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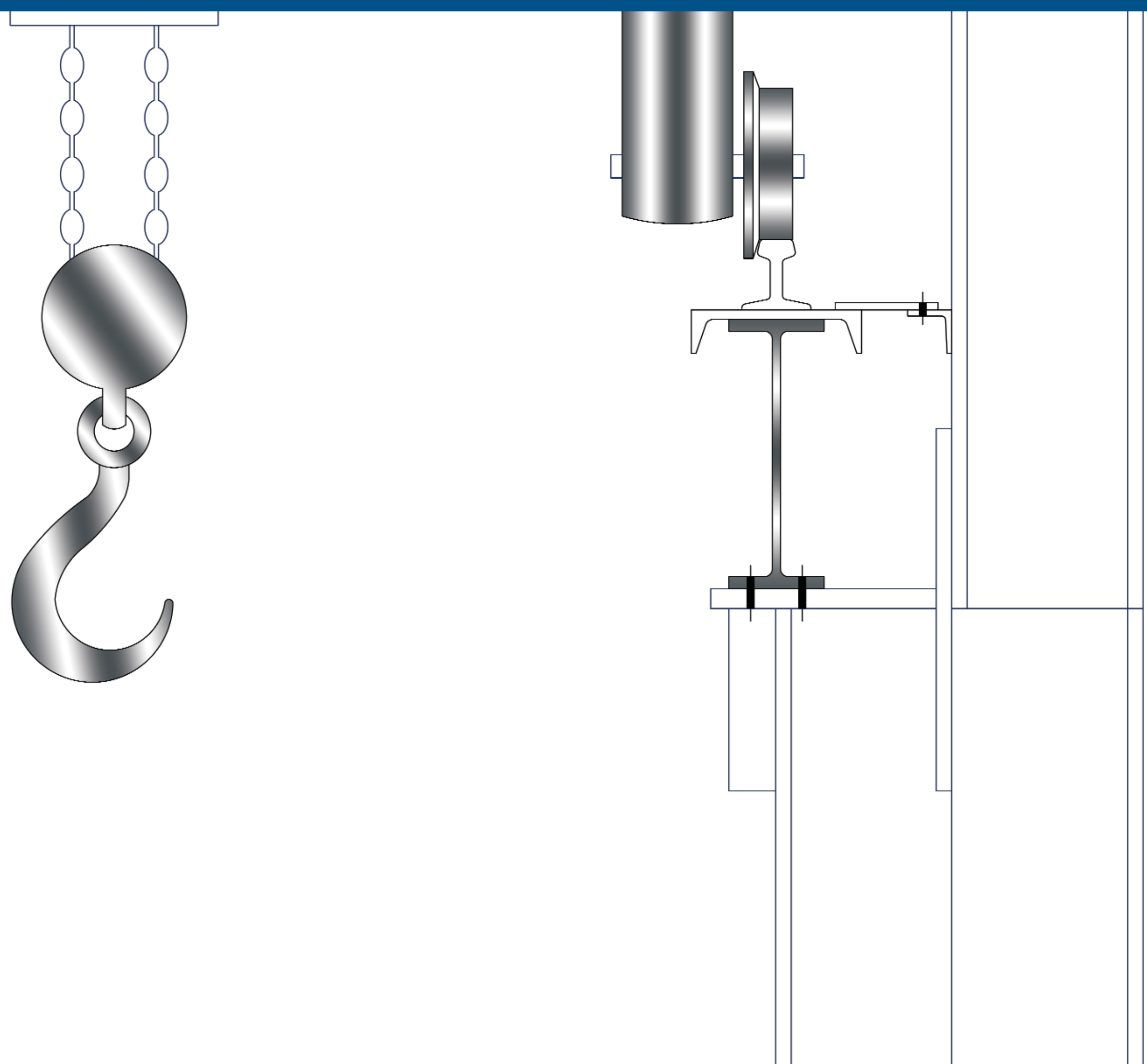
Steel Design Guide

Industrial Buildings

Roofs to Anchor Rods

Second Edition



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Roofs to Anchor Rods

Second Edition

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Part 1

INDUSTRIAL BUILDINGS—GENERAL

1. INTRODUCTION

Although the basic structural and architectural components of industrial buildings are relatively simple, combining all of the elements into a functional economical building can be a complex task. General guidelines and criteria to accomplish this task can be stated. The purpose of this guide is to provide the industrial building designer with guidelines and design criteria for the design of buildings without cranes, or for buildings with light-to-medium duty cycle cranes. Part 1 deals with general topics on industrial buildings. Part 2 deals with structures containing cranes. Requirements for seismic detailing for industrial buildings have not been addressed in this guide. The designer must address any special detailing for seismic conditions.

Most industrial buildings primarily serve as an enclosure for production and/or storage. The design of industrial buildings may seem logically the province of the structural engineer. It is essential to realize that most industrial buildings involve much more than structural design. The designer may assume an expanded role and may be responsible for site planning, establishing grades, handling surface drainage, parking, on-site traffic, building aesthetics, and, perhaps, landscaping. Access to rail and the establishment of proper floor elevations (depending on whether direct fork truck entry to rail cars is required) are important considerations. Proper clearances to sidings and special attention to curved siding and truck grade limitations are also essential.

2. LOADING CONDITIONS AND LOADING COMBINATIONS

Loading conditions and load combinations for industrial buildings without cranes are well established by building codes.

Loading conditions are categorized as follows:

1. *Dead load:* This load represents the weight of the structure and its components, and is usually expressed in pounds per square foot. In an industrial building, the building use and industrial process usually involve permanent equipment that is supported by the structure. This equipment can sometimes be represented by a uniform load (known as a collateral load), but the points of attachment are usually subjected to concentrated loads that require a separate analysis to account for the localized effects.

2. *Live load:* This load represents the force imposed on the structure by the occupancy and use of the building. Building codes give minimum design live loads in pounds per square foot, which vary with the classification of occupancy and use. While live loads are expressed as uniform, as a practical matter any occupancy loading is inevitably nonuniform. The degree of nonuniformity that is acceptable is a matter of engineering judgment. Some building codes deal with nonuniformity of loading by specifying concentrated loads in addition to uniform loading for some occupancies. In an industrial building, often the use of the building may require a live load in excess of the code stated minimum. Often this value is specified by the owner or calculated by the engineer. Also, the loading may be in the form of significant concentrated loads as in the case of storage racks or machinery.

3. *Snow loads:* Most codes differentiate between roof live and snow loads. Snow loads are a function of local climate, roof slope, roof type, terrain, building internal temperature, and building geometry. These factors may be treated differently by various codes.

4. *Rain loads:* These loads are now recognized as a separate loading condition. In the past, rain was accounted for in live load. However, some codes have a more refined standard. Rain loading can be a function of storm intensity, roof slope, and roof drainage. There is also the potential for rain on snow in certain regions.

5. *Wind loads:* These are well codified, and are a function of local climate conditions, building height, building geometry and exposure as determined by the surrounding environment and terrain. Typically, they're based on a 50-year recurrence interval—maximum three-second gust. Building codes account for increases in local pressure at edges and corners, and often have stricter standards for individual components than for the gross building. Wind can apply both inward and outward forces to various surfaces on the building exterior and can be affected by size of wall openings. Where wind forces produce overturning or net upward forces, there must be an adequate counterbalancing structural dead weight or the structure must be anchored to an adequate foundation.

6. *Earthquake loads:* Seismic loads are established by building codes and are based on:

   a. The degree of seismic risk
   b. The degree of potential damage
   c. The possibility of total collapse
   d. The feasibility of meeting a given level of protection

Earthquake loads in building codes are usually equivalent static loads. Seismic loads are generally a function of:

   a. The geographical and geological location of the building
   b. The use of the building
   c. The nature of the building structural system
   d. The dynamic properties of the building
   e. The dynamic properties of the site
   f. The weight of the building and the distribution of the weight

Load combinations are formed by adding the effects of loads from each of the load sources cited above. Codes or industry standards often give specific load combinations that must be satisfied. It is not always necessary to consider all loads at full intensity. Also, certain loads are not required to be combined at all. For example, wind need not be combined with seismic. In some cases only a portion of a load must be combined with other loads. When a combination does not include loads at full intensity it represents a judgment as to the probability of simultaneous occurrence with regard to time and intensity.

3. OWNER-ESTABLISHED CRITERIA

Every industrial building is unique. Each is planned and constructed to requirements relating to building usage, the process involved, specific owner requirements and preferences, site constraints, cost, and building regulations. The process of design must balance all of these factors. The owner must play an active role in passing on to the designer all requirements specific to the building such as:

1. Area, bay size, plan layout, aisle location, future expansion provisions.

2. Clear heights.

3. Relations between functional areas, production flow, acoustical considerations.

4. Exterior appearance.

5. Materials and finishes, etc.

6. Machinery, equipment and storage method.

7. Loads.

There are instances where loads in excess of code minimums are required. Such cases call for owner involvement. The establishment of loading conditions provides for a structure of adequate strength. A related set of criteria are needed to establish the serviceability behavior of the structure. Serviceability design considers such topics as deflection, drift, vibration and the relation of the primary and secondary structural systems and elements to the performance of nonstructural components such as roofing, cladding, equipment, etc. Serviceability issues are not strength issues but maintenance and human response considerations. Serviceability criteria are discussed in detail in *Serviceability Design Considerations for Steel Buildings* that is part of the AISC Steel Design Guide Series (Fisher, 2003). Criteria taken from the Design Guide are presented in this text as appropriate.

As can be seen from this discussion, the design of an industrial building requires active owner involvement. This is also illustrated by the following topics: slab-on-grade design, jib cranes, interior vehicular traffic, and future expansion.

3.1 Slab-on-Grade Design

One important aspect to be determined is the specific loading to which the floor slab will be subjected. Forklift trucks, rack storage systems, or wood dunnage supporting heavy manufactured items cause concentrated loads in industrial structures. The important point here is that these loadings are nonuniform. The slab-on-grade is thus often designed as a plate on an elastic foundation subject to concentrated loads.

It is common for owners to specify that slabs-on-grade be designed for a specific uniform loading (for example, 500 psf). If a slab-on-grade is subjected to a uniform load, it will develop no bending moments. Minimum thickness and no reinforcement would be required. The frequency with which the author has encountered the requirement of design for a uniform load and the general lack of appreciation of the inadequacy of such criteria by many owners and plant engineers has prompted the inclusion of this topic in this guide. Real loads are not uniform, and an analysis using an assumed nonuniform load or the specific concentrated loading for the slab is required. An excellent reference for the design of slabs-on-grade is *Designing Floor Slabs on Grade* by Ringo and Anderson (Ringo, 1996). In addition, the designer of slabs-on-grade should be familiar with the ACI *Guide for Concrete Floor and Slab Construction* (ACI, 1997), the ACI *Design of Slabs on Grade* (ACI, 1992).

3.2 Jib Cranes

Another loading condition that should be considered is the installation of jib cranes. Often the owner has plans to install such cranes at some future date. But since they are a purchased item—often installed by plant engineering personnel or the crane manufacturer—the owner may inadvertently neglect them during the design phase.

Jib cranes, which are simply added to a structure, can create a myriad of problems, including column distortion and misalignment, column bending failures, crane runway and crane rail misalignment, and excessive column base shear. It is essential to know the location and size of jib cranes in advance, so that columns can be properly designed and proper bracing can be installed if needed. Columns supporting jib cranes should be designed to limit the deflection at the end of the jib boom to boom length divided by 225.

3.3 Interior Vehicular Traffic

The designer must establish the exact usage to which the structure will be subjected. Interior vehicular traffic is a major source of problems in structures. Forklift trucks can accidentally buckle the flanges of a column, shear off anchor rods in column bases, and damage walls.

Proper consideration and handling of the forklift truck problem may include some or all of the following:

1. Use of masonry or concrete exterior walls in lieu of metal panels. (Often the lowest section of walls is made of masonry or concrete with metal panels used for the higher section.)

2. Installation of fender posts (bollards) for columns and walls may be required where speed and size of fork trucks are such that a column or load-bearing wall could be severely damaged or collapsed upon impact.

3. Use of metal guardrails or steel plate adjacent to wall elements may be in order.

4. Curbs.

Lines defining traffic lanes painted on factory floors have never been successful in preventing structural damage from interior vehicular operations. The only realistic approach for solving this problem is to anticipate potential impact and damage and to install barriers and/or materials that can withstand such abuse.

3.4 Future Expansion

Except where no additional land is available, every industrial structure is a candidate for future expansion. Lack of planning for such expansion can result in considerable expense.

When consideration is given to future expansion, there are a number of practical considerations that require evaluation.

1. The directions of principal and secondary framing members require study. In some cases it may prove economical to have a principal frame line along a building edge where expansion is anticipated and to design edge beams, columns and foundations for the future loads. If the structure is large and any future expansion would require creation of an expansion joint at a juncture of existing and future construction, it may be prudent to have that edge of the building consist of nonload-bearing elements. Obviously, foundation design must also include provision for expansion.

2. *Roof Drainage*: An addition which is constructed with low points at the junction of the roofs can present serious problems in terms of water, ice and snow piling effects.

3. Lateral stability to resist wind and seismic loadings is often provided by X-bracing in walls or by shear walls. Future expansion may require removal of such bracing. The structural drawings should indicate the critical nature of wall bracing, and its location, to prevent accidental removal. In this context, bracing can interfere with many plant production activities and the importance of such bracing cannot be overemphasized to the owner and plant engineering personnel. Obviously, the location of bracing to provide the capability for future expansion without its removal should be the goal of the designer.

3.5 Dust Control/Ease of Maintenance

In certain buildings (for example, food processing plants) dust control is essential. Ideally there should be no horizontal surfaces on which dust can accumulate. HSS as purlins reduce the number of horizontal surfaces as compared to C's, Z's, or joists. If horizontal surfaces can be tolerated in conjunction with a regular cleaning program, C's or Z's may be preferable to joists. The same thinking should be applied to the selection of main framing members (in other words, HSS or box sections may be preferable to wide-flange sections or trusses).

4. ROOF SYSTEMS

The roof system is often the most expensive part of an industrial building (even though walls are more costly per square foot). Designing for a 20-psf mechanical surcharge load when only 10 psf is required adds cost over a large area.

Often the premise guiding the design is that the owner will always be hanging new piping or installing additional equipment, and a prudent designer will allow for this in the

Table 4.1 Steel Deck Institute Recommended Spans (38)

Recommended Maximum Spans for Construction and Maintenance Loads Standard 1-1/2 in. and 3 in. Roof Deck

	Type	Span Condition	Span Ft -In.	Maximum Recommended Spans Roof Deck Cantilever
Narrow Rib Deck (Old Type A)	NR22	1	3′-10″	1′-0″
	NR22	2 or more	4′-9″	
	NR20	1	4′-10″	1′-2″
	NR20	2 or more	5′-11″	
	NR18	1	5′-11″	1′-7″
	NR18	2 or more	6′-11″	
Intermediate Rib Deck (Old Type F)	IR22	1	4′-6″	1′-2″
	IR22	2 or more	5′-6″	
	IR20	1	5′-3″	1′-5″
	IR20	2 or more	6′-3″	
	IR18	1	6′-2″	1′-10″
	IR18	2 or more	7′-4″	
Wide Rib (Old Type B)	WR22	1	5′-6″	1′-11″
	WR22	2 or more	6′-6″	
	WR20	1	6′-3″	2′-4″
	WR20	2 or more	7′-5″	
	WR18	1	7′-6″	2′-10″
	WR18	2 or more	8′-10″	
Deep Rib Deck	3DR22	1	11′-0″	3′-5″
	3DR22	2 or more	13′-0″	
	3DR20	1	12′-6″	3′-11″
	3DR20	2 or more	14′-8″	
	3DR18	1	15′-0″	4′-9″
	3DR18	2 or more	17′-8″	

NOTE: SEE SDI LOAD TABLES FOR ACTUAL DECK CAPACITIES

system. If this practice is followed, the owner should be consulted, and the decision to provide excess capacity should be that of the owner. The design live loads and collateral (equipment) loads should be noted on the structural plans.

4.1 Steel Deck for Built-up or Membrane Roofs

Decks are commonly 1½ in. deep, but deeper units are also available. The Steel Deck Institute (SDI, 2001) has identified three standard profiles for 1½ in. steel deck, (narrow rib, intermediate rib and wide rib) and has published load tables for each profile for thicknesses varying from 0.0299 to 0.0478 in. These three profiles, (shown in Table 4.1) NR, IR, and WR, correspond to the manufacturers' designations A, F, and B, respectively. The Steel Deck Institute identifies the standard profile for 3 in. deck as 3DR. A comparison of weights for each profile in various gages shows that strength-to-weight ratio is most favorable for wide rib and least favorable for narrow rib deck. In general, the deck selection that results in the least weight per ft^2 may be the most economical. However, consideration must also be given to the flute width because the insulation must span the flutes. In the northern areas of the U.S., high roof loads and thick insulation generally make the wide rib (B) profile predominant. In the South, low roof loads and thinner insulation make the intermediate profile common. Where very thin insulation is used narrow rib deck may be required, although this is not a common profile. In general the lightest weight deck consistent with insulation thickness and span should be used.

Table 4.2 Factory Mutual Data (3)			
Types 1.5A, 1.5F, 1.5B and 1.5BI Deck. Nominal 1½ in. (38mm) depth. No stiffening grooves			
	22g.	**20g.**	**18g.**
Type 1.5A	4′10″	5′3″	6′0″
Narrow Rib	(1.5m)	(1.6m)	(1.9m)
Type 1.5F	4′11″	5′5″	6′3″
Intermediate Rib	(1.5m)	(1.7m)	(2.0m)
Type 1.5B, BI	6′0″	6′6″	7′5″
Wide Rib	(1.8m)	(2.0m)	(2.3m)

In addition to the load, span, and thickness relations established by the load tables, there are other considerations in the selection of a profile and gage for a given load and span. First, the Steel Deck Institute limits deflection due to a 200-lb concentrated load at midspan to span divided by 240. Secondly, the Steel Deck Institute has published a table of maximum recommended spans for construction and maintenance loads (Table 4.1), and, finally Factory Mutual lists maximum spans for various profiles and gages in its Approval Guide (Table 4.2).

Factory Mutual in its Loss Prevention Guide (LPG) 1-28 *Insulated Steel Deck* (FM, various dates) provides a standard for attachment of insulation to steel deck. LPG 1-29 *Loose Laid Ballasted Roof Coverings* (FM, various dates) gives a standard for the required weight and distribution of ballast for roofs that are not adhered.

LPG 1-28 requires a side lap fastener between supports. This fastener prevents adjacent panels from deflecting differentially when a load exists at the edge of one panel but not on the edge of the adjacent panel. Factory Mutual permits an over span from its published tables of 6 in. (previously an overspan of 10 percent had been allowed) when "necessary to accommodate column spacing in some bays of the building. It should not be considered an original design parameter." The Steel Deck Institute recommends that the side laps in cantilevers be fastened at 12 in. on center.

Steel decks can be attached to supports by welds or fasteners, which can be power or pneumatically installed or self-drilling, self-tapping. The Steel Deck Institute in its *Specifications and Commentary for Steel Roof Deck* (SDI, 2000) requires a maximum attachment spacing of 18 in. along supports. Factory Mutual requires the use of 12-in. spacing as a maximum; this is more common. The attachment of roof deck must be sufficient to provide bracing to the structural roof members, to anchor the roof to prevent uplift, and, in many cases, to serve as a diaphragm to carry lateral loads to the bracing. While the standard attachment spacing may be acceptable in many cases, decks designed as diaphragms may require additional connections. Diaphragm capacities can be determined from the *Diaphragm Design Manual* (Steel Deck Institute, 1987)

Manufacturers of metal deck are constantly researching ways to improve section properties with maximum economy. Considerable differences in cost may exist between prices from two suppliers of "identical" deck shapes; therefore the designer is urged to research the cost of the deck system carefully. A few cents per ft^2 savings on a large roof area can mean a significant savings to the owner.

Several manufacturers can provide steel roof deck and wall panels with special acoustical surface treatments for specific building use. Properties of such products can be obtained from the manufacturers. The owner must specify special treatment for acoustical reasons.

4.2 Metal Roofs

Standing seam roof systems were first introduced in the late 1960s, and today many manufacturers produce standing seam panels. A difference between the standing seam roof and lap seam roof (through fastener roof) is in the manner in which two panels are joined to each other. The seam between two panels is made in the field with a tool that makes a cold-formed weather-tight joint. (Note: Some panels can be seamed without special tools.) The joint is made at the top of the panel. The standing seam roof is also unique in the manner in which it is attached to the purlins. The attachment is made with a clip concealed inside the seam. This clip secures the panel to the purlin and may allow the panel to move when experiencing thermal expansion or contraction.

A continuous single skin membrane results after the seam is made since through-the-roof fasteners have been eliminated. The elevated seam and single skin member provides a watertight system. The ability of the roof to experience unrestrained thermal movement eliminates damage to insulation and structure (caused by temperature effects which built-up and through fastened roofs commonly experience). Thermal spacer blocks are often placed between the panels and purlins in order to insure a consistent thermal barrier. Due to the superiority of the standing seam roof, most manufacturers are willing to offer considerably longer guarantees than those offered on lap seam roofs.

Because of the ability of standing seam roofs to move on sliding clips, they possess only minimal diaphragm strength and stiffness. The designer should assume that the standing seam roof has no diaphragm capability, and in the case of steel joists specify that sufficient bridging be provided to laterally brace the joists under design loads.

4.3 Insulation and Roofing

Due to concern about energy, the use of additional and/or improved roof insulation has become common. Coordination with the mechanical requirements of the building is necessary. Generally the use of additional insulation is warranted, but there are at least two practical problems that occur as a result. Less heat loss through the roof results in greater snow and ice build-up and larger snow loads. As a consequence of the same effect, the roofing is subjected to colder temperatures and, for some systems (built-up roofs), thermal movement, which may result in cracking of the roofing membrane.

4.4 Expansion Joints

Although industrial buildings are often constructed of flexible materials, roof and structural expansion joints are required when horizontal dimensions are large. It is not possible to state exact requirements relative to distances between expansion joints because of the many variables involved, such as ambient temperature during construction and the expected temperature range during the life of the buildings. An excellent reference on the topic of thermal expansion in buildings and location of expansion joints is the Federal Construction Council's Technical Report No. 65, *Expansion Joints in Buildings* (Federal Construction Council, 1974).

The report presents the figure shown herein as Figure 4.4.1 as a guide for spacing structural expansion joints in beam and column frame buildings based on design temperature change. The report includes data for numerous cities.

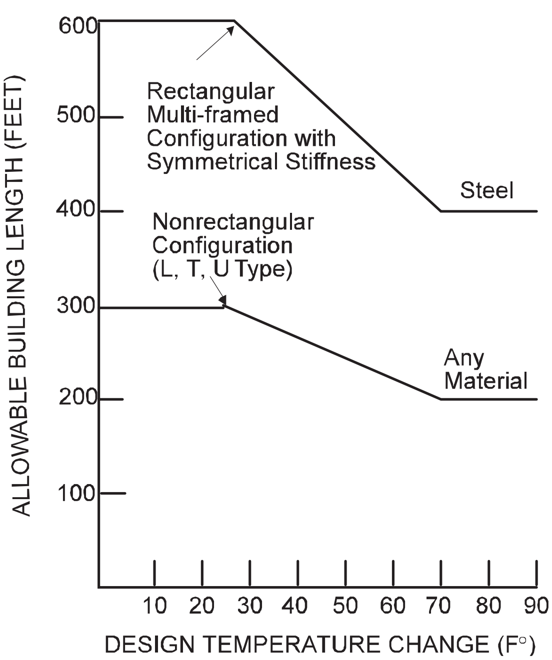


Fig. 4.4.1 Expansion Joint Spacing Graph
(Taken from F.C.C. Tech. Report No. 65, Expansion Joints in Buildings*)*

The report gives modifying factors that are applied to the allowable building length as appropriate.

The report indicates that the curve is directly applicable to buildings of beam-and-column construction, hinged at the base, and with heated interiors. When other conditions prevail, the following rules are applicable:

1. If the building will be heated only and will have hinged-column bases, use the allowable length as specified.

2. If the building will be air conditioned as well as heated, increase the allowable length 15 percent (if the environmental control system will run continuously).

3. If the building will be unheated, decrease the allowable length 33 percent.

4. If the building will have fixed column bases, decrease the allowable length 15 percent.

5. If the building will have substantially greater stiffness against lateral displacement in one direction decrease the allowable length 25 percent.

When more than one of these design conditions prevails in a building, the percentile factor to be applied should be the algebraic sum of the adjustment factors of all the various applicable conditions.

