.6 \text{ ksi}$$

17.7.8.1.3 Shrinkage of Concrete

Assume relative humidity, R = 70% (see also *AREMA* Fig. 17-1):

$$f_{\ell s} = 17,000 - 150 R \qquad \text{[AREMA Eq. 17-19]}$$

$$= \frac{17,000 - 150(70)}{1,000} = 6.5 \text{ ksi}$$

17.7.8.1.4 Relaxation of Prestressing Steel

For pretensioning tendons with 270 ksi low-relaxation strand:

$$f_{\ell r} = 5,000 - 0.10f_{\ell e} - 0.05(f_{\ell s} + f_{\ell c}) \qquad \text{[AREMA Eq. 17-21]}$$

$$= \frac{5,000}{1,000} - 0.10(8.6) - 0.05(6.5 + 13.6) = 3.1 \text{ ksi}$$

17.7.8.1.5 Total Losses at Service Loads

Total prestress losses = 8.6 + 13.6 + 6.5 + 3.1 = 31.8 ksi

Final prestressing force, P_{se} = (202.5 − 31.8)(0.153)(42) = 1,096.9 kips

$$\text{Percentage prestress losses} = \left(\frac{31.8}{202.5}\right)100 = 15.7\%$$

17.7.8.2 Total Losses at Service Loads

Losses due to elastic shortening, $f_{\ell e}$ = 8.6 ksi

Total initial prestress losses = 8.6 ksi

Initial prestress force after loss, P_{si} = (202.5 − 8.6)(0.153)(42) = 1,246.0 kips

$$\text{Percentage initial prestress losses} = \left(\frac{8.6}{202.5}\right)100 = 4.25\%$$

17.7.9 Concrete Stresses

Stresses need to be checked at several locations along the beam to ensure that the design satisfies permissible stresses at all locations at both transfer and service loads. For this design example, stresses will be checked at midspan and at the ends, which will govern straight strand designs without debonding.

17.7.9.1 Stresses at Transfer at Midspan

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = \frac{P_{si}}{A} - \frac{P_{si}e_c}{S_t} + \frac{M_g}{S_t}$$

M_g is based on overall length of 30 ft

$M_g = wL^2/8 = 1.513(30)^2/8 = 170.2$ ft-kips

$$f_t = \frac{1,246}{1,452} - \frac{(1,246)(7.96)}{11,248} + \frac{170.2(12)}{11,248}$$

$$= 0.858 - 0.882 + 0.182 = 0.158 \text{ ksi}$$

Compare with permissible values:

– 0.190 ksi < 0.158 ksi < 2.400 ksi O.K.

Compute concrete stress at the bottom fiber of the beam f_b:

$$f_b = \frac{P_{si}}{A} - \frac{P_{si}e_c}{S_b} + \frac{M_g}{S_b}$$

$$= \frac{1,246}{1,452} + \frac{(1,246)(7.96)}{11,248} - \frac{170.2(12)}{11,248}$$

$$= 0.858 + 0.882 - 0.182 = 1.558 \text{ ksi}$$

Compare with permissible values:

–0.190 ksi < 1.558 ksi < 2.400 ksi O.K.

17.7.9.2 Stresses at Transfer at End

Stresses should be checked at the end of the transfer length when designing a prestressed beam (see Section 9.1.8.2 for an example of this check). However, in this design example, a standard beam design is being checked. Therefore it is conservative to check the stresses at the very end of the member, assuming the full prestress force is effective at that location. Since the strands are straight and all strands are bonded for the full length of the beam, the concrete stresses at the end are simply the stresses at midspan without the stress due to dead load moment.

$f_t = -0.023$ ksi, which is within permissible values shown above O.K.

$f_b = 1.740$ ksi, which is within permissible values shown above O.K.

17.7.9.3 Stresses at Service Load at Midspan

Compute concrete stress at the top fiber of the beam, f_t:

$$f_t = \frac{P_{se}}{A} - \frac{P_{se}e_c}{S_t} + \frac{M_g + M_{SDL} + M_{LL+I}}{S_t}$$

$$= \frac{1{,}096.9}{1{,}452} - \frac{(1{,}096.9)(7.96)}{11{,}248} + \frac{1{,}338.1(12)}{11{,}248}$$

$= 0.755 - 0.776 + 1.428 = 1.407$ ksi < 2.800 ksi O.K.

Compute concrete stress at the bottom fiber of the beam, f_b:

$$f_b = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b} - \frac{M_g + M_{SDL} + M_{LL+I}}{S_b}$$

$$= \frac{1{,}096.9}{1{,}452} + \frac{(1{,}096.9)(7.96)}{11{,}248} - \frac{1{,}338.1(12)}{11{,}248}$$

$= 0.755 + 0.776 - 1.428 = 0.103$ ksi > 0.0 ksi O.K.

17.7.9.4 Stresses at Service Load at End

The prestress force is at its maximum value at release and service loads do not affect stresses at the end of the beam. Therefore, stresses at release will govern at the end of the beam, so there is no need to check stresses at the end at service loads.

17.7.10 Flexural Strength

17.7.10.1 Stress in Strands at Flexural Strength

In lieu of a more accurate determination of stress in pretensioning strands at nominal strength, f_{ps}, based on strain compatibility, the following approximate value of f_{ps} is used:

$$f_{ps} = f_{pu}\left(1 - 0.5\rho_p \frac{f_{pu}}{f'_c}\right), \text{ provided } f_{se} \text{ is greater than } 0.5\, f_{pu} \qquad \text{[AREMA Eq. 17-2]}$$

where

f_{se} = effective stress in pretensioning steel after losses

$= 202.5 - 31.8 = 170.7$ ksi $> 0.5(270) = 135.0$ ksi O.K.

$$\rho_p = \frac{A_{ps}}{bd}$$

where

A_{ps} = total area of pretensioning steel in tension zone
= 36 (0.153) = 5.508 in.^{2}

b = effective flange width = 7(12) = 84.0 in.

d = distance from extreme compression fiber to centroid of pretensioning force

$$= 30.5 - \frac{32(2.5) + 4(15.25)}{36} = 26.58 \text{ in.}$$

Note: In many cases, strands near or above midheight are neglected when computing d for calculating the average stress in strands at flexural strength. This is because, at the flexural strength, the strands located higher in the cross-section will not reach a strain (and stress) as high as the bottom strands. However, for this standard beam design, the strands at midheight have been included as shown above. A strain compatibility analysis (described in Sections 8.2.2.5 and 8.2.2.6) can be used to compute the strain and stress in the strands at midheight. Such an analysis for this beam indicates that the strands at midheight would reach a stress of approximately 251 ksi, which is reasonable when compared with the stress, f_{ps}, computed below. The same analysis indicates that the strands in the bottom row would reach a stress of nearly 260 ksi. Therefore, in this case, incorporating the strands at midheight has provided a reasonable result. If the strands at midheight are neglected, the strength of the section at midspan would prove to be inadequate.

$$\rho_p = \frac{A_{ps}}{bd} = \frac{5.508}{84(26.58)} = 0.00247$$

$$f_{ps} = 270\left(1 - (0.5)(0.00247)\frac{270}{7.0}\right) = 257.1 \text{ ksi}$$

17.7.10.2
Limits for Reinforcement

Assuming a rectangular section, compute the reinforcement ratio as:

$$\rho_p \frac{f_{ps}}{f'_c} = \frac{0.00247(257.1)}{7.0} = 0.0907 < 0.30 \quad \text{O.K.} \qquad \text{[AREMA Art. 17.5.4]}$$

When the reinforcement ratio exceeds 0.30, design moment strength shall not be taken greater than the moment strength based on the compression portion of the moment couple.

17.7.10.3
Design Moment Strength

Assuming beam acts as a rectangular section::

$$\phi M_n = \phi\left[A_{ps}f_{ps}d\left(1 - 0.6\rho_p \frac{f_{ps}}{f'_c}\right)\right] \qquad \text{[AREMA Eq. 17-3]}$$

$$= \phi\left[A_{ps}f_{ps}\left(d - \frac{a}{2}\right)\right] \qquad \text{[AREMA Eq. 17-4]}$$

where

M_n = nominal moment strength of a section

ϕ = strength reduction factor for flexure = 0.95 [AREMA Art.17.5.2]

$$a = \frac{A_{ps}f_{ps}}{0.85\,f'_c b} = \frac{5.508(257.1)}{0.85(7)(84)} = 2.83 \text{ in.}$$ [AREMA Art.17.5.4d]

Average depth of top flange = 6.5 in. > 2.83 in.

Therefore, rectangular section assumption is appropriate.

Using AREMA Eq. 17-4:

$$\phi M_n = 0.95\left[5.508(257.1)\left(26.58 - \frac{2.83}{2}\right)\right]\frac{1}{12} = 2{,}821.2 \text{ ft-kips}$$

Factored moment due to dead and live loads from **Table 17.7.5.4-1** = 2,837.5 ft-kips.

$$\text{Percentage over} = \frac{(2{,}837.5 - 2{,}821.2)}{2{,}821.2}(100) = 0.58\% \text{ (insignificant) say ok.}$$

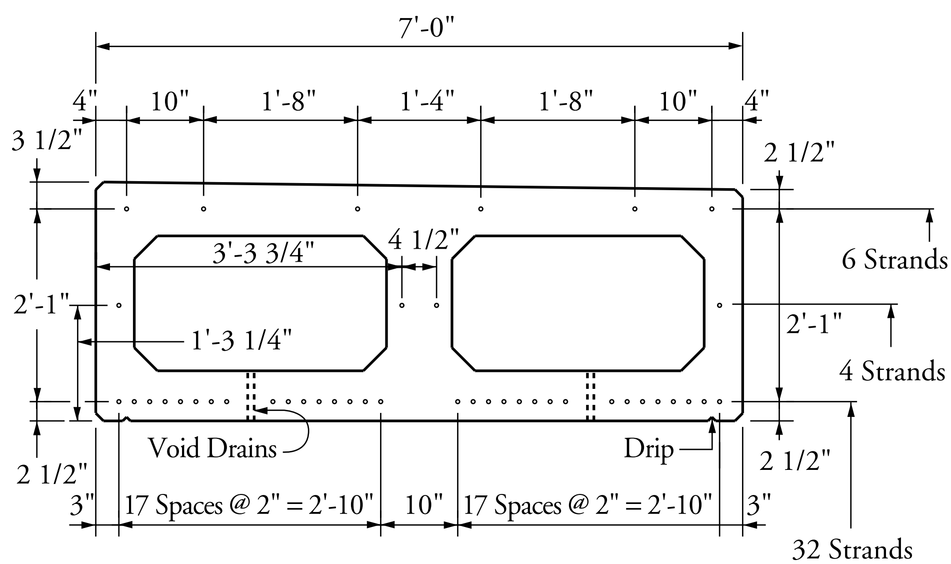
17.7.10.4 Minimum Reinforcement

The total amount of prestressed and non-prestressed reinforcement should be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment, M_{cr}: $\phi M_n \geq 1.2M_{cr}$. The calculation (not shown here but similar to the calculation in Section 9.1.10.2) yields 2,821.2 ft-kips > 2,427.3 ft-kips O.K.

17.7.10.5 Final Strand Pattern

Final strand locations are shown in **Figure 17.7.10.5-1**

Figure 17.7.10.5-1
Strand Pattern



17.7.11 Shear Design

17.7.11.1 Required Shear Strength

Prestressed concrete members subjected to shear are designed so that

$$V_u \le \phi\,(V_c + V_s) \qquad \text{[AREMA Eq. 17-8]}$$

where

V_u = factored shear force at section considered
V_c = nominal shear strength provided by concrete
V_s = nominal shear strength provided by shear reinforcement
ϕ = strength reduction factor for shear = 0.90 [AREMA Art. 17.5.2]

Per the *AREMA Manual*, Article 17.5.9b, the critical section for shear is located at a distance h/2 from face of support. In this design example, the critical section for shear is calculated from the centerline of the bearings since the pads are not rigid and have the potential to rotate.

$h/2 = 30.5/2 = 15.25$ in. $= 1.27$ ft

$V_u = 405.4$ kips (from Table 17.7.5.4-1)

17.7.11.2 Shear Strength Provided by Concrete

17.7.11.2.1 Simplified Approach

The shear strength provided by concrete, V_c, can be calculated by using *AREMA Manual* Eq. 17-9, provided that the effective prestress force is not less than 40% of the total tensile strength provided by the flexural reinforcement.

$$V_c = \left(0.6\sqrt{f_c'} + 700\frac{V_u d}{M_u}\right)b_w d \qquad \text{[AREMA Eq. 17-9]}$$

where

M_u = factored bending moment at the section

$$= 1.4\left(26.6 + 24.5 + \frac{5}{3}(194.5)\right) = 525.4 \text{ ft-kips}$$

b_w = total web width = 5 + 8.5 + 5 = 18.5 in.
d = 26.58 in. > 0.8h = (0.8)(30.5) = 24.4 in.
Therefore, use d = 26.58 in.

$$\frac{V_u d}{M_u} = \frac{405.4(26.58)}{525.4(12)} = 1.71 > 1.0, \text{ use } 1.0 \qquad \text{[AREMA Art. 17.5.9c]}$$

$$V_c = \left(0.6\sqrt{7{,}000} + 700(1.0)\right)18.5(26.58)/1{,}000 = 368.9 \text{ kips}$$

However, the maximum value of V_c is limited to:

$$5\sqrt{f_c'}\,b_w d = 5\sqrt{7{,}000}\,(18.5)\,(26.58)/1{,}000 = 205.7 \text{ kips} < V_c = 368.9 \quad \text{NO GOOD}$$

AREMA Manual Art. 17.5.9c allows higher values of V_c if a more detailed calculation is made. According to this method, V_c is the lesser of V_{ci} or V_{cw}.

where

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

17.7.11.2.2 Calculate V_{ci}

$$V_{ci} = 0.6\sqrt{f_c'}\,b_w d + V_D + \frac{V_i M_{cr}}{M_{max}} \qquad \text{[AREMA Eq. 17-10]}$$

but not less than $1.7\sqrt{f_c'}\,b_w d$

where

V_D = shear at section due to service dead load = $V_g + V_{SDL}$ = 20.0 + 18.4 = 38.4 kips

M_{cr} = moment causing flexural cracking at section due to externally applied loads

$$= S_b\left(6\sqrt{f_c'} + f_{pe} - f_d\right)$$

where

f_{pe} = compressive stress in concrete due to effective prestress force only, at the extreme fiber of section where tensile stress is caused by externally applied loads

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}e_c}{S_b}$$

$$= \frac{1{,}096.9}{1{,}452} + \frac{1{,}096.9(7.96)}{11{,}248} = 0.755 + 0.776 = 1.531 \text{ ksi}$$

f_d = stress due to unfactored dead load at extreme fiber of section where tensile stress is caused by externally applied loads

$$f_d = \frac{M_g + M_{SDL}}{S_b} = \frac{(26.6 + 24.5)12}{11{,}248} = 0.055 \text{ ksi}$$

$$M_{cr} = \left(\frac{6\sqrt{7{,}000}}{1{,}000} + 1.531 - 0.055\right)\frac{11{,}248}{12} = 1{,}854.0 \text{ ft-kips}$$

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max} = $V_u - V_D$ = 405.4 − 38.4 = 367.0 kips

M_{max} = maximum factored moment at the section due to externally applied loads = $M_u - M_g - M_{SDL}$ = 525.4 − 26.6 − 24.5 = 474.3 ft-kips

$$V_{ci} = 0.6\sqrt{f_c'}\,b_w d + V_D + \frac{V_i M_{cr}}{M_{max}} \qquad \text{[AREMA Eq. 17-10]}$$

$$= 0.6\frac{\sqrt{7{,}000}}{1{,}000}(18.5)(26.58) + 38.4 + \frac{367.0(1{,}854.0)}{474.3} = 1{,}497.7 \text{ kips}$$

but not less than $1.7\sqrt{f'_c}b_w d = 1.7\frac{\sqrt{7,000}}{1,000}(18.5)(26.58) = 69.9$ kips

Therefore,

V_{ci} = 1,497.7 kips

17.7.11.2.3 Calculate V_{cw}

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b_w d + V_p \qquad \text{[AREMA Eq. 17-11]}$$

where

f_{pc} = compressive stress in the concrete (after allowance for all pretension losses) at the centroid of cross section resisting externally applied loads

V_p = vertical component of effective prestress force at section
= 0 for straight strands.

Transfer length of strands = 50 strand diameters = 50(0.5) = 25 in. from end of beam.

Since the distance h/2 = 15.25 in. is closer to end of member than the end of the transfer length of the prestressing strands, a reduced pretensioning force will be considered when computing V_{cw}. [AREMA Art. 17.5.9c(2)(c)]

Effective prestress force at distance h/2 from centerline of the bearing,

$$P_{se} = \frac{(15.25 + 6.00)}{25}(1,096.9) = 932.4 \text{ kips}$$

$$f_{pc} = \frac{932.4}{1,452} = 0.642 \text{ ksi}$$

Therefore,

$$V_{cw} = \left(\frac{3.5\sqrt{7,000}}{1,000} + 0.3(0.642)\right)(18.5)(26.58) + 0 = 238.7 \text{ kips}$$

17.7.11.2.4 Calculate V_c

V_c = lesser of V_{ci} and V_{cw}

$V_c = V_{cw}$ = 238.7 kips

17.7.11.3 Calculate V_s and Shear Reinforcement

$$V_s = \frac{V_u}{\phi} - V_c = \frac{405.4}{0.9} - 238.7 = 211.7 \text{ kips} \qquad \text{[AREMA Eq. 17-8]}$$

17.7.11.3.1 Calculate V_s

17.7.11.3.2 Determine Stirrup Spacing

Required stirrup spacing is calculated as follows:

$$V_s = \frac{A_v f_y d}{s} \qquad \text{[AREMA Eq. 17-14]}$$

where A_v = area of shear reinforcement within a spacing, s

Try two closed stirrups, which provides (4) # 4 bars,
$A_v = 4(0.20)\text{ in.}^2 = 0.80\text{ in.}^2$

Stirrups are provided at 4 in. spacing to satisfy the minimum flexural requirements of the top slab of the box beam. Calculations for the top slab flexural reinforcement are not provided in this example.

$$\text{Spacing required, } s = \frac{A_v f_y d}{V_s} = \frac{0.80(60)(26.58)}{210.4} = 6.1\text{ in.} > 4\text{ in.} \qquad \text{O.K.}$$

Use # 4 stirrups (4 legs) at 4-in. centers.
A_v provided = 4(0.20) = 0.80 in.^{2}

Shear strength provided by stirrups,

$$V_s = \frac{0.80(60)(26.58)}{4} = 319.0 > 210.4\text{ kips} \qquad \text{O.K.}$$

17.7.11.3.3 Check V_s Limit

Allowable maximum shear strength provided by stirrups is:

$$8\sqrt{f'_c}\,b_w d = 8\sqrt{7{,}000}(18.5)(26.58)/1{,}000 \qquad \text{[AREMA Art. 17.5.9d(5)]}$$

= 329.1 kips > V_s O.K.

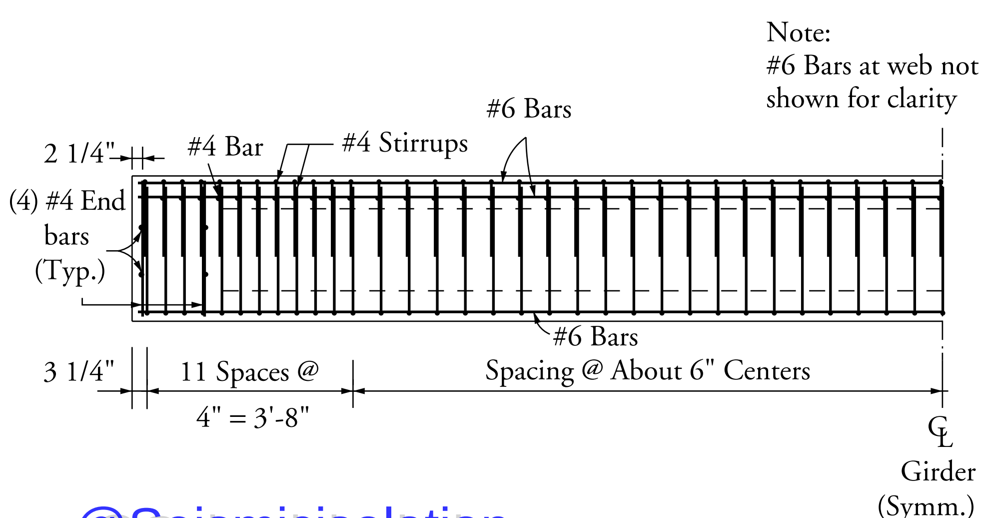
17.7.11.3.4 Check Stirrup Spacing Limits

Check for maximum spacing of stirrups

$$4\sqrt{f'_c}\,b_w d = 4\frac{\sqrt{7{,}000}}{1{,}000}(18.5)(26.58) = 164.6\text{ kips} < V_s \qquad \text{[AREMA Art. 17.5.9d(3)]}$$

Therefore, maximum spacing is lesser of 3/8h = 3/8(30.5) = 11.4 in. or 12 in.
Provide # 4 stirrups (4 legs) at 4-in. centers < 11.4 in. O.K.
Calculations for shear at other sections along the beam are not provided in this example.
For shear reinforcement details, see **Figures 17.7.4-1** and **17.7.11.3.4-1**

Figure 17.7.11.3.4-1
Elevation Showing
Non-Prestressed Reinforcement



17.7.12 Deflections

Camber and deflection calculations are required to determine the bridge seat elevations and maintain the minimum ballast depth. They are also required for the design of the elastomeric bearings.

17.7.12.1 Camber Due to Prestressing at Transfer

$$\Delta = \frac{P_{si}e_cL^2}{8E_{ci}I} = -\frac{1,219.7(7.96)(29(12))^2}{8(3,834)(171,535)} = -0.223 \text{ in.} \uparrow$$

17.7.12.2 Deflection Due to Beam Self-Weight at Transfer

$$\Delta = \frac{5wL^4}{384E_{ci}I} = \frac{5(1.513/12)(29(12))^4}{(384)(3,834)(171,535)} = 0.037 \text{ in.} \downarrow$$

17.7.12.3 Deflection Due to Superimposed Dead Load

$$\Delta = \frac{5wL^4}{384E_cI} = \frac{5(1.389/12)(29(12))^4}{(384)(5,072)(171,535)} = 0.025 \text{ in} \downarrow$$

17.7.12.4 Long-Term Deflection

According to *PCI Design Handbook* - 5th Edition, Table 4.8.2, long-term camber and deflection of prestressed concrete members can be calculated by an approximate method using multipliers. Calculations are shown in **Table 17.7.12.4-1**.

Table 17.7.12.4-1
Calculated Deflection, in.

	At Release (a)	Multiplier (b)	Erection (c) = (a)(b)	Multiplier (d)	Final (e) = (a)(d)
Prestress	− 0.223 ↑	1.80	− 0.401 ↑	2.45	− 0.546 ↑
Self-Weight	+ 0.037 ↓	1.85	+ 0.068 ↓	2.70	+ 0.100 ↓
Dead Load	N/A		+ 0.025 ↓	3.00	+ 0.075* ↓
Total	− 0.186 ↑		− 0.308 ↑		− 0.371 ↑

* This is the result of multiplying the dead load deflection at erection (c) by multiplier (d)

17.7.12.5 Deflection Due to Live Load

Live load deflection is generally calculated using influence lines. At this point, use of a computer program becomes very useful. However, for short span bridges, the designer can quickly calculate an approximate value for deflection by using the equivalent uniform load. The equivalent uniform live load, w_{equ}, for a simply supported beam can be derived from the maximum moment at midspan,

$$M_{LL+I} = \frac{w_{equ}L^2}{8}$$

$$w_{equ} = \frac{8M_{LL+I}}{L^2} = \frac{8(1,033.0)(12)}{(29(12))^2} = 0.819 \text{ kip/in.}$$

$$\Delta = \frac{5(0.819)(29(12))^4}{384(5,072)(171,535)} = 0.180 \text{ in.} \downarrow$$

$$\text{Maximum allowable deflection} = \frac{L}{640}$$

$$= \frac{29(12)}{640} = 0.544 \text{ in.} > 0.180 \text{ in.} \quad \text{O.K.}$$ [AREMA Art. 17.6.7a]

17.8 REFERENCES

AREMA Manual for Railway Engineering, 2000 Edition, American Railway Engineering and Maintenance-of-Way Association, Landover, MD, 2000

Marianos, W. N., Jr., "Railroad Use of Precast Concrete Bridge Structures," ACI Concrete International, V. 13, No. 9, September 1991, pp. 30-35

PCI Design Handbood, Fifth Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1999, 690 pp.

NOTATION

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
A	area of cross section of stringer or beam	A (4.6.2.2.1)	—
A	constant	—	—
A	maximum area of the portion of the supporting surface that is similar to the loaded area and concentric with it and does not overlap similar areas for adjacent anchorage devices	A (5.10.9.7.2)	—
A	effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires; when the flexural reinforcement consists of several bar sizes or wires, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used	A (5.7.3.4)	A (8.16.8.4)
A_b	net area of a bearing plate	A_b (5.10.9.7.2)	—
A_c	area of core of spirally reinforced compression member measured to the outside diameter of the spiral	A_c (5.7.4.6)	A_c (8.18.2.2.2)
A_c	total area of the composite section	—	—
A_c	area of concrete on the flexural tension side of the member	—	—
A_{cs}	cross-sectional area of a concrete strut in strut-and-tie model	A_{cs} (5.6.3.3.1)	—
A_{cv}	area of concrete section resisting shear transfer	A_{cv} (5.8.4.1)	A_{cv} (8.16.6.4.5)
A_g	gross area of section	A_g (5.5.4.2.1)	A_g (8.1.2)
A_g	gross area of bearing plate	A_g (5.10.9.7.2)	—
A_h	area of shear reinforcement parallel to flexural tension reinforcement	A_h (5.13.2.4.1)	A_h(8.15.5.8, 8.16.6.8)
A_o	area enclosed by centerlines of the elements of the beam	C4.6.2.2.1	—
A_{ps}, A_s^*	area of prestressing steel	A_{ps} (5.5.4.2.1)	A_s^* (9.17)
A_{PT}	transverse post-tensioning reinforcement	—	—
A_s	area of non-prestressed tension reinforcement	A_s (5.5.4.2.1)	A_s (9.7, 9.19)
A_s	total area of vertical reinforcement located within the distance (h/5) from the end of the beam	—	—
A_s'	area of compression reinforcement	A_s' (5.7.3.1.1)	A_s' (9.19)
A_{sf}	steel area required to develop the compressive strength of the overhanging portions of the flange	—	A_{sf} (9.17)
A_{sk}	area of skin reinforcement per unit height in one side face	A_{sk} (5.7.3.4)	A_{sk} (8.17.2.1.3)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
A_{sr}	steel area required to develop the compressive strength of the web of a flanged section	—	A_{sr} (9.17-9.19)
A_{ss}	area of reinforcement in an assumed strut of a strut-and-tie model	A_{ss} (5.6.3.3.4)	—
A_{st}	total area of longitudinal mild steel reinforcement	A_{st} (5.6.3.4.1)	A_{st} (8.16.4.1.2, 8.16.4.2.1)
A_t	area of one leg of closed transverse torsion reinforcement	A_t (5.8.3.6.2)	—
A_v	area of transverse reinforcement within a distance s	A_v (5.8.2.5)	A_v (9.20)
A_{vh}	area of web reinforcement required for horizontal shear	—	—
A_{vf}	area of shear-friction reinforcement	A_{vf} (5.8.4.1)	A_{vf} (8.15.5.4.3)
A_{vf}	total area of reinforcement, including flexural reinforcement	A_{vf} (5.10.11.4.4)	—
$A_{v\text{-}min}$	minimum area of web reinforcement	—	—
a	distance from the end of beam to drape point	—	—
a	depth of equivalent rectangular stress block	a (5.7.2.2)	a (8.16.2.7, 9.17.2)
a	lateral dimension of the anchorage device measured	a (5.10.9.6.2)	—
a_f	distance between concentrated load and face of support	a_f (5.13.2.5.1)	—
a_v	shear span, distance between concentrated load and face of support	a_v (5.13.2.4.1)	a_v (8.15.5.8, 8.16.6.8)
B	constant	—	—
B	buoyancy	—	B (3.22)
BR	vehicular braking force	BR (3.3.2)	—
b	the lateral dimension of the anchorage device measured parallel to the smaller dimension of the cross-section	b (5.10.9.6.2)	—
b	width of bottom flange of the beam	—	—
b	effective flange width	—	—
b	width of beam	b (4.6.2.2.1)	—
b	width of compression face of member	b (5.7.3.1.1)	b (8.1.2)
b	width of pier or diameter of pile	—	b (3.18.2.2.4)
b′	width of web of a flanged member	—	b′ (9.1.2)
b_v , b_e	effective web width of the precast beam	b_v (5.8.2.7)	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
b_v	width of cross section at the contact surface being investigated for horizontal shear	b_v (5.8.4.1)	b_v (9.20)
b_w	width of a web of a flanged member	b_w (5.7.3.1.1)	b_w (8.15.5.1.1)
b_w	width of web adjusted for the presence of ducts	b_w (5.8.2.5)	—
C	stiffness parameter = K(W/L)	—	C (3.23.4.3)
C	centrifugal force in percent of live load	—	C (3.10.1)
C_a	creep coefficient for deflection at time of erection due to loads applied at release	—	—
C_u	ultimate creep coefficient	—	—
C_u'	ultimate creep coefficient for concrete at time of application of the superimposed dead loads	—	—
CE	vehicular centrifugal force	CE (3.3.2)	—
CF	centrifugal force	—	CF (3.22)
CR	creep	CR (3.3.2)	—
CT	vehicular collision force	CT (3.3.2)	—
CV	vessel collision force	CV (3.3.2)	—
$C(t,t_o)$	creep coefficient at a concrete age of t days	—	—
c	cohesion factor	c (5.8.4.1)	—
c	vehicular braking force	—	—
c	distance from extreme compression fiber to neutral axis	c (5.7.2.2)	c (8.16.2.7)
D	parameter used in determination of load fraction of wheel load	—	D (3.23.4.3)
D	prestressing steel elongation	—	—
D	a constant that varies with bridge type and geometry	—	—
D	width of distribution per lane	—	—
D	dead load	D (3.3.2)	D (3.22)
DC	dead load of structural components and nonstructural attachments	DC (5.14.2.3.2)	—
DD	downdrag	DD (3.3.2)	—
D.F.	fraction of wheel load applied to beam	—	D.F. (3.28.1)
DFD	distribution factor for deflection	—	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
DFM	distribution factor for bending moment	—	—
DF_m	live load distribution factor for moment	—	—
DFV	distribution factor for shear force	—	—
DL	contributing dead load	—	DL (3.1)
DW	dead load of wearing surfaces and utilities	DW (3.3.2, 5.14.2.3.2)	—
d	distance from extreme compressive fiber to centroid of the pretensioning force	—	d (9.1.2)
d	depth of beam or stringer	d (4.6.2.2.1)	—
d	precast beam depth	—	—
d	distance from extreme compressive fiber to centroid of the reinforcing but not less than 0.8h . In negative moment section, the reinforcement is assumed to be located at the mid-height of the slab. For computing horizontal shear strength of composite members, d should be the distance from extreme compression fiber to centroid of tension reinforcement for entire composite section.	—	d (9.1.2)
d_b	nominal diameter of a reinforcing bar or wire	d_b (5.10.2.1)	d_b (8.1.2)
d_b	nominal diameter of prestressing steel	d_b (5.10.2.1)	D (9.17, 9.27)
d_c	thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto	d_c (5.7.3.4)	d_c (8.16.8.4)
d_e	distance between the center of exterior beam and interior edge of curb or traffic barrier	d_e (4.6.2.2.1)	—
d_e	effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement	d_e (5.7.3.3.1)	—
d_{ext}	depth of the extreme steel layer from extreme compression fiber	—	—
d_i	depth of steel layer from extreme compression fiber	—	—
d_p	distance from extreme compression fiber to the centroid of the prestressing tendons	d_p (5.7.3.1.1)	—
d_s	distance from extreme compression fiber to the centroid of the non-prestressed tensile reinforcement	d_s (5.7.3.2.2)	d_t (9.7, 9.17-9.19)
d_v	effective shear depth	d_v (5.8.2.7)	—
d_s'	distance from extreme compression fiber to centroid of compression reinforcement	d_s' (5.7.3.2.2)	d' (8.1.2)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
d"	distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement	—	d" (8.1.2)
E	earth pressure	—	—
E	width of slab over which a wheel load is distributed	—	E (3.24.3)
E_b	the modulus of elasticity of the bearing plate material	E_b (5.10.9.7.2)	—
E_c	modulus of elasticity of concrete	E_c (5.4.2.4)	E_c (3.26.3, 8.7.1)
E_{ci}	modulus of elasticity of concrete at transfer	E_{ci} (5.9.5.2.3a)	—
E_{eff}	effective modulus of elasticity	E_{eff} (C5.14.2.3.6)	—
E_p, E_s	modulus of elasticity of pretensioning reinforcement	E_p (5.4.4.2)	E_s (9.16.2.1.2)
E_s	modulus of elasticity of non-pretensioned reinforcement	E_s (5.4.3.2)	E_s (3.26.3, 8.7.2)
E_c^*	age adjusted effective modulus of concrete for a gradually applied load at the time of release of prestressing	—	—
EH	horizontal earth pressure load	EH (3.3.2)	—
EQ	earthquake	EQ (3.3.2)	EQ (3.22.1)
EQ	equivalent static horizontal force applied at the center of gravity of the structure	—	EQ (3.1)
ES	earth surcharge load	ES (3.3.2)	—
EV	vertical pressure from dead load of earth fill	EV (3.3.2)	—
e	eccentricity of the strands at h/2	—	—
e	eccentricity of strands at transfer length	—	—
e	correction factor for distribution	e (4.6.2.2.1)	—
e	eccentricity of a lane from the center of gravity of the pattern of girders	e (4.6.2.2.2d)	—
e	the eccentricity of the anchorage device or group of devices, with respect to the centroid of the cross-section, always taken as positive	e (5.10.9.6.3)	—
e'	difference between eccentricity of pretensioning steel at midspan and end span	—	—
e_c	eccentricity of the strand at the midspan	—	—
e_e	eccentricity of pretensioning force at end of beam	—	—
e_g	distance between the centers of gravity of the basic beam and deck	e_g (4.6.2.2.1)	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
e_p	eccentricity of the prestressing strands with respect to the centroid of the section	—	—
F_b	allowable tensile stress in the precompressed tensile zone at service loads	—	—
F_b	allowable bending stress	—	F_b (2.7.4.2)
F_{cj}	force in concrete for the j th component	—	—
FR	friction	FR (3.3.2)	—
F_{pi}	total force in strands before release	—	—
F_ε	reduction factor	F_ε (5.8.3.4.2)	—
f	stress	—	—
f_D	sum of dead load bending stresses	—	—
$f_{(L+I)}$	live load plus impact bending stress	—	—
f_b	concrete stress at the bottom fiber of the beam	—	—
f_b	average bearing stress in concrete on loaded area	—	f_b (8.15.2.1.3, 8.16.7.1)
f_c	extreme fiber compressive stress in concrete at service loads	—	f_c (8.15.2.1.1)
f'_c	specified compressive strength of concrete at 28 days, unless another age is specified	f'_c (5.4.2.1)	f'_c (8.1.2)
f_{ca}	concrete compressive stress ahead of the anchorage devices	f_{ca} (5.10.9.6.2)	—
f_{cds}	average concrete compressive stress at the c.g. of the prestressing steel under full dead load	—	f_{cds} (9.16)
f_{cgp}	concrete stress at the center of gravity of pretensioning tendons, due to pretensioning force at transfer and the self-weight of the member at the section of maximum positive moment	f_{cgp} (5.9.5.2.3a)	f_{cir} (9.16)
f'_{ci}	compressive strength of concrete at time of initial prestress	f'_{ci} (5.9.1.2)	f'_{ci} (9.15)
f_{ct}	average splitting tensile strength of lightweight aggregate concrete	f_{ct} (5.8.2.2)	f_{ct} (9.1.2)
$(f'_c)t$	compressive strength of concrete at t days	—	—
f_{cu}	the limiting concrete compressive stress for design by strut-and-tie model	f_{cu} (5.6.3.3.1)	—
f_d	stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads	—	f_d (9.20)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
f_f	fatigue stress range in reinforcement	f_f (5.5.3.2)	f_f (8.16.8.3)
f_{min}	algebraic minimum stress level in reinforcement	f_{min} (5.5.3.2)	f_{min} (8.16.8.3)
f_n	nominal concrete bearing stress	f_n (5.10.9.7.2)	
f_{pb}	compressive stress at bottom fiber of the beam due to prestress force	—	—
f_{pc}	compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.)	f_{pc}(C5.6.3.5)	f_{pc} (9.20)
f_{pe}	effective prestress after losses	f_{pe} (5.6.3.4.1)	f_{se}
f_{pe}	compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads	—	f_{pe} (9.20)
f_{pi}	initial stress immediately before transfer	—	—
f_{pj}	stress in the prestressing steel at jacking	f_{pj} (5.9.3)	—
f_{po}	stress in the prestressing steel when the stress in the surrounding concrete is 0.0	f_{po} (5.8.3.4.2)	—
f_{ps}	average stress in prestressing steel at the time for which the nominal resistance of member is required	f_{ps} (C5.6.3.3.3)	—
f_{pt}	stress in prestressing steel immediately after transfer	f_{pt} (5.9.3)	—
f_{pu}, f'_s	ultimate strength of prestressing steel	f_{pu} (5.4.4.1)	f'_s (9.15, 9.17)
f_{py}	yield point stress of prestressing steel	f_{py} (5.4.4.1)	f^*_y (9.15)
f_r	the modulus of rupture of concrete	f_r (5.4.2.6)	f_r (9.18, 8.15.2.1.1)
f_s	allowable stress in steel, but not taken greater than 20 ksi	—	—
f_s	tensile stress in reinforcement at service loads	—	f_s (8.15.2.2)
f_{sa}	tensile stress in the reinforcement at service loads	f_{sa} (5.7.3.4)	—
f_{se}	effective final pretension stress	—	—
f_{si}	effective initial pretension stress	—	—
f^*_{su}	stress in prestressing tension steel at ultimate load	—	f^*_{su}

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
f_t	extreme fiber tensile stress in concrete at service loads	—	f_t (8.15.2.1.1)
f_t	concrete stress at top fiber of the beam for the non-composite section	—	—
f_{tc}	concrete stress at top fiber of the slab for the composite section	—	—
f_{tg}	concrete stress at top fiber of the beam for the composite section	—	—
f_y	specified yield strength of non-prestressed conventional reinforcement	f_y (5.5.4.2.1), f_y'(5.7.3.1.1)	f_y (8.1.2), f_y' (9.19), f_{sy} (9.19, 9.20)
f_{yh}	specified yield strength of transverse reinforcement	f_{yh} (5.7.4.6)	—
g	A factor used to multiply the total longitudinal response of the bridge due to a single longitudinal line of wheel loads in order to determine the maximum response of a single girder	—	—
g	distribution factor	g (4.6.2.2.1)	—
H	average annual ambient mean relative humidity, percent	H (5.4.2.3.2)	RH (9.16.2.1.1)
H	height of wall	H (A13.4.2)	—
h	overall thickness or depth of a member	h (5.8.2.7)	h (9.20)
h_c	total height of composite section	—	—
h_f	compression flange thickness	h_f (5.7.3.1.1)	h_f (8.1.2)
I	moment of inertia about the centroid of the non-composite precast beam	I_g (5.7.3.6.2)	I (9.20)
I_c	moment of inertia of composite section	—	—
I_{cr}	moment of inertia of cracked section transformed to concrete	I_{cr} (5.7.3.6.2)	I_{cr} (8.13.3)
I_e	effective moment of inertia	I_e (5.7.3.6.2)	I_e (8.13.3)
I_g	moment of inertia of the gross concrete section about the centroidal axis, neglecting reinforcement	I_g (5.7.3.6.2)	I_g (3.23.4.3, 8.1.2, 9.20)
I_s	moment of inertia of reinforcement about centroidal axis of member cross section	I_s (5.7.4.3)	I_s (8.1.2)
IC	ice load	—	—
ICE	ice pressure	—	ICE (3.22.1)
IM	vehicular dynamic load allowance	IM (3.6.1.2.5)	I (3.8.2)

SYMBOL	***DESCRIPTION***	***CORRESPONDING AASHTO LRFD SPECIFICATIONS***	***CORRESPONDING AASHTO STANDARD SPECIFICATIONS***
J	gross St. Venant torsional constant of the precast member	J (4.6.2.2.1)	J (3.23.4.3)
K	a non-dimensional constant	—	—
K	effective length factor for compression members	K (5.7.4.1)	k (8.16.5.2.3)
K	factor used for calculating time-dependent losses	—	—
K	wobble friction coefficient	K (5.9.5.2.2b)	K (9.16)
K_g	longitudinal stiffness parameter	K_g (4.6.2.2.1)	K (3.23.4)
K_r	factor used for calculating relaxation loss occurs prior to transfer		
k	factor used in calculation of distribution factor for multi-beam bridges	k (4.6.2.2)	—
k	factor used in calculation of average stress in pretensioning steel for Strength Limit State	—	—
k	live load distribution constant for spread box girders	—	k (3.28.1)
k_c	product of applicable correction factors = $k_{\ell a}$ (k_h) (k_s)	—	—
k_c	a factor for the effect of the volume-to-surface ratio	k_c (5.4.2.3.2)	—
k_{cp}	correction factor for curing period	—	—
k_f	a factor for the effect of concrete strength	k_f (5.4.2.3.2)	—
k_h	correction factor for relative humidity	k_h (5.4.2.3.3)	—
k_{la}	correction factor for loading age	—	—
k_s	product of applicable correction factors = k_{cp} (k_h) (k_s)	—	—
k_s	correction factor for size	k_s (5.4.2.3.3)	—
L	length in feet of the span under consideration for positive moment and the average of two adjacent loaded spans for negative moment	—	L (3.8.2.1)
L	Overall beam length or design span	—	—
L	span length measured parallel to longitudinal beams	—	—
L	length of the loaded portion of span from section under consideration to the far reaction when computing shear due to truck loads.	—	—
L	loaded length of span	L (5.7.3.1.2)	L (3.8.2)
LL, L	live load	LL (3.3.2)	L (3.22)
L	total length of prestressing steel from anchorage to anchorage.	—	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
L_c	critical length of yield line failure pattern	L_c (A13.4.2)	—
LF	longitudinal force from live load	—	LF (3.22)
LS	live load surcharge	LS (3.3.2)	—
L_r	intrinsic relaxation of the strand	—	—
ℓ_d	development length	—	—
ℓ_t	transfer length	—	—
ℓ_u	unsupported length of compression member	ℓ_u (5.7.4.1)	ℓ_u (8.16.5.2.1)
M_b	unfactored bending moment due to barriers weight	—	—
M_c	flexural resistance of cantilevered wall	—	—
M_{CIP}	unfactored bending moment due to cast-in-place topping slab	—	—
M_{const}	unfactored bending moment due to construction load	—	—
M_{cr}	moment causing flexural cracking at section due to externally applied loads	M_{cr} (5.7.3.6.2)	M_{cr} (8.13.3, 9.20)
M_{cr}^*	cracking moment	—	M_{cr}^* (9.18)
M_D	unfactored bending moment due to diaphragm weight	—	—
M_d	bending moment at section due to unfactored dead load	—	—
$M_{d/nc}$	non-composite dead load moment	—	$M_{d/nc}$ (9.18)
M_f	unfactored bending moment due to fatigue truck per beam	—	—
M_g	unfactored bending moment due to beam self-weight	—	—
M_{LL}	unfactored bending moment due to lane load per beam	—	—
M_{LL+I}	unfactored bending moment due to live load + impact	—	—
M_{LT}	unfactored bending moment due to truck load with dynamic allowance per beam	—	—
M_{max}	maximum factored moment at section due to externally applied loads	—	M_{max} (9.20)
M_n	nominal flexural resistance	M_n (5.7.3.2.1)	M_n (9.1.2)
$M_{n/dc}$	non-composite dead load moment at the section	—	—
M_r	factored flexural resistance of a section in bending	M_r (5.7.3.2.1)	—
$M_{service}$	total bending moment for service load combination	—	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
M_S	unfactored bending moment due to slab and haunch weights	—	—
M_{SDL}	unfactored bending moment due to super-imposed dead loads	—	—
M_{SIP}	unfactored bending moment due to stay-in-place panel	—	—
M_u	factored moment at section $\leq \phi M_n$	M_u (C5.6.3.1)	M_u (9.17, 9.18)
M_{sw}	moment at section of interest due to self-weight of the member plus any permanent loads acting on the member at time of release	—	—
M_{ws}	unfactored bending moment due to wearing surface	—	—
M_x	bending moment at a distance (x) from the support	—	—
m	material parameter	—	—
m	stress ratio = (f_y / 0.85)	—	—
N	group number	—	N (3.22.1)
N_b, N_B	number of beams	N_b (4.6.2.2.1)	N_B (3.28.1)
N_L	number of traffic lanes	N_L (4.6.2.2.1, 3.6.1.1.1)	N_L (3.23.4)
NL	number of traffic lanes	—	—
N_u	applied factored axial force taken as positive if tensile	—	—
N_{uc}	factored axial force normal to the cross section, occurring simultaneously with V_u to be taken as positive for tension, negative for compression; includes effects of tension due to creep and shrinkage	N_{uc} (5.13.2.4.1)	N_u (8.16.6.2.2)
n	modular ratio of elasticity - E_s/E_c	n (5.7.1)	n (8.15.3.4)
P	concentrated wheel load	P (3.6.1.2.5)	—
P	live load intensity	P (C3.11.6.2)	—
P	live load on sidewalk	—	P (3.14.1.1)
P	load on one rear wheel of truck	—	P (3.24.3)
P	Diaphragm weight concentrated at quarter points	—	—
P_c	permanent net compression force	P_c (5.8.4.1)	—
P_{eff}	effective post-tensioning force	—	—
P_i	total pretensioning force immediately after transfer	—	—
P_n	nominal axial load strength at given eccentricity	P_n (5.5.4.2.1)	P_n (8.1.2)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
P_n	nominal axial resistance of strut or tie	P_n (5.6.3.2)	—
P_n	nominal bearing resistance	P_n (5.7.5)	—
P_{nx}	nominal axial load strength corresponding to M_{nx}, with bending considered in the direction of the x axis only	—	P_{nx} (8.16.4.3)
P_{ny}	nominal axial load strength corresponding to M_{ny}, with bending considered in the direction of the y axis only	—	P_{ny} (8.16.4.3)
P_{nxy}	nominal axial load strength with biaxial loading	—	P_{nxy} (8.16.4.3)
P_o	nominal axial load strength of a section at 0.0 eccentricity	P_o (5.7.4.5)	P_o (8.16.4.2.1)
P_{pe}	total pretensioning force after all losses	—	—
P_s	prestress force before initial losses	—	—
P_s	design jacking force	—	—
P_{se}	effective pretension force after allowing for all losses	—	—
P_{si}	effective pretension force after allowing for the initial losses	—	—
P_r	factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement	—	—
P_T	factored axial resistance of strut or tie	P_r (5.6.3.2)	—
P_T	factored bursting resistance of pretensioned anchorage t zone provided by transverse reinforcemen	P_r (5.10.10.1)	—
PL	pedestrian live load	PL (3.3.2)	—
p	fraction of truck traffic in a single lane	p (3.6.1.4.2)	—
p	A_s'/bd, ratio of non-prestressed tension reinforcement	—	p (9.7, 9.17-9.19)
p'	A_s'/bd, ratio of compression reinforcement	—	p' (9.19)
p^*	A_s^*/bd, ratio of prestressing steel	—	p^* (9.17, 9.19)
p_c	outside perimeter of the concrete section	p_c (5.8.2.1)	—
p_h	perimeter of the centerline of the closed transverse torsion reinforcement	p_h (5.8.3.6.2)	—
Q	first moment of inertia of the area above the fiber being considered	—	—
Q	statical moment of cross sectional area, above or below the level being investigated for shear, about the centroid	—	Q (9.20)
Q	total factored load	Q (3.4.1)	—
q	generalized load	q (3.4.1)	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
q_i	specified loads	q_i (3.4.1)	—
R	reaction on exterior beam in terms of lanes	—	—
R	rib shortening	—	R (3.22)
R_u	flexural resistance factor	—	—
R_w	total transverse resistance of the railing	R_w (A13.4.2)	—
r	radius of gyration of cross section of a compression member	r (5.7.4.1)	r (8.16.5.2.2)
r/h	ratio of base radius to height of rolled-on transverse deformations	r/h (5.5.3.2)	—
S	coefficient related to site conditions for use in determining seismic loads)	S (3.10.5)	—
S	surface area of concrete exposed to drying	—	—
S	shrinkage	—	S (3.22)
S	spacing of beams	—	S (3.23.3, 3.28.1)
S	effective span length of the deck slab	—	S (3.25.1.3)
S	width of precast member	—	S (3.23.4.3)
S_b	noncomposite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads	—	S_b (9.18)
S_{bc}	composite section modulus for extreme bottom fiber of the precast beam, I_c/y_{bc}	—	—
S_c	composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads	—	S_c (9.18)
S_t	section modulus for the extreme top fiber of the non-composite precast beam	—	—
S_{tc}	composite section modulus for top fiber of the slab, $I_c/(n)(y_{tc})$	—	—
S_{tg}	composite section modulus for top fiber of the precast beam, I_c/y_{tg}	—	—
S_u	ultimate free shrinkage strain in the concrete adjusted for member size and relative humidity	—	—
$S(t,t_o)$	shrinkage strain at a concrete age of t days	—	—
SE	settlement	SE (3.3.2)	—
SF	stream flow pressure	—	SF (3.22)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
SH	shrinkage	SH (3.3.2, 5.14.2.3.2)	S (3.22)
SR	fatigue stress range	—	—
s	spacing of shear reinforcement in direction parallel to the longitudinal reinforcement	s(5.8.4.1)	s (9.20)
s	effective deck span	—	s (3.25.1.3)
s	length of a side element	s (C4.6.2.2.1)	—
s	spacing of rows of ties	s (5.8.4.1)	—
T	collision force at deck slab level	—	—
T	mean daily air temperature	T (C3.9.2.2)	—
T_{burst}	the tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis	T_{burst} (5.10.9.6.3)	—
T_{cr}	torsional cracking resistance	T_{cr} (5.8.2.1)	—
T_n	nominal torsion resistance	T_n (5.8.2.1)	—
T_T	factored torsional resistance provided by circulatory shear flow	T_r (5.8.2.1)	—
T_u	factored torsional moment	T_u (C5.6.3.1)	—
TG	temperature gradient	TG (3.3.2, C4.6.6)	—
TU	uniform temperature	TU (3.3.2)	—
t	thickness of web	—	—
t	thickness of an element of the beam	—	—
t	time in days	t (5.4.2.3.2)	—
t	average thickness of the flange of a flanged member	—	t (9.17, 9.18)
t	deck thickness	—	t (3.25.1.3)
t_f	thickness of flange	—	—
t_i	age of concrete when load is initially applied	t_i (5.4.2.3.2)	—
t_{la}	loading ages in days	—	—
t_s	depth of concrete slab	t_s (4.6.2.2.1)	—
t_o	age of concrete in days at the end of the initial curing period	—	—
V	design shear force at section	—	V (8.15.5.1.1)