Regarding the type of structural expansion joint, most engineers agree that the best method is to use a line of double columns to provide a complete separation at the joints. When joints other than the double column type are employed, low friction sliding elements, such as shown in Figure 4.4.2, are generally used. Slip connections may

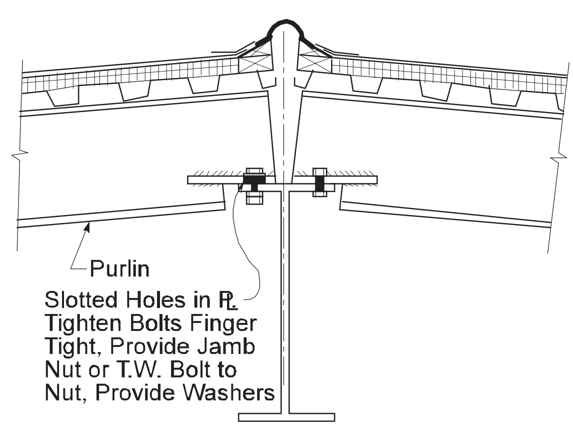


Fig. 4.4.2 Beam Expansion Joint

induce some level of inherent restraint to movement due to binding or debris build-up.

Very often buildings may be required to have firewalls in specific locations. Firewalls may be required to extend above the roof or they may be allowed to terminate at the underside of the roof. Such firewalls become locations for expansion joints. In such cases the detailing of joints can be difficult.

Figures 4.4.2 through 4.4.5 depict typical details to permit limited expansion. Additional details are given in architectural texts.

Expansion joints in the structure should always be carried through the roofing. Additionally, depending on membrane type, other joints called area dividers are necessary in the roof membrane. These joints are membrane relief joints only and do not penetrate the roof deck. Area divider joints are generally placed at intervals of 150 ft to 250 ft for adhered membranes, at somewhat greater intervals for ballasted membranes, and 100 ft to 200 ft in the case of steel roofs. Spacing of joints should be verified with manufacturer's requirements. The range of movement between joints is limited by the flexibility and movement potential of the anchorage scheme and, in the case of standing seam roofs, the clip design. Manufacturers' recommendations should be consulted and followed. Area dividers can also be used to divide complex roofs into simple squares and rectangles.

4.5 Roof Pitch, Drainage and Ponding

Prior to determining a framing scheme and the direction of primary and secondary framing members, it is important to decide how roof drainage is to be accomplished. If the structure is heated, interior roof drains may be justified. For unheated spaces exterior drains and gutters may provide the solution.

For some building sites it may not be necessary to have gutters and downspouts to control storm water, but their use is generally recommended or required by the owner. Significant operational and hazardous problems can occur where water is discharged at the eaves or scuppers in cold climates, causing icing of ground surfaces and hanging of ice from the roof edge. This is a special problem at overhead door locations and may occur with or without gutters. Protection from falling ice must be provided at all building service entries.

Performance of roofs with positive drainage is generally good. Due to problems (for example, ponding, roofing deterioration, leaking) that result from poor drainage, the International Building Code, (ICC, 2003) requires a roof slope of at least ¼ in. per ft.

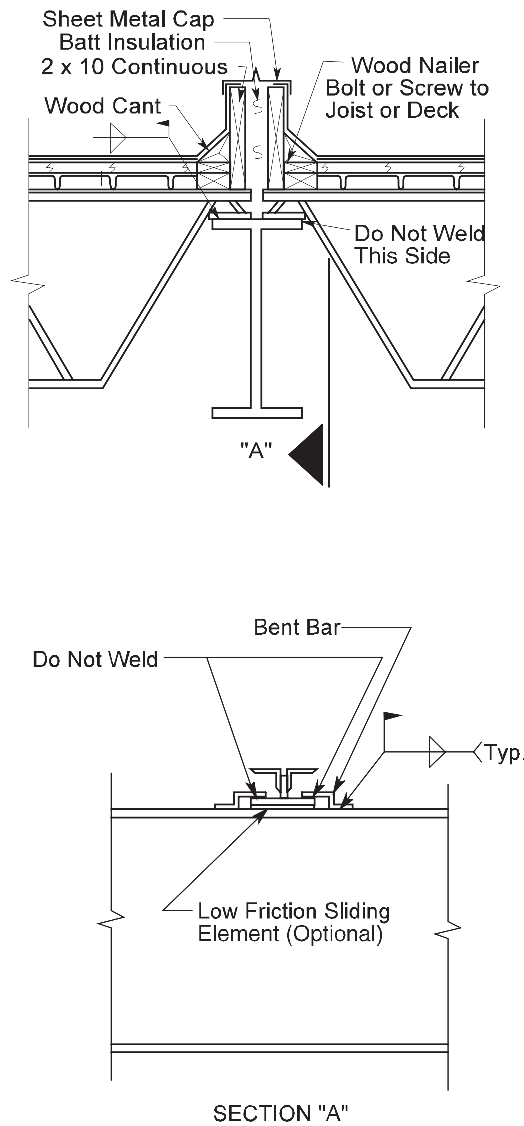


Fig. 4.4.3 Joist Expansion Joint

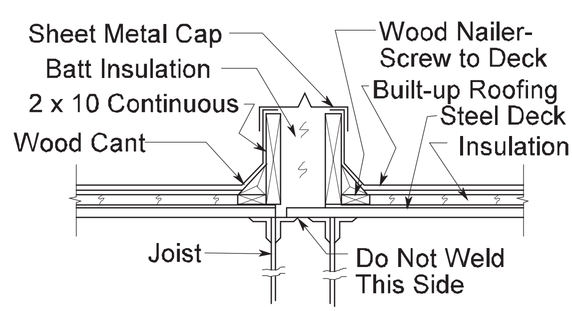


Fig. 4.4.4 Joist Expansion Joint

Ponding, which is often not understood or is overlooked, is a phenomenon that may lead to severe distress or partial or general collapse.

Ponding as it applies to roof design has two meanings. To the roofing industry, ponding describes the condition in which water accumulated in low spots has not dissipated within 24 hours of the last rainstorm. Ponding of this nature is addressed in roof design by positive roof drainage and control of the deflections of roof framing members. Ponding, as an issue in structural engineering, is a load/deflection situation, in which, there is incremental accumulation of rainwater in the deflecting structure. The purpose of a ponding check is to ensure that equilibrium is reached between the incremental loading and the incremental deflection. This convergence must occur at a level of stress that is within the allowable value.

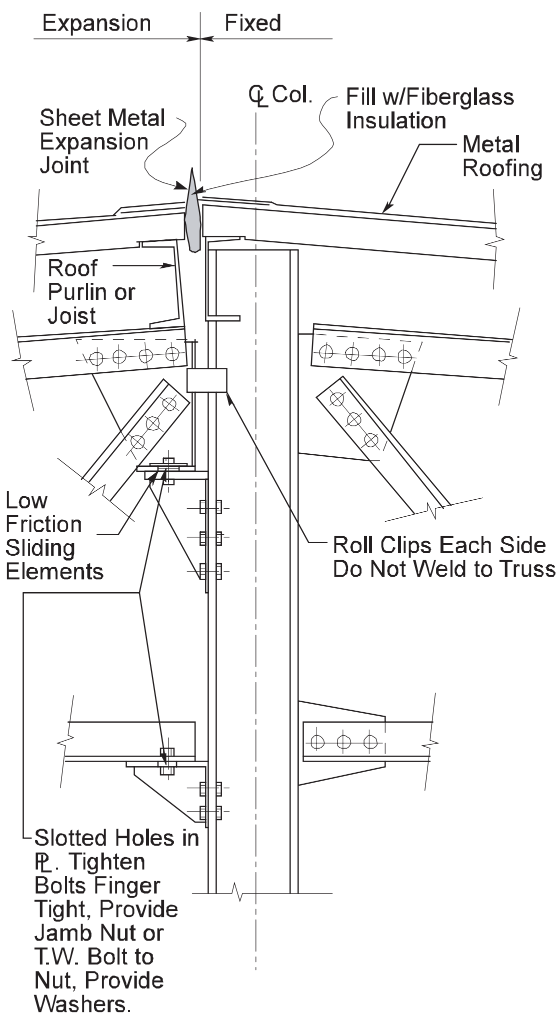


Fig. 4.4.5 Truss Expansion Joint

The AISC specifications for both LRFD (AISC, 1999) and ASD (AISC, 1989) give procedures for addressing the problem of ponding where roof slopes and drains may be inadequate. The direct method is expressed in Eq. K2-1 and K2-2 of the specifications. These relations control the stiffness of the framing members (primary and secondary) and deck. This method, however, can produce unnecessarily conservative results. A more exact method is provided in Appendix K of the *LRFD Specification* and in Chapter K in the *Commentary* in the *ASD Specification*.

The key to the use of the allowable stress method is the calculation of stress in the framing members due to loads present at the initiation of ponding. The difference between 0.8 F_y and the initial stress is used to establish the required stiffness of the roof framing members. The initial stress ("at the initiation of ponding") is determined from the loads present at that time. These should include all or most of the dead load and may include some portion of snow/rain/live load. Technical Digest No. 3 published by the Steel Joist Institute SJI (1971) gives some guidance as to the amount of snow load that could be used in ponding calculations.

The amount of accumulated water used is also subject to judgment. The AISC ponding criteria only applies to roofs which lack "sufficient slope towards parts of free drainage or adequate individual drains to prevent the accumulation of rain water..." However, the possibility of plugged drains means that the load at the initiation of ponding could include the depth of impounded water at the level of overflow into adjacent bays, or the elevation of overflow drains or, over the lip of roof edges or through scuppers. It is clear from reading the AISC *Specification* and *Commentary* that it is not necessary to include the weight of water that would accumulate after the "initiation of ponding." Where snow load is used by the code, the designer may add 5 psf to the roof load to account for the effect of rain on snow. Also, consideration must be given to areas of drifted snow.

It is clear that judgment must be used in the determination of loading "at the initiation of ponding." It is equally clear that one hundred percent of the roof design load would rarely be appropriate for the loading "at the initiation of ponding."

A continuously framed or cantilever system may be more critical than a simple span system. With continuous framing, rotations at points of support, due to roof loads that are not uniformly distributed, will initiate upward and downward deflections in alternate spans. The water in the uplifted bays drains into the adjacent downward deflected bays, compounding the effect and causing the downward deflected bays to approach the deflected shape of simple spans. For these systems one approach to ponding analysis could be based on simple beam stiffness, although a more refined analysis could be used.

The designer should also consult with the plumbing designer to establish whether or not a controlled flow (water retention) drain scheme is being used. Such an approach allows the selection of smaller pipes because the water is impounded on the roof and slowly drained away. This intentional impoundment does not meet the AISC criterion of "drains to prevent the accumulation of rainwater..." and requires a ponding analysis.

A situation that is not addressed by building code drainage design is shown in Figure 4.5.1. The author has investigated several roof ponding collapses where the accumulation of water is greater than would be predicted by drainage analysis for the area shown in Figure 4.5.1. As the water drains towards the eave it finds the least resistance to flow along the parapet to the aperture of the roof. Designers are encouraged to pay close attention these situations, and to provide a conservative design for ponding in the aperture area.

Besides rainwater accumulation, the designer should give consideration to excessive build-up of material on roof surfaces (fly ash, and other air borne material) from industrial operations. Enclosed valleys, parallel high- and low-aisle roofs and normal wind flows can cause unexpected build-ups and possibly roof overload.

4.6 Joists and Purlins

A decision must be made whether to span the long direction of bays with the main beams, trusses, or joist girders which support short span joists or purlins, or to span the short direction of bays with main framing members which support longer span joists or purlins. Experience in this regard is that spanning the shorter bay dimension with primary members will provide the most economical system. However, this decision may not be based solely on economics but rather on such factors as ease of erection, future expansion, direction of crane runs, location of overhead doors, etc.

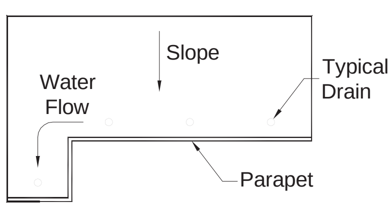


Fig. 4.5.1 Aperture Drainage

On the use of steel joists or purlins, experience again shows that each case must be studied. Standard steel joist specifications (SJI, 2002) are based upon distributed loads only. Modifications for concentrated loads should be done in accordance with the SJI Code Of Standard Practice. Hot-rolled framing members should support significant concentrated loads. However, in the absence of large concentrated loads, joist framing can generally be more economical than hot rolled framing.

Cold-formed C and Z purlin shapes provide another alternative to rolled W sections. The provisions contained in the American Iron and Steel Institute's *Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001) should be used for the design of cold-formed purlins. Additional economy can be achieved with C and Z sections because they can be designed and constructed as continuous members. However, progressive failure should be considered if there is a possibility for a loss in continuity after installation.

Other aspects of the use of C and Z sections include:

1. Z sections ship economically due to the fact that they can be "nested."

2. Z sections can be loaded through the shear center; C sections cannot.

3. On roofs with appropriate slope a Z section will have one principal axis vertical, while a C section provides this condition only for flat roofs.

4. Many erectors indicate that lap bolted connections for C or Z sections (bolted) are more expensive than the simple welded down connections for joist ends.

5. At approximately a 30-ft span length C and Z sections may cost about the same as a joist for the same load per foot. For shorter spans C and Z sections are normally less expensive than joists.

5. ROOF TRUSSES

Primary roof framing for conventionally designed industrial buildings generally consists of wide flange beams, steel joist girders, or fabricated trusses. For relatively short spans of 30- to 40-ft steel beams provide an economical solution, particularly if a multitude of hanging loads are present. For spans greater than 40 ft but less than 80-ft steel joist girders are often used to support roof loads. Fabricated steel roof trusses are often used for spans greater than 80 ft. In recent years little has been written about the design of steel roof trusses. Most textbooks addressing the design of trusses were written when riveted connections were used. Today welded trusses and field bolted trusses are used exclusively.

Presented in the following paragraphs are concepts and principles that apply to the design of roof trusses.

5.1 General Design and Economic Considerations

No absolute statements can be made about what truss configuration will provide the most economical solution. For a particular situation, however, the following statements can be made regarding truss design:

1. Span-to-depth ratios of 15 to 20 generally prove to be economical; however, shipping depth limitations should be considered so that shop fabrication can be maximized. The maximum depth for shipping is conservatively 14 ft. Greater depths will require the web members to be field bolted, which will increase erection costs.

2. The length between splice points is also limited by shipping lengths. The maximum shippable length varies according to the destination of the trusses, but lengths of 80 ft are generally shippable and 100 ft is often possible. Because maximum available mill length is approximately 70 ft, the distance between splice points is normally set at a maximum of 70 ft. Greater distances between splice points will generally require truss chords to be shop spliced.

3. In general, the rule "deeper is cheaper" is true; however, the costs of additional lateral bracing for more flexible truss chords must be carefully examined relative to the cost of larger chords which may require less lateral bracing. The lateral bracing requirements for the top and bottom chords should be considered interactively while selecting chord sizes and types. Particular attention should be paid to loads that produce compression in the bottom chord. In this condition additional chord bracing will most likely be necessary.

4. If possible, select truss depths so that tees can be used for the chords rather than wide flange shapes. Tees can eliminate (or reduce) the need for gusset plates.

5. Higher strength steels (F_y = 50 ksi or more) usually results in more efficient truss members.

6. Illustrated in Figures 5.1.1 and 5.1.2 are web arrangements that generally provide economical web systems.

7. Utilize only a few web angle sizes, and make use of efficient long leg angles for greater resistance to buckling. Differences in angle sizes should be recognizable. For instance avoid using an angle 4×3×¼ and an angle 4×3×5⁄16 in the same truss.

8. HSS, wide flange or pipe sections may prove to be more effective web members at some web locations, especially where subsystems are to be supported by web members.

9. Designs using the AISC *LRFD Specification* (AISC, 1999) will often lead to truss savings when heavy long span trusses are required. This is due to the higher DL to LL ratios for these trusses.

10. The weight of gusset plates, shim plates and bolts can be significant in large trusses. This weight must be considered in the design since it often approaches 10 to 15 percent of the truss weight.

11. If trusses are analyzed using frame analysis computer programs and rigid joints are assumed, secondary

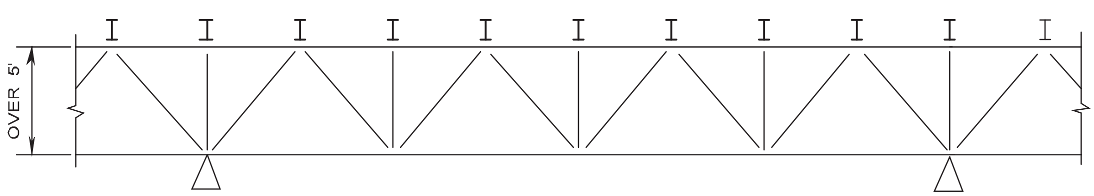


Fig. 5.1.1 Economical Truss Web Arrangement

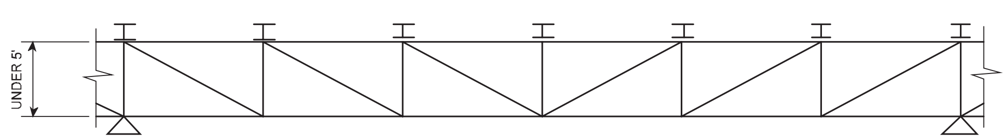


Fig. 5.1.2 Economical Truss Web Arrangement

bending moments will show up in the analysis. The reader is referred to (Nair, 1988a) wherein it is suggested that so long as these secondary stresses do not exceed 4,000 psi they may be neglected. Secondary stresses should not be neglected if the beneficial effects of continuity are being considered in the design process, for example, effective length determination. The designer must be consistent. That is, if the joints are considered as pins for the determination of forces, then they should also be considered as pins in the design process. The assumption of rigid joints in some cases may provide unconservative estimates on the deflection of the truss.

12. Repetition is beneficial and economical. Use as few different truss depths as possible. It is cheaper to vary the chord size as compared to the truss depth.

13. Wide flange chords with gussets may be necessary when significant bending moments exist in the chords (i.e. subsystems not supported at webs or large distances between webs).

14. The AISC *Manual of Steel Construction* can provide some additional guidance on truss design and detailing.

15. Design and detailing of long span joists and joist girders shall be in accordance with SJI specifications (SJI, 2002).

5.2 Connection Considerations

1. As mentioned above, tee chords are generally economical since they can eliminate gusset plates. The designer should examine the connection requirements to determine if the tee stem is in fact long enough to eliminate gusset requirements. The use of a deeper tee stem is generally more economical than adding numerous gusset plates even if this means an addition in overall weight.

2. Block shear requirements and the effective area in compression should be carefully checked in tee stems and gussets (AISC, Appendix B). Shear rupture of chord members at panel points should also be investigated since this can often control wide flange chords.

3. Intermediate connectors (stitch fasteners or fillers) may be required for double web members. Examples of intermediate connector evaluation can be found in the AISC *Manual*.

4. If wide flange chords are used with wide flange web members it is generally more economical to orient the chords with their webs horizontal. Gusset plates for the web members can then be either bolted or welded to the chord flanges. To eliminate the cost of fabricating large shim or filler plates for the diagonals, the use of comparable depth wide flange diagonals should be considered.

5. When trusses require field bolted joints the use of slip-critical bolts in conjunction with oversize holes will allow for erection alignment. Also if standard holes are used with slip-critical bolts and field “fit-up” problems occur, holes can be reamed without significantly reducing the allowable bolt shears.

6. For the end connection of trusses, top chord seat type connections should also be considered. Seat connections allow more flexibility in correcting column-truss alignment during erection. Seats also provide for efficient erection and are more stable during erection than “bottom bearing” trusses. When seats are used, a simple bottom chord connection is recommended to prevent the truss from rolling during erection.

7. For symmetrical trusses use a center splice to simplify fabrication even though forces may be larger than for an offset splice.

8. End plates can provide efficient compression splices.

9. It is often less expensive to locate the work point of the end diagonal at the face of the supporting member rather than designing the connection for the eccentricity between the column centerline and the face of the column.

5.3 Truss Bracing

Stability bracing is required at discrete locations where the designer assumes braced points or where braced points are required in the design of the members in the truss. These locations are generally at panel points of the trusses and at the ends of the web members. To function properly the braces must have sufficient strength and stiffness. Using standard bracing theory, the brace stiffness required (Factor of Safety = 2.0) is equal to $4P/L$, where P equals the force to be braced and L equals the unbraced length of the column. The required brace force equals $0.004P$. As a general rule the stiffness requirement will control the design of the bracing unless the bracing stiffness is derived from axial stresses only. Braces that displace due to axial loads only are very stiff, and thus the strength requirement will control. It should be noted that the AISE Technical Report No. 13 requires a $0.025P$ force requirement for bracing. More refined bracing equations are contained in a paper by Lutz

Design Forces (Kips)		
Horizontal Truss Web Member Forces		
Member	**Panel Shear**	**Force = (1.414)(Panel Shear)**
C1-D2 D1-C2	0.006(6X600) = 21.6	30.5
C2-D3 D2-C3	0.006(6X800) = 28.8	40.7
C3-D4 D3-C4	0.006(6X1000) = 36.0	50.9
Horizontal Truss Chord Forces		
Member	**Member Forces**	
C1-C2 D1-D2	21.6	
C2-C3 D2-D3	21.6 + 28.8 = 50.4	
C3-C4 D3-D4	50.4 + 36 = 86.4	
Strut Forces		
Member	**Force = (1.2%)(Ave. Chord Force)**	
A4-B4, E4-F4	12.0	
B4-C4, D4-E4	24.0	
C4-D4	36.0	
A3-B3, E3-F3	10.8	
B3-C3, D3-E3	21.6	
C3-D3	32.4	
A2-B2, E2-F2 B2-C2, D2-E2 C2-D2	8.4 16.8 25.2	
A1-B1, E1-F1 B1-C1, D1-E1 C1-D1	3.6 7.2 10.8	

Note: Forces not shown are symmetrical

and Fisher titled, *A Unified Approach for Stability Bracing Requirements* (Lutz, 1985). Requirements for truss bottom chord bracing are discussed in a paper by Fisher titled, *The Importance of Tension Chord Bracing* (Fisher, 1983). These requirements do not necessarily apply to long span joists or joist girders.

Designers are often concerned about the number of "out-of-straight" trusses that should be considered for a given bracing situation. No definitive rules exist; however, the Australian Code indicates that no more than seven out of straight members need to be considered. Chen and Tong (1994) recommend that $\sqrt{n}$ columns be considered in the out-of-straight condition where n = the total number of columns in a story. This equation suggests that $\sqrt{n}$ trusses could be considered in the bracing design. Thus, if ten trusses were to be braced, bracing forces could be based on four trusses.

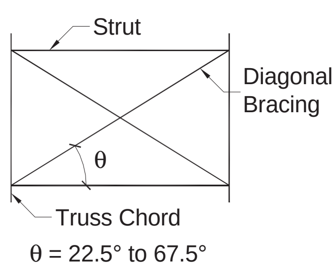


Fig. 5.3.1 Horizontal X-Bracing Arrangement

Common practice is to provide horizontal bracing every five to six bays to transfer bracing forces to the main force resisting system. In this case the brace forces should be calculated based on the number of trusses between horizontal bracing.

A convenient approach to the stability bracing of truss compression chords is discussed in a paper by entitled "Simple Solutions to Stability Problems in the Design Office" (Nair, 1988b). The solution presented is based upon the brace stiffness requirements controlled by an X-braced system. The paper indicates that as long as the horizontal X-bracing system comprises axially loaded members arranged as shown in Figure 5.3.1, the bracing can be designed for 0.6 percent of the truss chord axial load. Since two truss chord sections are being braced at each bracing strut location the strut connections to the trusses must be designed for 1.2 percent of the average chord axial load for the two adjacent chords. In the reference it is pointed out that the bracing forces do not accumulate along the length of the truss; however, the brace force requirements do accumulate based on the number of trusses considered braced by the bracing system.

In addition to stability bracing, top and bottom chord bracing may also be required to transfer wind or seismic lateral loads to the main lateral stability system. The force requirements for the lateral loads must be added to the stability force requirements. Lateral load bracing is placed in either the plane of the top chord or the plane of the bottom chord, but generally not in both planes. Stability requirements for the unbraced plane can be transferred to the laterally braced plane by using vertical sway braces.

EXAMPLE 5.3.1

Roof Truss Stability Bracing

For the truss system shown in Figure 5.3.2 determine the brace forces in the horizontal bracing system. Use the procedure discussed by (Nair, 1988b).