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
V	variable spacing of truck axles	—	V (3.7.6)
V	volume of concrete	—	—
V_c	nominal shear resistance provided by tensile stresses in the concrete	V_c (5.8.2.4)	V_c (9.20, 8.16.6.1)
V_{ci}	nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	—	V_{ci} (9.20)
V_{cw}	nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	—	V_{cw} (9.20)
V_d	shear force at section due to unfactored dead load	—	V_d (9.20)
V_i	factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	—	V_i (9.20)
V_{LL}	unfactored shear force due to lane load per beam	—	—
V_{LL+I}	unfactored shear force due to live load plus impact	—	—
V_{LT}	unfactored shear force due to truck load with dynamic allowance per beam	—	—
V_{mu}	ultimate shear force occurring simultaneously with M_u	—	—
V_n	nominal shear resistance of the section considered	V_n (5.8.2.1)	V_n (8.16.6.1)
V_{nh}	nominal horizontal shear strength	—	V_{nh}(8.16.6.5.3, 9.20)
V_p	component in the direction of the applied shear of the effective pretensioning force, positive if resisting the applied shear	V_p (C5.8.2.3)	V_p (9.20)
V_s	shear resistance provided by shear reinforcement	V_s (5.8.3.3)	V_s (8.16.6.1, 9.20)
V_T	factored shear resistance	V_r (5.8.2.1)	—
V_u	factored shear force at section	V_u (C5.6.3.1)	V_u (8.16.6.1, 9.20)
V_{uh}	factored horizontal shear force per unit length of the beam	—	—
V_x	shear force at a distance (x) from the support	—	—
v	factored design shear stress	v (5.8.3.4.2)	v (8.15.5.1.1)
v	permissible horizontal shear stress	—	v (9.20)
v_c	permissible shear stress carried by concrete	—	v_c (8.15.5.2)
WS, W	wind load on structure	WS (3.3.2)	W (3.22)
W	overall width of bridge	—	W (3.23.4.3)

NOTATION

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
W	roadway width between curbs	—	W (3.28.1)
WA	water load and stream pressure	WA (3.3.2)	—
WL	wind load on live load	WL (3.3.2)	WL (3.22)
w	width of clear roadway	w (3.6.1.1.1)	—
w	a uniformly distributed load	—	—
w_b	weight of barriers	—	—
w_c	unit weight of concrete	w_c (5.4.2.4)	w_c (8.1.2)
w_g	beam self-weight	—	—
w_s	slab and haunch weights	—	—
w_{ws}	weight of future wearing surface	—	—
X	distance from load to point of support	—	X (3.24.5.1)
X_{ext}	horizontal distance from the center of gravity of the pattern of girders to the exterior girder	X_{ext} (C4.6.2.2.2d)	—
x	the distance from the support to the section under question	—	—
x	horizontal distance from the center of gravity of the pattern of girders to each girder	x (C4.6.2.2.2d)	—
x	length of prestressing steel element from jack end to point x	x (5.9.5.2.2b)	L (9.16)
y_b	distance from centroid to the extreme bottom fiber of the non-composite precast beam	—	—
y_{bc}	distance from the centroid of the composite section to extreme bottom fiber of the precast beam	—	—
y_{bs}	distance from the center of gravity of strands to the bottom fiber of the beam	—	—
y_t	distance from centroid to the extreme top fiber of the non-composite precast beam	—	—
y_t	distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension	y_t (5.7.3.6.2)	y_t (8.13.3, 9.20)
y_{tc}	distance from the centroid of the composite section to extreme top fiber of the slab	—	—
y_{tg}	distance from the centroid of the composite section to extreme top fiber of the precast beam	—	—
Z	crack control parameter	Z (5.7.3.4)	—

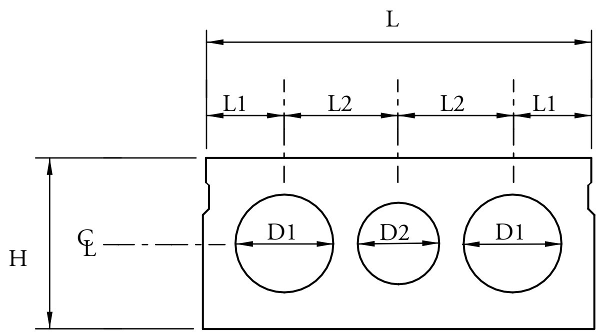
SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
Z	factor reflecting exposure conditions	—	—
α	angle of inclination of transverse reinforcement to longitudinal axis	α (5.8.3.3)	—
α	angle between inclined shear reinforcement and longitudinal axis of member	α (5.8.3.3)	α (8.1.2)
α	factor used in calculating elastic shortening loss	—	—
α	total angular change of prestressing steel path from jacking end to a point under investigation	α (5.9.5.2.2b)	α (9.16)
α	the angle of inclination of a tendon force, with respect to the centerline of the member	α (5.10.9.6.3)	—
α_h	total horizontal angular change of prestressing steel path from jacking end to a point under investigation	α_h (5.9.5.2.2b)	—
α_s	angle between compressive strut and adjoining tension tie	α_s (5.6.3.3.3)	—
α_v	total vertical angular change of prestressing steel path from jacking end to a point under investigation	α_v (5.9.5.2.2b)	—
β	factor indicating ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution)	β (5.8.3.3)	—
β	factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension	β (5.8.3.3)	—
β_b	ratio of area of reinforcement cut off to total area of reinforcement at the section	β_b (5.11.1.2.1)	β_b (8.24.1.4.2)
β_d	absolute value of ratio of maximum dead load moment to maximum total load moment, always positive	β_d (5.7.4.3)	β_d (8.1.2)
β_D	load combination coefficient for dead loads	—	β_D (3.22.1)
β_L	load combination coefficient for live loads	—	β_L (3.22.1)
β_1	ratio of depth of equivalent compression zone to depth	β_1 (5.7.2.2)	β_1(8.16.2.7,9.17-9.19)
Δ	deflection	—	—
Δ_{beam}	deflection due to beam self-weight	—	—
Δ_{b+ws}	deflection due to barrier and wearing surface weights	—	—
Δ_{fcdp}	change in concrete stress at c.g. of prestressing steel due to all dead loads, except dead load acting at the time the prestressing force is applied	Δ_{fcdp} (5.9.5.4.3)	—
Δf_{pA}	loss in prestressing steel stress due to anchorage set	Δf_{pA} (5.9.5.1)	—

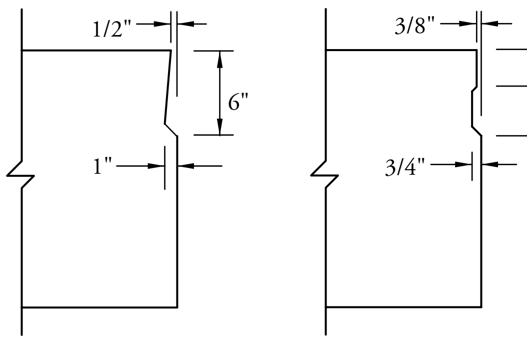
SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
Δf_{pCR}	loss in prestressing steel stress due to creep	Δf_{pCR} (5.9.5.1)	CR_c (9.16)
Δf_{pES}	loss in prestressing steel stress due to elastic shortening	Δf_{pES} (5.9.5.1)	ES (9.16)
Δf_{pF}	loss in prestressing steel stress due to friction	Δf_{pF} (5.9.5.1)	—
Δf_{pi}	total loss in pretensioning steel stress immediately after transfer	—	—
Δf_{pR}	total loss in prestressing steel stress due to relaxation of steel	Δf_{pR} (5.9.5.1)	CR_s (9.16)
Δf_{pR1}	loss in prestressing steel stress due to relaxation of steel at transfer	Δf_{pR1} (5.9.5.4.4b)	—
Δf_{pR2}	loss in prestressing steel stress due to relaxation of steel after transfer	Δf_{pR2} (5.9.5.4.4c)	—
Δf_{pSR}	loss in prestressing steel stress due to shrinkage	Δf_{pSR} (5.9.5.1)	SH (9.16)
Δf_{pT}	total loss in prestressing steel stress	Δf_{pT} (5.9.5.1)	—
Δf_s	total prestress loss, excluding friction	—	Δf_s (9.16)
Δ_D	deflection due to diaphragm weight	—	—
Δ_L	deflection due to specified live load	—	—
Δ_{LL+I}	deflection due to live load and impact	—	—
Δ_{LL}	deflection due to lane load	—	—
Δ_{LT}	deflection due to design truck load and impact	—	—
Δ_{max}	maximum allowable live load deflection	—	—
Δ_p	camber due pretension force at transfer	—	—
Δ_{SDL}	deflection due to barrier and wearing surface weights	—	—
Δ_{slab}	deflection due to the weights of slab and haunch	—	—
ε	strain	—	—
ε_{cu}	the failure strain of concrete in compression	ε_{cu} (5.7.3.1.2)	—
ε_{ps}	strain in prestressing steel	—	—
ε_s	tensile strain in cracked concrete in direction of tension tie	ε_s (5.6.3.3.3)	—
ε_{sh}	concrete shrinkage strain at a given time	ε_{sh} (5.4.2.3.3)	—
ε_{si}	strain in tendons corresponding to initial effective pretension stress	—	—
ε_x	longitudinal strain in the web reinforcement on the flexural tension side of the member	ε_x (5.8.3.4.2)	—

SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
ε_1	principal tensile strain in cracked concrete due to factored loads	ε_1 (5.6.3.3.3)	—
ϕ	resistance factor	ϕ (5.5.4.2.1)	ϕ (8.16.1.2)
ϕ_c	curvature at midspan	—	—
ϕ_0	curvature at support	—	—
γ	load factor	—	γ (3.22)
γ^*	factor for type of prestressing steel = 0.28 for low-relaxation steel = 0.40 for stress-relieved steel = 0.55 for bars	—	γ^* (9.17)
γ_i	load factor	γ_i (3.4.1)	—
γ_p	load factor for permanent loading	γ_p (3.4.1)	—
η	variable load modifier which depends on ductility, redundancy and operational importance	η (3.4.1)	—
κ	a correction factor for closely spaced anchorages	κ (5.10.9.6.2)	—
λ	parameter used to determine friction coefficient and it is related to unit weight for concrete	λ (5.8.4.2)	λ (8.15.5.4, 8.16.6.4)
μ	coefficient of friction	μ (5.8.4.1)	μ (8.15.5.4.3)
μ	Poisson's ratio	—	μ (3.23.4.3)
θ	skew angle	—	—
θ	angle of inclination of diagonal compressive stresses	θ (5.8.3.3)	—
θ_s	angle between compression strut and longitudinal axis of the member in a shear truss model of a beam	θ_s (5.6.3.3.2)	—
ρ	tension reinforcement ratio - A_s/b_wd, A_s/bd	—	ρ (8.1.2)
ρ'	compression reinforcement ratio = A_s'/bd	—	ρ' (8.1.2)
ρ^*	$\dfrac{A_s^*}{bd}$, ratio of pretensioning reinforcement	—	ρ^* (9.17, 9.19)
ρ_{actual}	actual ratio of nonpretensioned reinforcement	—	—
ρ_b	reinforcement ratio producing balanced strain conditions	—	ρ_b (8.16.3.1.1)
ρ_{min}	minimum ratio of tension reinforcement to effective concrete area	ρ_{min} (5.7.3.3.2)	—
ρ_v	ratio of area of vertical shear reinforcement to area of gross concrete area of a horizontal section	ρ_v (5.10.11.4.2)	—

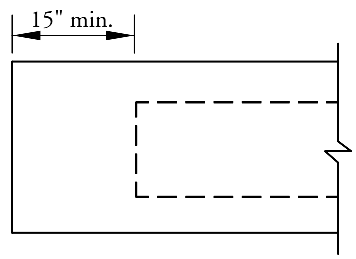
SYMBOL	DESCRIPTION	CORRESPONDING AASHTO LRFD SPECIFICATIONS	CORRESPONDING AASHTO STANDARD SPECIFICATIONS
ψ	a factor reflects the fact that the actual relaxation is less than the intrinsic relaxation	—	—
ψ	angle of harped pretensioned reinforcement	—	—
$\psi_{(t,ti)}$	creep coefficient - the ratio of the strain which exists t days after casting to the elastic strain caused when load p_i is applied t_i days after casting	$\psi_{(t,ti)}$ (5.4.2.3.2)	—
χ	aging coefficient	—	—

AASHTO Solid and Voided Slab Beams





Typical Keyway Details



Typical Longitudinal Section

Dimensions (inches)

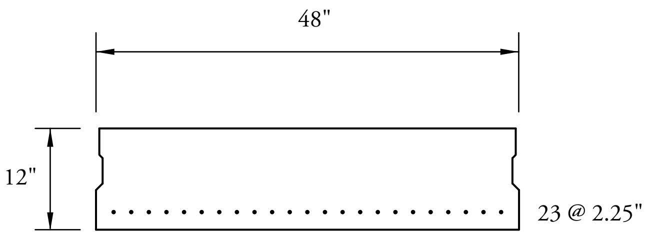
Type	L	H	L1	L2	No. of Voids	D1	D2
SI-36	36	12	-	-	0	-	-
SII-36	36	15	10.5	7.5	2	8	-
SIII-36	36	18	10.5	7.5	2	10	-
SIV-36	36	21	10.0	8.0	2	12	-
SI-48	48	12	-	-	0	-	-
SII-48	48	15	10.0	14.0	3	8	8
SIII-48	48	18	9.5	14.5	3	10	10
SIV-48	48	21	10.0	14.0	3	12	10

Properties

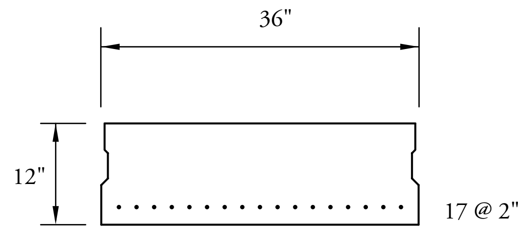
Type	Area in.^{2}	y_{bottom} in.	Inertia in.^{4}	Weight kip/ft	Max. Span* ft
SI-36	432	6.0	5,184	0.450	40
SII-36	439	7.5	9,725	0.457	47
SIII-36	491	9.0	16,514	0.511	52
SIV-36	530	10.5	25,747	0.552	57
SI-48	576	6.0	6,912	0.600	42
SII-48	569	7.5	12,897	0.593	49
SIII-48	628	9.0	21,855	0.654	53
SIV-48	703	10.5	34,517	0.732	57

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.

AASHTO Solid and Voided Slab Beams



Type SI-48



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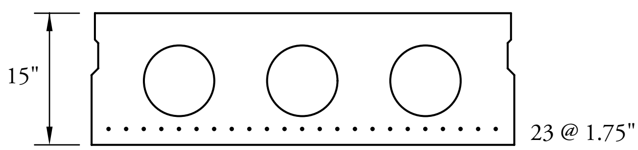

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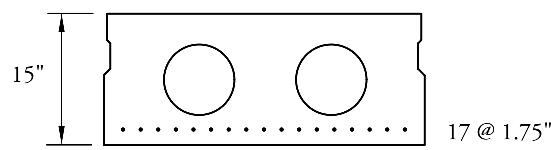

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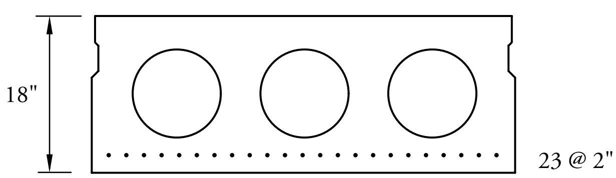

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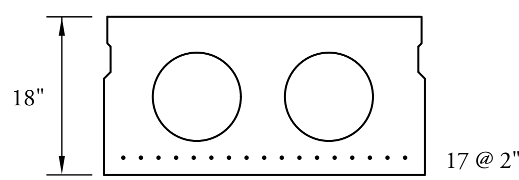

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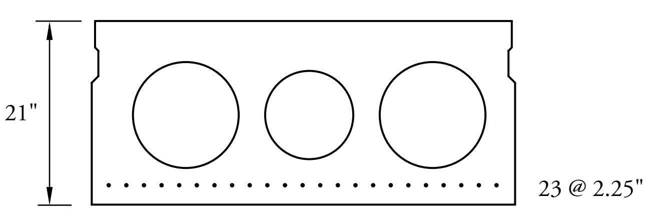
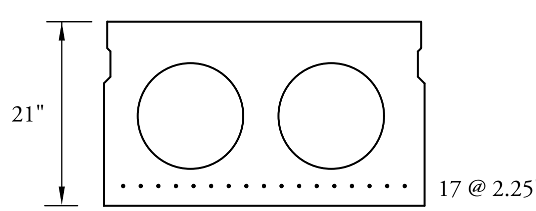
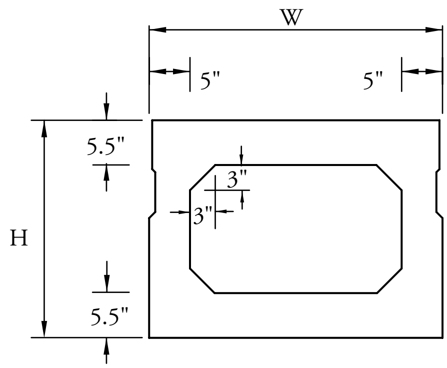


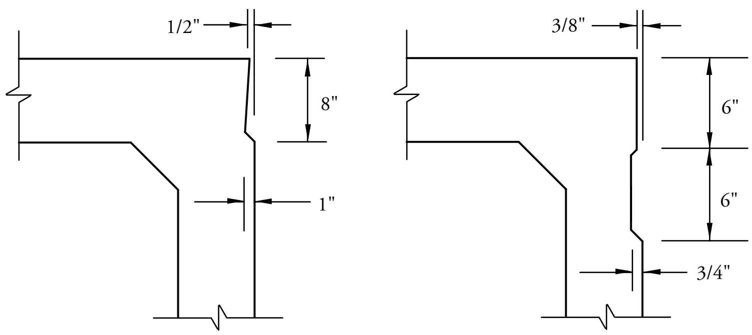
Type IV-48



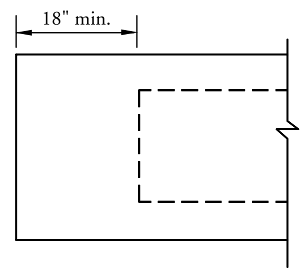
Type IV-36

AASHTO Box Beams





Typical Keyway Details



Typical Longitudinal Section

Dimensions (inches)

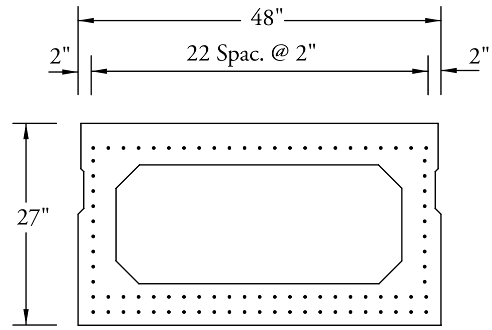
Type	W	H
BI-36	36	27
BI-48	48	27
BII-36	36	33
BII-48	48	33
BIII-36	36	39
BIII-48	48	39
BIV-36	36	42
BIV-48	48	42

Properties

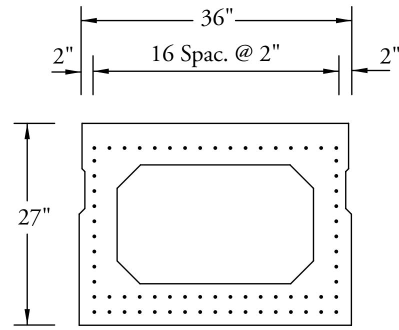
Type	Area in.^{2}	y_{bottom} in.	Inertia in.^{4}	Weight kip/ft	Max. Span* ft
BI-36	560.5	13.35	50,334	0.584	92
BI-48	692.5	13.37	65,941	0.721	92
BII-36	620.5	16.29	85,153	0.646	107
BII-48	752.5	16.33	110,499	0.784	108
BIII-36	680.5	19.25	131,145	0.709	120
BIII-48	812.5	19.29	168,367	0.846	125
BIV-36	710.5	20.73	158,644	0.740	124
BIV-48	842.5	20.78	203,088	0.878	127

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.

AASHTO Box Beams



Type BI-48



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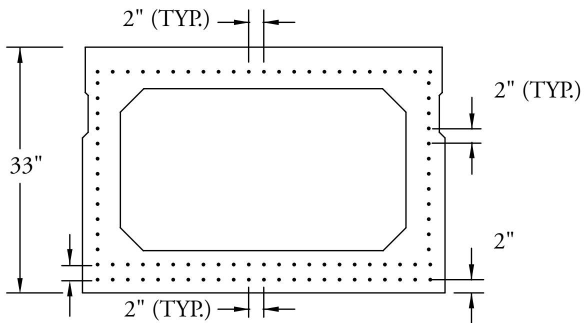
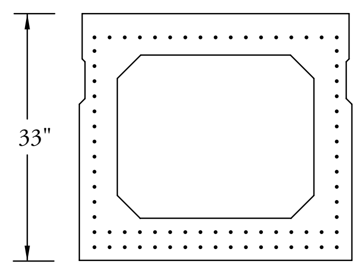


Type BII-48



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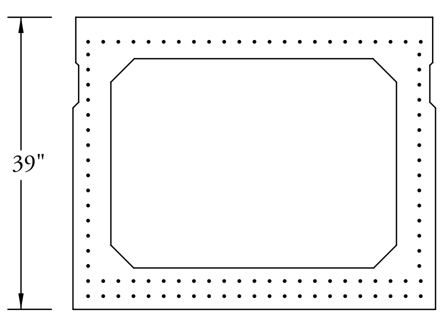
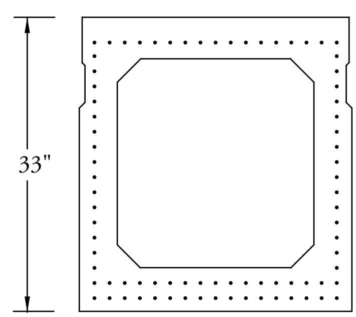


Type BIII-48



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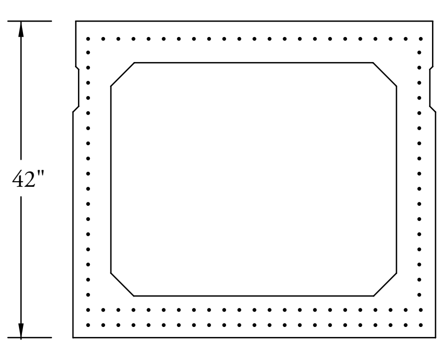
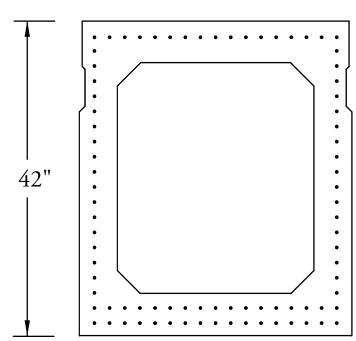
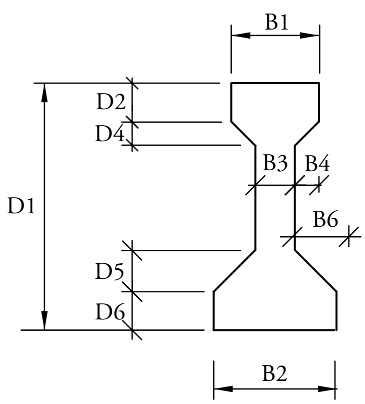


Type BIV-48

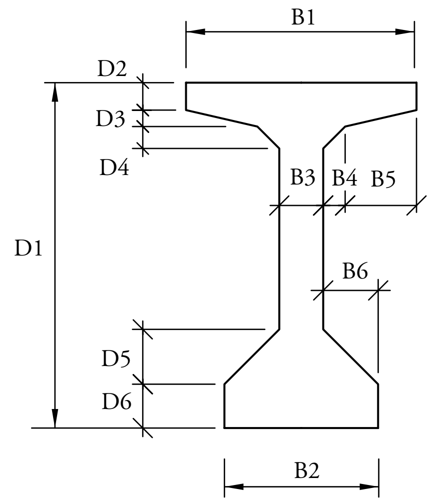


Type BIV-36

AASHTO I-Beams



Type I-IV



Type V-VI

Dimensions (inches)

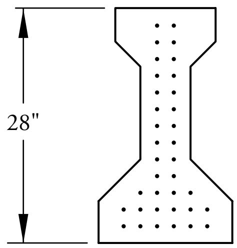
Type	D1	D2	D3	D4	D5	D6	B1	B2	B3	B4	B5	B6
I	28.0	4.0	0.0	3.0	5.0	5.0	12.0	16.0	6.0	3.0	0.0	5.0
II	36.0	6.0	0.0	3.0	6.0	6.0	12.0	18.0	6.0	3.0	0.0	6.0
III	45.0	7.0	0.0	4.5	7.5	7.0	16.0	22.0	7.0	4.5	0.0	7.5
IV	54.0	8.0	0.0	6.0	9.0	8.0	20.0	26.0	8.0	6.0	0.0	9.0
V	63.0	5.0	3.0	4.0	10.0	8.0	42.0	28.0	8.0	4.0	13.0	10.0
VI	72.0	5.0	3.0	4.0	10.0	8.0	42.0	28.0	8.0	4.0	13.0	10.0

Properties

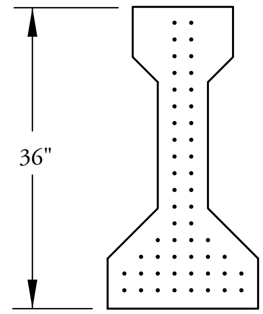
Type	Area in.^{2}	y_{bottom} in.	Inertia in.^{4}	Weight kip/ft	Maximum Span,* ft
I	276	12.59	22,750	0.287	48
II	369	15.83	50,980	0.384	70
III	560	20.27	125,390	0.583	100
IV	789	24.73	260,730	0.822	120
V	1,013	31.96	521,180	1.055	145
VI	1,085	36.38	733,320	1.130	167

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.

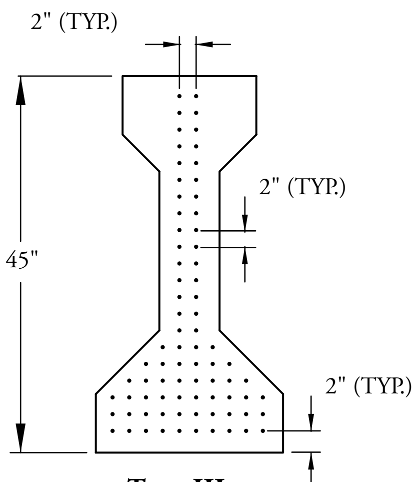
AASHTO I-Beams



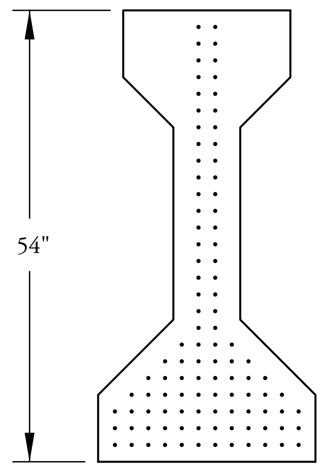
Type I



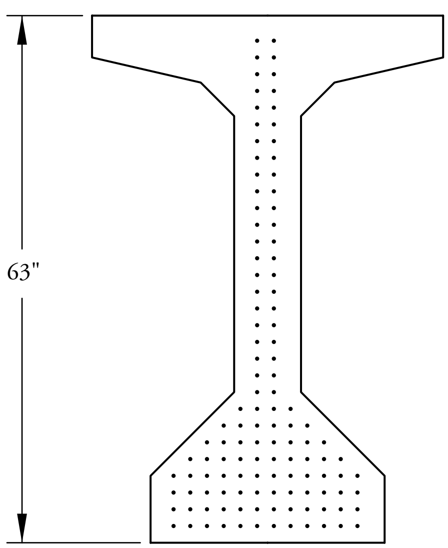
Type II



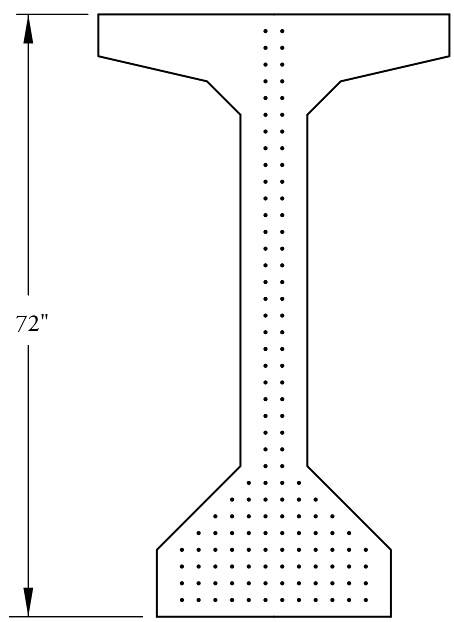
Type III



Type IV

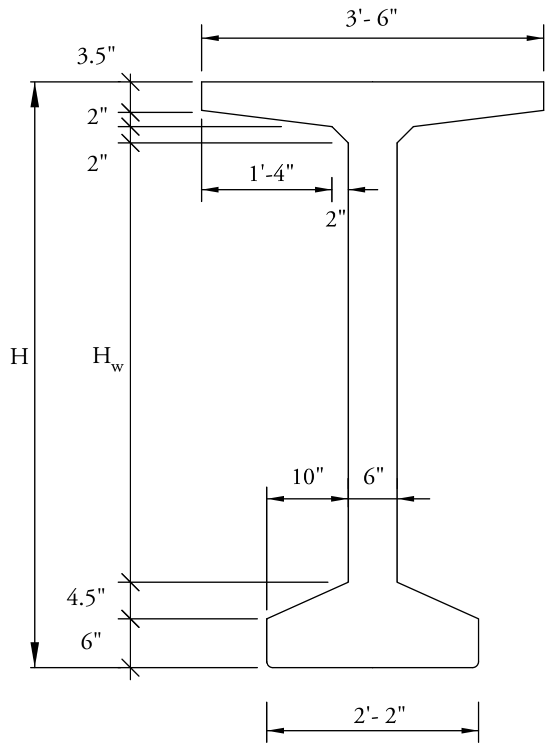


Type V



Type VI

AASHTO-PCI Bulb-Tees

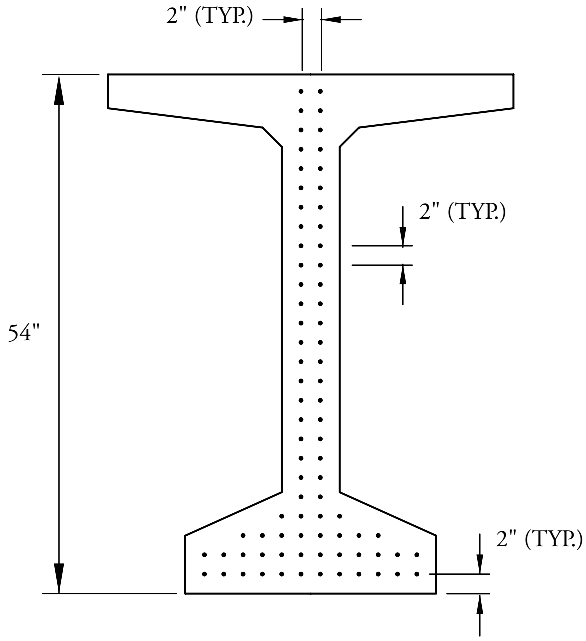


Properties

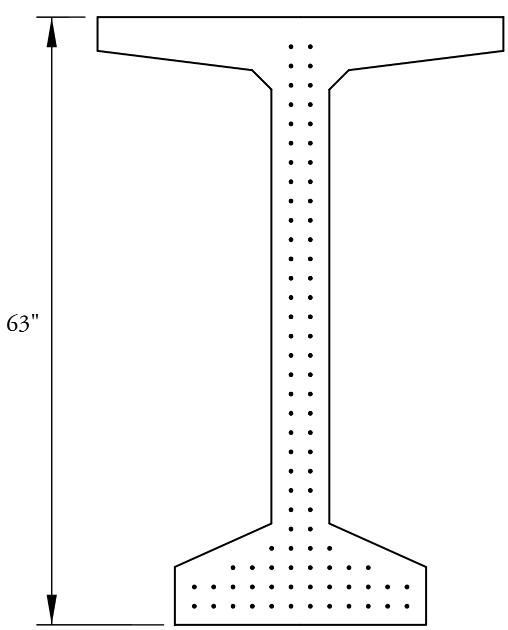
Type	H in.	H_w in.	Area in.^{2}	Inertia in.^{4}	y_{bottom} in.	Weight kip/ft	Maximum Span,* ft
BT-54	54	36	659	268,077	27.63	0.686	114
BT-63	63	45	713	392,638	32.12	0.743	130
BT-72	72	54	767	545,894	36.60	0.799	146

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.

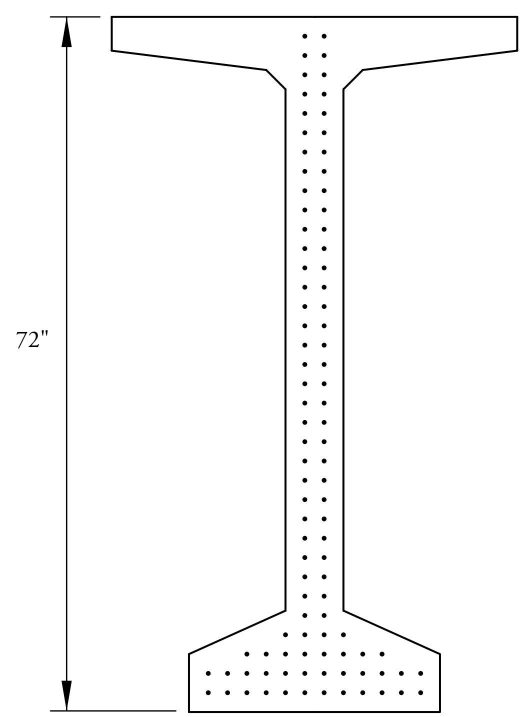
AASHTO-PCI Bulb-Tees



BT-54

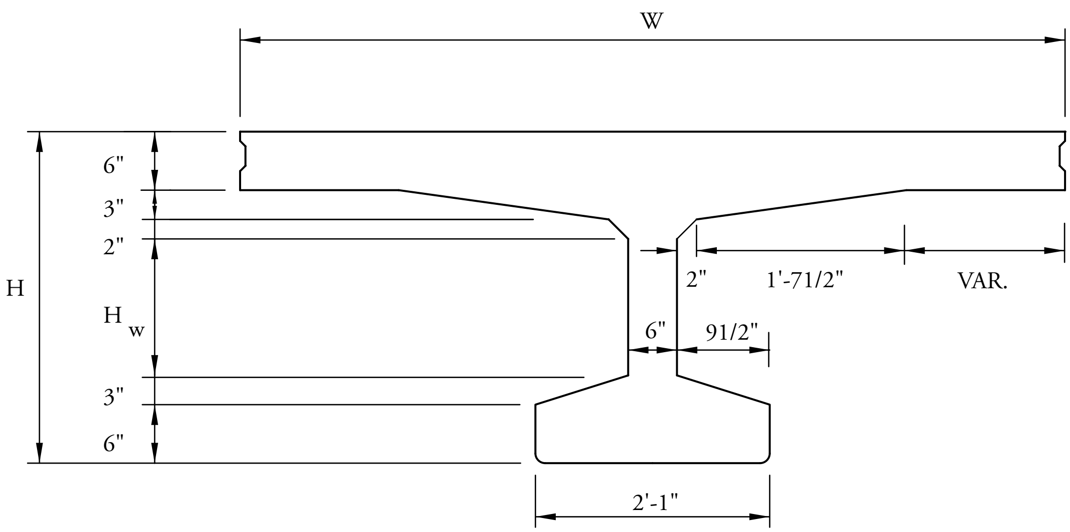


BT-63



BT-72

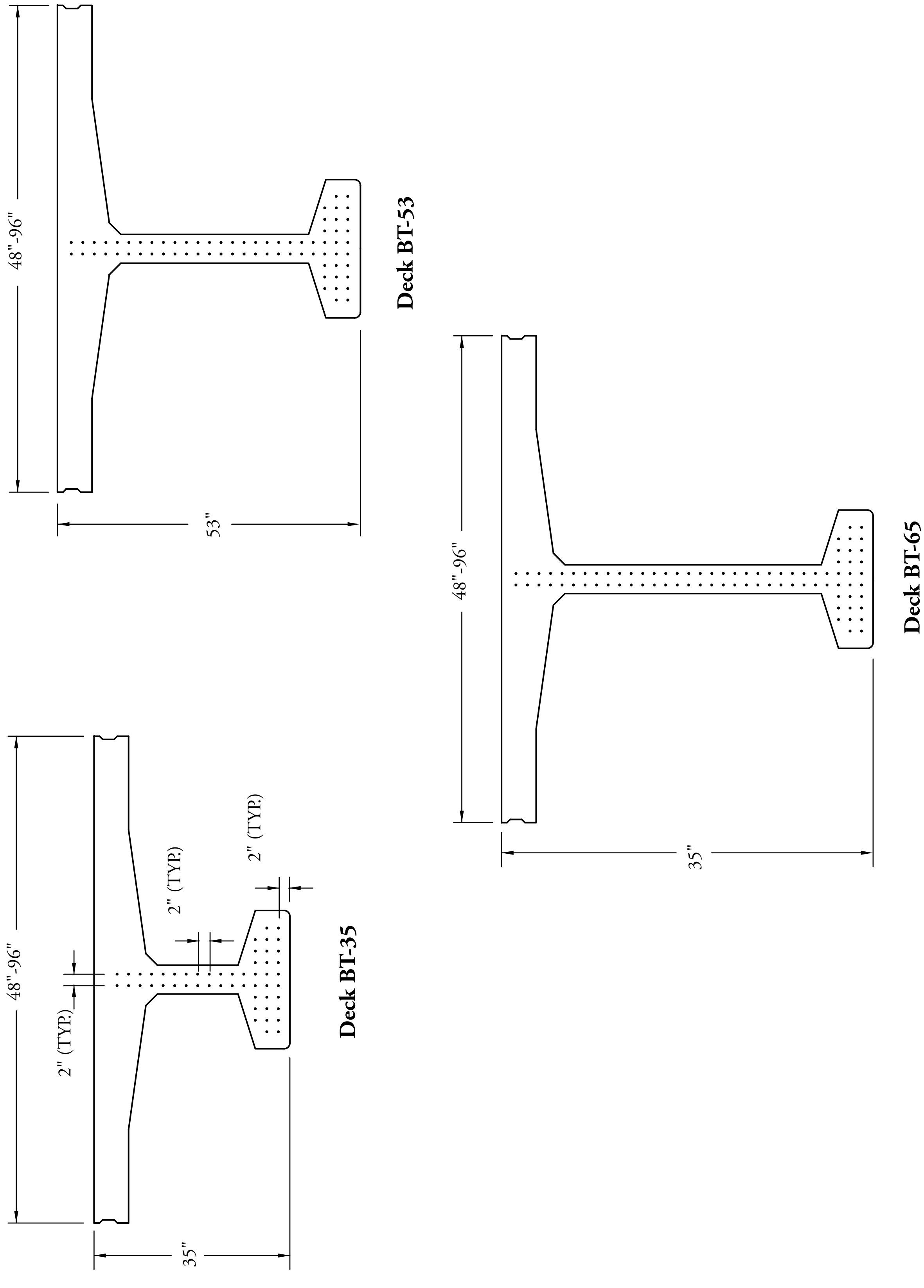
Deck Bulb-Tees



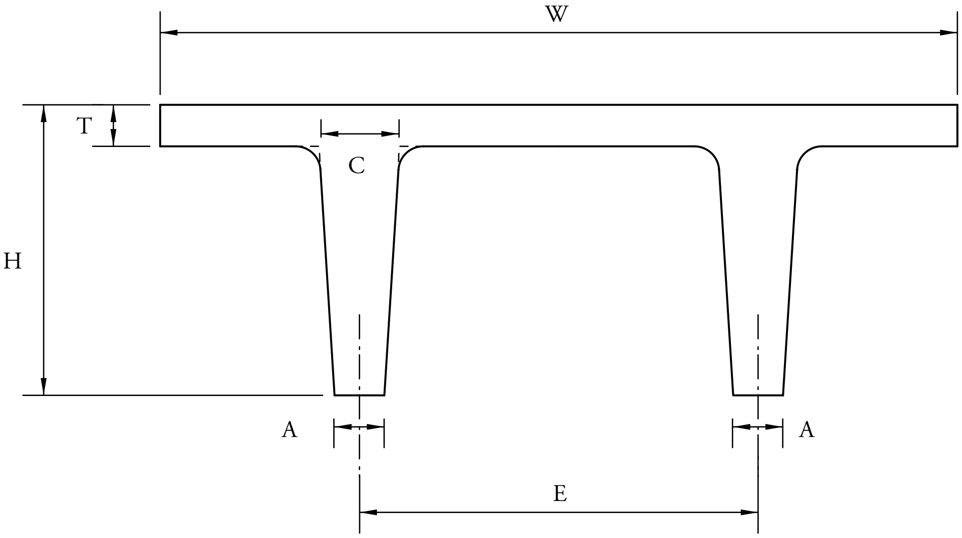
Dimensions and Properties

H in.	H_w in.	W in.	Area in.^{2}	Inertia in.^{4}	y_{bottom} in.	Weight kip/ft	Maximum Span* ft
35	15	48	677	101,540	21.12	0.75	100
		72	823	116,071	23.04	0.91	78
		96	967	126,353	24.37	1.07	65
53	33	48	785	294,350	31.71	0.87	145
		72	931	335,679	34.56	1.03	121
		96	1,075	365,827	36.63	1.19	105
65	45	48	857	490,755	38.55	0.95	168
		72	1,003	559,367	41.95	1.11	148
		96	1,147	610,435	44.46	1.27	130

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.


48"-96"
53"
Deck BT-53
48"-96"
35"
Deck BT-65
48"-96"
2" (TYP.)
2" (TYP.)
2" (TYP.)
35"
Deck BT-35

Double Tee Beams



Light Sections

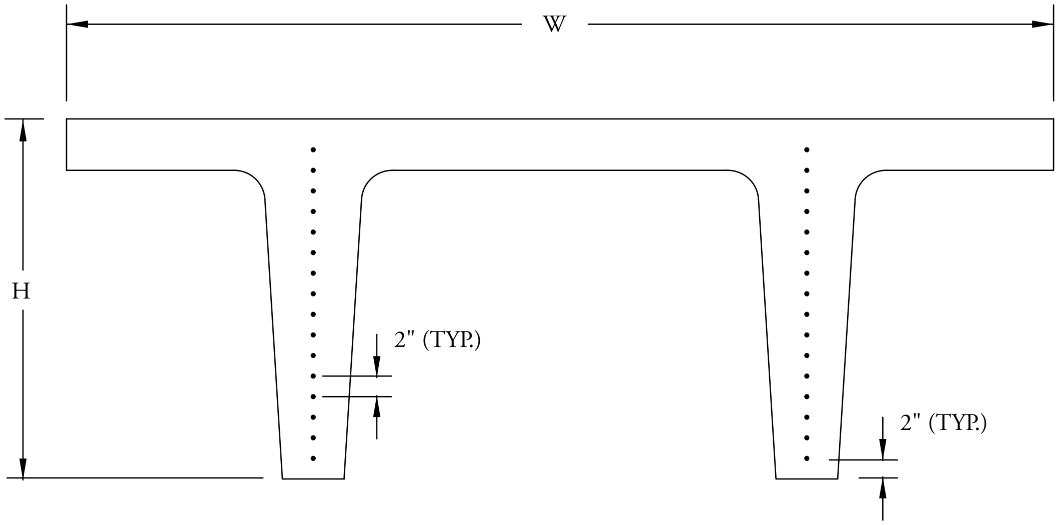
W ft	H in.	T in.	A in.	C in.	E in.	Area in.^{2}	Inertia in.^{4}	y_{bottom} in.	Weight kip/ft	Maximum Span* ft
5	27	5	4.50	8.00	36	575	33,740	18.60	0.599	40
6	23	5	4.50	6.50	36	558	21,366	16.61	0.582	30
6	27	5	4.50	8.00	36	635	35,758	19.15	0.662	37
8	27	5	3.75	5.75	48	689	32,888	20.64	0.718	40
8	35	5	3.75	6.50	48	787	72,421	26.20	0.820	35

Heavy Sections

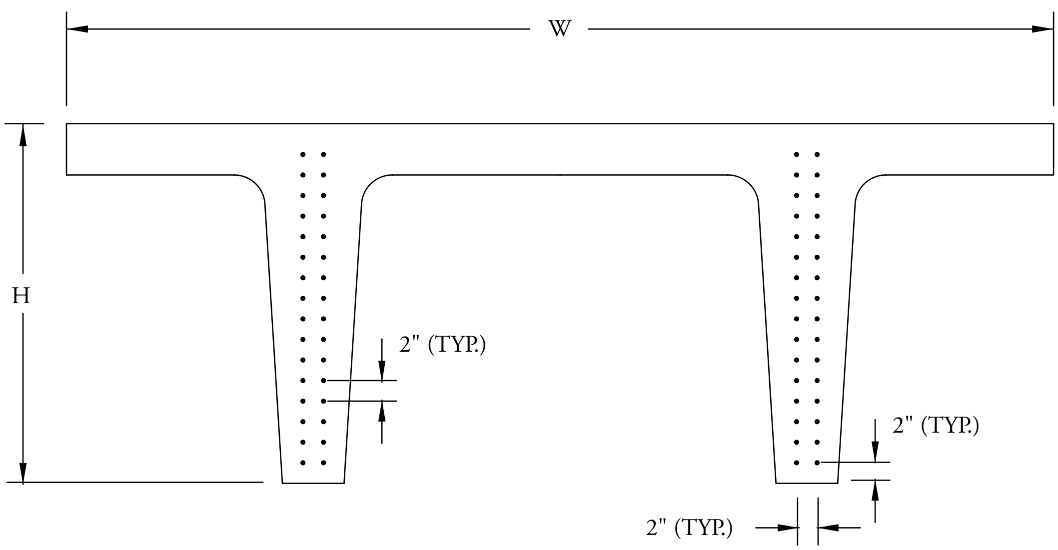
W ft	H in.	T in.	A in.	C in.	E in.	Area in.^{2}	Inertia in.^{4}	y_{bottom} in.	Weight kip/ft	Maximum Span* ft
5	36	6	6.00	8.00	30	780	90,286	23.69	0.812	87
6	35	5	6.00	9.75	48	840	90,164	23.30	0.876	70
7	35	5	6.00	9.75	48	900	95,028	23.91	0.938	64
8	35	5	6.00	9.75	48	960	99,299	24.45	1.001	62
6	27	5	7.00	9.75	48	731	45,084	18.09	0.761	52
7	27	5	7.00	9.75	48	791	47,486	18.58	0.824	47
8	27	5	7.00	9.75	48	851	49,566	19.00	0.886	45
6	21	5	7.75	9.75	48	644	22,720	14.11	0.671	37
7	21	5	7.75	9.75	48	704	23,903	14.48	0.733	35
8	21	5	7.75	9.75	48	764	24,920	14.80	0.796	32

*Based on simple span, HS-25 loading and f'_c = 7,000 psi. See chapter 6 for more information.

Double Tee Beams

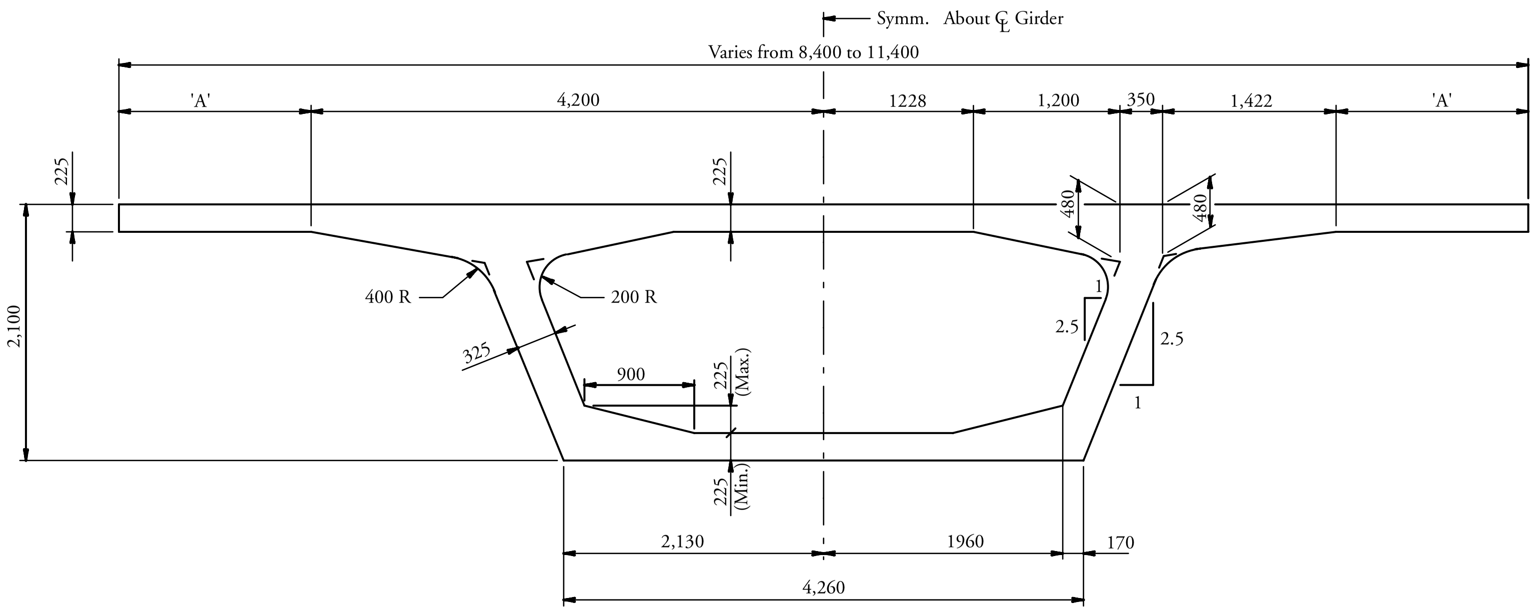


Light Sections



Heavy Sections

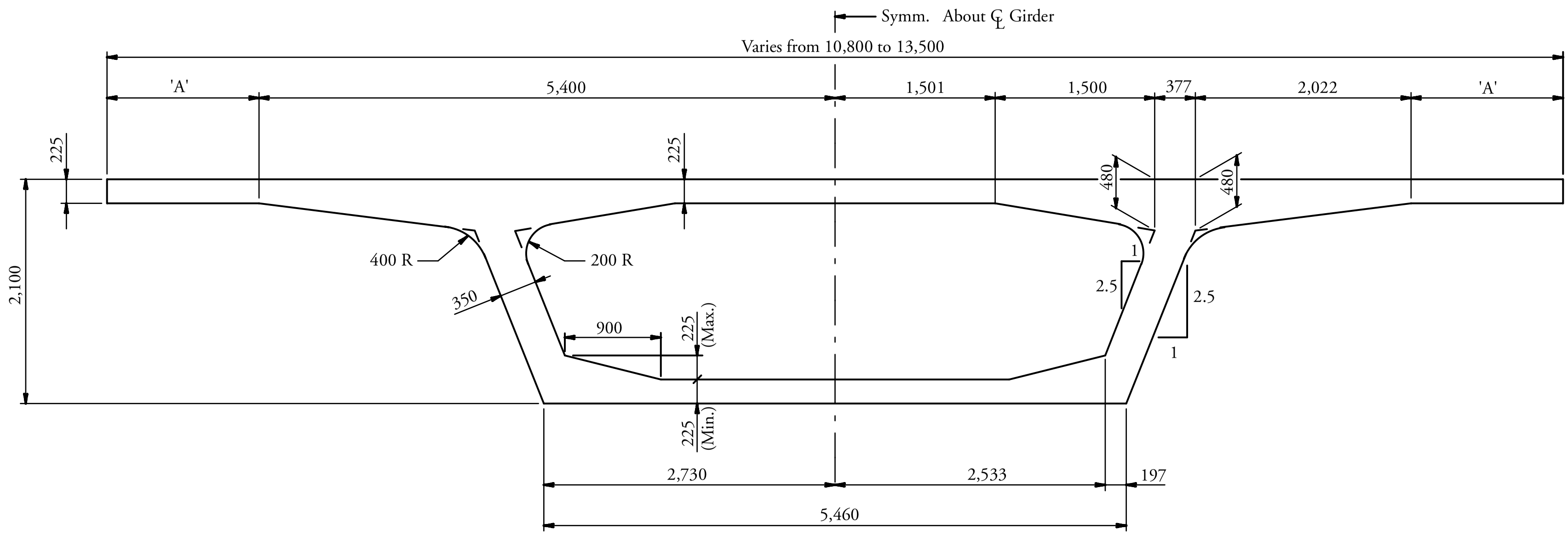
AASHTO-PCI-ASBI Standard Segment For Span-By-Span Construction