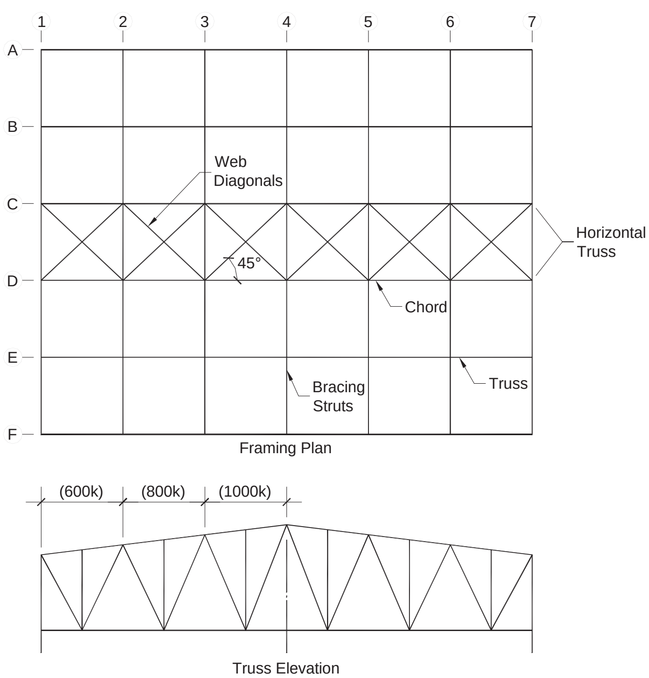


Fig. 5.3.2 Horizontal Bracing Systen

Solution:

Because the diagonal bracing layout as shown in Figure 5.3.2 forms an angle of 45 degrees with the trusses, the solution used in the paper by Nair, (1988b) is suitable. The bracing force thus equals 0.6 percent of the chord axial load. Member forces are summarized above.

5.4 Erection Bracing

The engineer of record is not responsible for the design of erection bracing unless specific contract arrangements incorporate this responsibility into the work. However, designers must be familiar with OSHA erection requirements (OSHA, 2001) relative to their designs.

Even though the designer of trusses is not responsible for the erection bracing, the designer should consider sequence and bracing requirements in the design of large trusses in order to provide the most cost effective system. Large trusses require significant erection bracing not only to resist wind and construction loads but also to provide stability until all of the gravity load bracing is installed. Significant cost savings can be achieved if the required erection bracing is incorporated into the permanent bracing system.

Erection is generally accomplished by first connecting two trusses together with strut braces and any additional erection braces to form a stable box system. Additional trusses are held in place by the crane or cranes until they can be "tied off" with strut braces to the already erected stable system. Providing the necessary components to facilitate this type of erection sequence is essential for a cost effective project.

Additional considerations are as follows:

1. Columns are usually erected first with the lateral bracing system (see Figure 5.4.1). If top chord seats are

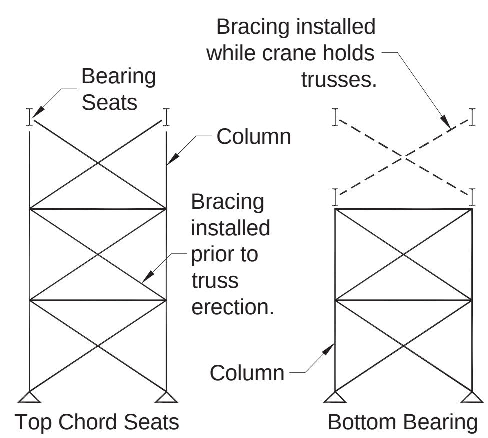


Fig. 5.4.1 Wall Bracing Erection Sequence

used, the trusses can be quickly positioned on top of the columns, braced to one another.

Bottom chord bearing trusses require that additional stability bracing be installed at ends of trusses while the cranes hold the trusses in place. This can slow down the erection sequence.

2. Since many industrial buildings require clear spans, systems are often designed as rigid frames. By designing rigid frames, erection is facilitated, in that, the sidewall columns are stabilized in the plane of the trusses once the trusses are adequately anchored to the columns. This scheme may require larger columns than a braced frame system; however, savings in bracing and erection time can often offset these costs.

3. Wide flange beams, HSS or pipe sections should be used to laterally brace large trusses at key locations during erection because of greater stiffness. Steel joists can be used; however, two notes of caution are advised:

   a. Erection bracing strut forces must be provided to the joist manufacturer; and it must be made clear whether joist bridging and roof deck will be in place when the erection forces are present. Large angle top chords in joists may be required to control the joist slenderness ratio so that it does not buckle while serving as the erection strut.
   b. Joists are often not fabricated to exact lengths and long slotted holes are generally provided in joist seats. Slotted holes for bolted bracing members should be avoided because of possible slippage. Special coordination with the joist manufacturer is required to eliminate the slots and to provide a suitable joist for bracing. In addition the joists must be at the job site when the erector wishes to erect the trusses.

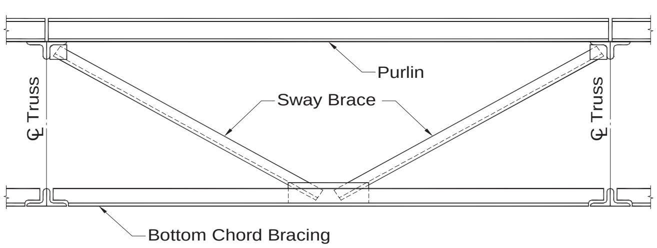


Fig. 5.4.2 Sway Frame

4. Wind forces on the trusses during erection can be considerable. See Design Loads on Structures During Construction, ASCE 37-02, ASCE (2002), for detailed treatment of wind forces on buildings during construction. The AISC Code of Standard Practice states that "These temporary supports shall be sufficient to secure the bare Structural Steel framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations." The projected area of all of the truss and other roof framing members can be significant, and in some cases the wind forces on the unsided structure are actually larger than those after the structure is enclosed.

5. A sway frame is normally required in order to plumb the trusses during erection. These sway frames should normally occur every fourth or fifth bay. An elevation view of such a truss is shown in Figure 5.4.2. These frames can be incorporated into the bottom chord bracing system. Sway frames are also often used to transfer forces from one chord level to another as discussed earlier. In these cases the sway frames must not only be designed for stability forces, but also the required load transfer forces.

5.5 Other Considerations

1. Camber large clear span trusses to accommodate dead load deflections. The fabricator accomplishes this by either adjusting the length of the web members in the truss and keeping the top chord segments straight or by curving the top chord. Tees can generally be easily curved during assembly whereas wide flange sections may require cambering prior to assembly. If significant top chord pitch is provided and if the bottom chord is pitched, camber may not be required. The engineer of record is responsible for providing the fabricator with the anticipated dead load deflection and special cambering requirements.

   The designer must carefully consider the truss deflection and camber adjacent to walls, or other portions of the structure where stiffness changes cause variations in deflections. This is particularly true at building endwalls, where differential deflections may damage continuous purlins or connections.

2. Connection details that can accommodate temperature changes are generally necessary. Long span trusses that are fabricated at one temperature and erected at a significantly different temperature can grow or shrink significantly.

3. Roof deck diaphragm strength and stiffness are commonly used for strength and stability bracing for joists. The diaphragm capabilities must be carefully evaluated if it is to be used for bracing of large clear span trusses.

For a more comprehensive treatment of erection bracing design, read *Serviceability Design Considerations for Steel Buildings*, (Fisher and West, 2003).

6. WALL SYSTEMS

The wall system can be chosen for a variety of reasons and the cost of the wall can vary by as much as a factor of three. Wall systems include:

1. Field assembled metal panels.
2. Factory assembled metal panels.
3. Precast concrete panels.
4. Masonry walls (part or full height).

A particular wall system may be selected over others for one or more specific reasons including:

1. Cost.
2. Appearance.
3. Ease of erection.
4. Speed of erection.
5. Insulating properties.
6. Fire considerations.
7. Acoustical considerations.
8. Ease of maintenance/cleaning.
9. Ease of future expansion.
10. Durability of finish.
11. Maintenance considerations.

Some of these factors will be discussed in the following sections on specific systems. Other factors are not discussed and require evaluation on a case-by-case basis.

6.1 Field-Assembled Panels

Field assembled panels consist of an outer skin element, insulation, and in some cases an inner liner panel. The panels vary in material thickness and are normally galvanized, galvanized prime painted suitable for field painting, or prefinished galvanized. Corrugated aluminum liners are also used. When aluminum materials are used their compatibility with steel supports should be verified with the manufacturer since aluminum may cause corrosion of steel. When an inner liner is used, some form of hat section interior subgirts are generally provided for stiffness. The insulation is typically fiberglass or foam. If the inner liner sheet is used as the vapor barrier all joints and edges should be sealed.

Specific advantages of field assembled wall panels include:

1. Rapid erection of panels.
2. Good cost competition, with a large number of manufacturers and contractors being capable of erecting panels.
3. Quick and easy panel replacement in the event of panel damage.
4. Openings for doors and windows that can be created quickly and easily.
5. Panels that are lightweight, so that heavy equipment is not required for erection. Also large foundations and heavy spandrels are not required.

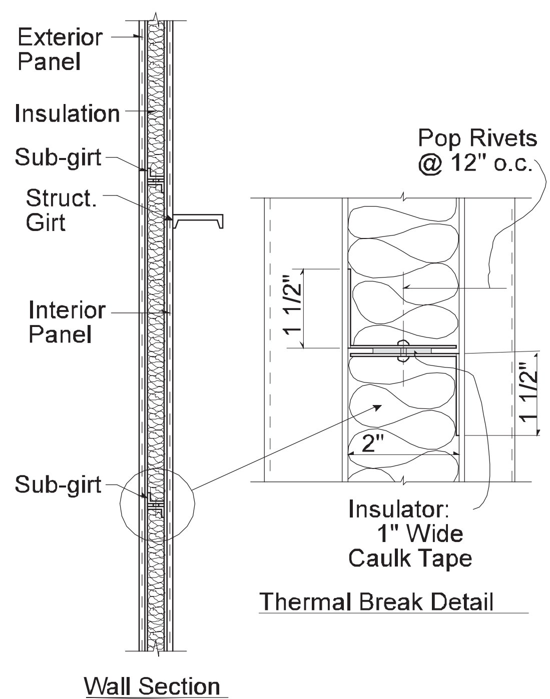


Fig. 6.1.1 Wall Thermal Break Detail

6. Acoustic surface treatment that can be added easily to interior panel wall at reasonable cost.

A disadvantage of field assembled panels in high humidity environments can be the formation of frost or condensation on the inner liner when insulation is placed only between the subgirt lines. The metal-to-metal contact (outside sheet-subgirt-inside sheet) should be broken to reduce thermal bridging. A detail that has been used successfully is shown in Figure 6.1.1. Another option may be to provide rigid insulation between the girt and liner on one side. In any event, the wall should be evaluated for thermal transmittance in accordance with (ASHRAE, 1989).

6.2 Factory-Assembled Panels

Factory assembled panels generally consist of interior liner panels, exterior metal panels and insulation. Panels providing various insulating values are available from several manufacturers. These systems are generally proprietary and must be designed according to manufacturer's recommendations.

The particular advantages of these factory-assembled panels are:

1. Panels are lightweight and require no heavy cranes for erection, no large foundations or heavy spandrels.

2. Panels can have a hard surface interior liner.

3. Panel side lap fasteners are normally concealed producing a "clean" appearance.

4. Documented panel performance characteristics determined by test or experience may be available from manufacturers.

Disadvantages of factory-assembled panels include:

1. Once a choice of panel has been made, future expansions may effectively require use of the same panel to match color and profile, thus competition is essentially eliminated.

2. Erection procedures usually require starting in one corner of a structure and proceeding to the next corner. Due to the interlocking nature of the panels it may be difficult to add openings in the wall.

3. Close attention to coordination of details and tolerances with collateral materials is required.

4. Thermal changes in panel shape may be more apparent.

6.3 Precast Wall Panels

Precast wall panels for industrial buildings could utilize one or more of a variety of panel types including:

1. Hollow core slabs.

2. Double-T sections.

3. Site cast tilt-up panels.

4. Factory cast panels.

Panels can be either load bearing or nonload bearing and can be obtained in a wide variety of finishes, textures and colors. Also, panels may be of sandwich construction and contain rigid insulation between two layers of concrete. Such insulated panels can be composite or noncomposite. Composite panels normally have a positive concrete connection between inner and outer concrete layers. These panels are structurally stiff and are good from an erection point of view but the "positive" connection between inner and outer layers may lead to exterior surface cracking when the panels are subjected to a temperature differential. The direct connection can also provide a path for thermal bridging.

True sandwich panels connect inner and outer concrete layers with flexible metal ties. Insulation is exposed at all panel edges. These panels are more difficult to handle and erect, but normally perform well.

Precast panels have advantages for use in industrial buildings:

1. A hard surface is provided inside and out.

2. These panels produce an architecturally "clean" appearance.

3. Panels have inherent fire resistance characteristics.

4. Intermediate girts are usually not required.

5. Use of load bearing panels can eliminate exterior framing and reduce cost.

6. Panels provide an excellent sound barrier.

Disadvantages of precast wall panel systems include:

1. Matching colors of panels in future expansion may be difficult.

2. Composite sandwich panels have "cold spots" with potential condensation problems at panel edges.

3. Adding wall openings can be difficult.

4. Panels have poor sound absorption characteristics.

5. Foundations and grade beams may be heavier than for other panel systems.

6. Heavier eave struts are required for steel frame structures than for other systems.

7. Heavy cranes are required for panel erection.

8. If panels are used as load bearing elements, expansion in the future could present problems.

9. Close attention to tolerances and details to coordinate divergent trades are required.

10. Added dead weight of walls can affect seismic design.

6.4 Masonry Walls

Use of masonry walls in industrial buildings is common. Walls can be load bearing or non-load bearing.

Some advantages of the use of masonry construction are:

1. A hard surface is provided inside and out.

2. Masonry walls have inherent fire resistance characteristics.

3. Intermediate girts are usually not required.

4. Use of load bearing walls can eliminate exterior framing and reduce cost.

5. Masonry walls can serve as shear walls to brace columns and resist lateral loads.

6. Walls produce a flat finish, resulting in an ease of both maintenance and dust control considerations.

Disadvantages of masonry include:

1. Masonry has comparatively low material bending resistance. Walls are normally adequate to resist normal wind loads, but interior impact loads can cause damage.

2. Foundations may be heavier than for metal wall panel construction.

3. Special consideration is required in the use of masonry ties, depending on whether the masonry is erected before or after the steel frame.

4. Buildings in seismic regions may require special reinforcing and added dead weight may increase seismic forces.

6.5 Girts

Typical girts for industrial buildings are hot rolled channel sections or cold-formed light gage C or Z sections. In some instances HSS are used to eliminate the need for compression flange bracing. In recent years, cold-formed sections have gained popularity because of their low cost. As mentioned earlier, cold-formed Z sections can be easily lapped to achieve continuity resulting in further weight savings and reduced deflections, Z sections also ship economically. Additional advantages of cold-formed sections compared with rolled girt shapes are:

1. Metal wall panels can be attached to cold-formed girts quickly and inexpensively using self-drilling fasteners.

2. The use of sag rods is often not required.

Hot-rolled girts are often used when:

1. Corrosive environments dictate the use of thicker sections.

2. Common cold-formed sections do not have sufficient strength for a given span or load condition.

3. Girts will receive substantial abuse from operations.

4. Designers are unfamiliar with the availability and properties of cold-formed sections.

Both hot-rolled and cold-formed girts subjected to pressure loads are normally considered laterally braced by the wall sheathing. Negative moment regions in continuous cold-formed girt systems are typically considered laterally braced at inflection points and at girt to column connections. Continuous systems have been analyzed by assuming:

1. A single prismatic section throughout.

2. A double moment of inertia condition within the lapped section of the cold-formed girt.

Research indicates that an analytical model assuming a single prismatic section is closer to experimentally determined behavior (Robertson, 1986).

The use of sag rods is generally required to maintain horizontal alignment of hot-rolled sections. The sag rods are often assumed to provide lateral restraint against buckling for suction loads. When used as bracing, the sag rods must be designed to take tension in either the upward or downward direction. The paneling is assumed to provide lateral support for pressure loads. Lateral stability for the girt based on this assumption is checked using Chapter F of the AISC *Specification*.

The typical design procedure for hot-rolled girts is as follows:

1. Select the girt size based on pressure loads, assuming full flange lateral support.

2. Check the selected girt for sag rod requirements based on deflections and bending stresses about the weak axis of the girt.

3. Check the girt for suction loads using Chapter F of the AISC Specification.

4. If the girt is inadequate, increase its size or add sag rods.

5. Check the girt for serviceability requirements.

6. Check the sag rods for their ability to resist the twist of the girt due to the suction loads. The sag rod and siding act to provide the torsional brace.

Cold-formed girts should be designed in accordance with the provisions of the American Iron and Steel Institute *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001). Many manufacturers of cold-formed girts provide design assistance, and offer load span tables to aid design.

Section C3.1.2 "Lateral Buckling Strength" of the AISI *Specification* provides a means for determining cold-formed girt strength when the compression flange of the girt is attached to sheeting (fully braced) or when discrete point braces (sag rods) are used. For lapped systems, the sum of the moment capacities of the two lapped girts is normally assumed to resist the negative moment over the support. For full continuity to exist, a lap length on each side of the column support should be equal to at least 1.5 times the girt depth (Robertson, 1986). Additional provisions are given in Section C3 for strength considerations relative to shear, web crippling, and combined bending and shear.

Section C3.1.3 "Beams with One Flange Attached to Deck or Sheathing" provides a simple procedure to design cold-formed girts subjected to suction loading. The basic equation for the determination of the girt strength is:

$$M_n = RS_eF_y$$

The values of R are shown below:

Simple Span C- or Z-Section R Values		
Depth Range, in.	**Profile**	**R**
d ≤ 6.5	C or Z	0.70
6.5 < d ≤ 8.5	C or Z	0.65
8.5 < d ≤ 11.5	Z	0.50
8.5 < d ≤ 11.5	C	0.40

S_e = Elastic section modulus, of the effective section, calculated with the extreme compression or tension fiber at F_y.

F_y = Specified minimum yield stress.

Other restrictions relative to insulation, girt geometry, wall panels, fastening systems between wall panels and girts, etc. are discussed in the AISI specifications.

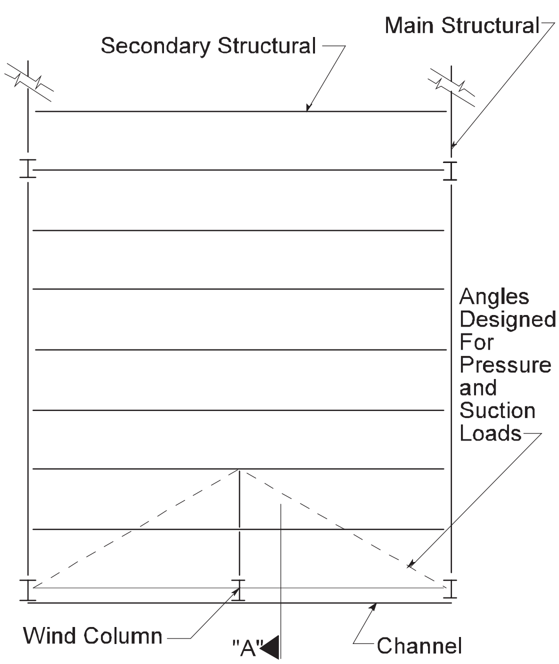


Fig. 6.6.1 Wind Column Reaction Load Transfer

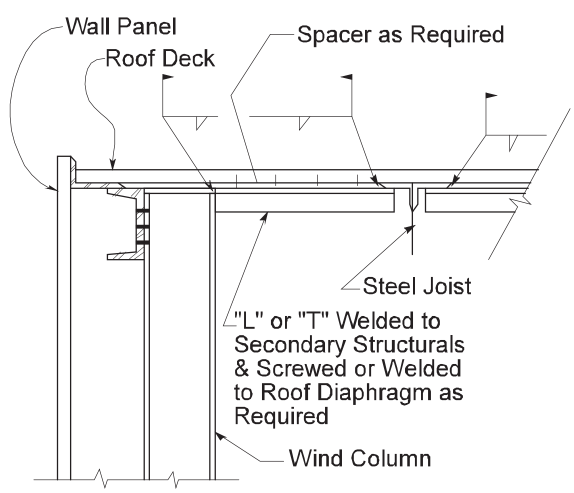


Fig. 6.6.2 Wind Column Reaction Load Transfer

It should also be mentioned that consideration should be given to tolerance differences between erected columns and girts. The use of slotted holes in girt to column attachments is often required.

6.6 Wind Columns

When bay spacings exceed 30 ft additional intermediate columns may be required to provide for economical girt design. Two considerations that should be emphasized are:

1. Sufficient bracing of the wind columns to accommodate wind suction loads is needed. This is normally accomplished by bracing the interior flanges of the columns with angles that connect to the girts.

2. Proper attention should be paid to the top connections of the columns. For intermediate sidewall columns, secondary roof framing members must be provided to transfer the wind reaction at the top of the column into the roof bracing system. Do not rely on "trickle theory" (in other words, "a force will find a way to trickle out of the structure"). A positive and calculable system is necessary to provide a traceable load path (in other words, Figure 6.6.1). Bridging systems or bottom chord extension on joists can be used to dissipate these forces, but the stresses in the system must be checked. If the wind columns have not been designed for axial load, a slip connection would be necessary at the top of the column.

Small wind reactions can be transferred from the wind columns into the roof diaphragm system as shown in Figure 6.6.2.

Allowable values for attaching metal deck to structural members can be obtained from screw manufacturers. Allowable stresses in welds to metal deck can be determined from the American Welding Society *Standard Specification for Welding Sheet Steel in Structures*, (AWS, 1998) or from the AISI specifications (AISI, 2001). In addition to determining the fastener requirements to transfer the concentrated load into the diaphragm, the designer must also check the roof diaphragm for its strength and stiffness. This can be accomplished by using the procedures contained in the Steel Deck Institute's *Diaphragm Design Manual* (SDI, 2001).

7. FRAMING SCHEMES

The selection of "the best" framing scheme for an industrial building without cranes is dependent on numerous considerations, and often depends on the owner's requirements. It may not be possible to give a list of rules by which the best such scheme can be assured. If "best" means low initial cost, then the owner may face major expenses in the future for operational expenses or problems with expansion. Extra dollars invested at the outset reduce potential future costs.

The economy of using of long span vs. short span joists and purlins has been mentioned previously in this guide. This section expands on the selection of the main framing system. No attempt has been made to evaluate foundation costs. In general, if a deep foundation system (for example, piles or drilled piers) is required, longer bay spacings are normally more economical.

The consideration of bay sizes must include not only roof and frame factors but also the wall system. The cost of large girts and thick wall panels may cancel the savings anticipated if the roof system alone is considered.

Additional aids in the design of efficient framing details can be found in *Detailing for Steel Construction* (AISC, 2002).

7.1 Braced Frames vs. Rigid Frames

The design of rigid frames is explained in numerous textbooks and professional journals and will not be covered here; however, a few concepts will be presented concerning the selection of a braced versus a rigid frame structural system. There are several situations for which a rigid frame system is likely to be superior.

1. Braced frames may require bracing in both the walls and roof. Bracing frequently interferes with plant operations and future expansion. If either consideration is important, a rigid frame structure may be the answer.

2. The bracing of a roof system can be accomplished through X-bracing or a roof diaphragm. In either case the roof becomes a large horizontal beam spanning between the walls or bracing which must transmit the lateral loads to the foundations. For large span to width ratios (greater than 3:1) the bracing requirements become excessive. A building with dimensions of 100 ft by 300 ft with potential future expansion in the long direction may best be suited for rigid frames to minimize or eliminate bracing that would interfere with future changes.

Use of a metal building system requires a strong interaction between the designer and the metal building manufacturer. That's because of much of the detailing process related to design is provided by the manufacturer, and the options open to the buyer may reflect the limits of the manufacturer's standard product line and details.

Experience has shown that there are occasions when braced frame construction may prove to be more economical than either standard metal building systems or special rigid frame construction when certain sacrifices on flexibility are accepted.

JOIST DATA		RELATIVE COSTS* MAIN FRAMING MEMBERS – W SECTIONS				
		Span (ft)				
Depth (in.)	Span (ft)	25	30	40	48	60
16	25	1.10	1.10	1.25	1.31	1.53
18	30	1.12	1.07	1.20	1.28	1.50
24	40	1.16	1.05	1.15	1.28	1.47
30	50	1.22	1.18	1.20	1.30	1.54
32	60	1.33	1.30	1.30	1.33	1.60

*Cost included fabrication and erection of primary and secondary framing (no deck). A total gravity load of 48 psf was used in all designs. Uplift and lateral bracing requirements were not included

7.2 HSS Columns vs. W Shapes

The design of columns in industrial buildings includes considerations that do not apply to other types of structures. Interior columns can normally be braced only at the top and bottom, thus square HSS columns are desirable due to their equal stiffness about both principal axes. Difficult connections with HSS members can be eliminated in single-story frames by placing the beams over the tops of the HSS. Thus a simple to fabricate cap plate detail with bearing stiffeners on the girder web may be designed. Other advantages of HSS columns include the fact that they require less paint than equivalent W shapes, and they are pleasing aesthetically.