2100-1

Deck Width mm	'A' mm	Area mm^2	Wt/3,000 mm Kn	I_x m^4	Y_t mm
8,400	0	4,916,000	360	2.967	799
8,700	150	4,984,000	365	2.999	790
9,000	300	5,051,000	370	3.030	781
9,300	450	5,119,000	375	3.060	772
9,600	600	5,186,000	380	3.089	763
9,900	750	5,254,000	385	3.118	755
10,200	900	5,321,000	390	3.145	747
10,500	1,050	5,389,000	394	3.172	739
10,800	1,200	5,456,000	399	3.199	731
11,100	1,350	5,524,000	404	3.225	723
11,400	1,500	5,591,000	409	3.250	716

AASHTO-PCI-ASBI Standard Segment For Span-By-Span Construction



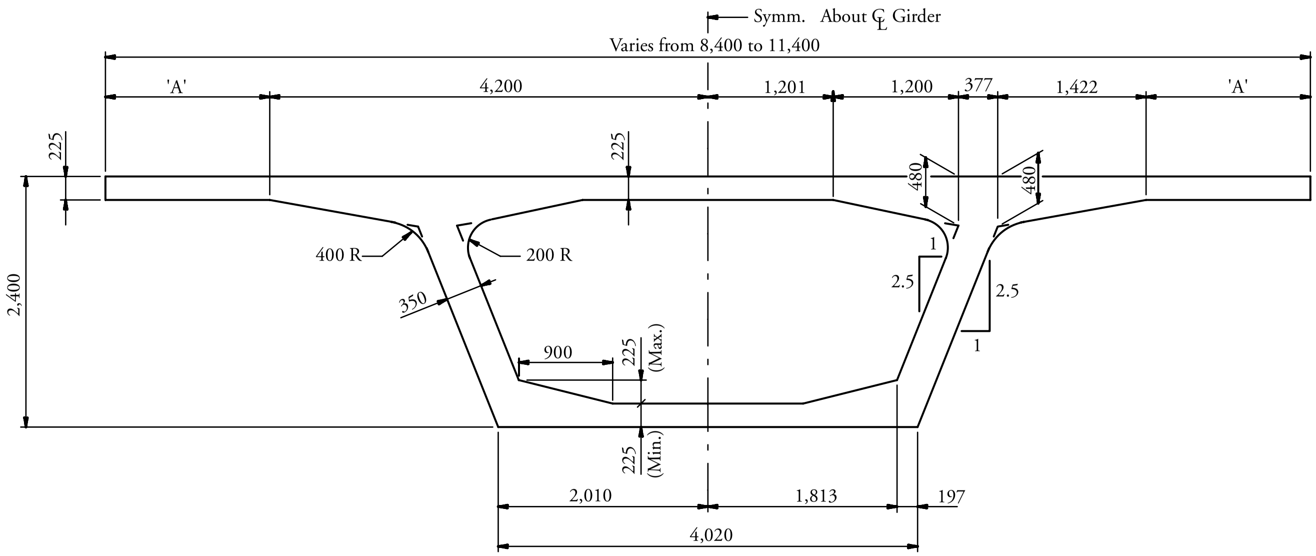
2100-2

Deck Width mm	'A' mm	Area mm^2	Wt/3,000 mm Kn	I_x m^4	Y_t mm
10,800	0	6,050,000	443	3.685	776
11,100	150	6,117,000	448	3.715	769
11,400	300	6,185,000	453	3.744	762
11,700	450	6,252,000	458	3.772	755
12,000	600	6,320,000	463	3.800	748
12,300	750	6,387,000	468	3.827	741
12,600	900	6,455,000	472	3.854	734
12,900	1,050	6,522,000	477	3.880	728
13,200	1,200	6,590,000	482	3.906	722
13,500	1,350	6,657,000	487	3.931	715

AASHTO/PCI STANDARD PRODUCTS

AASHTO-PCI-ASBI Standard Segment For Balanced Cantilever Construction Segment

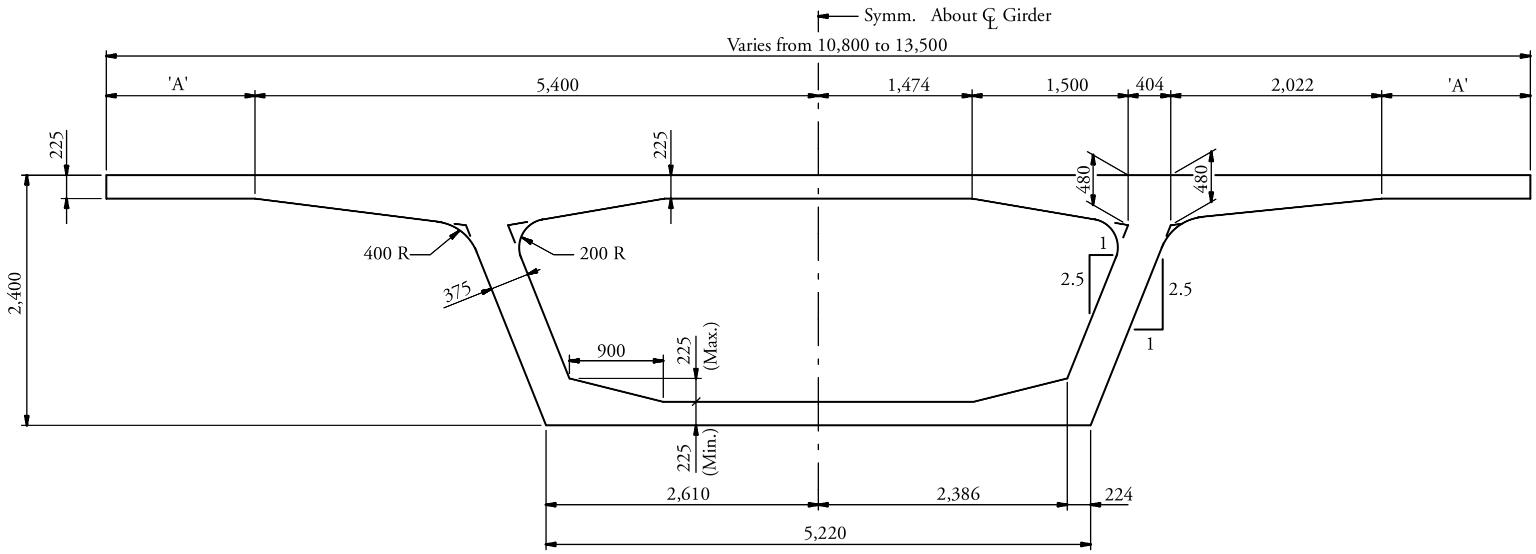
AASHTO-PCI-ASBI Standard Segment For Balanced Cantilever Construction Segment



2400-1

Deck Width mm	'A' mm	Area mm^2	Wt/3,000 mm Kn	I_x m^4	Y_t mm
8,400	0	5,177,000	379	4.064	908
8,700	150	5,245,000	384	4.107	898
9,000	300	5,312,000	389	4.148	888
9,300	450	5,380,000	394	4.188	878
9,600	600	5,447,000	399	4.228	869
9,900	750	5,515,000	404	4.266	859
10,200	900	5,582,000	409	4.304	850
10,500	1,050	5,650,000	414	4.340	841
10,800	1,200	5,717,000	419	4.376	833
11,100	1,350	5,785,000	423	4.411	824
11,400	1,500	5,852,000	428	4.445	816

AASHTO-PCI-ASBI Standard Segment For Balanced Cantilever Construction



2400-2

Deck Width mm	'A' mm	Area mm^2	Wt/3,000 mm Kn	I_x m^4	Y_t mm
10,800	0	6,327,000	463	5.045	882
11,100	150	6,395,000	468	5.085	874
11,400	300	6,462,000	473	5.124	866
11,700	450	6,530,000	478	5.162	858
12,000	600	6,597,000	483	5.199	851
12,300	750	6,665,000	488	5.236	843
12,600	900	6,732,000	493	5.272	836
12,900	1,050	6,800,000	498	5.307	829
13,200	1,200	6,867,000	503	5.342	821
13,500	1,350	6,935,000	508	5.376	815

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APPENDIX D

Introduction

Generic specifications for production of precast and prestressed concrete products have not been published by the Precast/Prestressed Concrete Institute (PCI). Such specifications require extensive committee involvement and a lengthy concensus approval process within the PCI structure. As a temporary substitute for generic specifications, this Appendix lists, without technical alteration, the specifications used by the states of Nebraska and Washington. Nebraska represents Midwestern conditions, while Washington represents coastal conditions. These two sets of specifications are not claimed to be model specifications, nor are they representative of conditions throughout the country as diverse geographically and economically as the United States. For example, critical evaluation of the validity of weather-related provisions for applications in hot, humid locations in the Southeast or hot, dry locations in the Southwest need to be assessed. Also, the characteristics of aggregates and other local raw materials need to be taken into account in adapting sample specifications to a particular location.

Both sets of specifications have recently been updated, and are thus reflective of most recent thinking of professionals at the Nebraska Department of Roads and the Washington State Department of Transportation. Both states are known to have leadership in the area of precast/prestressed concrete and are known to have a strong partnership with PCI and its producer members.

Nebraska Department of Roads Specifications

705 PRECAST/ PRESTRESSED CONCRETE STRUCTURAL UNITS

705.01 Description

1. This work consists of all labor, materials and equipment required in the production of precast/prestressed structural units.

705.02 Material Requirements

1. The materials used shall meet the requirements prescribed in this specification.
2. Precast/prestressed concrete structural units whose compressive strength does not achieve design strength shall be rejected.
3. The concrete class used in the manufacture of precast/prestressed structural units shall be shown in the Plans.
4. The Contractor is responsible for the concrete mix design and may use other concrete mixes which are proportioned in accordance with ACI Standard 318 and the following additional requirements:
   a. The mix designs shall be submitted to the Engineer 30 working days before beginning any concrete work.
   b. Concrete shall consist of Type I, Type II or Type III Portland cement, aggregate, air-entraining admixture and water. Concrete may also contain Class C or Class F fly ash, and ASTM C494 approved Type A, Type B, Type D and Type F admixtures.
   c. The minimum cement content shall be 335 kg per cubic meter.
   d. Coarse aggregate shall have a minimum limestone content of 30 percent of the total aggregate by mass.
   e. Fly ash cannot exceed 15 percent of cement by mass.
   f. Data from at least 15 individual batches shall be collected and given to the Engineer. The data collected shall include the following:
      (1) The 28-day compressive and flexural strength test results.
      (2) The water cement ratio.
      (3) The air content between 2.0 percent and 6.0 percent inclusive.
      (4) The cement and fly ash content.
      (5) The amount of fine aggregate, coarse aggregate, and sand and gravel.
5. No change shall be made in the concrete mix design during the progress of the work without the prior written permission of the Engineer.
6. Welding reinforcing steel is prohibited unless specifically authorized by the Engineer.

7. Prestressed steel other than that specified in the Plans or Special Provisions may be furnished with the approval of the Engineer. The yield and ultimate strength and other pertinent characteristics of this steel shall be submitted to the Engineer.
8. The area of broken wires shall not exceed 2 percent of the cross-sectional area of the stressing strands when the number of strands is 14 or less.
9. The area of broken wire shall not exceed 1 percent of the cross-sectional area of the stressing strands when the number of strands exceeds 14.
10. No more than one broken wire will be allowed in a single strand.
11. Bars for post-tensioning shall be of high tensile strength steel. They shall be equipped with wedge-type end anchorages which will develop the minimum specified ultimate bar stress on the nominal bar area. The physical properties of the bar steel determined by static tensile tests shall conform to the following:

    High-Strength Steel Post-Tensioning Requirements

    Ultimate Stress . . . . .1,000 MPa minimum

    Stress at 0.7% Elongation . . . . .896 MPa minimum

    Stress at 0.3% Elongation . . . . .517 MPa minimum

    Elongation in 20 Diameters . . . . .4% minimum

    Modulus of Elasticity . . . . .172 GPa minimum

    Diameter Tolerance . . . . .Plus or Minus 2.54 mm
12. Materials specified for testing shall be furnished 30 days before anticipated time of use. All materials required for testing shall be furnished by the Contractor to the Engineer without additional costs to the Department. The Engineer shall select a representative sample length for the various prestressed steel as follows:
    a. Two meters for wires requiring heading.
    b. For wires not requiring heading, sufficient length to make up one parallel-lay cable two meters long consisting of the same number of wires as the cable to be furnished.
    c. Two meters between near ends of fittings for a strand furnished with fittings.
    d. Two meters between threads at the ends of bars furnished with threaded ends.
13. If the anchorage assemblies are not attached to prestressing steel samples, two anchorage assemblies shall be furnished for testing, complete with distribution plates of each size or type of prestressing steel to be used.
14. Any defective material shall be rejected.
15. Concrete quality control shall be the responsibility of the Contractor. Concrete shall be sampled and tested as shown in **Table 705.02**.
16. Plant Approval Requirements:
    a. (1) All precast/prestressed concrete structural units shall be produced in a Precast/Prestressed Concrete Institute (PCI) Certified Plant.

       (2) The method of manufacture and quality of concrete are also subject to Department approval/inspection.
    b. (1) A Contractor proposing to furnish precast/prestressed structural units shall submit the following additional details to the Department concerning the method of manufacture:

Table 705.02

Required Concrete Sampling and Testing		
Correlation Test	Contractor Test Samples*	Department Test Samples
Yield ASTM C138 Air meter measuring bowl.	One per day.	One per 10 Contractor tests.
Air content ASTM C231 (0.8% variation allowed)	One per load.	One every five production days.
Concrete temperature. ASTM C1064	One per load.	One every five production days.
Concrete Compressive Strength		
28-day strength ASTM C31 Section 9.3 cure.	Two cylinders - each from a different load; and one from the last load.	One set of two cylinders every five production days.
56-day strength (only if 28-day strength is less than specified.) ASTM C31 Section 9.7 cure.	Two cylinders - each from a different load and from same load as 28-day break.	NA

* At least 6 cylinders shall be made each production day and at least 2 cylinders are required from each load.

* Cylinders shall be 150 mm by 300 mm.

(i) Type, number, size and location of the prestressing elements, and the name of the manufacturer of the post-tensioning or pretensioning elements.

(ii) Complete information as to type, size and method of installation of devices for anchoring post-tensioning elements.

(iii) The proposed manufacturing methods, and the Plans and design details of proposed casting beds and forms.

c. The use of portable pretensioning beds for the manufacture of concrete structural units or piles will not be allowed.

705.03
Construction Methods

1. The Contractor shall construct precast structures and piles as shown in the Plans.
2. The Contractor shall erect precast concrete structures and drive precast concrete piles as prescribed in the Plans.
3. a. When the precast superstructure units have been erected, the Contractor shall pack the shear key openings with grout.

   b. A pneumatic tool shall be used.

   c. Grout to be used for constructing shear keys in the precast concrete superstructure shall be composed of either Type I or Type II Portland cement, aggregate and water.

(1) The aggregate shall be fine aggregate as specified for the class of concrete being furnished.

(2) The Portland cement and aggregate shall be proportioned on the basis of 350 kg of dry aggregate per 100 kg of cement.

(3) The water content of the grout shall be limited to that necessary for proper mixing and placement. In no case shall the total water content exceed 45 kg per 100 kg of cement.

4. a. No live load shall be allowed on the superstructure units until the shear keys and tie bolts have been placed and the shear key grout cured, unless cross planking or mats not less than 190 mm in thickness and 4.2 m long are placed on the structure to distribute the load.

   b. In no case shall the live load vehicle mass exceed:

   (1) 23 Mg.

   (2) 9 Mg on any single axle.

   (3) 18 Mg on any tandem axle.

5. Stressing Requirements:

   a. (1) In all methods of tensioning, the stress induced in the prestressing elements shall be measured by the Contractor both with jacking gages and by elongation of the elements, and these results shall be the same within a five-percent tolerance.

   (i) Means shall be provided for measuring the elongation of reinforcement to at least the nearest 3 mm.

   (ii) All steel stressing devices, whether hydraulic jacks or screw jacks, shall be equipped with accurate reading calibrated pressure gages, rings or other devices as applicable to the jack being used.

   (iii) All devices shall be calibrated and, if necessary, recalibrated so as to allow the stress in the prestressing steel to be computed at all times.

   (iv) A certified calibration curve shall accompany each device.

   (v) Safety measures must be taken by the Contractor to prevent accidents due to possible breaking of the prestressing steel or the slipping of the grips during the prestressing process.

   (2) All calibrations and tests shall be performed at no additional cost to the Department.

   (3) Pressure gages, load cells, dynamometers, and any other devices used in determination of loads and/or pressures shall be accurate in their effective range within a two-percent tolerance.

   (i) Such equipment shall be calibrated by an approved testing laboratory.

   (ii) The laboratory shall furnish calibration curves for each device and shall certify the curves as being accurate and verifiable.

   (iii) The calibration of tensioning devices shall be accomplished in place.

   (iv) The configuration of jacks, gages and other components during calibration shall be exactly the same as during the actual stressing operation.

   (v) The method of calibration shall be as approved by the Engineer.

   (vi) Tensioning devices shall be calibrated at least once a year and anytime a system appears to be operating in an erratic or inaccurate manner, or gage pressure and elongation measurements fail to correlate.

(4) If the strand tension indicated by the gage pressure and by elongation method fail to agree within five percent, the operation shall be carefully checked and the source of error determined before proceeding further.

b. (1) The Contractor's elongation and jacking pressure measurements shall make appropriate allowance for friction and all possible slippage or relaxation of the anchorage.

(2) For pretensioned members, independent references shall be established adjacent to each anchorage by the Contractor to indicate any yielding or slippage that may occur between the time of initial stressing and final release of the cables.

(3) The Contractor may tension straight post-tensioned tendons from one end. The curved tendons shall generally be stressed by simultaneous jacking from both ends of the bar.

c. In all stressing operations, the Contractor shall keep stressing force symmetrical about the member's vertical axis.

6. Stressing Procedure:

a. Prestressing methods are shown in the Plans. When the Contractor elects to use a method other than that shown in the Plans, the Contractor shall submit complete shop Plans for the proposed method.

(1) Pretensioning method.

(2) Post-tensioning method.

(3) Combined method.

b. Pretensioning Method:

(1) The amount of stress to be given each strand by the Contractor shall be as shown in the Plans.

(2) All strands to be prestressed in a group shall be brought to a uniform initial tension before being given their full pretensioning. This uniform initial tension of approximately 4.5 to 9.0 kN shall be measured by a dynamometer or other approved means so that it can be used as a check against the computed and measured elongation.

(3) After initial tensioning, either single strand or multiple strand groups shall be stressed until the required elongation and jacking pressure are attained and reconciled within the five-percent tolerance.

(4) With the strand stressed in accordance with the Plan requirements and these Specifications, and with all other reinforcing in place, the Contractor shall cast the concrete to the lengths desired. Strand stress shall be maintained between anchorages until the concrete has reached the compressive strength specified in the Plans.

c. Post-tensioning Method: For all post-tensioned bars, the Contractor shall set the anchor plates exactly normal in all directions to the axis of the bar. Parallel wire anchorage cones shall be recessed within the beams. Tensioning shall not be done until the concrete has reached the compressive strength specified in the Plans.

d. Combined Method: In the event that the girders are manufactured with part of the reinforcement pretensioned and part post-tensioned, the applicable portions of the requirements listed above shall apply to each type.

7. Forms:

a. Forms for precast/prestressed concrete structural units shall conform to the requirements for concrete formwork as provided in Section 704.

b. Forms shall be accessible for the vibration and consolidation of concrete.

8. Placing Concrete:

a. The Contractor shall provide the Department a 4-week production schedule that is updated as necessary. Unscheduled production changes may delay fabrication when the Department elects not to reschedule inspectors.

b. The Engineer may observe any or all of the procedures and shall have access to all reported data anytime during fabrication. The Engineer shall report any inconsistencies to the job superintendent and note them in the plant diary.

c. Concrete shall not be placed before completing the forming and placing of reinforcement.

d. (1) Concrete shall be placed continuously in each unit, take care to avoid horizontal or diagonal planes of weakness.

(2) However, if there is a delay in delivery of concrete, or for some other reason placement is interrupted for more than 30 minutes, then the concrete shall be rejected.

e. (1) Special care shall be exercised to work and consolidate the concrete around the reinforcement and to avoid the formation of stone pockets, honeycombs and other defects.

(2) The concrete shall be consolidated by vibrating, or other means approved by the Engineer.

f. The forms shall be overfilled, the excess concrete screeded off, and the top surfaces finished to a uniform, even texture.

g. Each precast/prestressed concrete structural unit shall be stamped or marked with an identification number and its manufacture date.

h. (1) The optimum range of concrete temperatures from the time the concrete is completely mixed until the beginning of the presteam segment of the steam curing cycle shall be 10C to 35C. Failure to operate within the optimum range shall be cause for curtailment of operations to operate consistently within the range set forth. During the preset segment of the curing cycle, the temperature of the concrete shall not exceed 38C nor fall below 10C.

(2) When placing concrete under cold weather conditions (ambient air temperature less than 2C), follow the cold weather Specifications in Section 1002.

(3) Forms and reinforcing materials shall be preheated to a minimum temperature of 5C and a maximum temperature not to exceed that of the concrete at the time of placement.

(4) The Contractor may preheat the drums of the mixer-trucks to the limits set for forms and reinforcing, but under no condition shall heat be applied to the drums while they contain any of the batch materials or concrete.

9. Curing:

a. General:

(1) The Contractor shall cure the concrete with wet burlap, waterproof covers, polyethylene sheets, or liquid membrane-forming compound. Liquid membrane-forming compound shall not be used on that portion of precast/prestressed concrete girders, twin tees, or bridge beams upon which concrete will be cast later.

(2) Water spray curing, or other moist curing methods may be used, subject to the approval of the Engineer.

(3) The period of curing shall be determined by the results of the compressive strength test on cylinders made during the progress of the work and cured to closely approximate the concrete strength of the product it represents.

(4) Side forms may be removed 12 hours after placing the concrete, provided curing is continued with one of the approved NDOR curing procedures.

b. Steam or radiant heat will be allowed for accelerated curing provided the following procedure is adhered to:

(1) Curing chambers shall be reasonably free of leakage and shall have a minimum clearance of 75 mm between the enclosure and restricting portions of the forms in order to ensure adequate circulation of heat. The relative humidity within the curing enclosure shall be maintained between 70 and 100 percent.

(2) One approved continuous recording thermometer for each 35 meters of casting bed with a minimum of two continuous recording thermometers shall be located in each enclosure or curing chamber.

(i) Continuous temperature record charts for each casting shall be available to the Engineer for his examination and approval at anytime.

(ii) If the temperature records or other temperature readings taken by the Engineer indicate that hand control of heat is producing temperature changes in excess of those specified, the Engineer may direct that automatic controls which can be activated by the recording thermometers or by separate temperature switches be installed. These automatic controls are to control the rate of temperature change and maximum curing temperature according to a preset plan.

(3) Temperature of the curing concrete shall be 10C to 30C and shall be maintained near placement temperature until the concrete has reached initial set as determined by ASTM C403 "Time of Setting of Concrete Mixture by Penetration Resistance."

(i) The temperature rate of rise shall not exceed 30C per hour.

(ii) The concrete shall be completely covered with a waterproof curing chamber during accelerated curing periods.

(4) Steam jets shall not be directed at the concrete or the steel forms.

(5) When the heat has been applied for a minimum of 3 hours and the desired concrete temperature has been reached (not to exceed 80C), the heat source may be turned off. Should the temperature within the concrete rise above 80C, the concrete shall be rejected.

(6) The temperature in the concrete shall be maintained so that at any given time the difference between the highest and lowest temperature station readings will not be more than 15C. If the temperature varies more than 15C, the product shall be rejected.

(7) Eight hours after placing the concrete, individual sections may be uncovered to remove their forms. The curing may be discontinued during this operation. The section shall not be left uncovered longer than necessary and never longer than 30 minutes. Waterproofed covers shall be used to recover the product.

(8) After the heat source has been turned off, the curing cover shall be maintained in place during the soaking period until the release strength has been reached.

(9) Detensioning shall be accomplished before the temperatures of the units drop below 40C and while they are still moist.

(10) An automatic master slave heat curing system may be used for curing quality control cylinders.

10. Defects and Repair Procedures:

a. After the forms are removed, stone pockets, honeycombs, or other defects may be exposed. The Engineer shall determine if these defects affect the item's structural integrity and in which case the item will be rejected.

b. Precast/prestressed concrete structural units which have chips, spalls, honeycomb or otherwise defective areas which are not considered detrimental to structural integrity may be used after being repaired in the following manner:

(1) Remove all unsound concrete.

(2) Coat the affected area with epoxy resin binder. Care shall be taken to prevent getting epoxy on the exposed surface.

(3) Fill the prepared area with Class "47B-XX" concrete mix (Aggregate larger than 10 mm is not allowed) with the type of cement used in placing the unit. Where the unit is exposed to view, white cement shall be added to give a uniform appearance with the concrete surrounding the patch.

(4) Place and secure formwork to ensure all required configurations.

(5) Cure 24 hours with wet burlap. Steam curing at 25°C will be allowed.

(6) The patch shall be ground smooth to remove all joints.

11. Surface Finish:

a. The exterior face of all exterior girders or beams plus the bottoms and chamfers on all lower flanges shall be given the following finish:

(1) All uneven form joints in excess of 3 mm shall be ground smooth.

(2) The surface shall be steel brushed to remove scale and laitance, and to open partially obstructed holes.

(3) Dampen surface.