W shapes may be more economical than HSS for exterior columns for the following reasons:

1. The wall system (girts) may be used to brace the weak axis of the column. It should be noted that a stiffener or brace may be required for the column if the inside column flange is in compression and the girt connection is assumed to provide a braced point in design.

2. Bending moments due to wind loads predominate about one axis.

3. It is easier to frame girt connections to a W shape than to a HSS section. Because HSS have no flanges, extra clip angles are required to connect girts.

7.3 Mezzanine and Platform Framing

Mezzanines and platforms are often required in industrial buildings. The type of usage dictates design considerations. For proper design the designer needs to consider the following design parameters:

1. Occupancy or Use.

2. Design Loads (Uniform and Concentrated).

3. Deflection Criteria.

4. Surface Type.
   a. Raised pattern plate.
   b. Smooth plate.
   c. Concrete composite slab.
   d. Concrete non-composite slab.
   e. Hollow core slabs (topped or untopped).
   f. Plywood.

5. Guard rail requirements, including removable sections.

6. Future Expansion.

7. Vibration Control.

8. Lateral Stability Requirements.

7.4 Economic Considerations

As previously mentioned, bay sizes and column spacing are often dictated by the function of the building. Economics, however, should also be considered. In general, as bay sizes increase, the weight of the horizontal framing increases. This will mean additional cost unless offset by savings in foundations or erection. Studies have indicated that square or slightly rectangular bays usually result in more economical structures.

In order to evaluate various framing schemes, a prototype general merchandise structure was analyzed using various spans and component structural elements. The structure was a 240-ft × 240-ft building with a 25-ft eave height. The total roof load was 48 psf, and beams with $F_y = 50$ ksi were used. Plastic analysis and design was used. Columns were HSS with a yield strength of 46 ksi.

Variables in the analysis were:

1. Joist spans: 25, 30, 40, 50 and 60 ft.

2. Girder spans, W sections: 25, 30, 40, 48 and 60 ft.

Cost data were determined from several fabricators. The data did not include sales tax or shipping costs. The study yielded several interesting conclusions for engineers involved in industrial building design.

An examination of the tabular data shows that the most economical framing scheme was the one with beams spanning 30 ft and joists spanning 40 ft.

Another factor that may be important is that for the larger bays (greater than 30 ft) normal girt construction becomes less efficient using C or Z sections without intermediate "wind columns" being added. For the 240-ft × 240-ft building being considered, wind columns could add \$0.10 per square ft of roof to the cost. Interestingly, if the building were 120 ft × 120 ft, the addition of intermediate wind columns would add \$0.20 per ft^2 because the smaller building has twice the perimeter to area ratio as the larger structure.

Additional economic and design considerations are as follows:

1. When steel joists are used in the roof framing it is generally more economical to span the joists in the long direction of the bay.

2. K series joists are more economical than LH joists; thus an attempt should be made to limit spans to those suitable for K joists.

3. For 30-ft to 40-ft bays, efficient framing may consist of continuous or double-cantilevered girders supported by columns in one direction and joists spanning the other direction.

4. If the girders are continuous, plastic design is often used. Connection costs for continuous members may be higher than for cantilever design; however, a plastically designed continuous system will have superior behavior when subjected to unexpected load cases. All flat roof systems must be checked to prevent ponding problems. See Section 4.5.

5. Simple-span rolled beams are often substituted for continuous or double-cantilevered girders where spans are short. The simple span beams often have adequate moment capacity. The connections are simple, and the savings from easier erection of such systems may overcome the cost of any additional weight.

6. For large bay dimensions in both directions, a popular system consists of cold-formed or hot-rolled steel purlins or joists spanning 20 ft to 30 ft to secondary trusses spanning to the primary trusses. This framing system is particularly useful when heavily loaded monorails must be hung from the structure. The secondary trusses in conjunction with the main trusses provide excellent support for the monorails.

7. Consideration must be given to future expansion and/or modification, where columns are either moved or eliminated. Such changes can generally be accomplished with greater ease where simple span conditions exist.

8. BRACING SYSTEMS

8.1 Rigid Frame Systems

There are many considerations involved in providing lateral stability to industrial structures. If a rigid frame is used, lateral stability parallel to the frame is provided by the frame. However, for loads perpendicular to the main frames and for wall bearing and "post and beam" construction, lateral bracing is not inherent and must be provided. It is important to re-emphasize that future expansion may dictate the use of a rigid frame or a flexible (movable) bracing scheme.

Since industrial structures are normally light and generally low in profile, wind and seismic forces may be relatively low. Rigid frame action can be easily and safely achieved by providing a properly designed member at a column line. If joists are used as a part of the rigid frame the designer is cautioned on the following points:

1. The design loads (wind, seismic, and continuity) must be given on the structural plans so that the joist manufacturer can provide the proper design. The procedure must be used with conscious engineering judgment and full recognition that standard steel joists are designed as simple span members subject to distributed loads. (See the Steel Joist Institute's Standard Specifications for Standard Steel Joists and Long Span Joists (SJI, 2002). Bottom chords are normally sized for tension only. The simple attachment of the bottom chord to a column to provide lateral stability will cause gravity load end moments that cannot be ignored. The designer should not try to select member sizes for these bottom chords since each manufacturer's design is unique and proprietary.

2. It is necessary for the designer to provide a well-designed connection to both the top and bottom chords to develop the induced moments without causing

excessive secondary bending moments in the joist chords.

3. The system must have adequate stiffness to prevent drift related problems such as cracked walls and partitions, broken glass, leaking walls and roofs, and malfunctioning or inoperable overhead doors.

8.2 Braced Systems

Roof Diaphragms

The most economical roof bracing system is achieved by use of a steel deck diaphragm. The deck is provided as the roofing element and the effective diaphragm is obtained at little additional cost (for extra deck connections). A roof diaphragm used in conjunction with wall X-bracing or a wall diaphragm system is probably the most economical bracing system that can be achieved. Diaphragms are most efficient in relatively square buildings; however, an aspect ratio up to three can be accommodated.

A cold-formed steel diaphragm is analogous to the web of a plate girder. That is, its main function is to resist shear forces. The perimeter members of the diaphragm serve as the "flanges."

The design procedure is quite simple. The basic parameters that control the strength and stiffness of the diaphragm are:

1. Profile shape.
2. Deck material thickness.
3. Span length.
4. The type and spacing of the fastening of the deck to the structural members.
5. The type and spacing of the side lap connectors.

The profile, thickness, and span of the deck are typically based on gravity load requirements. The type of fastening (i.e., welding, screws, and power driven pins) is often based on the designer's or contractor's preference. Thus the main design variable is the spacing of the fasteners. The designer calculates the maximum shear per ft of diaphragm and then selects the fastener spacing from the load tables. Load tables are most often based on the requirements set forth in the Department of Army, Navy and Air Force TM 5-80-10, *Seismic Design for Buildings* (Department of Army, 1992), and the Steel Deck Institute's *Diaphragm Design Manual* (SDI, 1987).

Deflections are calculated and compared with serviceability requirements.

The calculation of flexural deformations is handled in a conventional manner. Shear deformations can be obtained mathematically, using shear deflection equations, if the shear modulus of the formed deck material making up the diaphragm is known. Deflections can also be obtained using empirical equations such as those found in (Department of Army, 1992) and (SDI, 1987). In addition, most metal deck manufacturers publish tables in which strength and stiffness (or flexibility) information is presented. In order to illustrate the diaphragm design procedure a design example is presented below. The calculations presented are based on the Steel Deck Institute's procedure (SDI, 1987)

EXAMPLE 8.2.1

Diaphragm Design (ASD)

Design the roof diaphragm for the structure shown in Figure 8.2.1. The eave wind loads are shown in the figure.

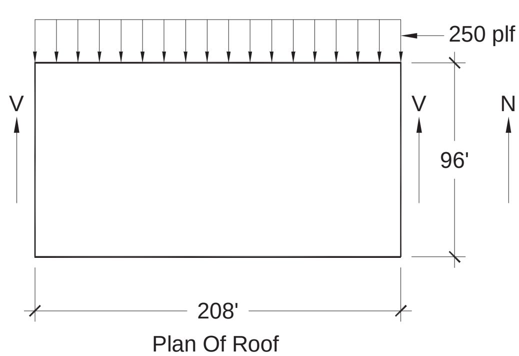


Fig. 8.2.1 Example

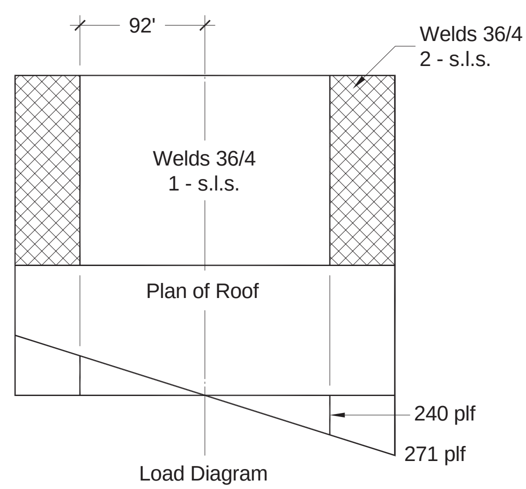


Fig. 8.2.1.1

Note that the length to width ratio of the diaphragm does not exceed 3, which is the generally accepted maximum for diaphragms.

Assume that a 0.0358 in. thick intermediate rib deck spanning 5 in-6 in is used to support the gravity loads. Steel joists span in the north-south direction. Use welds to connect the deck to the structural members and #10 screws for the side laps.

Solution:

1. Calculate the maximum diaphragm shear.

$$V = \frac{WL}{2} = \frac{(250)(208)}{2} \cong 26{,}000 \text{ lb}$$

$$v = \frac{V}{96} = \frac{26{,}000}{96} \cong 271 \text{ lb/ft}$$

2. Obtain the shear capacity of the deck from the SDI *Diaphragm Design Manual* (SDI, 1987).

For a 20-gage (0.0358 in. thickness) deck, spanning 5 ft-6 in. the allowable shear is:

a. 240 lb/ft with a 36/4 weld pattern and one side lap screw.
b. 285 lb/ft with a 36/4 weld pattern and two sidelap screws.
c. 300 lb/ft with a 36/5 weld pattern and one sidelap screw.

Use patterns a. and b. as shown in Figure 8.2.1.1:

3. Compute the lateral deflection of the diaphragm.

For simplicity assume one sidelap screw for the entire roof.

The deflection equations are:

a. For bending: $\Delta_b = \dfrac{5wL^4}{384EI}$

b. For shear: $\Delta_s = \dfrac{wL^2}{8DG'}$

where

w = the eave force (kips/ft)
L = the diaphragm length (ft)
D = the diaphragm depth (ft)

$$G' = \frac{K2}{3.78 + \left(\dfrac{0.3D_{xx}}{span}\right) + 3(K1)(span)}$$

From the SDI tables:

$K2$ = 1056
D_{xx} = D_{ir} = 909 (intermediate rib)
$K1$ = 0.561
$K1$ = 0.509 corresponds to 22-gage deck.

$$\therefore G' = \frac{1056}{3.78 + 0.3(909)/5.5 + 3(0.561)5.5} = 16.9$$

The moment of inertia, I, can be based on an assumed area of the perimeter member. Assuming the edge member has an area of 3.0 in.^{2}, the moment of inertia equals:

$$I = 2Ad^2 = (2)(3.0)(48\times12)^2 = 1.99\times10^6 \text{ in.}^4$$

The bending deflection equals:

$$\Delta_b = \frac{(5)(0.25)(208)^4(1728)}{(384)(29000)(1.99\text{x}10^6)} = 0.18 \text{ in.}$$

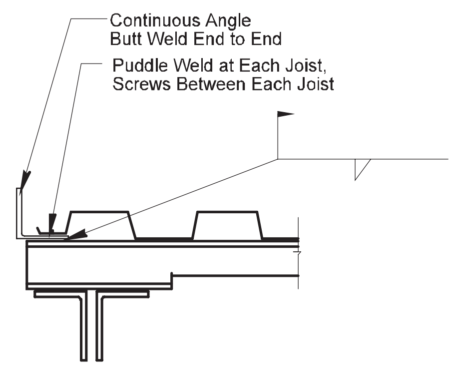


Fig. 8.2.2 Eave Angle

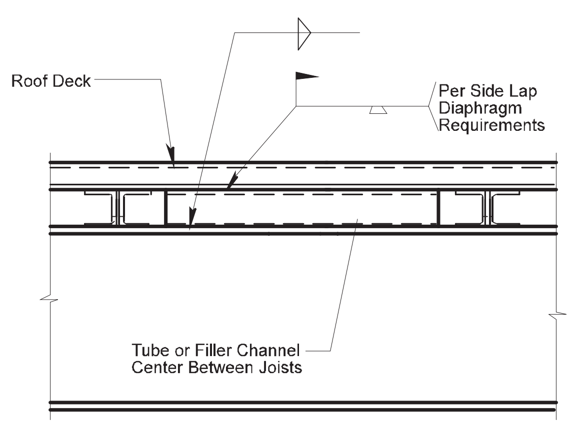


Fig. 8.2.3 Shear Collector

The shear deflection equals:

$$\Delta_s = \frac{(0.25)(208)^2}{(8)(96)(16.9)} = 0.83 \text{ in.}$$

The total deflection equals:

$$\Delta = \Delta_b + \Delta_s = 0.18 + 0.83 = 1.01 \text{ in.}$$

To transfer the shear forces into the east and west walls of the structure the deck can be welded directly to the perimeter beams. The deck must be connected to the perimeter beams with the same number of fasteners as required in the field of the diaphragm. Thus, ⅝ in. diaphragm arc spot welds 9 in. on center should be specified at the east and west walls.

The reader is cautioned regarding connecting steel deck to the end walls of buildings. If the deck is to be connected to a shear wall and a joist is placed next to the wall, allowance must be made for the camber in the edge joist in order to connect the deck to the wall system. If proper details are not provided, diaphragm connection may not be possible, and field adjustments may be required. Where the edge joist is eliminated near the endwall, the deck can often be pushed down flat on an endwall support. If the joist has significant camber, it may be necessary to provide simple span pieces of deck between the wall and the first joist. A heavier deck thickness may be required due to the loss in continuity. The edge should be covered with a sheet metal cap to protect the roofing materials. This can present an additional problem since the sharp edge of the deck will stick up and possibly damage the roofing.

Along the north and south walls, a diaphragm chord can be provided by attaching an angle to the top of the joists as shown in Figure 8.2.2. The angle also stiffens the deck edge and prevents tearing of roofing materials along the edge where no parapet is provided under foot traffic. In some designs an edge angle may also be required for the side lap connections for wind forces in the east-west direction. Also, shear connectors may be required to transfer these forces into the perimeter beam. Shown in Figure 8.2.3 is a typical shear collector.

Roof X-Bracing

An alternative to the roof diaphragm is to use X-bracing to develop a horizontal truss system. As with the metal deck diaphragm, as the length to width ratio of the building becomes larger than 3 to 1 the diagonal forces in the truss members may require consideration of an alternate bracing method.

An especially effective way to develop an X-braced roof is to utilize flat bar stock resting on the roof joists. The use of ¼ in. bar stock does not usually interfere with deck placement and facilitates erection.

Vertical Bracing

In braced buildings the roof diaphragm loads or the roof X-bracing loads are transferred to a vertical braced frame, which in turn transfers the loads to the foundation level. In most cases the vertical bracing is located at the perimeter of the structure so as not interfere with plant operations. The vertical bracing configuration most frequently used is an x-braced system using angles or rods designed only to function as tension members. However, in areas of high seismicity, a vertical bracing system that incorporates tension/compression members is often required. In these cases, other bracing forms may be used, such as, chevron bracing or eccentrically braced frames.

In buildings with large aspect ratios, bracing may be required in internal bays in order to reduce the brace forces, and to reduce foundation-overturning forces.

8.3 Temporary Bracing

Proper temporary bracing is essential for the timely and safe erection and support of the structural framework until the permanent bracing system is in place. The need for temporary bracing is recognized in Section M4.2 of the AISC specifications (AISC, 1989), (AISC, 1999), and in Section 7.10 of the AISC *Code of Standard Practice* (AISC, 2000).

The *Code of Standard Practice* places the responsibility for temporary bracing solely with the erector. This is appropriate since temporary bracing is an essential part of the work of erecting the steel framework.

While the general requirements of the *Code of Standard Practice* are appropriate in establishing the responsibility for temporary erection bracing, two major issues have the potential to be overlooked in the process.

First, it is difficult to judge the adequacy of temporary bracing in any particular situation using only the general requirements as a guide. There is no “codified” standard that can be applied in judging whether or not a minimum level of conformity has been met. However, ASCE 37-02, *Design Loads on Structures During Construction*, (ASCE, 2002) and AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Frames* (AISC, 1997) can be useful in making evaluations of the adequacy of proposed temporary bracing and in establishing the need for such bracing.

Secondly, the *Code of Standard Practice* does not emphasize that the process of erection can induce forces and stresses into components and systems such as footings and piers that are not part of the structural steel framework. Unless otherwise specified in the contract documents, it is the practice of architects and engineers to design the elements and systems in a building for the forces acting upon the completed structure only. An exception to this is the requirement in OSHA, Subpart R (OSHA, 2001) that column bases be designed to resist a 300-lb downward load acting at 18 in. from the faces of columns.

Without a detailed erection bracing plan it is difficult for anyone in the design/construction process to evaluate the performance of the erector relative to bracing without becoming involved in the process itself. This is inconsistent with maintaining the determination of temporary bracing as the sole responsibility of the erector. The lack of emphasis on the necessity that the erector must check the effect of erection induced forces on other elements has at times allowed erection problems to be erroneously interpreted as having been caused by other reasons. This is most obvious in the erection of steel columns.

To begin and pursue the erection of a steel framework it is necessary to erect columns first. This means that at one time or another each building column is set in place without stabilizing framing attached to it in two perpendicular directions. Without such framing the columns must cantilever for a time from the supporting footing or pier unless adequate guys brace them or unless the columns and beams are designed and constructed as rigid frames in both directions. The forces induced by the cantilevered column on the pier or footing may not have been considered by the building designer unless this had been specifically requested. It is incumbent upon the steel erector to make a determination of the adequacy of the foundation to support cantilevered columns during erection.

Trial calculations suggest that large forces can be induced into anchor rods, piers and footings by relatively small forces acting at or near the tops of columns. Also wind forces can easily be significant, as can be seen in the following example. Figure 8.3.1 shows a section of unbraced frame consisting of three columns and two beams. The beams are taken as pin ended. Wind forces are acting perpendicular to the frame line.

Using a shape factor of 2.0 for a 40 mph wind directed at the webs of the W12 columns, a base moment of approximately 18,000 ft-lbs occurs. If a 5 in. by 5 in. placement pattern were used with four anchor rods and an ungrouted base plate, a tension force of approximately 21.6 kips would be applied to the two anchor rods. The allowable force for a ¾ in. Grade 36 anchor rod is 8.4 kips. Even if the bolts were fully in the concrete, they would be severely overstressed and would likely fail. Four 1⅛ in. anchor rods would be required to resist the wind force. Of course not only the size of the anchor rod is affected, but the design of the base plate and its attachment to the column, the spacing of the anchor rods and the design of the pier and footing must also be checked.

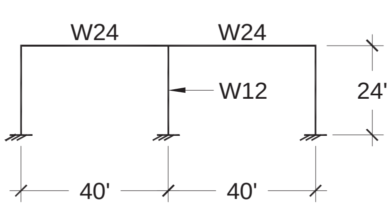


Fig. 8.3.1 Erection Bracing Example

Guying can also induce forces into the structure in the form of base shears and uplift forces. These forces may not have been provided for in the sizing of the affected members. The erector must also check this. The placement of material such as decking on the incomplete structure can induce unanticipated loadings. This loading must also be considered explicitly. OSHA, Subpart R states that no decking bundles may be placed on the frame until a qualified person has documented that a structure or portion is capable of supporting the load.

Erection bracing involves other issues as well. First, the *Code of Standard Practice* distinguishes between frames in which the frame is stabilized by construction in the control of the Erector versus those frames in which other non-structural steel elements are required for the stability of the frame. The distinction is drawn because the timing of the removal of bracing is affected. In a structural steel frame, where lateral stability is achieved in the design and detailing of the framework itself, the bracing can be removed when the erector's work is complete. A steel framework that relies on elements other than the structural steel to provide lateral stability should have the necessary elements providing the stability identified in the contract documents along with the schedule of their completion. The coordination of the installation of such elements is a matter that must be addressed by the General Contractor.

Temporary support beyond the requirements discussed above would be the responsibility of the owner according to the *Code of Standard Practice*. For example, if the steel frame and its temporary bracing are to support other non-structural elements, the responsibility for this must be clearly identified and the reactions from the elements are to be provided to the erector. Otherwise the responsibility for this falls to others, not the erector.

The timing of column base grouting affects the performance of column bases during erection. The *Code of Standard Practice* establishes the timing of grouting and assigns the responsibility for grouting to the owner. The erector should be aware of the schedule for this work.

All of the foregoing points to the need for care, attention and thoroughness on the part of the erector in preparing and following a temporary bracing and erection scheme.

Table 9.1.1 Allowable Bolt Fatigue Stress	
Number of Loading Cycles [a]	Allowable Tensile Stress (psi)
20,000 to 100,000	40,000
100,000 to 500,000	25,000
500,000 to 2,000,000	15,000
Over 2,000,000	10,000
[a] – These categories correspond to the loading conditions indicated in Appendix K of the AISC Specification.	

9. COLUMN ANCHORAGE

Building columns must be anchored to the foundation system to transfer tension forces, shear forces, and overturning moments. This discussion will be limited to the design of column anchorages for shear and tension forces. The principles discussed here can be applied to the design of anchorages for overturning moments.

Tension forces are typically transferred to the foundation system with anchor rods. Shear forces can be transferred to the foundation system through bearing, friction, or shear friction. The principal means of shear transfer considered in this section is through bearing of the anchor rods and through bearing of embedded components of the column. Friction should not be considered if seismic conditions exist. Design for these various anchorage methods is addressed in the following text.

Improper design, detailing and installation of anchor rods have caused numerous structural problems in industrial buildings. These problems include:

1. Inadequate sizing of the anchor rods,
2. Inadequate development of the anchor rods for tension,
3. Inadequate design or detailing of the foundation for forces from the anchor rods,
4. Inadequate base plate thickness,
5. Inadequate design and/or detailing of the anchor rod - base plate interface,
6. Misalignment or misplacement of the anchor rods during installation, and
7. Fatigue.

The reader should be familiar with the OSHA requirements contained in *Safety and Health Standards for the Construction Industry*, 29 CFR 1926 *Part R Safety Standards for Steel Erection*, (OSHA, 2001). This document was partially produced to prevent construction accidents associated with column base plates. For example, OSHA requires that all column based have four anchor rods.

The following discussion presents methods of designing and detailing column bases.

9.1 Resisting Tension Forces with Anchor Rods

The design of anchor rods for tension consists of four steps:

1. Determine the maximum net uplift for the column.
2. Select the anchor rod material and number and size of anchor rods to accommodate this uplift
3. Determine the appropriate base plate size, thickness and welding to transfer the uplift forces. Refer to AISC Design Guide 1 (AISC, 1990).
4. Determine the method for developing the anchor rod in the concrete (i.e. transferring the tension force from the anchor rod to the concrete foundation).

Step 1

The maximum net uplift for the column is obtained from the structural analysis of the building for the prescribed building loads. The use of light metal roofs on industrial buildings is very popular. As a result of this, the uplift due to wind often exceeds the dead load; thus the supporting columns are subjected to net uplift forces. In addition, columns in rigid bents or braced bays may be subjected to net uplift forces due to overturning.

Step 2

Anchor rods should be specified to conform to ASTM F1554. Grades 36, 55 and 105 are available in this specification where the grade number represents the yield stress of the anchor. Unless otherwise specified, the end of anchor will be color coded to identify its grade. Welding is permitted to the Grade 36 and also to the Grade 55 if it conforms to the S1 supplement.

Anchor rods should no longer be specified to A307 even if the intent is to use the A307 Grade C anchor that conforms to A36 properties. Anchor rods conforming to the ASTM specifications listing of Anchor Rods and Threaded Bolts in the 1999 AISC *LRFD Specification* can be used as well as 304 and 316 stainless steels.