(4) The grout shall consist of 1.5 parts of fine sand, 1 part of Portland cement and sufficient water to produce a consistency of thick paint. The cement used in the grout shall be a blend of regular Type I and white Portland cement to duplicate the lighter appearance of the steam-cured units.

(5) If necessary, an admixture which will not discolor the concrete may be used in the grout to reduce shrinkage if approved by the Engineer. Admixtures containing iron particles shall not be used.

(6) Apply grout to the surface.

(7) The surface shall be float finished with a cork or other suitable float. This operation shall completely fill all holes and depressions on the surface.

(8) When the grout is of such plasticity that it will not be pulled from holes or depressions, sponge rubber or burlap shall be used to remove all excess grout.

(9) Surface finishing during cold weather shall not be performed unless the temperature is 50C and rising. The surface shall be protected against temperature drops below 5C for a period of 12 hours after finishing.

(10) A uniform appearance will be required. In the event the appearance produced by the above procedure is not uniform, both in texture and coloration, other methods approved by the Engineer shall be employed.

b. The interior face of an exterior girder or beam and all interior girders or beams shall be finished from the lower flange to the fillet of the web in accordance with subparagraphs (3), (4), (5) and (6) above.

12. Grouting for Post-Tensioned Units:

a. The Contractor shall install steel in flexible or other approved tubes which shall be cast in the concrete and shall be pressure-grouted after the prestressing process has been completed.

b. Bonding grout shall be made to the consistency of thick paint and shall be mixed in the proportions as follows: Portland cement (Type I), 45 kg; Fly ash (ASTM C618), 15 kg; Water, 20 to 27 kg (adjust at site); and admixture (Interplast B), 0.5 kg.

c. The final grouting pressure shall be at least 550 kPa.

d. The Contractor shall make provisions to demonstrate to the Engineer that grouting material has completely filled all areas within the conduit.

13. The Contractor shall paint all exposed metal, except weathering grade steel.

14. Handling, Transporting and Storing:

a. (1) After precast structural units have attained a compressive strength of 20 Mpa, the Engineer shall approve the method used to remove the units from the casting beds.

(2) Prestressed concrete structural units shall attain the “release” strength specified in the Plans before being delivered to the site. Prestressed concrete structural units will not be incorporated in the final product until the minimum age and strength specified in the Plans is attained.

(3) All precast/prestressed concrete structure units shall be supported at or within 150 mm of all lifting or bearing devices. When supported at the proper positions, no part of the units shall be allowed to rest on the ground. Prestressed concrete bridge girders shall be set on a level area to prevent field bowing, and adequate supports shall be placed under their lifting or bearing devices to prevent settlement into the ground.

(4) The girders shall be transported in an upright position and the points of support and direction of the reactions with respect to the girder shall be approximately the same during the transportation and storage, as when the girder is in its final position. If the Contractor finds it necessary to transport or store the precast girders in some other position, the Contractor should be prepared to prove no internal damage resulted.

(i) Adequate padding shall be provided between tile chains and cables to prevent chipping of the concrete.

(ii) Live loads shall not be allowed on the superstructure units until the floor slab is placed and has attained the design strength shown in the Plans.

15. Inspection Facilities:

a. The Contractor shall arrange with the producer of precast/prestressed concrete structural units to provide an office laboratory, and a bathroom for the Department’s inspector. The areas shall meet the following requirements:

(1) Thermostatically controlled heating and air conditioning shall be provided so that temperature can be maintained between 20C and 25C.

(2) The floors shall be tile or a similar floor covering.

(3) Interior and exterior walls shall be well maintained and painted.

(4) All exterior doors shall have cylinder locks and all keys shall be turned over to the Engineer.

(5) Ceiling lighting shall provide a minimum of 5,000 lx of light on all working surfaces.

(6) Electrical outlets shall be spaced no more than 2 m apart with no less than one outlet on any wall of the office or lab.

(7) A single trunk telephone shall be installed in the office and the installation charges shall be paid by the Contractor. The monthly charges shall be paid by the State.

(8) A fire extinguisher and first aid kid shall be provided.

(9) A ventilated bathroom with a toilet and sink shall be provided in the structure. A fresh water supply and drain will be required in the lab area.

(10) The lab, office and bathroom shall be separate rooms with interconnecting doors.

(11) The minimum lab area is 21 m^2 .

(12) The minimum toilet area is 2 m^2 .

(13) The minimum office area is 15 m^2 .

(14) The Contractor shall clean and maintain the rooms and shall supply all heating fuel, electricity and water.

(15) The Contractor shall also supply for the sole use of the inspectors, all desks, work tables, chairs, files, lockers and sanitary supplies necessary and commensurate with the inspection of his/her plant. It is anticipated that the following minimum amount of office and lab equipment will be required: One desk with approximately 1 m x 2 m top; one upright locker or wardrobe, with shelves, approximately 1.5 m deep; two four-drawer file cabinets; one chair per inspector; one square meter of work surface per inspector in the office area; and a 1 m x 15 m lab counter with storage space beneath.

705.04 Method of Measurement and Basis of Payment

1. Precast/prestressed concrete superstructures will be measured for payment by the lump sum.
2. The cost of furnishing and maintaining the inspection facilities will not be paid for directly, but shall be considered subsidiary to the items for which the contract provides that direct payment will be made.
3. If a precast or prestressed structural item's 56-day compressive strength is less than the design strength, then the Engineer will determine if the item can be used. It the item is to be used, a payment deduction of 25 percent will be taken if the 56-day compressive strength is less than 95 percent of the design strength.
4. Payment is considered full compensation for all work prescribed in this Section including the cost of prestressing and precasting.

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6-02.3(25) PRESTRESSED CONCRETE GIRDERS

The Contractor shall be required to perform quality control inspection. The manufacturing plant of prestressed concrete girders shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of prestressed member to be produced and shall be approved by WSDOT as a Certified Prestress Concrete Fabricator prior to the start of production. WSDOT certification will be granted at, and renewed during, the annual prestressed plant review and approval process.

Prior to the start of production of girders, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the work. If the Inspector observes any nonspecification work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the girder(s) will be rejected.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the work and verify the quality of that work. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

The various types of girders are:

- Prestressed Concrete Girder - Refers to prestressed concrete girders including Series W42G, W50G, W58G, and W74G girders, bulb-tee girders, and deck bulb-tee girders.
- Bulb-Tee Girder - Refers to a bulb-tee girder or a deck bulb-tee girder.
- Deck Bulb-Tee Girder - Refers to a bulb-tee girder with a top flange designed to support traffic loads (i.e., without a cast-in-place deck). This type of bulb-tee girder is mechanically connected to adjacent girders at the job site.

6-02.3(25)A Shop Plans

The Plans show design conditions and details for prestressed girders. Deviations will not be permitted, except as specifically allowed by these Specifications and by manufacturing processes approved by the annual plant approval process.

Shop plans shall allow the size and location of all cast-in holes for installation of deck formwork hangers and/or temporary bracing. Holes for formwork hangers shall match approved deck formwork plans designed in accordance with Section 6-02.3(16). There shall be no field-drilled holes in prestressed girders.

The Contractor shall have the option to furnish Series W74G prestressed concrete girders with minor dimensional differences from those shown in the Plans. The 2-5/8-inch top flange taper may be reduced to 1-5/8 inches and the bottom flange may be increased to 2 feet 2 inches. Other dimensions of the girder shall be adjusted as necessary to accommodate the above mentioned changes. Reinforcing steel shall be adjusted as necessary. The overall height and top flange width shall remain unchanged.

If the Contractor elects to provide a Series W74G girder with an increased web thickness, shop plans along with supporting design calculations shall be submitted to the Engineer for approval prior to girder fabrication. The girder shall be designed for at least the same load carrying capacity as the girder shown in the Plans. The load carrying capacity of the mild steel reinforcement shall be the same as that shown in the Plans.

The Contractor may alter bulb-tee girder dimensions as specified from that shown in the Plans if:

1. The girder has the same or higher load carrying capacity (using current AASHTO Design Specification);
2. The Engineer approves, before the girder is made, complete design calculations for the girder;
3. The Contractor adjusts substructures to yield the same top of roadway elevation shown in the Plans;
4. The depth of the girder is not increased by more than 2 inches and is not decreased;
5. The web thickness is not increased by more than 1 inch and is not decreased;
6. The top flange minimum thickness of the girder is not increased by more than 2 inches, providing the top flange taper section is decreased a corresponding amount;
7. The top flange taper depth is not increased by more than 1 inch; and
8. The bottom flange width is not increased by more than 2 inches.

The Contractor shall provide four copies of the shop plans to the Engineer for approval. Only steel side forms will be approved, except plywood forms are acceptable on the end bulkheads. Approval of shop plans means only that the Engineer accepts the methods and materials. Approval does not imply correct dimensions.

6-02.3(25)B Casting

Before casting girders, the Contractor shall have possession of an approved set of shop drawings.

All concrete mixes to be used shall be pre-approved in the WSDOT plant certification process and must meet the requirements of Section 9-19.1. The temperature of the concrete when placed shall be between 50 F and 90 F. The temperature limits in Section 6-02.3(6)A do not apply to prestressed concrete girders.

Air entrainment is not required in the concrete placed into prestressed precast concrete girders unless otherwise noted. The Contractor shall use air-entrained concrete in the entire roadway deck flange of deck bulb-bee girders. Maximum and minimum air content shall be as specified in Section 6-02.3(3)A.

No welds will be permitted on steel within prestressed girders. Once the prestressing steel has been installed, no welds or grounds for welders shall be made on the forms or the steel in the girder, except as specified.

The Contractor may form circular block-outs in the girder top flanges to receive falsework hanger rods. These block-outs shall:

1. Not exceed 1 inch in diameter,
2. Be spaced no more than 72 inches apart longitudinally on the girder,

3. Be located 3 inches or more from the outside edge of the top flange on Series W42G, W50G, and W58G girders, and 6 inches or more for Series W74G girders, and
4. Be located within 1 foot 3 inches of the web centerline for bulb-tee girder.

The Contractor may form circular block-outs in the girder webs to support brackets for roadway slab falsework. These block-outs shall:

1. Not exceed 1 inch in diameter,
2. Be spaced no more than 72 inches apart longitudinally on the girder, and
3. Be positioned so as to clear the girder reinforcing and prestressing steel.

6-02.3(25)C Prestressing

Each stressing system shall have a pressure gauge or load cell that will measure jacking force. Any gauge shall display pressure accurately and readably with a dial at least 6 inches in diameter or with a digital display. Each jack and its gauge shall be calibrated as a unit and shall be accompanied by a certified calibration chart. The Contractor shall provide one copy of this chart to the Engineer. The cylinder extension during calibration shall be in approximately the position it will occupy at final jacking force.

Jacks and gauges shall be recalibrated and recertified:

1. Annually,
2. After any repair or adjustment, and
3. Any time there are indications that the jack calibration is in error.

The Engineer may use pressure cells to check jacks, gauges and calibration charts before and during tensioning.

All load cells shall be calibrated and shall have an indicator that shows prestressing force in the strand. The range of this cell shall be broad enough that the lowest 10 percent of the manufacturer's rated capacity will not be used to measure jacking force.

From manufacture to encasement in concrete, all reinforcement used in girders shall be protected against dirt, oil, grease, damage, rust and all corrosives. If strands in the stressing bed are exposed before they are encased in concrete, the Contractor shall protect them from contamination or corrosion. The protection method requires the Engineer's approval. If steel has been damaged or if it shows rust or corrosion, it will be rejected.

6-02.3(25)D Curing

During curing, the Contractor shall keep the girder in a saturated curing atmosphere until the girder concrete has reached the required release strength. If the Engineer approves, the Contractor may shorten curing time by heating the outside of impervious forms. Heat may be radiant, convection, conducted steam or hot air. With steam, the arrangement shall envelop the entire surface with saturated steam. The Engineer will not permit hot air curing until after approving the Contractor's proposed method to envelop and maintain the girder in a saturated atmosphere. Saturated atmosphere means a relative humidity of at least 90 percent. The Contractor shall never allow dry heat to touch the girder surface at any point.

Under heat curing methods, the Contractor shall:

1. Keep all unformed girder surfaces in a saturated atmosphere throughout the curing time;
2. Embed a thermocouple (linked with a thermometer accurate to plus or minus 5° F) 6 to 8 inches from the top or bottom of the girder on its centerline and near its midpoint;
3. Monitor with a recording sensor (accurate to plus or minus 5° F) arranged and calibrated to continuously record, date and identify concrete temperature throughout the heating cycle;
4. Make this temperature record available for the Engineer to inspect;
5. Heat concrete to no more than 100° F during the first two hours after pouring the concrete, and then increase no more than 25° F per hour to a maximum of 175° F;
6. Cool concrete, after curing is complete, no more than 25° F per hour, to 100° F; and
7. Keep the temperature of the concrete above 60 F until the girder reaches release strength.

The Contractor may strip side forms once the concrete has reached a minimum compressive strength of 3,000 psi. All damage from stripping is the Contractor's responsibility.

6-02.3(25)E Contractor's Control Strength

Concrete strength shall be measured on test cylinders cast from the same concrete as that in the girder. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the girder. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time-temperature relationships for the cylinders for each girder to the Engineer. When two or more girders are cast in a continuous line and in a continuous pour, a single set of test cylinders may represent all girders provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer.

The Contractor shall mold, cure, and test enough of these cylinders to satisfy specification requirements for measuring concrete strength. The Contractor may use 4-inch by 8-inch or 6-inch by 12-inch cylinders. The required design strength shall be increased 5 percent when using 4-inch by 8-inch cylinders. This 5 percent increase will not be applied for the determination of the release strength. If heat is used to shorten curing time, the Contractor shall let cylinders cool for at least 1/2 hour before testing.

Test cylinders may be cured in a moist room or water tank in accordance with AASHTO T-23 after the girder concrete has obtained the required release strength. If, however, the Contractor intends to ship the girder prior to the standard 28-day strength test, the design strength for shipping shall be determined from cylinders placed with the girder and cured under the same conditions as the girder. These cylinders may be placed in a noninsulated, moisture-proof envelope.

To measure concrete strength in the girder, the Contractor shall randomly select two test cylinders and average their compressive strengths. The compressive strength in either cylinder shall not fall more than 5 percent below the specified strength. If these two cylinders do not pass the test, two other cylinders shall be selected and tested.

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If too few cylinders were molded to carry out all required tests on the girder, the Contractor shall remove and test cores from the girder under the surveillance of the Engineer. If the Contractor casts cylinders to represent more than one girder, all girders in that line shall be cored. These cores shall measure 4 inches in diameter by web thickness high and shall be removed from just below the top flange, one approximately 3 feet to the left and the other approximately 3 feet to the right of the midpoints of the girder's length. The Engineer may accept the girder if these cores have the required compressive strength.

If the girder is cored to determine the release strength, the required patching and curing of the patch shall be done prior to shipment. If there are more than two holes or if they are not in a neutral location, they shall be patched prior to the release of prestress.

The Contractor shall coat cored holes with a Type II, Grade 2 epoxy and patch the holes using the same type concrete as that in the girder, or a mix approved during the annual plant review and approval. The girder shall not be shipped until tests show the patches have reached the required design strength of the girder.

6-02.3(25)F Prestress Release

Side and flange forms that restrain deflection shall be removed before release of the prestressing reinforcement.

All harped and straight strands shall be released in a way that will produce the least possible tension in the concrete. This release shall not occur until tests show each girder has reached the minimum compressive strength required by the Plans.

6-02.3(25)G Protection of Exposed Reinforcement

When a girder is removed from its casting bed, all bars and strands projecting from the girder shall be cleaned and painted with a minimum dry film thickness of 1 mil of paint Formula No. A-9-73. During handling and shipping, projecting reinforcement shall be protected from bending or breaking. Just before placing concrete around the painted projecting bars or strands, the Contractor shall remove from them all dirt, oil and other foreign matter.

6-02.3(25)H Finishing

The Contractor shall apply a Class 2 finish, as defined in Section 6-02.3(14), to:

1. The vertical exterior surfaces of the outside girders;
2. The bottoms, sides, and tops of the lower flanges on all girders; and
3. The bottom of the outside roadway flange of each outside bulb-tee girder section.

All other girder surfaces shall receive a Class 3 finish.

The interface on I-girders and other girders that contact the cast-in-place deck shall have a finish of dense, screeded concrete without a smooth sheen or laitance on the surface. After vibrating and screeding, and just before the concrete reaches initial set, the Contractor shall texture the interface. This texture shall be applied with a steel brooming tool that etches the surface transversely leaving grooves 1/2-inch to 1/4-inch wide, between 1/2-inch and 1/4-inch deep, and spaced 1/4-inch to 1/2-inch apart.

On the deck bulb-tee girder section, the Contractor shall test the roadway deck surface portion for flatness. This test shall occur after floating but while the concrete remains plastic. Testing shall be done with a 10-foot straightedge parallel to the girder centerline and with a flange width straightedge at right angles to the girder centerline. The Contractor shall fill depressions, cut down high spots, and refinish to correct any deviation of more than 1/4 inch within the straightedge length. This section of the roadway surface shall be finished to meet the requirements for finishing roadway slabs, as defined in Section 6-02.3(1).

The Contractor may repair rock pockets and other defects in the girder provided the repair is covered in the annual plant approval package. All other repairs and repair procedures shall be documented and approved by the Engineer prior to acceptance of the girder.

6-02.3(25)I Tolerances

The girders shall be fabricated as shown in the Plans and shall meet the dimensional tolerances listed below. Actual acceptance or rejection will depend on how the Engineer believes a defect outside these tolerances will affect the structure's strength or appearance.

1. Length (overall): ±1/4 inch per 25 feet of beam length, up to a maximum of ±1 inch
2. Width (flanges): +3/8 inch, −1/4 inch
3. Width (narrow web section): +3/8 inch, −1/4 inch
4. Girder Depth (overall): ±1/4 inch
5. Flange Depth: +1/4 inch, −1/8 inch
6. Strand Position: ±1/4 inch from the center of gravity of a strand group and of an individual strand
7. Longitudinal Position of the Harping Point: ±18 inches
8. Bearing Recess (center recess to beam end): ±1/4 inch
9. Beam Ends (deviation from square or designated skew): Horizontal: ±1/2 inch from web centerline to flange edge; Vertical: ±1/8 inch per foot of beam depth
10. Bearing Area Deviation from Plane (in length or width of bearing): 1/16 inch
11. Stirrup Reinforcing Spacing: ±1 inch
12. Stirrup Projection from Top of Beam: ±3/4 inch
13. Mild Steel Concrete Cover: −1/8, +3/8 inch
14. Offset at Form Joints (deviation from a straight line extending 5 feet on each side of joint): ±1/4 inch
15. Differential Camber Between Girders in a Span (measured in place at the job site)

    For I-girders: 1/8 inch per 10 feet of beam length (Series W42G, W50G, W58G and W74G).

    For deck bulb-tee girders: Cambers shall be equalized by an approved method when the difference in cambers between adjacent girders or stages measured at mid-span exceeds 1/4 inch.
16. Position of Inserts for Structural Connections: ±1/2 inch
17. Position of Lifting Loops: ±3 inches longitudinal, ±1 inch transverse
18. Weld plates for bulb-tee girders shall be placed ±1/2 inch longitudinal, and ±1/8 inch vertical

6-02.3(25)J Horizontal Alignment

The Contractor shall check and record the horizontal alignment of both top and bottom flanges of each girder upon removal of the girder from the casting bed. The Contractor shall also check and record the horizontal alignment within a two-week period prior to shipment, but no less than three days prior to shipment. If the girder remains in storage for a period exceeding 120 days, the Contractor shall check and record the horizontal alignment at approximately 120 days. Each check shall be made by measuring the distance between each flange and a chord that extends the full length of the girder. The Contractor shall perform and record each check at a time when the alignment of the girder is not influenced by temporary differences in surface temperature. These records shall be available for the Engineer's inspection and included in the Contractor's Prestressed Concrete Certificate of Compliance.

Immediately after the girder is removed from the casting bed, neither flange shall be offset more than 1/8 inch for each 10 feet of girder length. During storage and prior to shipping, the offset (with girder ends plumb and upright and with no external force) shall not exceed 1/4 inch per 10 feet of girder length. Any girder within this tolerance may be shipped, but must be corrected at the job site to the 1/8 inch maximum offset per 10 feet of girder length before concrete is placed into the diaphragms.

The Engineer may permit the use of external force to correct girder alignment at the plant or job site if the Contractor provides stress calculations and a proposed procedure. If external force is permitted, it shall not be released until after the roadway slab has been placed and cured ten days.

6-02.3(25)K Girder Deflection

The Contractor shall check and record the vertical deflection (camber) of each girder upon removal of the girder from the casting bed. The Contractor shall also check and record the vertical deflection (camber) within a two-week period prior to shipment, but no less than three days prior to shipment. If the girder remains in storage for a period exceeding 120 days, the Contractor shall check and record the vertical deflection (camber) at approximately 120 days. The Contractor shall perform and record each check at a time when the alignment of the girder is not influenced by temporary differences in surface temperature. These records shall be available for the Engineer's inspection and included in the Contractor's Prestressed Concrete Certification of Compliance.

The "D" dimensions shown in the Plans are computed girder deflections at midspan based on a time elapse of 120 days after release of the prestressing strands. A positive (+) "D" dimension indicates upward deflection.

The Contractor shall control the deflection of prestressed concrete girders that are to receive a cast-in-place slab by scheduling fabrication or other means. The actual girder deflection at the midspan may vary from the "D" dimension at the time of slab forming by a maximum of plus 1/2 inch for girder lengths up to 80 feet and plus 1 inch for girder lengths over 80 feet. The method used by the Contractor to control the girder deflection shall not cause damage to the girders or any over stress when checked in accordance with the AASHTO Specifications.

All costs, including any additional Contracting Agency engineering expenses, in connection with controlling the girder deflection shall be borne by the Contractor.

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6-02.3(25)L Handling and Storage

During handling and storage, each girder shall always be kept plumb and upright. It shall be lifted only by the lifting strands at either end. Series W42G, W50G and W58G girders can be picked up at a maximum angle of 30 degrees to the vertical (measured in the longitudinal plane of the girder). All other prestressed girders shall be picked up vertical. Girders shall be braced laterally to prevent tipping or buckling as specified in the Plans.

Before moving a long girder, the Contractor shall check it for any tendency to buckling. Each girder that may buckle shall be braced on the sides to prevent buckling. This bracing shall be attached securely to the top flanges of the girder. The lateral bracing shall be in place during all lifting or handling necessary for transportation from the manufacturing plant to the job site and erection of the girder. The Contractor is cautioned that for some delivery routes more conservative guidelines for lateral bracing may be required. Before removing the bracing to cast diaphragms, the Contractor shall fasten all girders in place by other means.

If the Contractor wishes to deviate from these handling and bracing requirements, the vertical pickup, or the pickup location, the proposed method shall be analyzed by the Contractor's engineer and submitted with the supporting calculations to the Engineer for approval. The Contractor's analysis shall conform to Articles 5.2 and 5.3 of the PCI *Design Handbook, Precast and Prestressed Concrete,* Third Edition, or other approved methods. The Contractor's calculations shall verify that the concrete stresses in the prestressed girder do not exceed those listed in Section 6-02.3(25)M.

If girders are to be stored, the Contractor shall place them on a stable foundation that will keep them in a vertical position. Stored girders shall be supported at the bearing recesses or, if there are no recesses, approximately 18 inches from the girder ends. For long-term storage of girders with initial horizontal curvature, the Contractor may wedge one side of the bottom flange, tilting the girders to control curvature. If the Contractor elects to set girders out of plumb during storage, the Contractor shall have the proposed method analyzed by the Contractor's engineer to ensure against damaging the girder.

6-02.3(25)M Shipping

After the girder has reached its 28-day design strength, and the fabricator believes it to comply with the specification, the girder and a completed Certification of Compliance (DOT 350-151), signed by a Precast/Prestressed Concrete Institute Certified Technician or a professional engineer, acceptable to the Contracting Agency, shall be presented to the Engineer for inspection. If the Engineer finds the certification and the girder to be acceptable, the Engineer will stamp the girder "Approved for Shipment."

No prestressed girders shall be shipped that are not stamped "Approved for Shipment."

No bulb-tee girder shall be shipped for at least seven days after concrete placement. No other girder shall be shipped for at least ten days after concrete placement.

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Girder support during shipping shall meet these requirements unless otherwise shown in the Plans:

Type of Girder	**Centerline Support Within This Distance From Either End**
Series W42G and W50G and all bulb-tee girders	3 feet
Series W58G	4 feet
Series W74G	5 feet

If the Contractor wishes to use other support locations, they may be proposed to the Engineer for review and approval in accordance with Section 6-02.3(25)L. The Contractor's proposal shall include calculations showing that concrete stresses in the girders will not exceed those listed below:

Criteria for Checking Girder Stresses at the Time of Lifting or Transporting and Erecting

Stresses at both support and harping points shall be satisfied based on these criteria:

1. Allowable compression stress, $f'_c - 0.60\, f'_{cm}$
   a. f'_{cm} = compressive strength at time of lifting or transporting verified by test but shall not exceed design compressive strength (f'_c) at 28 days in psi + 1,000 psi
2. Allowable tension stress, f_t
   a. With no bonded reinforcement = $3\sqrt{f'_{cm}}$
   b. With bonded reinforcement to resist total tension force in the concrete computed on the basis of an uncracked section = $7.5\sqrt{f'_{cm}}$. The allowable tensile stress in reinforcement is 30 ksi (AASHTO M-31, Gr. 60)
3. Prestress losses
   a. 1 day to 1 month = 20,000 psi
   b. 1 month to 1 year = 35,000 psi
   c. 1 year or more = 45,000 psi (max.)
4. Impact on dead load
   a. Lifting from casting beds = 0 percent
   b. Transporting and erecting = 20 percent

6-02.3(25)N Prestress Concrete Girder Erection

Before beginning to erect any prestressed concrete girders, the Contractor shall submit to the Engineer for review and shall have received approval for the erection plan and procedure describing the methods the Contractor intends to use. The erection plan and procedure shall provide complete details of the erection process including but not limited to:

1. Temporary falsework support, bracing, guys, deadmen and attachments to other structure components or objects;
2. Procedure and sequence of operation;
3. Girder stresses during progressive stages of erection;

4. Girder weights, lift points, and lifting devices, spreaders, angle of lifting strands in accordance with Section 6-02.3(25)L, etc.;
5. Crane(s) make and model, weight, geometry, lift capacity, outrigger size and reactions;
6. Girder launcher or trolley details and capacity (if intended for use); and
7. Locations of cranes, barges, trucks delivering girders, and the location of cranes and outriggers relative to other structures, including retaining walls and wing walls.

The erection plan shall include drawings, notes, catalog cuts, and calculations clearly showing the above listed details, assumptions, and dimensions. Material properties and specifications, structural analysis, and any other data used shall also be included. The plan shall be prepared by (or under the direct supervision of) a Professional Engineer, licensed under Title 18 RCW, State of Washington, in the branch of Civil or Structural, and shall carry the engineer's seal and signature, in accordance with Section 6-02.3(16).

The Contractor shall submit the erection plans, calculations and procedures directly to the Bridge and Structures Office, Construction Support Engineer, in accordance with Section 6-02.3(16). After the plan is approved and returned to the Contractor, all changes that the Contractor proposes shall be submitted to the Engineer for review and approval.

When prestressed girders arrive on the project, the Project Engineer will confirm that they are stamped "Approved for Shipment" and that they have not been damaged in shipment before accepting them.

The concrete in piers and crossbeams shall reach at least 80 percent of design strength before girders are placed on them. The Contractor shall hoist girders only by the lifting strands at the ends, always keeping the girders plumb and upright.

Instead of the oak block wedges shown in the Plans, the Contractor may use Douglas fir blocks if the grain is vertical.

Before the grout pads are placed, the concrete beneath them shall be thoroughly cleaned, roughened, and wetted with water to ensure proper bonding. Pads shall be kept wet continuously until they reach a compressive strength of at least 2,000 psi. Grout pads shall reach this strength before girders are set on them. Grout compressive strength will be determined by fabricating cubes in accordance with WSDOT Test Method 813 and testing in accordance with AASHTO T-106.

The Contractor shall check the horizontal alignment of both the top and bottom flanges of each girder before placing concrete in the bridge diaphragms as described in Section 6-02.3(25)J.

The Contractor shall fill all block-out holes and patch any damaged area caused by the Contractor's operation, with an approved mix, to the satisfaction of the Engineer.

6-02.3(25)O Deck Bulb-Tee Girder Flange Connection

The Contractor shall submit a method of equalizing deck bulb-tee girder deflections to the Engineer for approval. This submittal shall be prepared by, or under the direction of, a professional engineer licensed under Title 18 RCW, State of Washington, and shall carry the engineer's signature and seal. This submittal shall be made a minimum of 60 days prior to field erection of the deck bulb-tee girder. Deflection equalizing methods approved for previous Contracting Agency contracts will be acceptable providing the bridge configuration is similar and the previous method was satisfactory. A listing of the previous Contracting Agency contract numbers for which the method was used shall be included with the submittal.

On the deck bulb-tee girders, girder deflection shall be equalized utilizing the approved method before girders are weld-tied and before keyways are filled. Keyways between the nonshrink grout shall have a compressive strength of 4,000 psi before the equalizing equipment is removed. Compressive strength shall be determined by fabricating cubes in accordance with WSDOT Test Method 813 and testing in accordance with AASHTO T-106.

Welding grounds shall be attached directly to the steel plates being welded when welding the weld-ties on bulb-tee girders.

No construction equipment shall be placed on the structure, other than equalizing equipment, until the girders have been weld-tied and the keyway grout has attained a compressive strength of 4,000 psi.

6-02.3(26) Cast-In-Place Prestressed Concrete

Intentionally Omitted.

6-02.3(27) Concrete for Precast Units

Concrete for precast non-prestressed units shall meet all the requirements for a Contractor-provided mix design and the following acceptance limits:

Maximum Slump	4 inches
Minimum Entrained Air	4 1/2 percent
Compressive Strength	Specified Design Strength

If the design strength of the precast concrete is 4,000 psi or less, the Contractor may use Contracting Agency provided mix design Class 4000 with air.

Precast units shall not be removed from forms until the concrete has attained a minimum compressive strength of 70 percent of the specified design strength as verified by rebound number determined in accordance with ASTM C805.

Precast units shall not be shipped until the concrete has reached the specified design strength as determined by testing cylinders made from the same concrete as the precast units. The cylinders shall be made, handled, and stored in accordance with WSDOT Test Method 809 Method 2 and compression tested in accordance with WSDOT Test Methods 801 and 811.

6-02.3(28) Precast Concrete Panels

The Contractor shall perform quality control inspection. The manufacturing plant for precast concrete units shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of precast member to be produced and shall be approved by WSDOT as a Certified Precast Concrete Fabricator prior to the start of production. WSDOT Certification will be granted at, and renewed during, the annual precast plant review and approval process. Products which shall conform to this requirement include noise barrier panels, wall panels, floor and roof panels, marine pier deck panels, retaining walls, pier caps and bridge deck panels.

Prior to the start of production of the precast concrete units, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the work. If the Inspector observes any nonspecification work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the units(s) will be rejected.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the work and verify the quality of that work. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

If products are prestressed, all prestressing materials and methods shall be in accordance with Section 6-02.3(25).

6-02.3(28)A Shop Drawings

Before casting the structural elements, the Contractor shall submit:

1. Seven sets of shop drawings for approval by the Bridge and Structures Engineer, Department of Transportation, Transportation Building, Olympia, WA 98504; and,
2. Two sets of shop drawings to the Project Engineer.

These shop drawings shall show complete details of the methods, materials and equipment the Contractor proposes to use in prestressing/precasting work. The shop drawings shall follow the design conditions shown the Plans unless the Engineer approves equally effective variations.

The shop drawings shall contain as a minimum:

1. Unit shapes (elevations and sections) and dimensions.
2. Finishes and method of constructing the finish (i.e., forming, rolling, etc.)
3. Reinforcing, joint, and connection details.
4. Lifting, bracing, and erection inserts.
5. Locations and details of hardware attached to the structure.
6. Relationship to adjacent material.

Approval of these shop drawings shall not relieve the Contractor of responsibility for accuracy of the drawings or conformity with the Contract. Approval will not indicate a check on dimensions.

The Contractor may deviate from the approved shop drawings only after obtaining the engineer's approval of a written request that describes the proposed changes. Approval of a change in method, material or equipment shall not relieve the Contractor of any responsibility for completing the work successfully.

Before completion of the Contract, the Contractor shall provide the Engineer with reproducible originals of the shop drawings (and any approved changes). These shall be clear, suitable for microfilming, and on permanent sheets that conform with the size requirements of Section 6-01.9.

6-02.3(28)B Casting

Before casting precast concrete units, the Contractor and Fabrication Inspector shall have possession of an approved set of shop drawings.

All concrete mixes to be used shall be preapproved in the WSDOT plant certification process, and shall meet all the requirements for a Contractor provided mix design with the following acceptance limits:

Minimum Slump	4 inches
Maximum Slump	6 inches
Entrained Air	Per Section 6-02.3(3)A
Compressive Strength	Specified Design Strength

If the design strength of the precast concrete is 4,000 psi or less, the Contractor may use Contracting Agency-provided mix design Class 4000 with air.

Precast units shall not be removed from forms until the concrete has attained a minimum compressive strength of 70 percent of the specified design strength.

Forms may be steel or plywood faced, providing they impart the required Finish to the concrete.

6-02.3(28)C Curing

Concrete in the precast units shall be cured by either moist or accelerated curing methods. The methods to be used shall be preapproved in the WSDOT plant certification process.

1. For moist curing, the surface of the concrete shall be kept covered or moist until such time as the compressive strength of the concrete reaches the strength specified for stripping. Exposed surfaces shall be kept continually moist by fogging, spraying or covering with moist burlap or cotton mats. Moist curing shall commence as soon as possible following completion of surface finishing.
2. For accelerated curing, heat shall be applied at a controlled rate following the initial set of concrete in combination with an effective method of supplying or retaining moisture. Moisture may be applied by a cover of moist burlap, cotton matting or other effective means. Moisture may be retained by covering the unit with an impermeable sheet.

Heat may be radiant or convection or conducted stream of hot air. Heat the concrete to no more than 100°F during the first two hours after pouring the concrete, and then increase no more than 25°F per hour to a maximum of 175°F. After curing is complete, cool the concrete no more than 25°F per hour to 100°F. Maintain the concrete temperature above 60°F until the unit reaches stripping strength.

Concrete temperature shall be monitored by means of a thermocouple embedded in the concrete (linked with a thermometer accurate to plus or minus 5 F). The recording sensor (accurate to plus or minus 5 F) shall be arranged and calibrated to continuously record, date and identify concrete temperature throughout the heating cycle. This temperature record shall be made available to the Engineer for inspection and become a part of the documentation required.

The Contractor shall never allow dry heat to directly touch exposed unit surfaces at any point.

6-02.3(28)D Contractor's Control Strength

The concrete strength at stripping and the verification of design strength shall be determined by testing cylinders made from the same concrete as that in the precast units. The cylinders shall be made, handled, and stored in accordance with WSDOT Test Method 809 Method 2 and compression tested in accordance with WSDOT test Methods 801 and 811.

For accelerated-cured units, concrete strength shall be measured on test cylinders cast from the same concrete as that in the unit. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the unit. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time-temperature relationship for the cylinders for each unit to the Engineer. When two or more units are cast in a continuous line and in a continuous pour, a single set of test cylinders may represent all units provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer.

The Contractor shall mold, cure and test enough of these cylinders to satisfy specification requirements for measuring concrete strength. The Contractor may use 4-inch by 8-inch or 6-inch by 12-inch cylinders. The required design strength shall be increased 5 percent when using 4-inch by 8-inch cylinders. This 5 percent increase will not be applied for the determination of the stripping strength. The Contractor shall let cylinders cool for at least one-half hour before testing for release strength.

Test cylinders may be cured in a moist room and water tank in accordance with AASHTO T-23 after the unit concrete has obtained the required release strength. If, however, the Contractor intends to ship the unit prior to standard 28-day strength test, the design strength for shipping shall be determined from cylinders placed with the unit and cured under the same conditions as the unit. These cylinders may be placed in a noninsulated, moisture-proof envelope.

To measure concrete strength in the precast unit, the Contractor shall randomly select two test cylinders and average their compressive strengths. The compressive strength in either cylinder shall not fall more than 5 percent below the specified strength. If these two cylinders do not pass the test, two other cylinders shall be selected and tested.

6-02.3(28)E Finishing

The Contractor shall provide a finish on all relevant concrete surfaces as defined in Section 6-02.3(14), unless the Plans or Special Provisions require otherwise.

6-02.3(28)F Tolerances

The units shall be fabricated as shown in the Plans, and shall meet the dimensional tolerances listed in PCI MNL-116-85, unless otherwise required by the Plans or Special Provisions.

6-02.3(28)G Handling and Storage

The Contractor shall lift all units only by adequate devices at locations designated on the shop drawings. When these devices and locations are not shown in the Plans, Section 6-02.3(25)L shall apply.

Precast units shall be stored off the ground on foundations suitable to prevent differential settlement or twisting of the units. Stacked units shall be separated and supported by dunnage of uniform thickness capable of supporting the units. Dunnage shall be arranged in vertical planes. The upper units of a stacked tier shall not be used as storage areas for shorter units unless substantiated by engineering analysis and approved by the Engineer.

6-02.3(28)H Shipping

Precast units shall not be shipped until the concrete has reached the specified design strength. The units shall be supported in such a manner that they will not be damaged by anticipated impact on their dead load. Sufficient padding material shall be provided between tie chains and cables to prevent chipping or spalling of the concrete.

6-02.3(28)I Erection

When the precast units arrive on the project, the Project Engineer will confirm that they are stamped "Approved for Shipment." The Project engineer will evaluate the units for damage before accepting them.

The Contractor shall lift all units by suitable devices at locations designated on the shop drawings. Temporary shoring or bracing shall be provided, if necessary. Units shall be properly aligned and leveled as required by the Plans. Variations between adjacent units shall be leveled by a method approved by the Engineer.

BRIDGE DESIGN MANUAL GLOSSARY

Accelerated curing Curing of concrete, mortar, grout or neat cement paste at an elevated temperature using live steam or radiant heat.

Active earth pressure Lateral pressure resulting from the earth being retained by a structure or component, which is tending to move away from the soil mass.

Active earth wedge Wedge of earth with a tendency to move if not retained by a structure or component.

Adjacent box beams Precast, prestressed concrete box beams that are set side-by-side.

Admixture A material other than water, aggregates, hydraulic cement and fiber reinforcement, used as an ingredient of concrete or mortar and added to the batch immediately before or during its mixing.

Aeroelastic vibration Periodic, elastic response of a structure to wind pressure.

Agency A public entity that issues design/construction specifications. Usually the federal government or a state, county or municipal government or entity created by legislation such as a tollroad, through-way or port authority.

Alkali-aggregate reaction Chemical reaction in either mortar or concrete between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates; under certain conditions, deleterious expansion of concrete or mortar may result.

Allowable stress In working stress design, the maximum stress permitted for a specified condition.

Analysis A mathematical process by which structural deformations, forces and stresses are determined.

Anchorage In post-tensioning, a mechanical device used to anchor the stressed tendon to the concrete; in pretensioning, a device used to anchor the tendon until the concrete has reached a predetermined strength and the prestressing force has been transferred to the concrete; for reinforcing bars, a length of reinforcement, or a mechanical anchor or hook, or combination thereof, at the end of a bar needed to transfer the force carried by the bar into the concrete.

Anchorage blister A built-out area in the web, flange or flange-web junction for the incorporation of tendon anchorage fittings.

Anchorage seating Deformation of anchorage or seating of tendons in anchorage device when prestressing force is transferred from jack to anchorage device.

Anchorage zone The portion of the structure in which the prestressing force is transferred from the anchorage device onto the local zone of the concrete, and then distributed more widely into the general zone of the structure.

Aspect ratio Ratio between length and width of a rectangle.

At Jacking See Jacking.

At loading See Loading.

At transfer See Transfer.

Axle unit Single axle or tandem axle.

Beam A longitudinal stringer used primarily for relatively short spans; used interchangeably with stringer or girder.

Berm An earthwork used to redirect or slow down impinging vehicles or vessels; also used to stabilize fill, embankment or soft ground and cut slopes.

Blanketing See Debonding.

Bleeding The autogenous flow of mixing water within, or its emergence from, newly placed concrete or mortar, caused by the settlement of the solid materials within the mass.

Bonded tendon Prestressing tendon that is bonded to concrete either directly or through grouting.

BRIDGE DESIGN MANUAL GLOSSARY

Bracket (or Corbel) — Short (haunched) cantilever that projects from the face of a column or wall to support a concentrated load or beam reaction.

Bridle strand (lead strand) — In a production plant, prestressing strand which is not embedded in a precast concrete member but extends from near the end of the member(s), through the end of the casting bed. It is reused for each cast by splicing it to the end of the "production" strand, which is embedded in, and extends a short distance out of the member.

Bursting force — Tensile forces in the concrete in the vicinity of post-tensioning anchorages caused by the transfer of prestressing forces.

Cast-in-place concrete — Concrete placed in its final location in the structure while still in a plastic state.

Cementitious materials — Materials having cementing properties such as portland cement, blast furnace slag, fly ash, silica fume, and metakaolin.

Centrifugal force — A lateral force resulting from a change in direction of the movement of a vehicle.

Closely spaced anchorages — Anchorage devices are defined as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage devices in the direction considered.

Closure — A placement of cast-in-place concrete used to connect two or more previously cast portions of a structure.

Coating — Material used to protect prestressing tendons against corrosion, to reduce friction between tendon and duct, or to debond prestressing tendons.

Coefficient of thermal expansion — Increase or decrease in linear dimension per unit length or increase or decease in volume per unit volume per degree of temperature increase or decrease, respectively.

Composite construction — Concrete components or concrete and steel components interconnected to respond to force effects as a unit.

Compressive strength of concrete (f'_c) — Specified compressive strength of concrete in pounds per square inch, psi.

Concrete cover — The specified distance between the surface of the reinforcing bars, strands, post-tensioning ducts, anchorages or other embedded items, and the surface of the concrete.

Concrete, structural lightweight — A concrete containing lightweight aggregate having an air-dry unit weight as determined by "Method of Test for Unit Weight of Structural Lightweight Concrete" (ASTM C 567), not exceeding 115 pcf. In this specification, a lightweight concrete without natural sand is termed "all-lightweight concrete" and one in which all fine aggregate consists of normal weight sand is termed "sand-lightweight concrete."

Confinement — A condition where the disintegration of the concrete under compression is prevented by lateral and/or circumferential reinforcement.

Confinement, anchorage — Anchorage for a post-tensioning tendon that functions on the basis of containment of the concrete in the local anchorage zone by special reinforcement.

Corbel — see Bracket.

Couplers (couplings) — Means by which prestressing force is transmitted from one partial-length prestressing tendon to another.

Creep of concrete — Time-dependent deformation of concrete under sustained load.

Curvature friction — Friction resulting from the tendon moving against the duct when tensioning, due to bends or curves in the specified prestressing tendon profile.

Debonding (blanketing) — Wrapping, sheathing, or coating prestressing strand to prevent bond between strand and surrounding concrete.

Deck — A component of a bridge superstructure, with or without wearing surface.

Deck slab — A solid concrete slab, resisting and distributing wheel loads to the supporting components.

BRIDGE DESIGN MANUAL GLOSSARY

Decompression	The stage at which the compressive stresses, induced by prestress, are overcome by the tensile stresses.
Deep component	Components in which the distance from the point of 0.0 shear to the face of the support is less than 2d, or components in which a load causing more than one-third of the shear at a support is closer than 2d from the face of the support.
Deep draft waterways	A navigable waterway used by merchant ships with loaded drafts of 14-60 ft.
Deformed reinforcement	Deformed reinforcing bars, deformed wire, welded smooth wire fabric, and welded deformed wire fabric.
Design lane	A notional traffic lane positioned transversely on the roadway.
Design load	All applicable loads and forces or their related internal moments and forces used to proportion members. For design by SERVICE LOAD DESIGN, design load refers to loads without load factors. For design by STRENGTH DESIGN METHOD, design load refers to loads multiplied by appropriate load factors.
Design strength	Nominal strength multiplied by a strength reduction factor, ϕ.
Design water depth	Depth of water at mean high water.
Development length	Length of embedded reinforcement required to develop the design strength of the reinforcement at a critical section.
Deviation saddle	A concrete block built-out in a web, flange or web-flange junction, cast monolithically with the segment, to control the geometry of, or to provide a means for changing direction of external tendons.
Distortion	Change in structural geometry.
Dolphin	Protective object, which may have its own fender system, usually circular in plan, and structurally independent from the bridge.
Duct	Hole or void formed in a prestressed concrete member to accommodate a tendon for post-tensioning.
Durability	The ability of concrete to resist weathering action, chemical attack, abrasion and other conditions of service.
Dynamic load allowance	An increase in the applied static force effects to account for the dynamic interaction between the bridge and moving vehicles.
Edge distance	The minimum distance between the centerline of reinforcement or other embedded elements and the edge of the concrete.
Effective depth	The depth of a component effective in resisting flexural or shear forces; as for d and d_v.
Effective prestress	Stress remaining in concrete due to prestressing after all calculated losses have been deducted, excluding effects of superimposed loads and weight of member; stress remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load.
Elastic shortening of concrete	Shortening of member caused by application of forces induced by prestressing.
Embedment	An object, usually metal, plastic or wood, intentionally cast into concrete and used to lift the member, provide a hole or sleeve in the member, or to make a welded or bolted attachment to the member.
Embedment length	The length of reinforcement or anchorage provided beyond a critical section over which transfer of force between concrete and reinforcement may occur.
End anchorage	Length of reinforcement, or mechanical anchor, or hook, or combination thereof, beyond point of zero stress in reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member.
End block	Enlarged end section of a member designed to reduce anchorage stresses.

BRIDGE DESIGN MANUAL GLOSSARY

Equivalent fluid — A notional substance whose density is such that it would exert the same pressure as the soil it is seen to replace for computational purposes.

Exposed — A condition in which a portion of a bridge's substructure or superstructure is subject to physical contact by any portion of a colliding vessel's bow, deckhouse or mast.

External tendon — A post-tensioning tendon placed outside of the body of concrete, usually inside a box girder.

Extreme — A maximum or a minimum.

Factored load — Load, multiplied by appropriate load factors, used to proportion members by the strength design method.

Fatigue strength — The greatest stress which can be sustained for a given number of stress cycles without failure.

Fender — Protection hardware attached to the structural component to be protected, or used to delineate channels or to redirect aberrant vessels.

Fiber reinforced plastic reinforcement — Reinforcement made with a resin matrix containing continuous fibers that work together as a single element.

Finite element method — A method of analysis in which a structure is discretized into elements connected at nodes, the shape of the element displacement field is assumed, partial or complete compatibility is maintained among the element interfaces, and nodal displacements are determined by using energy variational principles or equilibrium methods.

Flexible duct — A loosely interlocked duct which can be coiled into a 4.0 ft diameter without damage.

Fly ash — The finely divided residue resulting from the combustion of ground or powdered coal and which is transported from the firebox through the boiler by flue gases.

Force effects — A deformation or a stress resultant, i.e., thrust, shear, torque and/or moment, caused by applied loads, imposed deformations or volumetric changes.

Form — A mold into which fresh concrete is placed to fabricate a specified shape.

Frazil ice — Ice resulting from turbulent water flow.

Friction (post-tensioning) — Surface resistance between tendon and duct in contact during tensioning (also, see Curvature friction).

General zone — Region adjacent to a post-tensioning anchorage within which the prestressing force spreads out to an essentially linear stress distribution over the cross-section of the component.

Girder — The main longitudinal superstructural element. The term is used primarily for I- and box-section bridges of relatively long span; used interchangeably with stringer or beam.

Global — Pertinent to the entire superstructure or to the whole bridge.

Grillage analysis method — A method of analysis in which all or part of the superstructure is discretized into orthotropic components that represent the characteristics of the structure.

Grout — A mixture of cementitious material and water, with or without aggregate, proportioned to produce a consistency without segregation of the constituents. The consistency may range from that required for pouring to that required for dry packing.

Grout opening (vent) — Inlet, outlet, vent, or drain in post-tensioning duct for grout, water, or air.

Heat of hydration — Heat evolved by chemical reactions with water, such as that evolved during the setting and hardening of portland cement, or the difference between the heat of solution of dry cement and that of partially hydrated cement.

High performance concrete — Concrete that meets special performance and uniformity requirements including: ease of placement and consolidation without affecting strength; long-term mechanical properties; early high strength; toughness; volume stability; and, longer life in severe environments.

BRIDGE DESIGN MANUAL GLOSSARY

Honeycomb Voided or unsound area of concrete resembling the cellular structure of a honeycomb. May exist at the face of formed surfaces or internally and is caused by incomplete vibration or consolidation often in combination with restrictions to the placement of concrete caused by the close spacing of reinforcement or other obstructions.

Influence surface A continuous or discretized function over a bridge deck whose value at a point, multiplied by a load acting normal to the deck at that point, yields the force effect being sought.

Intermediate anchorage Anchorage not located at the end surface of a member or segment for tendons that do not extend over the entire length of the member or segment; usually in the form of embedded anchors, blisters, ribs, or recess pockets.

Internal tendon A post-tensioning tendon placed within the body of concrete.

Isotropic reinforcement An arrangement of reinforcement in which the bars are orthogonal and the reinforcement ratios in the two directions are equal.

Jacking (at jacking) Tensioning of the prestressing tendons; at the time of tensioning.

Jacking force The force exerted by the device, which introduces tension into the prestressing tendons.

Knot A velocity of 1.1508 mph.

Lane live load The combination of tandem axle and uniformly distributed loads, or the combination of the design truck and design uniformly distributed load.

Launching bearing Temporary bearings with low friction characteristics used for construction of bridges by the segmental launching method.

Launching nose Temporary steel assembly attached to the front of an incrementally launched bridge to reduce superstructure force effects during launching.

Lever rule The statical summation of moments about one point to calculate the reaction at a second point.

Lightweight concrete Concrete containing lightweight aggregate and having an air-dry unit weight not exceeding 135 pcf, as determined by ASTM C-567.

Liquefaction The loss of shear strength in a saturated soil due to excess hydrostatic pressure. In saturated, cohesionless soils, such a strength loss can result from loads that are applied instantaneously or cyclically, particularly in loose fine to medium sands that are uniformly graded.

Load The effect of acceleration, including that due to gravity, imposed deformation or volumetric change.

Load factor A coefficient expressing the probability of variations in the nominal load for the expected service life of the bridge.

Loading (at loading) The time when loads are applied. Such loads include prestressing forces and permanent loads, but generally not live loads; refers to the maturity of the concrete at the time of loading.

Local Pertinent to a component or subassembly or components.

Local zone The volume of concrete surrounding and immediately ahead of the anchorage device, subjected to high compressive stresses.

Loss of prestress Reduction in prestressing force resulting from combined effects of strains in concrete and steel, including effects of elastic shortening, creep and shrinkage of concrete, relaxation of steel stress, and for post-tensioned members, friction and anchorage seating.

Low-relaxation steel Prestressing strand in which the steel relaxation losses have been substantially reduced by stretching at an elevated temperature during manufacture.

Method of analysis A mathematical process by which structural deformations, forces and stresses are determined.