The number of anchor rods required is a function of the maximum net uplift on the column and the allowable tensile load per rod for the anchor rod material chosen. Prying forces in anchor rods are typically neglected. This is usually justified when the base plate thickness is calculated assuming cantilever bending about the web and/or flange of the column section (as described in Step 3 below). However, calculations have shown that prying forces may not be negligible when the rods are positioned outside the column profile and the rod forces are large. A conservative estimate for these prying forces can be obtained using a method similar to that described for hanger connections in the AISC *Manual of Steel Construction*.

Another consideration in selection and sizing of anchor rods is fatigue. For most building applications, where uplift loads are generated from wind and seismic forces, fatigue can be neglected because the maximum design wind and seismic loads occur infrequently. However, for anchor rods used to anchor machinery or equipment where the full design loads may occur more often, fatigue should be considered. In addition, in buildings where crane load cycles are significant, fatigue should also be considered. AISE Technical Report No. 13 for the design of steel mill buildings recommends that 50 percent of the maximum crane lateral loads or side thrust be used for fatigue considerations.

In the past, attempts have been made to pretension or preload anchor rods in the concrete to prevent fluctuation of the tensile stress in anchor rods and, therefore, eliminate fatigue concerns. This is not recommended, unless the anchor rods are re-tensioned to accommodate creep in the supporting concrete foundation. If setting nuts are employed below the base plate, pretensioning can be employed to provide a tight connection between the base plate and the anchors.

Table 9.1.1 shows recommended allowable fatigue stresses for non-pretensioned steel bolts. These values are based on S-N (stress verse number of cycles) data for a variety of different types of bolts. (These data were obtained from correspondence with Professor W. H. Munse of the University of Illinois and are based on results from a number of test studies.) By examining these values, it can be ascertained that, for the AISE loading condition fatigue will not govern when ASTM 1554 Grade 36 anchor rods are used. However, fatigue can govern the design of higher strength anchor rods for this load case.

Step 3

Base plate thickness may be governed by bending associated with compressive loads or tensile loads. For compressive loads, the design procedure illustrated in the "Column Base Plates" section of Part 3 of the AISC 9th Edition *Manual of Steel Construction*, and Part 14 of the Third Edition of the *LRFD Manual of Steel Construction*, may be followed. However, for lightly loaded base plates where the dimensions "m" and "n" (as defined in this procedure) are small, thinner base plate thickness can be obtained using yield line theory.

For tensile loads, a simple approach is to assume the anchor rod loads generate bending moments in the base plate consistent with cantilever action about the web or flanges of the column section (one-way bending). If the web is taking the anchor load from the base plate, the web and its attachment to the base plate should be checked. A more refined analysis for anchor rods positioned inside the column flanges would consider bending about both the web and the column flanges (two-way bending). For the two-way bending approach, the derived bending moments should be consistent with compatibility requirements for deformations in the base plate. In either case, the effective bending width for the base plate can be conservatively approximated using a 45° distribution from the centerline of the anchor rod to the face of the column flange or web. Calculations for required base plate thickness for uplift (tensile) loads are illustrated in Examples 9.4.1 and 9.4.2.

Step 4

Appendix D of ACI 318-02 (ACI 2002) and Appendix B of ACI 349-01 (ACI 2001) both address the anchoring to concrete of cast-in or post-installed expansion or undercut anchors. These appendices do not cover adhesive anchors and grouted anchors. The provisions in both appendices are based on the Concrete Capacity Design (CCD) Method. The current ACI 349-01 Appendix B provisions represent a

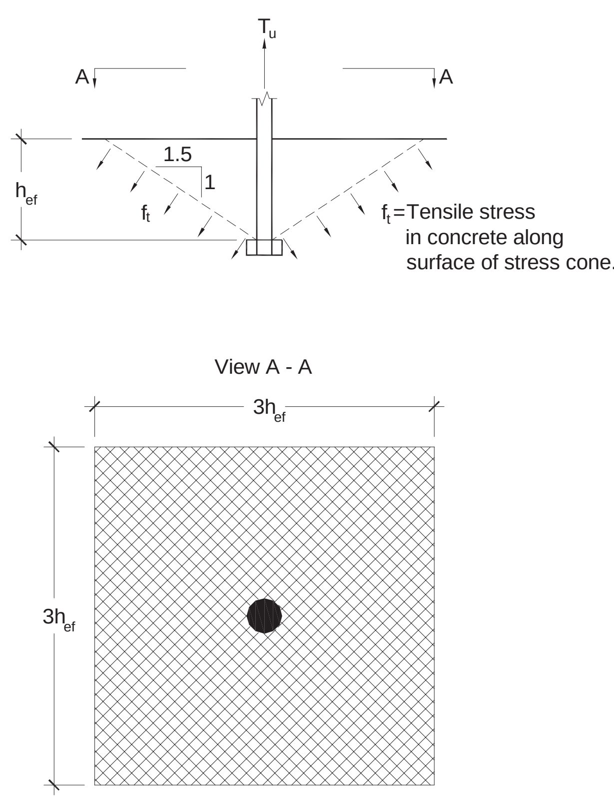


Fig. 9.1.1 Full Breakout Cone in Tension per ACI 318-02

significant change to the previous (ACI 349-97) criteria for anchoring.

In the CCD method the concrete cone is considered to be formed at an angle of approximately 34 degrees (1 to 1.5 slope) rather than the previously assumed 45. For simplification of application, the cone is considered to be square rather than round in plan. See Figure 9.1.1.

The concrete breakout stress (f_t in Figure 9.1.1) in the CCD method is considered to decrease with increase in size of the breakout surface. Consequently, the increase in strength of the breakout in the CCD method is proportional to the embedment depth to the power of 1.5 (or to the power of $\frac{5}{3}$ for deeper embedments). With a constant breakout stress on the failure surface, as was considered in ACI 349-97, the breakout strength is proportional to the square of the embedment depth.

Appendix D of ACI 318-02 permits non-ductile design except for anchor rods used in regions of moderate or high seismic risk. In Appendix B of ACI 349-01 three alternative embedment design methodologies are provided:

1. The design concrete breakout tensile strength, side blowout strength, or pullout strength, of the anchor and 65 percent of the concrete breakout shear strength must exceed the ultimate strength of the embedment steel.

2. The design strength of the concrete must exceed the yield strength of the anchor by 33 percent.

3. Non-ductile anchor design is permitted provided that the design strength of the concrete is limited to 60 percent of the design strength.

AISC in Section J10. (AISC, 1999) defers anchor design to ACI 318. Section 15.8.3.3 of ACI 318-02 requires that anchor rods and mechanical connections reach their design strength before anchorage failure or failure of the surrounding concrete. It is suggested in this design guide that the design generally follow the second and third approaches given above. For strength design, it is presumed that ASCE-7 load factors are employed. Thus, the ϕ factors used in this document will differ from those used in Appendix D of ACI 349-01. ACI 349-01 uses load factors of 1.4D and 1.7L, and f factors that conform in general to those in Appendix C of ACI 318-02. The ϕ factors used herein correspond to those in D4.4 of Appendix D and 9.3 of ACI 318-02.

If an anchor is designed to lap with reinforcement, the anchor capacity can be taken as $\phi A_{se}F_y$ as the lap splice length will ensure that ductile behavior will occur. A_{se} is the effective cross-sectional area that is the tensile stress area for threaded rods. ϕ equals 0.9 as prescribed in Chapter 9 of ACI 318-02. If the anchor is resisted solely by concrete, one needs to have the concrete designed with additional capacity in order to insure ductility in the connection. ACI 318 in Section 15.8.3.3 does not define what is meant by achieving anchor rod (and mechanical connection) design strength before anchorage or concrete failure. In order to achieve this, it is proposed to have the concrete reach a capacity of 1.25 ($\phi A_{se}F_y$). This is based on the requirement in ACI 318 Section 12.14.3.2 that a full mechanical splice shall develop 1.25 F_y. Alternately, the author suggests limiting the non-ductile anchorage capacity to 70 percent of the typical design strength, which is somewhat less restrictive than the 60 percent reduction used in Appendix B of ACI 349-01.

Hooked anchor rods usually fail by straightening and pulling out of the concrete. This failure is precipitated by a localized bearing failure in the concrete above the hook. Calculation of the development load provided by a hook is illustrated in Example 9.4.1. As indicated in Example 9.4.1, a hook is generally not capable of developing the recommended tensile capacity mentioned in the previous paragraph. Therefore, hooks should only be used when tension in the anchor rod is small.

Appendix D of ACI 318-02 has a pullout capacity for a hooked anchor of $\phi\psi_4(0.9\ f'_c e_h d_o)$ which is based on an anchor with diameter d_o bearing against the hook extension of e_h. ϕ is taken as 0.70. The hook extension is limited to a maximum of 4.5d_o. ψ_4 equals 1.0 if the anchor is located

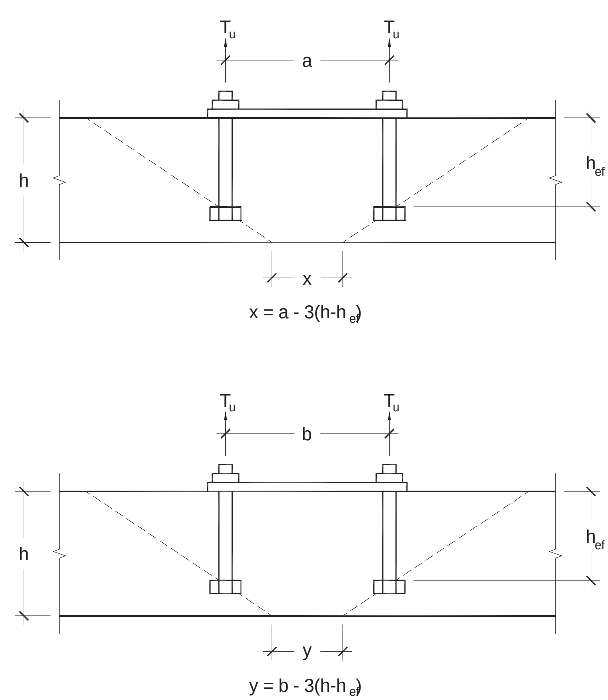


Fig. 9.1.2 Breakout Cone for Group Anchors in Thin Slab

where the concrete is cracked at service load, or ψ_4 equals 1.4 if it is not cracked.

Tests have shown that a heavy bolt head, or a heavy hex nut on a threaded rod, will develop the full tensile capacity of normal strength anchor rods when properly embedded and confined in concrete. With high strength anchor rods, washer plates may be necessary to obtain the full capacity of the anchors. Therefore, the design for development for headed anchor rods (typically threaded rods with heavy hex nuts) is a matter of determining the required embedment depths, edge distances and/or steel reinforcement to prevent concrete breakout failure prior to the development of the recommended tensile capacity for the rod.

As presented in Appendix B of ACI 349-01, failure occurs in the concrete when tensile stresses along the surface of a stress cone surrounding the anchor rod exceed the tensile strength of the concrete. The extent of this stress cone is a function of the embedment depth, the thickness of the concrete, the spacing between adjacent anchors and the location of adjacent free edges in the concrete. The shapes of these stress cones for a variety of situations are illustrated in Figures 9.1.1, 9.1.2 and 9.1.3.

The stress cone checks rely upon the strength of plain concrete for developing the anchor rods and typically apply when columns are supported directly on spread footings, concrete mats or pile caps. However, in some instances the projected area of the stress cones or overlapping stress cones is extremely limited due to edge constraints. Consequently the tensile strength of the anchor rods cannot be fully developed with plain concrete. This is often the case with concrete piers. In these instances, steel reinforcement in the concrete is used to carry the force from the anchor rods. This reinforcement often doubles as the reinforcement required to accommodate the tension and/or bending forces in the pier. The reinforcement must be sized and developed for the required tensile capacity of the anchor rods on both sides of the potential failure plane described in Figure 9.1.4.

The anchor rod embedment lengths are determined from the required development lengths for this reinforcing steel. Hooks or bends can be added to this reinforcement to minimize development length in the breakout cone.

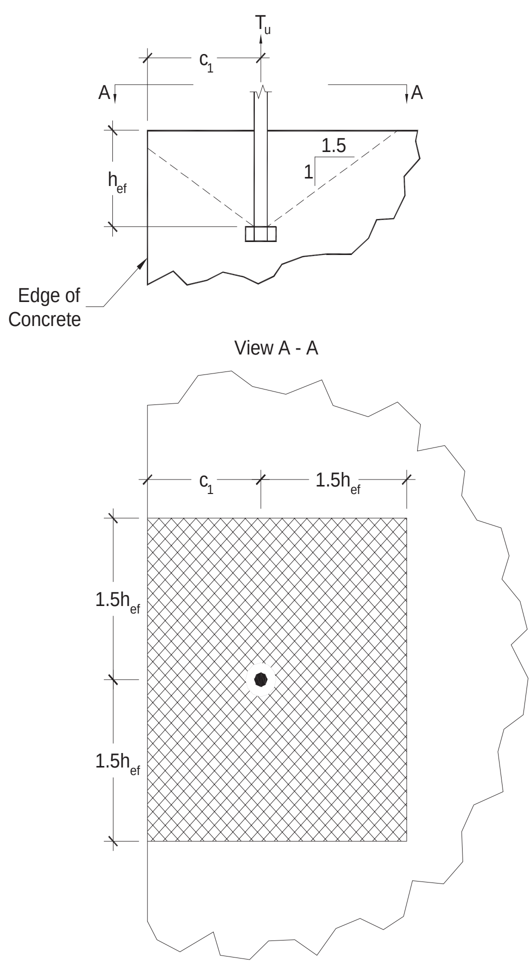


Fig. 9.1.3 Breakout Cone in Tension Near an Edge

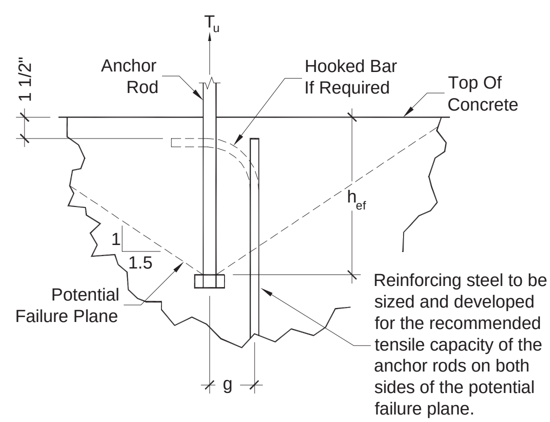


Fig. 9.1.4 The Use of Steel Reinforcement for Developing Anchor Rods

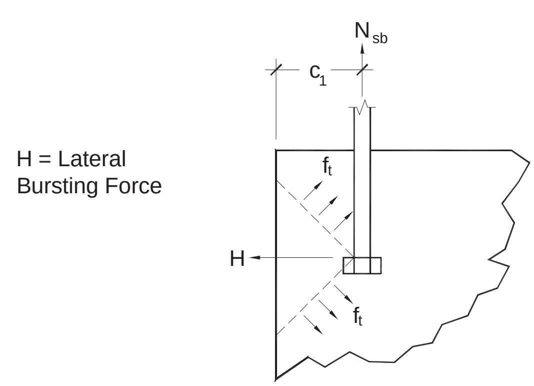


Fig. 9.1.5 Lateral Bursting Forces for Anchor Rods in Tension Near an Edge

Appendix D of ACI 318-02 also lists criteria for anchor rods to prevent "failure due to lateral bursting forces at the anchor head." These lateral bursting forces are associated with tension in the anchor rods. The failure plane or surface in this case is assumed to be cone shaped and radiating from the anchor head to the adjacent free edge or side of the concrete structure. This is illustrated in Figure 9.1.5. It is recommended to use a minimum side cover c_1 of 6 anchor diameters for anchor rods conforming to ASTM F1554 Grade 36 to avoid problems with side face breakout. As with the pullout stress cones, overlapping of the stress cones associated with these lateral bursting forces is considered in Appendix D. Use of washer plates can be beneficial by increasing the bearing area that increases the side-face blowout strength.

For the common case of four anchor rods in tension in a footing, a mat, or a wide pier, where a full breakout cone can be achieved, Figure 9.1.6 provides a means of determining the anchor size, and then determining the needed anchor depth following the proposed limit states described earlier. The concrete breakout capacities assume the concrete to be uncracked. The designer should refer to ACI 318-02 to determine if the concrete should be taken as cracked or uncracked. If the concrete is considered cracked,

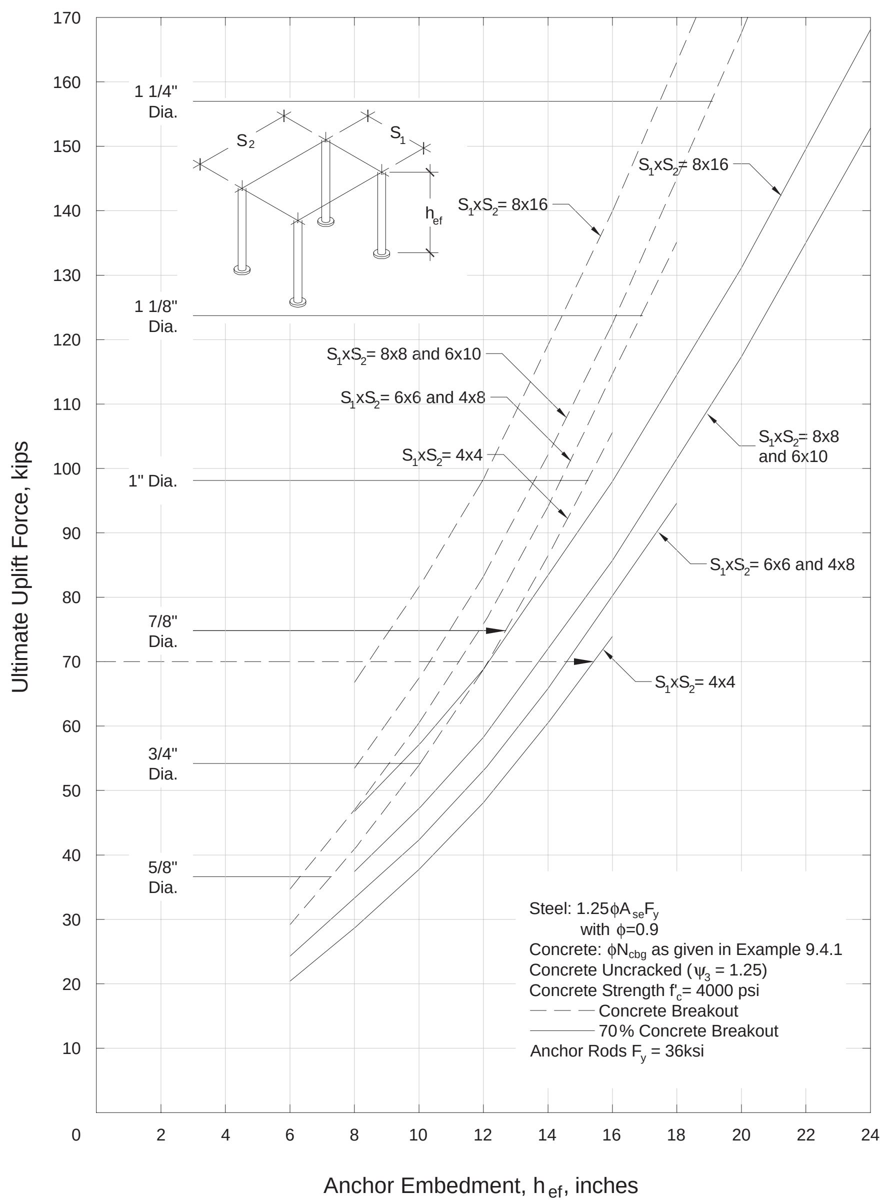


Fig. 9.1.6 Capacity of 4-Anchor Rods Without Edge Distance Reduction

such that ψ_3 equals 1.0, then eighty percent of the concrete capacity values should be used. Application of this Figure is illustrated in Example 9.4.1.

9.2 Resisting Shear Forces Using Anchor Rods

Appendix B of ACI 349-85 (ACI, 1985) and ACI 349-97 (ACI, 1997) used 'shear-friction' for transferring shear from the anchor rods to the concrete. This procedure was used in the previous version of this design guide. Appendix B of ACI 349-01 and Appendix D of ACI 318-02 both employ the CCD method to evaluate the concrete breakout capacity from shear forces resisted by anchor rods. For the typical cast-in-place anchor group used in building construction the shear capacity determined by concrete breakout as illustrated in Figure 9.2.1 is evaluated as

$$\phi V_{cbg} = \phi \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b$$

where $V_b = 7\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_c'}\, c_1^{1.5}$

c_1 = the edge distance in the direction of load as illustrated in Figure 9.2.1.
ℓ = the embedment depth.
d_o = the bar diameter.

Typically ℓ/d_o becomes 8 since the load bearing length is limited to $8d_o$.

ϕ = 0.70
ψ_5 = 1.0 (all anchors at same load).

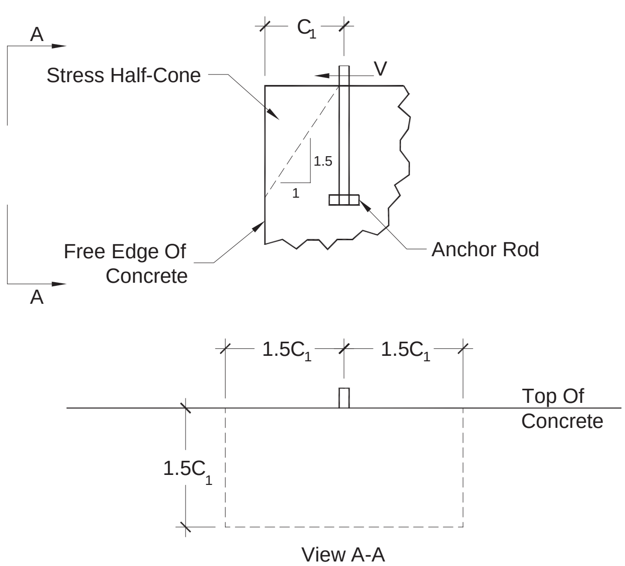


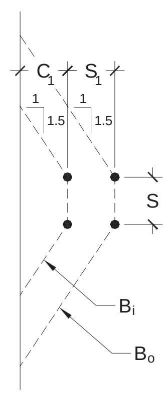
Fig 9.2.1 Concrete Breakout Cone for Shear

ψ_7 = 1.4 (uncracked or with adequate supplementary reinforcement).

$$\phi V_{cbg} = 10.4 \frac{A_v}{A_{vo}} \psi_6 \sqrt{d_o}\sqrt{f_c'}\, c_1^{1.5}$$

A_{vo} = $4.5c_1^2$ (the area of the full shear cone for a single anchor as shown in View A-A of Figure 9.2.1).
A_v = the total breakout shear area for a single anchor or a group of anchors.
ψ_6 = a modifier to reflecting the capacity reduction when side cover limits the size of the breakout cone.

It is recommended that the bar diameter, d_o, used in the square root term of the V_b expression, be limited to a maximum of 1.25 in. based on recent research results. If the edge distance c_1 is large enough, then the anchor rod shear capacity will govern. This capacity is given as $\phi n 0.6 A_{se} f_{ut}$ = $0.39 n A_{se} f_{ut}$ with ϕ = 0.65 where f_{ut} is the specified tensile strength of the anchor steel, and n is the number of anchors. Where anchors are used with a built-up grout pad, the anchor capacity should be multiplied by 0.8 which results in an anchor shear capacity of $0.31 n A_{se} f_{ut}$. Appendix B of ACI 349-01 does permit the sharing of the anchor shear integrity with the friction developed from factored axial and flexural



S/S_1	C_1/S_1 For B_o To Control
0.5	> 1.59
2/3	> 1.56
1.0	> 1.50
1.5	> 1.40
2.0	> 1.30

Fig. 9.2.2 Concrete Breakout Surfaces for Group Anchors

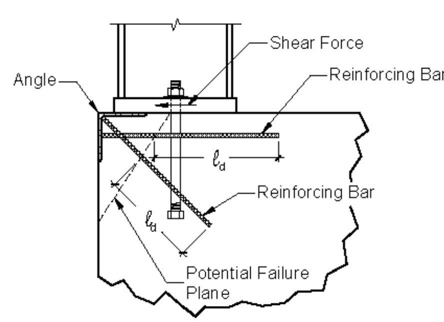


Fig. 9.2.3 Concrete Reinforcement to Improve Shear Capacity Where Edge Distance is Limited

load. A coefficient of friction of 0.4 is used. ACI 318-02 Appendix D does not recognize the benefit of the friction.

In evaluating the concrete breakout, one should check the breakout either from the most deeply embedded anchors or breakout on the anchors closer to the edge. When breakout is being determined on the inner two anchors, the outer two anchors should be considered to carry the same load. When the concrete breakout is considered from the outer two anchors, all of shear is to be taken by the outer anchors. Shown in Figure 9.2.2 are the two potential breakout surfaces and an indication of which will control, based on anchor location relative to the edge distance.

To ensure that shear yield of the anchor will control, design the concrete breakout shear capacity to meet or exceed the minimum of $1.25\phi V_y$ using $\phi = 0.9$ to obtain $1.25(0.9)(0.6A_{se}\,F_y) = 0.675\ A_{se}\,F_y$. An appreciable edge distance is required to achieve a ductile shear failure. For example, with 4 anchor rods, with F_y = 36 ksi, with a 4 in. by 4 in. pattern and a 4 in. edge distance (c_1 in Figure 9.2.2), full anchor shear capacity can be reached for ½ in. diameter anchors provided that no benefit exists from the frictional shear resistance. For full shear capacity of ⅝ in. diameter (F_y = 36 ksi) anchors a 5 in. edge distance is required while a 7 in. edge distance is required for ¾ in. diameter anchors with no frictional benefit.

In many cases it is necessary to use reinforcement to anchor the breakout cone in order to achieve the shear capacity as well as the ductility desired. An example of this is illustrated in Figure 9.2.3. The ties placed atop piers as required in Section 7.10.5.6 of ACI 318-02 and illustrated in Example 9.4.2 can also be used structurally to transfer the shear from the anchors to the piers. If the shear is small, the best approach is to simply design for the non-ductile concrete breakout using the 70 percent factor noted earlier.

Careful consideration should be given to the size of the anchor rod holes in the base plate, when transferring shear forces from the column base plate to the anchor rods. The designer should use the recommended anchor rod hole diameters and minimum washer diameters, which can be found on page 14-27 of the AISC 3rd edition *LRFD Manual of Steel Construction* (AISC, 2001). These recommended hole sizes vary with rod diameter, and are considerably larger than normal bolt hole sizes. If slip of the column base, before bearing, against the anchor rods is of concern, then the designer should consider using plate washers between the base plate and the anchor rod nut. Plate washers, with holes $^1/_{16}$ in. larger than the anchor rods, can be welded to the base plate so that minimal slip would occur. Alternatively, a setting plate could be used, and the base plate of the column welded to the setting plate. The setting plate thickness must be determined for proper bearing against the anchor rods.

9.3 Resisting Shear Forces Through Bearing and with Reinforcing Bars

Shear forces can be transferred in bearing by the use of shear lugs or by embedding the column in the foundation. These methods are illustrated in Figure 9.3.1.

Appendix B of ACI 349-01 does permit the use of confinement and of shear friction in combination with bearing for transferring shear from anchor rods into the concrete. The commentary to ACI 349-02 suggests that this mechanism is developed as follows:

1. Shear is initially transferred through the anchor rods to the grout or concrete by bearing augmented by shear resistance from confinement effects associated with tension anchors and external concurrent axial load.

2. Shear then progresses into a shear-friction mode.

The recommended bearing limit ϕP_{urbg} per Section B.4.5.2 of ACI 349-01 Appendix B is $\phi 1.3 f'_c A_\ell$. Using a ϕ consistent with ASCE-7 load factors use $\phi P_{urbg} = 0.80 f'_c A_\ell$ for shear lugs.

A_ℓ = embedded area of the shear lug (this does not include the portion of the lug in contact with the grout above the pier).

For bearing against an embedded base plate or column section where the bearing area is adjacent to the concrete surface it is recommended that $\phi P_{ubrg} = 0.55 f'_c A_{brg}$ consistent with ACI 318-02.

According to the Commentary of Appendix B of ACI 349-01, the anchorage shear strength due to confinement can be taken as $\phi K_c(N_y - P_a)$, with ϕ equal to 0.75, where N_y

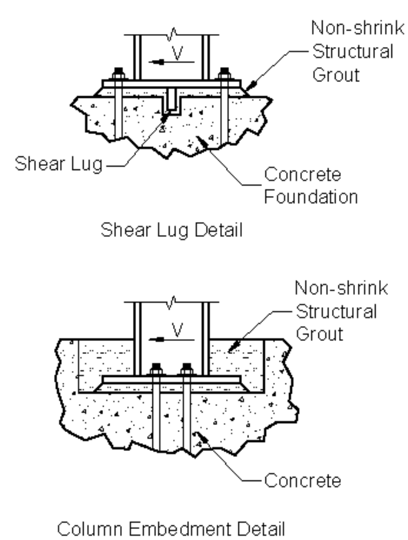


Fig. 9.3.1 Transfer of Base Shears Through Bearing

is the yield strength of the tension anchors equal to $nA_{se}F_y$, and P_a is the factored external axial load on the anchorage. (P_a is positive for tension and negative for compression). This shear strength due to confinement considers the effect of the tension anchors and external loads acting across the initial shear fracture planes. When P_a is negative, one must be assured that the P_a will actually be present while the shear force is occurring. Based on ACI 349-01 Commentary use Kc = 1.6.

In summary the lateral resistance can be expressed as:

$\phi P_n = 0.80\, f'_c A_\ell + 1.2(N_y - P_a)$ for shear lugs and

$\phi P_n = 0.55 f'_c A_{brg} + 1.2(N_y - P_a)$ for bearing on a column or the side of a base plate.

If the designer wishes to use shear-friction capacity as well, the provisions of ACI 349-01 can be followed.

Additional comments related to the use of shear lugs are provided below:

1. For shear lugs or column embedments bearing in the direction of a free edge of the concrete, Appendix B of ACI 349-01 states that in addition to considering bearing failure in the concrete, "the concrete design shear strength for the lug shall be determined based on a uniform tensile stress of $4\phi\sqrt{f_c'}$ acting on an effective stress area defined by projecting a 45° plane from the bearing edge of the shear lug to the free surface." The bearing area of the shear lug (or column embedment) is to be excluded from the projected area. Use a ϕ equal to 0.75. This criterion may control or limit the shear capacity of the shear lug or column embedment details in concrete piers.

2. Consideration should be given to bending in the base plate resulting from forces in the shear lug. This can be of special concern when the base shears (most likely due to bracing forces) are large and bending from the force on the shear lug is about the weak axis of the column. As a rule of thumb, the author generally requires the base plate to be of equal or greater thickness than the shear lug.

3. Multiple shear lugs may be used to resist large shear forces. Appendix B of ACI 349-01 provides criteria for the design and spacing of multiple shear lugs.

A typical design for a shear lug is illustrated in Example 9.4.3. The designer may want to consider resisting shear forces with the shear lugs welded to a setting template. The setting templates are cast with the anchor rods. The columns are then set with conventional shim stacks. To complete the shear transfer, shear transfer bars are welded to the base plate and to the setting template. The setting template has grout holes and thus allows good consolidation of the concrete around the shear lugs.

To complete the discussion on anchorage design, transfer of shear forces to reinforcement using hairpins or tie rods will be addressed. Hairpins are typically used to transfer load to the floor slab. The friction between the floor slab and the subgrade is used in resisting the column base shear when individual footings are not capable of resisting horizontal forces. The column base shears are transferred from the anchor rods to the hairpin (as shown in Figure 9.3.2)

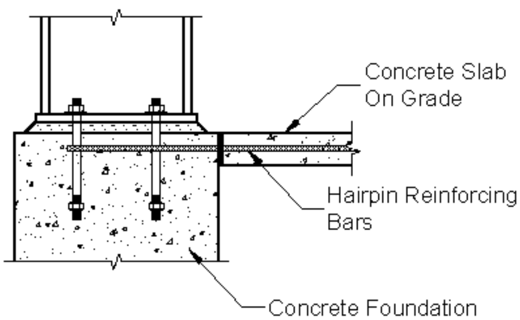


Fig. 9.3.2 Typical Detail Using Hairpin Bars.

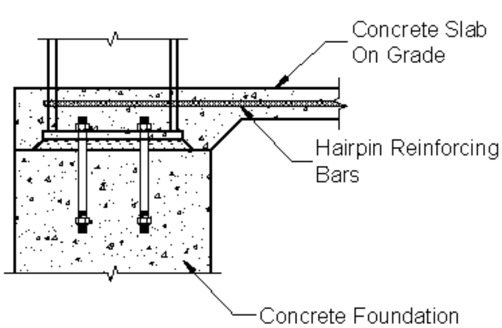


Fig. 9.3.3 Alternate Hairpin Detail

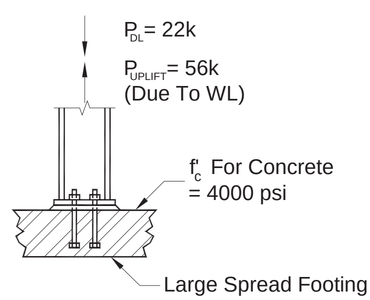


Fig. 9.4.1 Example 9.4.1

through bearing. Problems have occurred with the eccentricity between the base plate and the hairpin due to bending in the anchor rods after the friction capacity is exceeded. This problem can be avoided as shown in Figure 9.3.3 or by providing shear lugs. Because hairpins rely upon the frictional restraint provided by the floor slab, special consideration should be given to the location and type of control and construction joints used in the floor slab to assure no interruption in load transfer, yet still allowing the slab to move.

Tie rods are typically used to counteract large shear forces associated with gravity loads on rigid frame structures. When using tie rods with large clear span rigid frames, consideration should be given to elongation of the tie rods and to the impact of these elongations on the frame analysis and design. In addition significant amounts of sagging or bowing should be removed before tie rods are encased or covered, since the tie rod will tend to straighten when tensioned.

9.4 Column Anchorage Examples (Pinned Base)

EXAMPLE 9.4.1:

Column Anchorage for Tensile Loads (LRFD)

Design a base plate and anchorage for a W10×45 column subjected to a net uplift as a result of the loadings shown in Figure 9.4.1:

Procedure:

1. Determine the design uplift on the column.

2. Select the type and number of anchor rods.

3. Determine the appropriate base plate thickness and welding to transfer the uplift forces from the column to the anchor rods.

4. Determine the method for developing the anchor rods in the concrete in the spread footing.

5. Re-evaluate the anchorage if the column is on a 20 in. by 20 in. pier.

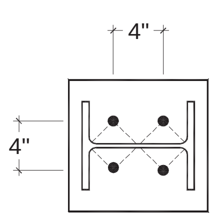


Fig. 9.4.2 Rod Load Distribution

Solution:

1. Factored uplift = 1.6(56)-0.9(22) = 69.8 kips

2. Use four anchor rods (minimum per OSHA requirements).

   T/Rod = 69.8/4 = 17.45 kips.

   Using an ASTM F1554 Grade 36 material, select a ⅞ in. diameter rod.

   The design strength is the lower value of:

   $\phi F_y A_q$ = (0.9)(36)(0.60) = 19.44 kips

   or $\phi F_u A_e$ = (0.75)(58)(0.462) = 20.10 kips o.k.

   Note: The anchor rods are positioned inside the column profile and rod forces are not extremely large; therefore, prying forces are negligible.

3. The rods are positioned inside the column profile with a 4 in. square pattern. To simplify the analysis, conservatively assume the tensile loads in the anchor rods generate one-way bending in the base plate about the web of the column. This assumption is illustrated by the rod load distributions shown in Figure 9.4.2.

   M_y in the base plate equals the rod force times the lever arm to the column web face.

$$M_y = 17.45\left(2 - \frac{0.350}{2}\right) = 31.85 \text{ in.-kips}$$

   The effective width of base plate for resisting M_y at the face of web = b_{eff}.

   Using a 45° distribution for the rod loads,

$$b_{eff.} = \left(2 - \frac{0.350}{2}\right)(2) = 3.65 \text{ in.}$$

$$Z_y = \frac{b_{eff}\, t^2}{4}$$

$$F_y = 36 \text{ ksi}$$

$$t_{req'd.} = \sqrt{\frac{M_y(4)}{b_{eff}\,\phi(F_y)}}$$

$$t_{req'd.} = \sqrt{\frac{31.9(4)}{3.65(0.9)(36)}} = 1.04 \text{ in.}$$

Use a 1⅛ in. thick plate (Fy = 36 ksi).

For welding of the column to the base plate:

$$\text{Maximum weld load} = \frac{T/Bolt}{b_{eff}} = \frac{17.5}{3.65} = 4.78\text{k/in.}$$

Minimum weld for a 1⅛ in. thick base plate = 5⁄16 in. (Table J2.4 of *LRFD Specification*).

Design weld load per in. for a 5⁄16 in. fillet weld with E70 electrode:

= (5/16)(0.707)(0.75)(70) = 11.6 k/in.

4.78 < 11.60

Check web:

$$\begin{aligned}\phi P_n &= \phi b_{eff}(2)F_y t_w\\ &= (0.9)(3.65)(2)(36)(0.35)\\ &= 82.8 \text{ kips}\end{aligned}$$

82.8 > (4)(17.5) o.k.

4. As noted earlier, this column is anchored in the middle of a large spread footing. Consequently, there are no edge constraints on the concrete tensile cones and there is no concern regarding edge distance to prevent lateral breakout of the concrete.

To ensure a ductile failure in the case of overload, design the embedment of the anchor rods to yield some prior to concrete breakout. For ⅞ in. diameter F1554 Grade 36 rods, this is equal to (1.25)(0.9)(0.462)(36) = 18.7 kips/rod.

Try using a 3.5 in. hook on the embedded end of the anchor rod to develop the rod.

Based on uniform bearing on the hook, the hook bearing capacity per ACI 318-02 Appendix D

$$= \phi\,(0.9)(f'_c)(d_o)(e_h)(\psi_4)$$

where

ϕ = 0.70
f'_c = concrete compressive strength
d_o = hook diameter
e_h = hook projection
ψ_4 = cracking factor (1.0 for cracked, 1.4 for uncracked concrete)

$$\begin{aligned}\text{Hook bearing capacity} &= 0.70(0.9)(4000)(7/8)(3.5-0.875)(1.4)\\ &= 8100 \text{ lb}\\ &= 8.10 \text{ kips} < 18.7 \text{ kips N.G.}\end{aligned}$$

Thus a 3.5 in. hook is not capable of developing the required tensile force in the rod. Therefore, use a heavy hex nut to develop the anchor rod.

To achieve a concrete breakout strength, ϕN_{cbg}, that exceeds the desired 4(18.7) = 74.8 kip steel capacity, the embedment depth must be at least 13 in. determined by trial and error or from Figure 9.1.6.

Per ACI 318-02 Appendix D, the concrete breakout strength

$$\phi N_{cbg} = \phi\psi_3 24\sqrt{f'_c}\, h_{ef}^{1.5}\frac{A_N}{A_{No}} \text{ for } h_{ef} \le 11 \text{ in.}$$

and

$$\phi N_{cbg} = \phi\psi_3 16\sqrt{f'_c}\, h_{ef}^{5/3}\frac{A_N}{A_{No}} \text{ for } h_{ef} > 11 \text{ in.}$$

where

ϕ = 0.70
ψ_3 = 1.25 considering the concrete to be uncracked
h_{ef} = 13 in.
A_N = concrete breakout cone area for group
= (3(13) + 4)(3(13) + 4) = 1849
A_{No} = concrete breakout cone area for single anchor
= $9(13)^2$ = 1521

$$\phi N_{cbg} = 0.70(1.25)(16)\sqrt{0.004}(13)^{5/3}\left(\frac{1849}{1521}\right)$$

$$= 77.4 \text{ kips}$$

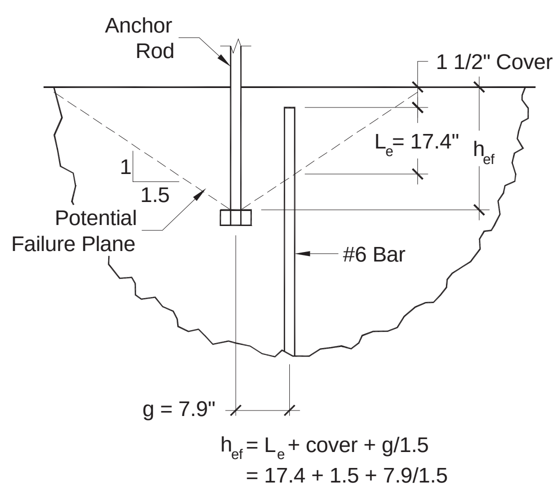


Fig. 9.4.3 Embedment Depth for Transfer to Reinforcing Bars

From Figure 9.1.6 for a 4 in. by 4 in. spacing of anchors, with the ultimate tension of 69.8 kips, an anchor embedment of 15.5 in. would be required to achieve the 70 percent breakout capacity in which case a ductile anchor failure would not be required. This embedment would be satisfactory if the anchors were 1 in. diameter F1554 Grade 36 or larger. With the ⅞ in. diameter anchors a 13 in. embedment is adequate to achieve the anchor capacity considering the full breakout capacity shown as dashed lines in Figure 9.1.6.

5. If the anchors were installed in a 20 in. square pier the concrete breakout strength would be limited by the pier cross section. With an 8 in. maximum edge distance the effective h_{ef} need be only 8/1.5 = 5.33 in. to have the breakout cone area equal this pier cross sectional area. This leads to a

$$\phi N_{cb} = 0.75(1.25)(24)\sqrt{0.004}(5.33)^{1.5}\left(\frac{20^2}{9(5.33)^2}\right)$$

$$= 27.4 \text{ kips}$$

Therefore the uplift strength is 0.7(27.4) = 19.2 kips based on the concrete only. Thus, it is necessary to transfer the anchor load to the vertical reinforcing steel in the pier. The required $A_s = \dfrac{69.8 \text{ kips}}{0.9(60)} = 1.29 \text{ in.}^2$

The minimum 4-#7 bars required per ACI 318-02 in the pier are adequate to take this tension. With the bars located in the corners of the piers use a lateral offset distance g = [(20 in.– 4 in.)/2 – 2.4 in.]$\sqrt{2}$ = 7.9 in. Using a Class B splice factor with a 1.3 value and with a development length of the #7 bar equal to 24.9 in., compute l_e from the ratio

$$\frac{\ell_e}{69.8} = \frac{1.3\ell_d}{nA_s\phi F_y} = \frac{1.3(24.9)}{4(0.6)(0.9)(60)}$$

ℓ_e = 17.4 in. Therefore minimum required h_{ef} = 17.4 + 1.5 + 7.9/1.5 = 24.2 in. as illustrated on Figure 9.4.3. Select 25 in. embedment for anchors.

EXAMPLE 9.4.2:

Column Anchorage for Combined Tension and Shear Loads (Pinned Base) (LRFD)

Design a base plate and anchorage for the W10×45 column examined in Example 9.4.1, but with an additional nominal base shear of 23 kips due to wind. Assume a 2 in. thick grout bed is used beneath the base plate. For this example, the column is assumed to be supported on a 20 in. square pier. See Figure 9.4.4.

Procedure:

1. Determine the maximum net tension in the anchor rods. Decide whether the tension can be transferred to the concrete or whether the anchors must be lapped with the vertical pier reinforcement.

2. Select the type and number of anchor rods.

3. Determine the appropriate base plate thickness and welding to transfer the uplift and shear forces from the column to the anchor rods.

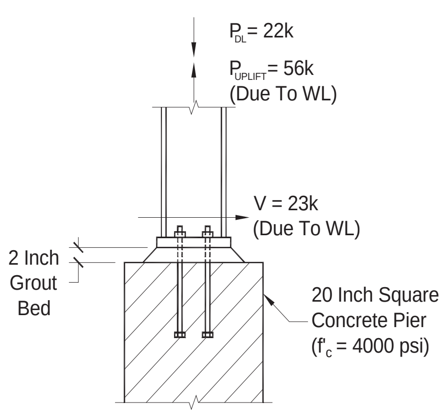


Fig. 9.4.4 Column pier support

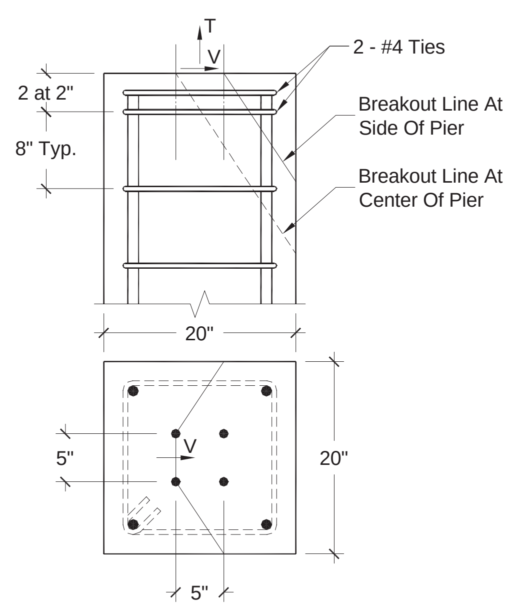


Fig. 9.4.5 Pier for Ex. 9.4.2 Showing Shear Breakout Cone

4. Determine whether the shear can be transferred directly to the concrete, or whether the shear must be transferred to ties.

Solution:

1. As determined in Example 9.4.1, the net uplift on the column = 69.8 kips, and as determined from the last part of Example 9.4.1, it is necessary to transfer the tensile loading to the pier vertical reinforcement. The vertical reinforcement in the pier will be larger in this case due to the moment introduced into the pier from the applied shear.

2. A total of four anchor rods are to be used. The same four ⅞ in. diameter rods used in Example 9.4.1 could be used here as well, provided the

$$f_t \le \phi F_t = \phi\sqrt{45^2 - 6.25\, f_v^2}.$$

However, 1⅛ in. diameter F1554 Grade 36 anchors are required in this case.

$$f_v = \frac{(1.6)(23)}{4(0.994)} = 9.26 \text{ ksi}; \qquad f_t = \frac{69.8}{4(0.994)} = 17.56 \text{ ksi}$$

$$\phi F_t = 0.75\sqrt{45^2 - 6.25\,(9.26)^2}$$

$$= 28.94 \text{ ksi} \quad \text{o.k.}$$

3. Position the rods within the profile of the column with a 5 in. square pattern. Conservatively assume the tensile loads in the anchor rods generate one-way bending in the base plate about the web of the column or assume that two way bending occurs by considering bending of the base plate between flanges.

4. The shear breakout cone as viewed from the top of the pier is shown in the Figure 9.4.5.The shear breakout force is based on all shear on the back anchors.

$$\phi V_b = 10.4\sqrt{1.125}\sqrt{0.004}\ \ c_1^{1.5} = 29.0 \text{ kips}$$
with $c_1 - 12$ in.

$$\psi_6 = \left(0.7 + 0.3\frac{7.5}{1.5(12.5)}\right) = 0.82$$

$$\therefore \phi V_{cbg} = \phi V_b \frac{A_v}{A_{vo}} \psi_6 = 29.0 \frac{20 \times 1.5 \times 12}{4.5(12)^2} 0.82 = 13.21 \text{ kips}$$

$$< 1.6(23)$$

The maximum shear of concrete pier without stirrups per ACI 318-02 is

$$\phi V_c / 2 = \phi \frac{2\sqrt{f_c'} b_w d}{2}$$

$$\frac{0.85\left(2\sqrt{0.004}\right)}{2}(20 \times 17.5) = 18.82 \text{ kips} \quad < 1.6(23)$$

From this calculation it is obvious that the applied shear of 23 kips must be transferred to tie reinforcement at the top of the pier and then transferred down the pier with the aid of tie reinforcement, since the shear is greater than that which can be taken by concrete alone.

ACI 318-02 in section 7.10.5.6 requires the use of either 2-#4 or 3-#3 ties as lateral reinforcement within the top 5 in. of the pier.