BRIDGE DESIGN MANUAL GLOSSARY

Mode of vibration	A shape of dynamic deformation associated with a frequency of vibration.
Modulus of elasticity	The ratio of uniaxial normal stress to corresponding strain for tensile or compressive stress below the proportional limit of a material.
Modulus of rupture	A measure of the ultimate load-carrying capacity of a plain concrete beam and sometimes referred to as rupture modulus or rupture strength. It is calculated as the apparent tensile stress in the extreme fiber of a test specimen under the load which produces rupture.
Multi-beam bridge deck	A bridge constructed with precast, prestressed concrete beams that are placed side-by-side on the supports.
Navigable waterway	A waterway, determined by the U.S. Coast Guard as being suitable for interstate or foreign commerce, as described in 33CFR205-25.
Node	A point where finite elements or grid components meet; in conjunction with finite differences, a point where the governing differential equations are satisfied.
Nominal load	An arbitrarily selected design load level.
Nominal strength	Strength of a member or cross-section calculated in accordance with provisions and assumptions of the STRENGTH DESIGN METHOD before application of any strength reduction factors.
Normally consolidated soil	A soil for which the current overburden pressure is the greatest that has been experienced.
Normal-weight concrete	Concrete having a weight between 135 and 160 pcf.
Orthotropic	Perpendicular to each other; having physical properties which differ in two or more orthogonal directions.
Overconsolidated soil	A soil which has been under greater overburden pressure than currently exists.
Overconsolidation ratio	OCR equals Maximum Preconsolidation Pressure divided by Overburden Pressure.
Overlay	A layer of portland cement or asphaltic concrete placed on a new or existing bridge deck or roadway and used as a wearing or leveling course or both. Portland cement concrete overlays may be non-composite or bonded to the underlying deck, with or without connecting reinforcement, to increase the structural depth and capacity of the section.
Partially debonded strand	A prestressing strand that is bonded for a portion of its length and intentionally debonded elsewhere through the use of mechanical or chemical means. Also called shielded or blanketed strand.
Partially prestressed concrete	Concrete with a combination of tensioned prestressing strands and reinforcing bars.
Passive earth pressure	Lateral pressure resulting from the earth resisting the lateral movement of a structure or component into the soil mass.
Permanent loads	Loads and forces which are, or assumed to be, constant upon completion of construction.
Permeability	The ability of concrete to resist penetration of liquids and gases.
Permit vehicle	Any vehicle whose right to travel is administratively restricted in any way due to its weight or size.
Plain reinforcement	Reinforcement that does not conform to the definition of deformed reinforcement.
Post-tensioning	A method of prestressing concrete whereby the tendon is kept from bonding to the plastic (wet) concrete, then elongated and anchored directly against the hardened concrete, imparting stresses through end bearing.
Post-tensioning duct	A form device used to provide a path for post-tensioning tendons or bars in hardened concrete. The following types are in general use: see flexible, rigid, semi-rigid duct.

BRIDGE DESIGN MANUAL GLOSSARY

Pozzolan	A siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value in finely divided form and in the presence of moisture, chemically reacts with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.
Precast members	Concrete elements cast in a location other than their final position.
Precompressed zone	Portion of flexural member cross-section compressed by prestressing force.
Prestressed concrete	Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.
Prestressing steel	High strength steel used to prestress concrete and consisting of seven-wire strands, single wires, bars, rods, or groups of wires or strands.
Pretensioning	A method of prestressing concrete whereby the tendons are elongated, anchored while the concrete in the member is cast, and released when the concrete is strong enough to receive the forces from the tendon through bond.
Quality assurance	Actions taken by an owner or his representative to provide assurance that what is being done and what is being provided are in accordance with the applicable standards of good practice for the work.
Quality control	Actions taken by a manufacturer or contractor to provide control over what is being done and what is being provided so that the applicable standards of good practice for the work are followed.
Radiant heat curing	Curing of concrete, mortar, grout or neat cement paste at an elevated temperature using heat applied by means of pipes circulating steam, oil or hot water, by electric blankets or heating elements or by circulating warm air.
Reinforcement	Bats, wires, strands, or other slender members, which are embedded in concrete in such a manner that they and the concrete act together in resisting forces.
Relaxation (of tendon stress)	Time-dependent reduction of stress in prestressing tendon at constant strain.
Release strength (prestress release, prestress release strength)	The strength of concrete when the strands are "detensioned" in pretensioned members.
Reliability index	A quantitative assessment of safety expressed as the ratio of the difference between the mean resistance and mean force effect to the combined standard deviation of resistance and force effect.
Required strength	Strength of a member or cross-section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in Article 3.22.
Rigid duct (post-tensioning)	Seamless tubing stiff enough to limit the deflection of a 20.0 ft length supported at its ends to not more than 1.0 in.
Roadway width	Clear space between barriers and/or curbs.
Sand-lightweight concrete	A class of lightweight concrete containing lightweight coarse aggregate and natural sand fine aggregate.
Segmental component	A component made up of individual elements, either precast or cast-in-place and post-tensioned together to act as a monolithic unit under loads.
Semi-rigid duct (post-tensioning)	A corrugated duct of metal or plastic sufficiently stiff to be regarded as not coilable into conventional shipping coils without damage.
Service load	Loads without load factors.
Setting temperature	An average temperature for the structure used to determine the dimensions of a structure when a component is added or set in place.
Shallow draft waterways	A navigable waterway used primarily by barge vessels with loaded drafts of less than 9-10 ft.

BRIDGE DESIGN MANUAL GLOSSARY

Shear friction A recognized concept used in the design of areas with shear forces that achieves ductility by placing reinforcement across an anticipated crack so that the tension developed by the reinforcing bars will provide a force normal to the crack. This normal force in combination with "friction" at the crack interface provides the shear resistance.

Shear lag Nonuniform distribution of stress over the cross-section.

Shrinkage of concrete Time-dependent deformation of concrete caused by drying and chemical changes (hydration process).

Skew angle Smaller angle between the centerline of a support and a line normal to the roadway centerline.

Slab A component having a width of at least four times its effective depth.

Spacing of beams The center-to-center distance between beams.

Special anchorage device Anchorage device whose adequacy should be proven in a standardized acceptance test. Most multi-plane anchorages and all bond anchorages are Special Anchorage Devices.

Specified strength of concrete The nominal compressive strength of concrete specified for the work and assumed for design and analysis of new structures.

Splice Connection of one reinforcing bar to another by lapping, welding, mechanical couplers or other means; connection of welded wire reinforcement by lapping; connection of a length of prestressing strand to another using special chucks; connection of piles by mechanical couplers.

Spiral reinforcement Continuously wound reinforcement in the form of a cylindrical or rectangular helix.

Splitting tensile strength The tensile strength of concrete, determined by a splitting test made in accordance with ASTM C496.

Spread box beams Precast, prestressed concrete box beams that are placed on the substructure with a space between them that requires a formed structural topping (deck slab).

Steam curing Curing of concrete, mortar, grout or neat-cement paste in water vapor at atmospheric pressure and at a maximum temperature between about 100 F and 200 F.

Stirrups or ties Lateral reinforcement formed of individual units, open or closed, or of continuously wound reinforcement. The term "stirrups" is usually applied to lateral reinforcement in horizontal members and the term "ties" to those in vertical members.

Stress range The algebraic difference between the maximum and minimum stresses due to transient loads.

Stringer A bridge superstructure element which is repeated in the superstructure, primarily in the longitudinal direction but occasionally in the transverse direction; used interchangeably with beam or girder.

Structural mass concrete Any large volume of concrete where special materials or procedures are required to cope with the generation of heat of hydration and attendant volume change to minimize cracking or concrete degradation.

Structurally continuous barrier A barrier, or any part thereof, which is interrupted only at deck points.

Strut-and-tie model A model used principally in regions of concentrated forces and geometric discontinuities to determine concrete proportions and reinforcement quantities and patterns based on assumed compression struts in the concrete, tensile ties in the reinforcement and the geometry of nodes at their points of intersection.

Substructure Structural parts of the bridge, which support the horizontal span.

Sulfate attack Either a chemical or a physical reaction or both between sulfates usually in soil or ground water and concrete or mortar; the chemical reaction is primarily with calcium aluminate hydrates in the cement-paste matrix, often causing deterioration.

Superstructure Structural parts of the bridge, which provide the horizontal span.

BRIDGE DESIGN MANUAL GLOSSARY

Surcharge	A load used to model the weight of earth fill or other loads applied to the top of the retained material.
Tandem	Two closely spaced axles usually connected to the same under-carriage which enhances the equalization of load between the axles.
Temperature Gradient	Variation of temperature of the concrete throughout the cross-section.
Tendon	A tensioned element, generally high-strength steel wires, strands, or bars, used to impart prestress to the concrete. In post-tensioned concrete, the complete assembly of prestressing steel, anchorages and sheathing, when required, is also called a tendon.
Tension tie member	Member having an axial tensile force sufficient to create tension over the entire cross-section and having limited concrete cover on all sides. Examples include: arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.
Thermal expansion	See Coefficient of thermal expansion.
Ties	See Stirrups.
Ton (short)	2,000 lb (U.S. measure).
Tonne (metric)	2,205 lb.
Transfer (at transfer)	Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member; immediately after the transfer of prestressing force to the concrete.
Transfer length	Length over which prestressing force is transferred to concrete by bond in pretensioned members.
Transverse reinforcement	Reinforcement used to resist shear, torsion, and lateral forces or provide confinement of concrete in a structural member. The terms “stirrups” and “web reinforcement” are usually applied to transverse reinforcement in flexural members and the terms “ties”, “hoops” and “spirals” are applied to transverse reinforcement in compression members.
Type A joints	Cast-in-place joints of wet concrete and/or epoxy between precast units.
Type B joints	Dry joints between precast units.
Vent	See Grout opening.
Wall friction angle	An angle whose arctangent represents the apparent friction between a wall and a soil mass.
Water-cement ratio	The ratio of the amount of water, exclusive only of that absorbed by the aggregates, to the amount of cement in a concrete, mortar, grout, or cement paste mixture; preferably stated as a decimal by mass and abbreviated w/c.
Water-cementitious materials ratio	The ratio of the amount of water, exclusive only of that absorbed by the aggregate, to the amount of cementitious materials in a concrete or mortar mixture.
Welded wire reinforcement	A series of longitudinal and transverse wires arranged substantially at right angles to each other and welded together at all points of intersection.
Wheel	Single or dual tire at one end of an axle.
Wheel line	A transverse or longitudinal grouping of wheels.
Wheel load	One-half of a specified design axle load.
Wobble friction	Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment.
Workability	That property of freshly mixed concrete or mortar which determines the ease with which it can be mixed, placed, consolidated and finished.
Wrapping or sheathing	Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.
Yield strength	The specified yield strength of reinforcement.

PCI CERTIFICATION PROGRAMS

INTRODUCTION

Since 1967, the Precast/Prestressed Concrete Institute (PCI) has been a leader in the development of innovative quality programs. It was 1967 that saw the beginning of the PCI Plant Certification Program, a program that would set the pace for other construction-related certification programs that followed in later years (Duggleby, 1992). In 1985, PCI implemented its Plant Quality Personnel Certification Program and in 1999, introduced the Certified Field Auditor and Field Qualification Programs for erectors of precast concrete (Shutt, 2000).

With the ever-increasing demand for quality, the certification of manufacturers, erectors and personnel provides the customer the assurance that quality systems are being followed, personnel are qualified and control is practiced through each step of the construction process. Independent, unannounced audits help to assure process control.

PCI PLANT CERTIFICATION

The certification of a manufacturing plant by PCI ensures that the plant has developed and documented an in-depth, in-house quality system that is based on time-tested national industry standards.

Standards

Production and quality standards are contained in the PCI publication, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products* (MNL-116). This manual has been recognized by the construction industry as the standard for the manufacture of precast and prestressed concrete since it was first printed in 1970. MNL-116 is the only such recognized national standard for the industry.

Production Experience

Each company must have at least one year of production experience in order to qualify for certification.

Quality System Manual

Every plant must document their specific practices in a custom Quality Systems Manual (QSM). The requirements for the QSM are contained in Division 1 and Appendix A of MNL-116. Fifteen separate sections require that all operations in the plant be addressed thoroughly by management. Each QSM must be approved by PCI prior to certification and must then be reviewed annually and updated if necessary. Plants can obtain additional assistance for compiling a QSM in the PCI publication *Preparation Guidelines for a Structural Plant Quality System Manual* (2000).

Audits/Auditors

Nearly all new plants undergo a "Precertification Evaluation" after which a plant is audited twice each year. These audits are not announced in advance. Auditors are independent, specially trained engineers. They are employed by a single consulting engineering firm under contract to PCI, which ensures consistency for every plant (Shutt, 1994).

Closing Meeting/Reports

Every audit ends with a closing meeting. Auditors and key plant personnel meet to review preliminary results. If improvements are needed, they can be started right away. Later, a detailed written analysis documents observations and reasons for required improvements. The report also includes a numerical grade sheet that indicates the level of compliance with the standards.

Grades

The numerical grade sheet is organized with each section of the gradesheet corresponding to a division (chapter) of MNL-116. During an audit, each division is evaluated separately. Grades in each division must meet or exceed an established minimum value. Then, the grades for all divisions are combined into an overall grade. A minimum overall grade must also be achieved for certification. Audits are evaluated on a strict pass-fail criteria. A failing grade requires a Special Immediate Audit. Failure of that audit or the subsequent Regular Audit results in loss of Certification.

Product Groups

A plant is evaluated and classified according to the type of products produced. This allows for a more product-specific inspection and analysis of a plant's specialized capabilities. It provides specifiers with more information about the production experience of precast plants.

Plants, including bridge products producers, may be certified in up to four general groups of products. The manuals listed in parenthesis contain the standards for certification in that Group.

Group A Architectural Concrete Products (MNL-117)

Group B or Group BA Bridge Products (MNL-116) or the combination of the A and B Product Groups (MNL-116)

Group C or Group CA Commercial (Structural) Products (MNL-116) or the combination of the A and C Product Groups (MNL-116)

Group G Glass Fiber Reinforced Concrete Products (MNL-130)

Product Categories

The Product Groups are further divided into Categories that define a product's reinforcement or the ways in which the products are manufactured or used. Product Categories that include prestressing may incorporate pretensioning or post-tensioning or both. Bridge products producers must be certified in one Category from Group B or Group BA.

Group B Categories

B1 – Precast Bridge Products (no prestressed reinforcement)
Examples include pile caps, retaining wall components, three-sided boxes or arches, median barriers, parapet walls, railings, fascia panels, abutment panels, sound barriers, pier columns, pier caps, precast diaphragms and conventionally-reinforced segmental units, and partial- and full-depth deck panels.

B2 – Prestressed Deck and Miscellaneous Bridge Products (non-superstructure)
Examples include prestressed (pretensioned or plant post-tensioned) pier columns, pier beams, sound walls, fascia panels, piles, sheet piles, partial- and full-depth deck panels.

B3 – Prestressed Straight-Strand Bridge Beams (superstructure)
Examples include solid-slab beams, voided slabs, box beams, I-beams, bulb-tees, double tees, multiple-stemmed units, box beam segments with pretensioned or plant post-tensioned prestressing.

B4 – Prestressed Deflected-Strand Bridge Beams (superstructure)
Examples include box beams, I-beams, bulb-tees, double tees, multiple-stemmed units and plant post-tensioned precast beams with draped tendons.

Group BA Categories

Group B Category products with an architectural finish
(see additional information that follows)

B1A – Precast Bridge Products with Architectural Finish (no prestressed reinforcement)

B2A – Prestressed Miscellaneous Bridge Products with Architectural Finish (non-superstructure)

B3A – Prestressed Straight-Strand Bridge Beams with Architectural Finish (superstructure)

B4A – Prestressed Deflected-Strand Bridge Beams with Architectural Finish (superstructure)

Producers may also be certified in one or more of the following Groups and Categories.

Group A Categories

AT – Miscellaneous Architectural Trim Units

A1 – Architectural Precast Concrete Products

Group C Categories

C1 – Precast Concrete Products (no prestressed reinforcement)

C2 – Prestressed Hollow-Core and Repetitively-Produced Products

C3 – Prestressed Straight-Strand Structural Members

C4 – Prestressed Deflected-Strand Structural Members

Group CA Categories

Group C Category products with an architectural finish
(see additional information that follows)

C1A – Precast Concrete Products with Architectural Finish (no prestressed reinforcement)

C2A – Prestressed Hollow-Core and Repetitively-Produced Products with Architectural Finish

C3A – Prestressed Straight-Strand Structural Members with Architectural Finish

C4A – Prestressed Deflected-Strand Structural Members with Architectural Finish

Within a Product Group, the Categories listed above are intended to be in ascending order of production complexity. A producer qualified to produce products in a given Category is automatically qualified in the preceding Categories but not in succeeding Categories. See the following Guide Qualification Specifications and accompanying notes for more details.

For more descriptive information about the types of products and projects that are represented by these Categories, contact PCI, visit the PCI website at www.pci.org, or refer to other more-detailed program literature from PCI.

Architectural Finishes – Product Groups BA and CA

Beginning with the Fourth Edition of MNL-116 (1999), an additional product distinction was made available to the specifier. The new classification defines products that have architectural finishes applied to more traditional structural products. Before now, these products were not addressed in either MNL-116 or MNL-117. The special requirements for finish, texture, color, tolerances and quality control are included at the end of each division of manual MNL-116.

Identification of BA and CA Producers

The architectural finishes designation may be applied to any “B” or “C” category. Qualified producers will be identified with the suffix “A” following their normal designation of “B1” through “B4” and “C1” through “C4.” For example, if a precaster is certified to produce precast sound barrier wall panels with conventional steel reinforcement and with an exposed aggregate surface finish, the appropriate designation will be “B1A.” A bridge-products producer that manufactures prestressed fascia panels with an architectural finish for a bridge would be required to hold "B2A" certification. Refer to the Guide Qualification Specification near the end of this appendix for information about how to specify this and other Bridge Groups and Categories.

List of Certified Plants

A current listing of all PCI Certified Plants is published quarterly in PCI's ASCENT magazine. A convenient searchable list is regularly updated on the PCI website at www.pci.org, or contact Director of Certification Programs at PCI.

Endorsements

PCI holds a National Evaluation Report (NER QA-105) as an approved Quality Assurance Agency by the National Evaluation Service (NES). The NES includes:

- Building Officials and Code Administrators Evaluation Service Inc. (BOCA ES)
- Southern Building Code Congress International, Public Safety Testing and Evaluation Services Inc. (SBBCI PST&ESI)

PCI holds a separate registration (AA-658) as an approved Quality Assurance Agency by the International Conference of Building Officials Evaluation Service Inc. (ICBO ES).

PCI Plant Certification is included in the MasterSpec of the American Institute of Architects and is required in the specifications of the following federal agencies:

- U.S. Army Corps of Engineers, Civil Works Division & Military Programs
- U.S. Naval Facilities Engineering Command (NAVFEC)
- Federal Aviation Administration
- General Services Administration
- U.S. Department of Agriculture, FSIS
- U.S. Department of Interior, Bureau of Reclamation

Plant Certification is strongly endorsed in correspondence by the Federal Highway Administration (Kane, 1996) for precast concrete bridge products and is required or accepted by more than two-thirds of the individual state departments of transportation (Merwin, 1995).

PCI PLANT QUALITY PERSONNEL CERTIFICATION

Conducting an effective quality control program requires knowledgeable and motivated testing and inspection personnel. Each must understand quality basics, the necessity for quality control, how products are manufactured, and precisely how to conduct tests and inspections. PCI has been training quality control personnel since 1974. In 1985, the first technician training manual was published by PCI and the first qualified personnel attained certification.

There are three levels of Plant Quality Personnel Certification.

PCI Plant Quality Personnel Certification, Level I

Level I requires six months or equivalent of approved industry experience. It requires a basic level of understanding of the many quality control issues normally encountered in a precast plant, such as:

- Quality and quality control programs, testing and measuring
- Prestressing concepts and tensioning procedures for straight strands, including basic elongation calculations

- Basic concepts about concrete – water-cementitious materials ratio (w/cm), types of cements, accelerated curing concepts
- Control of purchased materials
- Precast production procedures
- Welding practices including welding of reinforcing bars and studs
- Interpretation of basic shop drawings

Certification at Level I requires current certification in the American Concrete Institute (ACI) Concrete Field Testing Technician Program, Grade I. The ACI certification requires a closed-book written test and precise field demonstration of seven ASTM methods to test properties of fresh concrete.

Certification at Level I is accomplished by passing a closed-book written examination. Examinations may be administered locally by a PCI-approved proctor or at a PCI-conducted training school. A manual for training and self-study (TM-101) is available from PCI. Level I must be renewed by testing every five years unless a higher level of PCI certification has been attained.

PCI Plant Quality Personnel Certification, Level II

Level II certification requires one year of approved industry experience or equivalent plus PCI Level I and current ACI Level I as prerequisites. Other requirements for Level II include a greater level of knowledge of most of the topics previously described for PCI Level I, as well as:

- Tensioning and elongation corrections that account for temperature effects, chuck seating, abutment movement and bed shortening. Calculations for elongation and corrections are required.
- Effects of accelerated curing and importance of w/cm are further emphasized. Corrections to mix proportions must be calculated to account for excess moisture in the aggregates.
- Material control tests are further explored including aggregate gradations and analysis. Calculations are required for gradation analysis.
- Plant topics include more detail in reading shop drawings and in procedures for welding reinforcing bars and studs.

Certification through Level II is accomplished by passing a closed-book written examination. Examinations may be administered locally by a PCI-approved proctor or at a PCI-conducted training school. A manual for training and self-study for Level II (TM-101) is available from PCI. Level II must be renewed every five years by testing unless Level III has been attained.

PCI Plant Quality Personnel Certification, Level III

Level III provides significant instruction in concrete materials and technology. Certification at this level requires two years of approved industry experience (or equivalent) and attendance at a four-day PCI school. The candidate must pass a closed-book written examination at the school. PCI Level II certification is a prerequisite. Certification at Level III is valid for life. There is a training manual (TM-103) available from PCI that covers all course topics, including:

- Properties of:
  - Basic concrete materials
  - Admixtures
  - Fresh concrete
  - Hardened Concrete

- Mix designs using normal and lightweight aggregates
- Architectural concrete
- Troubleshooting and fine-tuning concrete mixes
- Finished product evaluation
- Stud welding and welding of reinforcing steel
- Deflected prestressing strands and the calculation of deflection forces

Agency Requirement

Plant Quality Personnel Certification is required by nearly a third of the individual state departments of transportation. They require certification for plant quality personnel and for their own materials inspectors and quality assurance personnel.

SUMMARY

The precast, prestressed concrete industry, through PCI, has taken bold steps to establish industry quality standards. The standards apply to personnel, to production and operations, to quality control, and to field operations. The standards have been published and widely disseminated and are open for evaluation and written comments, all of which will be given consideration.

The PCI industry standards for quality production are demanding to achieve. Once attained and practiced regularly, adherence to these standards contributes to improved and continuing customer satisfaction. Following these standards has been shown to reduce the "cost of quality" for the owner as well as the producer.

Certification by PCI assures compliance to the published standards for quality production. Certified personnel and producers choose to demonstrate their proficiency by voluntarily undergoing examinations and audits by accredited third-party assessors.

PCI Plant and Personnel Certification are reliable means for qualifying personnel and precast concrete producers. Use the Guide Qualification Specification that follows to require PCI Certification Programs for your projects.

GUIDE QUALIFICATION SPECIFICATION

The following guide specification can be used to qualify a precast concrete manufacturer to submit a bid on your project. Generally, the easiest procedure would be to list the precast product or the various precast products included in your project. Then, determine the appropriate Product Group and Category for each product, considering the use of the product, the method of reinforcement and special required surface finishes, if any. Show each of the products and the required Group and Category in the project specifications. Refer to the following "Notes to Specifiers" for additional discussion.Product categories that include prestressing may incorporate pretensioning or post-tensioning or both.

Further, it is recommended that the manufacturer employ trained and certified personnel according to the Personnel Qualifications guide specification that follows.

Manufacturer Qualifications – Structural Precast Concrete

The precast concrete manufacturing plant shall be certified under the Precast/Prestressed Concrete Institute Plant Certification Program. The Manufacturer shall be certified at the time of bidding. Certification shall be in the following Product Group(s) and Category(ies):

[Select and insert one or more of the following applicable groups and categories]

Group B – Bridge-Related Products

B1 – Precast Bridge Products (no prestressed reinforcement)

B2 – Prestressed Miscellaneous Bridge Products (non-superstructure)

B3 – Prestressed Straight-Strand Bridge Beams (superstructure)

B4 – Prestressed Deflected-Strand Bridge Beams (superstructure)

Group BA – Bridge-Related Products that Require Architectural Finishes

B1A – Precast Bridge Products with Architectural Finish (no prestressed reinforcement)

B2A – Prestressed Miscellaneous Bridge Products with Architectural Finish (non-superstructure)

B3A – Prestressed Straight-Strand Bridge Beams with Architectural Finish (superstructure)

B4A – Prestressed Deflected-Strand Bridge Beams with Architectural Finish (superstructure)

Notes to Specifiers:

1. Additional guide specifications, not shown here, are available from PCI for Product Groups "A," "C" and "G."

2. Categories in Product Group B are listed in ascending order of production complexity. For example, a plant certified to produce products in Category B4 is automatically certified to produce products in the preceding Categories B1, B2 and B3. However, a plant certified to produce products in Category B2, while certified for Category B1, is not certified for Categories B3 or B4.

3. Categories in Group BA are also listed in ascending order. See Notes 2 & 4.

4. Group BA supercedes Group B in the same Category. For example, a plant certified to produce products in Category B4A is automatically certified to produce products in the preceding Categories B1A, B2A, B3A, and in categories B1, B2, B3 and B4. However, a plant certified to produce products in Category B2A, while also certified for Categories B1A, B1 and B2, is not certified for Categories B3A, B4A, B3 or B4.

5. A Product Group and Category should be determined and shown in the specifications for each type of precast concrete product used in a project. Separating products will enable precasters to submit bids on specific products. For example, on a project that included both prestressed piling and beams, a precaster with expertise in producing prestressed piling (with Certification B2) could submit a price on piles only. On the same project, a producer with Certification B4 could submit a price for the beams and decide to either include or exclude the piling.

6. Specify the most appropriate Product Group and Category for the project. Do not select a higher Category than necessary. Similarly, do not add "A" to a listing when not necessary to meet project requirements. Selecting an inappropriate Group or Category could result in unnecessary cost or could restrict the number of available bidders.

Personnel Qualifications

The Manufacturer shall employ a minimum of one person, regularly present in the plant, who is certified by the Precast/Prestressed Concrete Institute for Plant Quality Personnel, Level II. All other personnel regularly engaged in the measuring, testing or evaluation of products or materials shall be similarly certified, or actively pursuing certification for Plant Quality Personnel, Level I.

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