Per Section 12.13.2.1 of ACI 318-02, the #4 bar can be developed by hooking around a vertical bar. Therefore 4-#4 hooks can develop 4(0.20)(60)(0.9) = 43.2 kips. Since V_u = 1.6 (23 kips) = 36.8 kips is less than the 43.2 kips the 2-#4 ties at the top of the pier can transfer the shear into the pier. With #4 ties at the minimum required spacing in shear (use 8 in.), the ϕV_n for the pier is $\phi\left[2\sqrt{f_c'} b_w d + A_v f_y d / s\right]$, which equals

$$0.85\left(2\sqrt{0.004}(20)(17.5) + 0.2(2)(60)(17.5)/8\right)$$

$$= 82.2 \text{ kips} > 36.8 \text{ kips.}$$

The vertical reinforcement in the pier at 1 percent would require the use of 4-#9 bars. If the provisions of ACI-318-02 Section 10.8.4 and 15.8.2.1 are applicable, 0.5 percent reinforcement ratio could be used

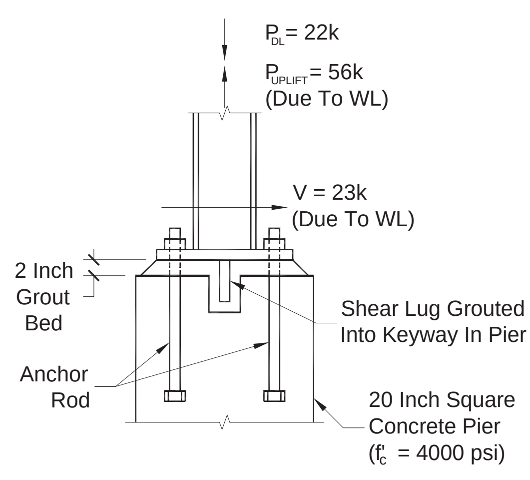


Fig. 9.4.6 Example 9.4.3

which would permit use of 4-#7 bars. T_u = 1.6(56) − 0.9(22) = 69.8 kips which produces 17.5 kips per bar. A single Grade 60 #9 bar has a ϕN_n = 0.9(60)(1.0) = 54.0 kips. The vertical rebar selected is a function of the pier height due to the tension from moment requirements at the base of the pier in addition to the uplift tension. Since there is a significant shear in this example, it may be prudent to place hooks at the top of the vertical reinforcing bars as illustrated in Figure 9.1.4.

EXAMPLE 9.4.3:

Design for Shear Lugs (Pinned Base)

Design a shear lug detail for the W10×45 column considered in Example 9.4.2. See Figure 9.4.6.

The anchor rods in this example will be designed only to transfer the net uplift from the column to the pier and the shear lug will be designed to transfer the entire shear load to the pier with the confinement component being ignored.

The design for the anchor rods will be identical to that in Example 9.4.1 where ⅞ in. diameter anchor rods were selected. Therefore, calculations for the anchor rods are not included in this example. As shown, the anchor rods are positioned outside the column flanges to prevent interference with the lug detail.

Procedure:

1. Determine the required embedment for the lug into the concrete pier.

2. Determine the appropriate thickness for the lug.

3. Size the welds between the lug and the base plate.

Solution:

1. Two criteria are used to determine the appropriate embedment for the lug. These criteria are the bearing strength of the concrete and the shear strength of the concrete in front of the lug. As discussed in Section 9.3, the shear strength of the concrete in front of the lug is evaluated (in ultimate strength terms) as a uniform tensile stress of $4\phi\sqrt{f'_c}$ with ϕ = 0.75 acting on an effective stress area defined by projecting a 45° plane from the bearing edge of the shear lug to the free surface (the face of the pier). The bearing area of the lug is to be excluded from the projected area. Because this criterion is expressed in ultimate strength terms, the bearing strength of the concrete is also evaluated with an ultimate strength approach. The ultimate bearing strength of the concrete in contact with the lug is evaluated as $0.8f'_c A_\ell$ as discussed in Section 9.3.

Because the anchor rods were sized for just the required uplift tension the $1.2(N_y - P_a)$ term addressed in Section 9.3 will be small and thus is being ignored in this example.

The factored shear load = (1.6)(23) = 36.8 kips.

Equating this load to the bearing capacity of the concrete, the following relationship is obtained:

$(0.8)(4000)(A_\ell)_{req'd.} = 36{,}800$

$(A_\ell)_{req'd.} = 11.5 \text{ in.}^2$

Assuming the base plate and shear lug width to be 9 in., the required embedded depth (d) of the lug (in the concrete) is calculated as:

d = 115/9 = 1.28 in.   Use 1½ in.

See Figure 9.4.7.

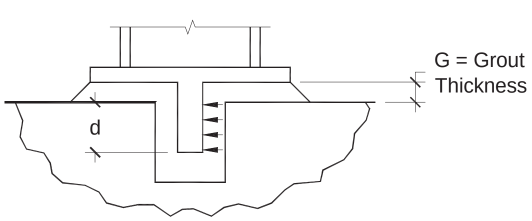


Fig. 9.4.7 Shear Lug Depth

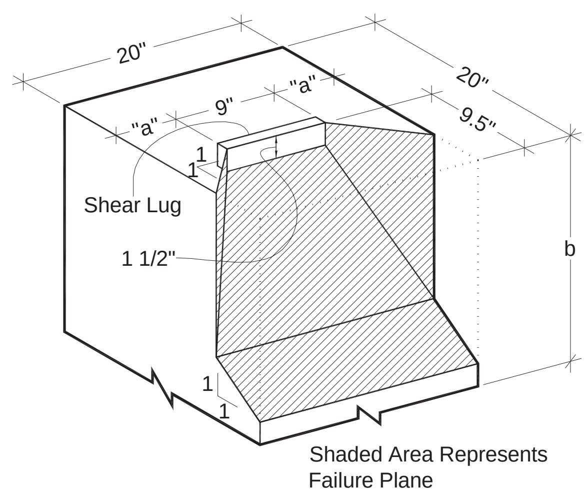


Fig. 9.4.8 Lug Failure Plane

Using this embedment, the shear strength of the concrete in front of the lug is checked. The projected area of the failure plane at the face of the pier is shown in Figure 9.4.8.

Assuming the lug is positioned in the middle of the pier and the lug is 1 in. thick,

a = 5.5 in. in 20 in. wide pier
b = 1.5 in. + 9.5 in. = 11.0 in.

The projected area of this plane (Av), excluding the area of the lug, is then calculated as:

A_v = (20)(11.0) −1.5(9) = 207 in.^{2}

Using this area, the shear capacity of the concrete in front of the lug (V_u) is calculated as:

$$V_u = 4\phi\sqrt{f_c'}A_v$$

$$= 4(0.75)\sqrt{4000}(207)/1000$$

$$= 39.2 \text{ kips} > 36.8 \text{ kips.} \quad \text{o.k.}$$

With a shear lug, the concrete is capable of resisting the shear, as compared to Example 9.4.2, where the anchor rods needed to have their shear transferred to the top-of-pier tie reinforcement.

2. Using working loads and a cantilever model for the lug,

M_ℓ = $V(G+d/2)$
= 23(2+1.5/2) = 63.3 kip in.

Note: G = 2 in. = thickness of grout bed.

For A36 steel

F_b = 0.75(36) = 27 ksi

M_ℓ = 27(9t^2/6) = 40.5t^2

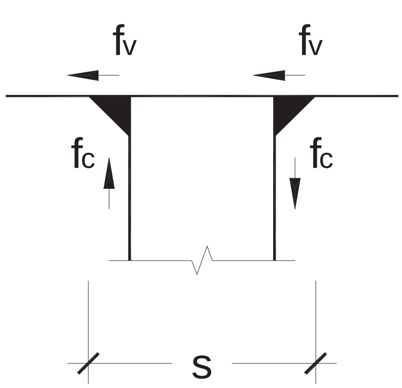


Fig. 9.4.9 Forces on Shear Lug Welds

Req'd t = 1.25 in.

Use a 1¼ in. thick lug (F_y = 36 ksi)

Based on the discussion in Section 9.3 it is recommended to use base plate of 1¼ in. minimum thickness with this shear lug.

3. Most steel fabricators would rather use heavy fillet welds than partial or full penetration welds to attach the lug to the base plate. The forces on the welds are as shown in Figure 9.4.9.

Consider ⁵⁄₁₆ in. fillet welds,

s = 1.25 + 0.3125(1/3)(2) = 1.46 in.

$$f_c = \frac{63.3}{(1.46)(9)} = 4.82 \text{ kips/in.}$$

$$f_v = \frac{23}{(9)(2)} = 1.28 \text{ kips/in.}$$

The resultant weld load (f_r) is calculated as:

$$f_r = \sqrt{(4.82)^2 + (1.28)^2} = 4.99 \text{ kips/in.}$$

For a ⁵⁄₁₆ in. fillet weld using E70 electrode, the allowable load ($f_{allow.}$) is calculated as:

$f_{allow.}$ = 0.3125(0.707)(21)
= 4.64 kips/in. < 4.99 kips/in.

Use ⅜ in. fillet welds

9.5 Partial Base Fixity

In some cases the designer may want to consider designing a column base that is neither pinned nor fixed. These may be cases where full fixity cannot be obtained, or where the designer wants to know the effect of partial fixity. The treatment of partial fixity is beyond the scope of this design guide; however, an excellent treatment of partial fixity can be found in the paper, *Stiffness Design of Column Bases* (Wald, 1998).

10. SERVICEABILITY CRITERIA

The design of the lateral load envelope (in other words, the roof bracing and wall support system) must provide for the code-imposed loads, which establish the required strength of the structure. A second category of criteria establishes the serviceability limits of the design. These limits are rarely codified and are often selectively applied project by project based on the experience of the parties involved.

In AISC Design Guide 3 (Fisher, 2003) several criteria are given for the control of building drift and wall deflection. These criteria, when used, should be presented to the building owner as they help establish the quality of the completed building.

To be useful, a serviceability criterion must set forth three items:

a. loading,

b. performance limit, and

c. an analysis approach.

Concerning lateral forces, the loading recommended by Design Guide 3 is the pressure due to wind speeds associated with a 10-year recurrence interval. These pressures are approximately 75 percent of the pressures for strength design criteria, based on a 50-year return period. The establishment of deflection limits is explained below, with criteria given for each of the wall types previously presented. The author recommends that frame drift be calculated using the bare steel frame only. Likewise the calculations for deflection of girts would be made using the bare steel section. The contribution of non-structural components acting compositely with the structure to limit deflection is often difficult to quantify. Thus the direct approach (neglecting non-structural contribution) is recommended and the loads and limits are calibrated to this analysis approach. The deflection limits for the various roof and wall systems are as follows.

10.1 Serviceability Criteria for Roof Design

In addition to meeting strength criteria in the design of the roof structure, it is also necessary to provide for the proper performance of elements and systems attached to the roof, such as roofing, ceilings, hanging equipment, etc. This requires the control of deflections in the roof structure. Various criteria have been published by various organizations. These limits are:

1. American Institute of Steel Construction (AISC, 1989):
   a. Depth of fully stressed roof purlins should not be less than approximately span/20.
2. Steel Deck Institute (SDI, 2000):
   a. Maximum deflection of deck due to uniformly distributed live load: span over 240.
   b. Maximum deflection of deck due to a 200-lb concentrated load at midspan on a one-foot section of deck: span over 240.
3. Steel Joist Institute (SJI, 2002):
   a. Maximum deflection of joists supporting plaster ceiling due to design live load: span over 360.
   b. Maximum deflection of joists supporting ceilings other than plaster ceilings due to design live load: span over 240.
4. National Roofing Contractors Association (NRCA 2001):
   a. Maximum deck deflection due to full uniform load: span over 240.
   b. Maximum deck deflection due to 300-lb load at midspan: span over 240.
   c. Maximum roof structure deflection due to total load: span over 240.
5. Factory Mutual (FM, 2000):
   a. Maximum deck deflection due to a 300-lb concentrated load at midspan: span over 200.

AISC Design Guide 3 also presents deflection limits for purlins supporting structural steel roofs (both through fastener types and standing seam types). First, a limiting deflection of span over 150 for snow loading is recommended. Secondly, attention is drawn to conditions where a flexible purlin parallels nonyielding construction such as at the building eave. In this case deflection should be controlled to maintain positive roof drainage. The appropriate design load is suggested as dead load plus 50 percent of snow load or dead load plus 5 psf live load to check for positive drainage under load.

Mechanical equipment, hanging conveyors, and other roof supported equipment has been found to perform adequately on roofs designed with deflection limits in the range of span over 150 to span over 240 but these criteria should be verified with the equipment manufacturer and building owner. Consideration should also be given to differential deflections and localized loading conditions.

10.2 Metal Wall Panels

Relative to serviceability metal wall panels have two desirable attributes: 1) Their corrugated profiles make them fairly limber for out of plane distortions and 2) their material and fastening scheme are ductile (i.e., distortions and possible yielding do not produce fractures). Also, the material for edge and corner flashing and trim generally allows moment and distortion without failure. Because of this the deflection limits associated with metal panel buildings are relatively generous. They are:

1. Frame deflection (drift) perpendicular to the wall surface of frame: eave height divided by 60 to 100.

2. The deflection of girts and wind columns should be limited to span over 120, unless wall details and wall-supported equipment require stricter limits.

10.3 Precast Wall Panels

Non-load bearing precast wall panels frequently span from grade to eave as simple span members. Therefore drift does not change the statics of the panel. The limitation on drift in the building frame is established to control the amount of movement in the joint at the base of the panel as the frame drifts. This limit has been proposed to be eave height over 100. A special case exists when precast panels are set atop the perimeter foundations to eliminate a grade wall. The foundation anchorage, the embedment of the panel in the soil and the potential of the floor slab to act as a fulcrum mean that the frame deflections must be analyzed for compatibility with the panel design. It is possible to tune frame drift with panel stresses but this requires interaction between frame designer and panel designer. Usually the design of the frame precedes that of the panel. In this case the frame behavior and panel design criteria should be carefully specified in the construction documents.

10.4 Masonry Walls

Masonry walls may be hollow, grouted, solid, or grouted and reinforced. Masonry itself is a brittle, non-ductile material. Masonry with steel reinforcement has ductile behavior overall but will show evidence of cracking when subjected to loads which stress the masonry in tension. When masonry is attached to a supporting steel framework, deflection of the supports may induce stresses in the masonry. It is rarely feasible to provide sufficient steel (stiffness) to keep the masonry stresses below cracking levels, thus flexural tension cracking in the masonry is likely and when properly detailed is not considered a detriment. The correct strategy is to impose reasonable limits on the support movements and detail the masonry to minimize the impact of cracking.

Masonry should be provided with vertical control joints at the building columns and wind columns. This prevents flexural stresses on the exterior face of the wall at these locations from inward wind. Because the top of the wall is generally free to rotate, no special provisions are required there. The base of the wall joint is most difficult to address. To carry the weight of the wall the base joint must be solid, not caulked. Likewise, the mortar in the joints make the base of the wall a fixed condition until the wall cracks.

Frame drift recommendations are set to limit the size of the inevitable crack at the base of the wall. Because reinforced walls can spread the horizontal base cracks over several joints, separate criteria are given for them. If proper base joints are provided, reinforced walls can be considered as having the behavior of precast walls; in other words, simple span elements with pinned bases. In that case the limit for precast wall panels would be applicable. Where wainscot walls are used, consideration must be given to the joint between metal wall panel and masonry wainscot. The relative movements of the two systems in response to wind must be controlled to maintain the integrity of the joint between the two materials.

The recommended limits for the deflection of elements supporting masonry are:

1. Frame deflection (drift) perpendicular to an unreinforced wall should allow no more than a $^1/_{16}$ in. crack to open in one joint at the base of the wall. The drift allowed by this criterion can be conservatively calculated by relating the wall thickness to the eave height and taking the crack width at the wall face as $^1/_{16}$ in. and zero at the opposite face.

2. Frame deflection (drift) perpendicular to a reinforced wall is recommended to be eave height over 100.

3. The deflection of wind columns and girts should be limited to span over 240 but not greater than 1.5 in.

Part 2
INDUSTRIAL BUILDINGS—GENERAL

11. INTRODUCTION

This section of the guide deals with crane buildings, and will include coverage of those aspects of industrial buildings peculiar to the existence of overhead and underhung cranes. In that context, the major difference between crane buildings and other industrial buildings is the frequency of loading caused by the cranes. Thus, crane buildings should be classified for design purposes according to the frequency of loading.

Crane building classifications have been established in the AISE Technical Report No. 13 (AISE, 2003) as classes A, B, C and D. Classifications for cranes have been established by the Crane Manufacturers Association of America (CMAA, 2002). These designations should not be confused with the building designations.

11.1 AISE Technical Report 13 Building Classifications

Class A are those buildings in which members may experience either 500,000 to 2,000,000 repetitions or over 2,000,000 repetitions in the estimated life span of the building of approximately 50 years. The owner must analyze the service and determine which load condition may apply. It is recommended that the following building types be considered as Class A:

- Batch annealing buildings
- Scrap yards
- Billet yards
- Skull breakers
- Continuous casting buildings
- Slab yards
- Foundries
- Soaking pit buildings
- Mixer building
- Steelmaking buildings
- Mold conditioning buildings
- Stripper buildings
- Scarfing yards
- Other buildings as based on predicted operational requirements

Class B are those buildings in which members may experience a repetition from 100,000 to 500,000 cycles of a specific loading, or 5 to 25 repetitions of such load per day for a life of approximately 50 years.

Class C are those buildings in which members may experience a repetition of from 20,000 to 100,000 cycles of a specific loading during the expected life of a structure, or 1 to 5 repetitions of such load per day for a life of approximately 50 years.

Class D are those buildings in which no member will experience more than 20,000 repetitions of a specific loading during the expected life of a structure.

11.2 CMAA 70 Crane Classifications

The following classifications are taken directly from CMAA 70.

“10-2 CRANE CLASSIFICATIONS

2.1 Service classes have been established so that the most economical crane for the installation may be specified in accordance with this specification.

The crane service classification is based on the load spectrum reflecting the actual service conditions as closely as possible.

Load spectrum is a mean effective load, which is uniformly distributed over a probability scale and applied to the equipment at a specified frequency. The selection of the properly sized crane component to perform a given function is determined by the varying load magnitudes and given load cycles which can be expressed in terms of the mean effective load factor.

where

$$K = \sqrt[3]{W_1^3 P_1 + W_2^3 P_2 + W_3^3 + W_n^3 P_n}$$

W = Load magnitude; expressed as a ratio of each lifted load to the rated capacity. Operation with no lifted load and the weight of any attachment must be included.

P = Load probability; expressed as a ratio of cycles under each load magnitude condition to the total cycles. The sum total of the load probabilities P must equal 1.0.

Table 11.2.1 Crane Loading Conditions	
CMAA 70 Crane Classification	AISC Loading Condition
A, B	1
C, D	2
E	3
F	4

Table 11.2.2 AISC Loading Cycles		
Loading Condition	From	To
1	20,000[a]	100,000[b]
2	100,000	500,000[c]
3	500,000	2,000,000[d]
4	Over 2,000,000	

[a]Approximately equivalent to two applications every day for 25 years.
[b]Approximately equivalent to 10 applications every day for 25 years.
[c]Approximately equivalent to 50 applications every day for 25 years.
[d]Approximately equivalent to 200 applications every day for 25 years.

K = Mean effective load factor. (Used to establish crane service class only.)

All classes of cranes are affected by the operating conditions, therefore, for the purpose of the classifications it is assumed that the crane will be operating in normal ambient temperature of 0 °F to 104 °F (-17.7 °C to 40 °C) and normal atmospheric conditions (free from excessive dust, moisture and corrosive fumes).

The cranes can be classified into loading groups according to the service conditions of the most severely loaded part of the crane. The individual parts which are clearly separate from the rest, or forming a self contained structural unit, can be classified into different loading groups if the service conditions are fully known.

2.2 CLASS A (STANDBY OR INFREQUENT SERVICE) This service class covers cranes which may be used in installations such as powerhouses, public utilities, turbine rooms, motor rooms and transformer stations where precise handling of equipment at slow speeds with long, idle period between lifts are required. Capacity loads may be handled for initial installation of equipment and for infrequent maintenance.

2.3 CLASS B (LIGHT SERVICE) This service covers cranes which may be used in repair shops, light assembly operations, service buildings, light warehousing, etc., where service requirements are light and the speed is slow. Loads may vary from no load to occasional full rated loads with two to five lifts per hour, averaging 10 ft per lift.

2.4 CLASS C (MODERATE SERVICE) This service covers cranes that may be used in machine shops or paper mill machine rooms, etc., where service requirements are moderate. In this type of service the crane will handle loads which average 50 percent of the rated capacity with 5 to 10 lifts per hour, averaging 15 ft, not more than 50 percent of the lift at rated capacity.

2.5 CLASS D (HEAVY SERVICE) This service covers cranes which may be used in heavy machine shops, foundries, fabricating plants, steel warehouses, container yards, lumber mills, etc., and standard duty bucket and magnet operations where heavy duty production is required. In this type of service, loads approaching 50 percent of the rated capacity will be handled constantly during the working period. High speeds are desirable for this type of service with 10 to 20 lifts per hour averaging 15 feet, not more than 65 percent of the lifts at rated capacity.

2.6 CLASS E (SEVERE SERVICE) This type of service requires a crane capable of handling loads approaching a rated capacity throughout its life. Applications may include magnet/bucket combination cranes for scrap yards, cement mills, lumber mills, fertilizer plants, container handling, etc., with 20 or more lifts per hour at or near the rated capacity.

2.7 CLASS F (CONTINUOUS SEVERE SERVICE) This type of service requires a crane capable of handling loads approaching rated capacity continuously under severe service conditions throughout its life. Applications may include custom designed specialty cranes essential to performing the critical work tasks affecting the total production facility. These cranes must provide the highest reliability with special attention to ease of maintenance features."

The class of crane, the type of crane, and loadings all affect the design. The fatigue associated with crane class is especially critical for the design of crane runways and connections of crane runway beams to columns. Classes E and F produce particularly severe fatigue conditions. The determination of fatigue stress levels and load conditions is discussed in more detail in the next section.

The CMAA 70 crane classifications do not relate directly to the AISC loading conditions for fatigue. Loading condition refers to the fatigue criteria given in Appendix K of the AISC ASD *Specifications* (AISC, 1989). Based on the average number of lifts for each CMAA 70 crane classification, the crane classes corresponding to the AISC ASD loading conditions are shown in Table 11.2.1.

The approximate number of loading cycles for each loading condition is given in the AISC ASD Specification Table A-K4.1. The Table is repeated below as Table 11.2.2.

The AISC *LRFD Specification* (AISC, 1999) no longer refers to loading conditions. The *LRFD Specification* uses equations to determine an allowable stress range for a given number of stress cycles. The *LRFD Specification* states that, "The Engineer of Record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections." To use the LRFD equations, the designer must enter the value of *N*, which is the stress range fluctuations in design life, into the appropriate design equations provided in the *Specification*. The LRFD fatigue provisions are the most up to date AISC provisions and are recommended for use by the author.

12. FATIGUE

Proper functioning of the bridge cranes is dependent upon proper crane runway girder design and detailing. The runway design must account for the fatigue effects caused by the repeated passing of the crane. Runway girders should be thought of as a part of a system comprised of the crane rails, rail attachments, electrification support, crane stops, crane column attachment, tie back and the girder itself. All of these items should be incorporated into the design and detailing of the crane runway girder system.

Based on the author's experience it is estimated that 90 percent of crane runway girder problems are associated with fatigue cracking.

Engineers have designed crane runway girders that have performed with minimal problems while being subjected to millions of cycles of loading. The girders that are performing successfully have been properly designed and detailed to:

- Limit the applied stress range, to acceptable levels.
- Avoid unexpected restraints at the attachments and supports.
- Avoid stress concentrations at critical locations.
- Avoid eccentricities due to rail misalignment or crane travel and other out-of plane distortions.
- Minimize residual stresses.

Even when all state of the art design provisions are followed building owners can expect to perform periodic maintenance on runway systems. Runway systems that have performed well have been properly maintained by keeping the rails and girders aligned and level.

Some fatigue damage should be anticipated eventually even in "perfectly designed" structures since fabrication and erection cannot be perfect. Fabricating, erecting, and maintaining the tolerances required in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2000), the American Welding Society, *Structural Welding Code—Steel*, AWS D1.1, (AWS 2002), and the AISE *Technical Report 13*, *Guide for the Design and Construction of Mill Buildings* (AISE, 2003), should be followed in order to provide predicted fatigue behavior.

Fatigue provisions have a 95 percent reliability factor (two standard deviations below mean curve of test data) for a given stress range, and expected life condition. Thus, it is reasonable to expect that 5 percent of similar details can experience fatigue failure before the expected fatigue life is expired. However, if the designer chooses a design life of the structure to be shorter than the expected fatigue life per AISC criteria, the reliability of a critical detail should be higher than 95 percent.

12.1 Fatigue Damage

Fatigue damage can be characterized as progressive crack growth due to fluctuating stress on the member. Fatigue cracks initiate at small defects or imperfections in the base material or weld metal. The imperfections act as stress risers that magnify the applied elastic stresses into small regions of plastic stress. As load cycles are applied, the plastic strain in the small plastic region advances until the material separates and the crack advances. At that point, the plastic stress region moves to the new tip of the crack and the process repeats itself. Eventually, the crack size becomes large enough that the combined effect of the crack size and the applied stress exceed the toughness of the material and a final fracture occurs.

Fatigue failures result from repeated application of service loads, which cause crack initiation and propagation to final fracture. The dominant variable is the tensile stress range imposed by the repeated application of the live load—not the maximum stress that is imposed by live plus dead load. Fatigue damage develops in three stages. These are crack initiation, stable crack growth and unstable crack growth to fracture. Of these the crack initiation phase takes up about eighty percent of the total fatigue life; thus when cracks are of detectible size the fatigue life of a member or detail is virtually exhausted and prompt remedial action should be taken. Abrupt changes in cross section, geometrical discontinuities such as toes of welds, unintentional discontinuities from lack of perfection in fabrication, effects of corrosion and residual stresses all have a bearing on the localized range of tensile stress at details that lead to crack initiation. These facts make it convenient and desirable to structure fatigue design provisions on the basis of categories, which reflect the increase in tensile stress range due to the severity of the discontinuities introduced by typical details. Application of stress concentration factors to stresses determined by usual analysis is not appropriate.

However, fluctuating compressive stresses in a region of tensile residual stress may cause a net fluctuating tensile stress or reversal of stress, which may cause cracks to initiate.

The 1999 AISC *LRFD Specification* provides continuous functions in terms of cycles of life, and stress range, in lieu of the previous criteria for fatigue life that reflected the database only at the break points in the step-wise format. The 1999 AISC provisions use a single table that is divided into sections, which describe various conditions. The sections are:

1. Plain material away from any welding.
2. Connected material in mechanically fastened joints.
3. Welded joints joining components of built-up members.
4. Longitudinal fillet welded end conditions.
5. Welded joints transverse to direction of stress.
6. Base metal at welded transverse member connections.
7. Base metal at short attachments.
8. Miscellaneous.

The 1999 AISC provisions use equations to calculate the design stress range for a chosen design life, N, for various conditions and stress categories. The point of potential crack initiation is identified by description, and shown in the table figures. The tables contain the threshold design stress, F_{TH}, for each stress category, and also provide the detail constant, C_f, applicable to the stress category that is required for calculating the design stress range, F_{SR}. For example, for the majority of stress categories:

$$F_{SR} = \left[\frac{C_f}{N}\right]^{0.333} \geq F_{TH}$$

where

F_{SR} = the Design Stress Range for a defined load condition (number of cycles) and a stress category of the fatigue sensitive detail.

C_f = Constant from AISC Table A-K3.1

Table 12.1.1 CMAA 70 Classification v. Design Life

CMAA 70 Crane Classification	Design Life
A	20,000
B	50,000
C	100,000
D	500,000
E	1,500,000
F	>2,000,000

N = Number of stress range fluctuations in design life,
= Number of stress range fluctuations per day × 365 × years of design life.

F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life.

The standard fatigue design equation applies:

$$f_{sr} \leq F_{SR}$$

where

f_{sr} = the service fatigue stress range based on the cyclic load range, an analytical model, and the section properties of the particular member at the fatigue sensitive detail location.

The 1999 AISC *LRFD Specification*, as well as previous AISC specifications, limit the allowable stress range for a given service life based on an anticipated severity of the stress riser for a given fabricated condition.

Consideration of fatigue requires that the designer determine the anticipated number of full uniform amplitude load cycles. To properly apply the AISC *Specification* (1999) fatigue equations to crane runway girder fatigue analyses, one must understand the difference between the AISC fatigue provisions determined using data from cyclic constant amplitude loading tests, and crane runway variable amplitude cyclic loadings. It is a common practice for the crane runway girder to be designed for service life that is consistent with the crane classification. The Crane Manufacturers Association of America, *Specifications for Electric Overhead Traveling Cranes* (CMAA, 2002) includes crane designations that define the anticipated number of full uniform amplitude load cycles for the life of the crane. Correlating the CMAA 70 crane designations for a given crane to the required fatigue life for the structure cannot be directly determined. The crane does not lift its maximum load, or travel at the same speed, every day or every hour.

Shown in Table 12.1.1 are estimates of the number of cycles of full uniform amplitude for CMAA 70 crane classifications A through F over a 40-year period. It must be emphasized that these are only guidelines and actual duty cycles can only be established from the building's owner and the crane manufacturer.

12.2 Crane Runway Fatigue Considerations

The 1999 AISC *Specification* provisions as they relate to crane runway design are discussed below. A complete design example is provided in a paper by Fisher and Van de Pas titled, "New Fatigue Provisions for the Design of Crane Runway Girders," (Fisher, 2002). The fatigue provisions discussed below assume that the girders are fabricated using the AWS provisions for cyclically loaded structures. In a few instances, additional weld requirements are recommended by AISE *Technical Report 13*. These are pointed out in the sections below.

Tension Flange Stress

When runway girders are fabricated from plate material, fatigue requirements are more severe than for rolled shape girders. The 1999 AISC *Specification* Appendix K3, Table A-K3.1, Section 3.1, applies to the design of the plate material and Section 1.1 applies to plain material. Stress Category B is required for plate girders as compared to stress Category A for rolled shapes.

Web to Flange Welds

Appendix K3, Table A-K3.1, Section 8.2 of the 1999 AISC *Specification* controls the shear in fillet welds, which connect the web to the tension and the compression flanges, stress Category F. Cracks have been observed in plate girders at the junction of the web to the compression flange of runway girders when fillet welds are used to connect the web to the compression flange. Such cracking has been traced to localized tension bending stresses in the bottom side of compression flange plate with each wheel load passage. Each wheel passage may occur two or four (or more) times with each passage of the crane; thus, the life cycles for this consideration is generally several times greater than the life cycles to be considered in the girder live load stress ranges, due to passage of the loaded crane. The calculation of such highly localized tensile bending stresses is so complex and unreliable that the problem is buried in conservative detail requirements. To reduce the likelihood of such cracks the AISE Technical Report No. 13 recommends that the top flange to web joint be a full penetration weld, with fillet reinforcement.

Tiebacks

Tiebacks are provided at the end of the crane runway girders to transfer lateral forces from the girder top flange into the crane column and to laterally restrain the top flange of the crane girder against buckling. The tiebacks must have adequate strength to transfer the lateral crane loads. However, the tiebacks must also be flexible enough to allow for longitudinal movement of the top of the girder caused by girder end rotation. The amount of longitudinal movement due to the end rotation of the girder can be significant. The end rotation of a 40-foot girder that has undergone a deflection of span over 600 is about 0.005 radians. For a 36-inch deep girder this results in 0.2 in. of horizontal movement at the top flange. The tieback must also allow for vertical movement due to axial shortening of the crane column. This vertical movement can be in the range of ¼ in. In general, the tieback should be attached directly to the top flange of the girder. Attachment to the web of the girder with a diaphragm plate should be avoided. The lateral load path for this detail causes bending stresses in the girder web perpendicular to the girder cross section. The diaphragm plate also tends to resist movement due to the axial shortening of the crane column. Various AISC fatigue provisions are applicable to the loads depending on the exact tieback configurations.

Bearing Stiffeners

Bearing stiffeners should be provided at the ends of the girders as required by the AISC *Specification* (1999) Paragraphs K1.3 and K1.4. Fatigue cracks have occurred at the connection between the bearing stiffener and the girder top flange. The cracks occurred in details where the bearing stiffener was fillet welded to the underside of the top flange. Passage of each crane wheel produces shear stress in the fillet welds. The AISC (1999) fatigue provisions contain fatigue criteria for fillet welds in shear; however, the determination of the actual stress state in the welds is extremely complex, thus the AISE *Technical Report No. 13* recommends that full penetration welds be used to connect the top of the bearing stiffeners to the top flange of the girder. The bottom of the bearing stiffeners may be fitted (preferred) or fillet welded to the bottom flange. All stiffeners to girder webs welds should be continuous. Horizontal cracks have been observed in the webs of crane girders with partial height bearing stiffeners. The cracks start between the bearing stiffeners and the top flange and run longitudinally along the web of the girder. There are many possible causes for the propagation of these cracks. One possible explanation is that eccentricity in the placement of the rail on the girder causes distortion of the girder cross-section and rotation of the girder cross-section.

Intermediate Stiffeners

If intermediate stiffeners are used, the AISE *Technical Report No. 13* also recommends that the intermediate stiffeners be welded to the top flange with full penetration welds for the same reasons as with bearing stiffeners. Stiffeners should be stopped short of the tension flange in accordance with the AISC *Specification* (1999) provisions contained in Chapter G. The AISE Technical Report No. 13 also recommends continuous stiffener to web welds for intermediate stiffeners.

Fatigue must be checked where the stiffener terminates adjacent to the tension flange. This condition is addressed in Section 5.7, Table A-K3.1, of the 1999 AISC specifications.

Channel Caps and Cap Plates

Channel caps or cap plates are frequently used to provide adequate top flange capacity to transfer lateral loads to the crane columns and to provide adequate lateral torsional stability of the runway girder cross section. It should be noted that the cap channel or plate does not fit perfectly with 100 percent bearing on the top of the wide flange. The tolerances given in ASTM A6 allow the wide flange member to have some flange tilt along its length, or the plate may be cupped or slightly warped, or the channel may have some twist along its length. These conditions will leave small gaps between the top flange of the girder and the top plate or channel. The passage of the crane wheel over these gaps will tend to distress the channel or plate to top flange welds. Calculation of the stress condition for these welds is not practical. Because of this phenomenon, cap plates or channels should not be used with Class E or F cranes. For less severe duty cycle cranes, shear flow stress in the welds can be calculated and limited according to the AISC (1999) fatigue provisions in Appendix K3, Table A-K3.1, Section 8.2. The channel or plate welds to the top flange can be continuous or intermittent. However, the AISC design stress range for the base metal is reduced from Category B (Section 3.1) for continuous welds to Category E (Section 3.4) for intermittent welds.

Crane Column Cap Plates

The crane column cap plate should be detailed so as to not restrain the end rotation of the girder. If the cap plate girder bolts are placed between the column flanges, a force couple between the column flange and the bolts resists the girder end rotation. This detail has been known to cause bolt failures. Preferably, the girder should be bolted to the cap plate outside of the column flanges. The column cap plate should be extended outside of the column flange with the bolts to the girder placed outside of the column flanges. The column cap plate should not be made overly thick, as this detail requires the cap plate to distort to allow for the end rotation of the girder. The girder to cap plate bolts should be adequate to transfer the tractive or bumper forces to the longitudinal crane bracing. Traction plates between girder webs may be required for large tractive forces or bumper forces. The engineer should consider using finger tight bolts with upset threads as a means of reducing bolt fatigue in crane column cap plates (Rolfes, 2001).

Miscellaneous Attachments

Attachments to crane runway girders should be avoided. The AISE *Technical Report No. 13* specifically prohibits welding attachments to the tension flange of runway girders. Brackets to support the runway electrification are often necessary. If the brackets are bolted to the web of the girder, fatigue consequences are relatively minor, i.e. stress category B, Section 1.3 of the AISC (1999) fatigue specifications. However, if the attachment is made with fillet welds to the web Appendix K3, Table A-K3.1, Section 7.2 of the AISC *Specification* applies. This provision places the detail into stress category D or E depending on the detail. If transverse stiffeners are present, the brackets should be attached to the stiffeners.

13. CRANE INDUCED LOADS AND LOAD COMBINATIONS

It is recommended that the designer shows on the drawings the crane wheel loads, wheel spacing, bumper forces, and the design criteria used to design the structure.

Although loading conditions for gravity, wind, and seismic loads are well defined among building codes and standards, crane loading conditions generally are not.

As mentioned previously, crane fatigue loadings are primarily a function of the class of service, which in turn is based primarily on the number of cycles of a specific loading case. This classification should be based on the estimated life span, rate of loading, and the number of load repetitions. The owner should specify or approve the classification for all portions of a building. A maximum life span of 50 years is generally accepted.

The provisions of the American Society of Civil Engineers (ASCE, 2002) and the Association of Iron and Steel Engineers (AISE, 2003) on crane runway loads are summarized in the following discussion. ASCE 7 is referenced by the International Building Code (ICC 2003), and is a legal requirement. AISE *Technical Report No. 13* is a guideline and can be used for situations not covered by ASCE 7, or when specified by project specifications. In addition, the MBMA *Low Rise Building Systems Manual* (MBMA, 2002) provides a comprehensive discussion on crane loads.

AISE *Technical Report 13* recommendations are based on ASD design provisions, whereas ASCE 7 provisions are

Table 13.2.1 AISE Report 13 Crane Side Thrusts	
Crane Type	**Total side thrust percent of lifted load**
Mill crane	40
Ladle cranes	40
Clamshell bucket and magnet cranes (including slab and billet yard cranes)	100
Soaking pit cranes	100
Stripping cranes	100↑
Motor room maintenance cranes, etc.	30
Stacker cranes (cab-operated)	200

↑ ingot and mold

given for both Strength Design and Allowable Stress Design. ASCE 7 indicates that the live load of a crane is the rated capacity. No comments are made about appropriate load factors relative to the trolley, hoist, or bridge weight. The author recommends using a 1.2 load factor for the bridge weight and a 1.6 load factor for the hoist and trolley weight.

13.1 Vertical Impact

ASCE 7

ASCE 7 defines the maximum wheel load as follows: "The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum." Vertical impact percentages are then multiplied by the maximum wheel loads. The percentage factors contained in ASCE 7 are as follows:

Monorail cranes (powered)	25
Cab-operated or remotely operated bridge cranes (powered)	25
Pendant-operated bridge cranes (powered)	10
Bridge cranes or monorail cranes with hand-geared bridge, trolley, and hoist	0

AISE Technical Report No. 13

The allowances for vertical impact are specified as 25 percent of the maximum wheel loads for all crane types, except a 20 percent impact factor is recommended for motor room maintenance cranes, etc.

In all cases, impact loading should be considered in the design of column brackets regardless of whether ASCE 7 or AISE *Technical Report 13* requirements are being used.

13.2 Side Thrust

Horizontal forces exist in crane loadings due to a number of factors including:

1. Runway misalignment
2. Crane skew
3. Trolley acceleration
4. Trolley braking
5. Crane steering

ASCE 7

"The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface on a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure."

AISE Technical Report No. 13

The AISE *Technical Report 13* requires that "The recommended total side thrust shall be distributed with due regard for the lateral stiffness of the structures supporting the rails and shall be the greatest of:

1. That specified in Table 3.2 [Shown here as Table 13.2.1].

2. 20 percent of the combined weight of the lifted load and trolley. For stacker cranes this factor shall be 40 percent of the combined weight of the lifted load, trolley and rigid arm.

3. 10 percent of the combined weight of the lifted load and crane weight. For stacker cranes this factor shall be 15 percent of the combined weight of the lifted load and the crane weight."

In the AISE *Technical Report 13* lifted load is defined as: "a total weight lifted by the hoist mechanism, including working load, all hooks, lifting beams, magnets or other appurtenances required by the service but excluding the weight of column, ram or other material handling device which is rigidly guided in a vertical direction during hoisting action."

Table 13.3.1 ASCE 7/AISE Report 13 Side Thrust Comparison		
SIDE THRUST COMPARISON ASCE 7 V. AISE Report 13 100T MILL CRANE TROLLEY WT = 60,000 LBS. (Includes Hoist), ENTIRE CRANE WT = 157,000 LBS.		
CRITERIA	EQUATION (TOTAL FORCE)	TOTAL FORCE
ASCE 7 (ASD)	0.20 (Rated Capacity + Trolley Wt)	52.00 kips
AISE Report 13	(1) 0.40 (Lifted Load) (2) 0.20 (Lifted Load + Trolley Wt) (3) 0.10 (Lifted Load + Entire Crane Wt)	80.00 kips 52.00 kips 35.72 kips

For pendant operated cranes, the AISE *Technical Report 13* side thrust is taken as 20 percent of the maximum load on the driving wheels. In most cases one half of the wheels are driving wheels.

AISE *Technical Report 13* requires that radio-operated cranes be considered as cab-operated cranes with regard to side thrusts.

Table 13.3.1 is provided to illustrate the variation between the AISC *Specification* and AISE *Technical Report No. 13* for a particular crane size.

13.3 Longitudinal or Tractive Force

ASCE 7

The longitudinal force on crane runway beams is calculated as 10 percent of the maximum wheel loads of the crane. ASCE 7 excludes bridge cranes with hand-geared bridges from this requirement, thus the author presumes that it is ASCE's position, that tractive forces are not required for hand-geared bridge cranes. It is further stated in ASCE 7 that the longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam in either direction parallel to the beam.

AISE Technical Report 13

The tractive force is taken as 20 percent of the maximum load on driving wheels.

13.4 Crane Stop Forces

The magnitude of the bumper force is dependent on the energy-absorbing device used in the crane bumper. The device may be linear such as a coil spring or nonlinear such as hydraulic bumpers. See Section 18.6 for additional information on the design of the runway stop.

The crane stop, crane bracing, and all members and their connections that transfer the bumper force to the ground, should be designed for the bumper force. It is recommended that the designer indicate on the structural drawings the magnitude of the bumper force assumed in the design. The owner or crane supplier generally specifies the bumper force. If no information can be provided at the time of design Section 6.6 of the MBMA Manual (MBMA, 2002) can provide some guidance.

13.5 Eccentricities

The bending of the column due to eccentricity of the crane girder on the column seat must be investigated. The critical bending for this case may occur when the crane is not centered over the column but located just to one side as illustrated in Figure 13.5.1. Additional consideration for other eccentricities is discussed in Sections 17.2 and 18.2.

13.6 Seismic Loads

Although cranes do not induce seismic loads to a structure, the crane weight should be considered in seismic load determination. The seismic mass of cranes and trolleys that lift a suspended load need include only the empty weight of

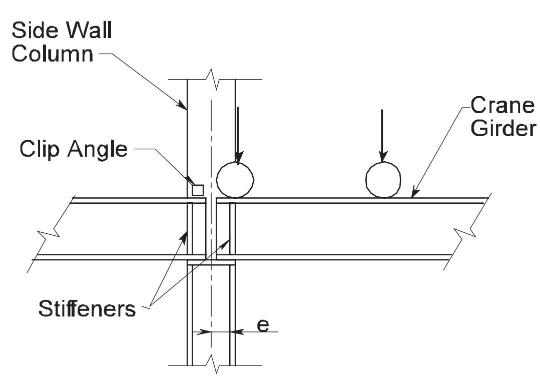


Fig. 13.5.1 Possible Critical Crane Location

the equipment. The designer should carefully evaluate the location of the cranes within the building in the seismic analysis.

Where appropriate, a site investigation should be performed in order to determine the soil profile type for seismic response.

Seismic response interaction between the crane building and equipment should be taken into account.

Special consideration should also be given to design requirements beyond those specified in the building code for buildings, structures, and equipment that must remain serviceable immediately after a design level earthquake. This may include the examination of vertical accelerations and their affect on the cranes ability to not "bounce" off the runway during a seismic event.

Also, the designer is cautioned to verify seismic limitations that may be imposed on the structural system, and to determine the need for special detailing requirements based on the Seismic Design Category.

13.7 Load Combinations

In addition to the applicable building code, the owner may require conformance with AISE *Technical Report 13* rules. However, in the absence of such rules, the designer should consider the usage of the structure in determining the criteria for the design. Building codes generally may not contain information on how to combine the various crane loads; in other words which crane loads, and how many cranes should be considered loaded at one time, but generally they do address how crane loadings should be combined with wind, snow, live, seismic, and other loads.

For one crane, each span must be designed for the most severe loading with the crane in the worst position for each element that is affected. As mentioned, when more than one crane is involved in making a lift most codes are silent on a defined procedure. Engineering judgment on the specific application must be used.

AISE *Technical Report No. 13* (which is based on ASD) includes the following provisions for the design of members subject to multiple crane lifts. These provisions are to be used in the design of the supporting elements.

The design of members (and/or frames), connection material and fasteners shall be based on whichever one of the three cases listed hereinafter may govern. Moments and shears for each type loading shall be listed separately (in other words, dead load, live load, crane eccentricity, crane thrust, wind, etc.). The permissible stress range under repeated loads shall be based on fatigue considerations with the estimated number of load repetitions in accordance with the Building Classification. The owner shall designate an increase in the estimated number of load repetitions for any portion of the building structure for which the projected workload or possible change in building usage warrants.

Case 1:

"$D + C_{vs} + 0.5C_{ss} + C_i$

This case applies to load combinations for members designed for repeated loads. The number of load repetitions used as a basis for design shall be 500,000 to 2,000,000 (Loading Condition 3) or over 2,000,000 (Loading Condition 4), as determined by the owner, for Class A construction. Class B and Class C constructions shall be designed for 100,000 to 500,000 (Load Condition 2) and 20,000 to 100,000 (Load Condition 1), respectively. This case does not apply to Class D buildings." It should be noted that the inclusion of D (dead load) should not be included with this load case since the dead load does not cause a cyclic stress condition. AISE *Technical Report 13* does allow a more sophisticated variable stress range spectrum to be used.

Case 2:

"(1) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C_i + C_{ss} + C_{ls}$
(Single Crane)

(2) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vm} + C_{ss} + C_{ls}$
(Multiple Cranes)

This case applies to all classes of building construction. Full allowable stresses may be used."

Case 3:

"(1) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C_i + W$

(2) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C_i + C_{ss} + 0.5W$

(3) $D + L + (L_r \text{ or } R \text{ or } S) + C_{vs} + C_i + 0.67C_{bs}$

(4) $D + L + (L_r \text{ or } R \text{ or } S) + C_d + E$

This case applies to all classes of building construction. The total of the combined load effects may be multiplied by 0.75, with no increase in allowable stresses"

For the above equations AISE *Technical Report 13* has the following symbols and notations:

C_{vs} = vertical loads due to a single crane in one aisle only
C_{ss} = side thrust due to a single crane in one aisle only
C_i = vertical impact due to a single crane in one aisle only
C_{ls} = longitudinal traction due to a single crane in one aisle only
C_{vm} = vertical loads due to multiple cranes

C_{bs} = bumper impact due to a single crane in one aisle only at 100 percent speed
C_d = dead load of all cranes, parked in each aisle, positioned for maximum seismic effects
D = dead load
E = earthquake load
F = loads due to fluids
L = live loads due to use and occupancy including roof live loads, with the exception of snow loads and crane runway loads
L_r = roof live loads
S = snow loads
R = rain loads (inadequate drainage)
H = loads due to lateral pressure of soil and water in soil
P = loads due to ponding
T = self-straining forces as from temperature changes, shrinkage, moisture changes, creep, or differential settlement
W = wind load

Because the standard AISE *Technical Report 13* building classifications were based upon the most frequently encountered situations, they should be used with engineering judgment. The engineer, in consultation with the owner, should establish the specific criteria. For example, other load combinations that have been used by engineers include:

1. A maximum of two cranes coupled together with maximum wheel loads, 50 percent of the specified side thrust from each crane, and 90 percent of the specified traction. No vertical impact.

2. One crane in the aisle and one in an adjacent aisle with maximum wheel loads, specified vertical impact, and with 50 percent combined specified side thrust and specified traction from each crane.

3. A maximum of two cranes in one aisle and one or two cranes in an adjacent aisle with maximum wheel loads, and 50 percent of the specified side thrust of the cranes in the aisle producing the maximum side thrust, with no side thrust from cranes in the adjacent aisle. No vertical impact or traction.

Additional information relative to loading combinations is contained in the MBMA *Low Rise Building Systems Manual*. The crane combinations contained in the Manual agree very closely with the AISE *Technical Report 13* combinations.

14. ROOF SYSTEMS

The inclusion of cranes in an industrial building will generally not affect the basic roof covering system. Crane buildings will "move" and any aspect in the roof system that might be affected by such a movement must be carefully evaluated. This generally means close examination of details (for example, flashings, joints, etc.).

A difference in the roof support system design for crane buildings, as opposed to industrial buildings without cranes, is that the use of a roof diaphragm system should only be used after careful consideration of localized forces that may be imparted into the diaphragm from the crane forces. Whereas wind loads apply rather uniformly distributed forces to the diaphragm, cranes forces are localized, and cause concentrated repetitive forces to be transferred from the frame to the diaphragm. These concentrated loads combined with the cyclical nature of the crane loadings (fatigue) should be carefully examined before selecting a roof diaphragm solution.

15. WALL SYSTEMS

The special consideration, which must be given to wall systems of crane buildings, relates to movement and vibration. Columns are commonly tied to the wall system to provide bracing to the column or to have the column support the wall. (The latter is applicable only to lightly loaded columns.) For masonry and concrete wall systems it is essential that proper detailing be used to tie the column to the wall. Figure 15.1 illustrates a detail that works well for masonry walls. The bent anchor rod has flexibility to permit movement perpendicular to the wall but is "stiff" parallel to the wall, enabling the wall to brace the column about its weak axis. The use of the wall as a lateral bracing system for columns should be avoided if future expansion is anticipated.

If a rigid connection is made between column and wall and crane movements and vibrations are not accounted for, wall distress is inevitable.

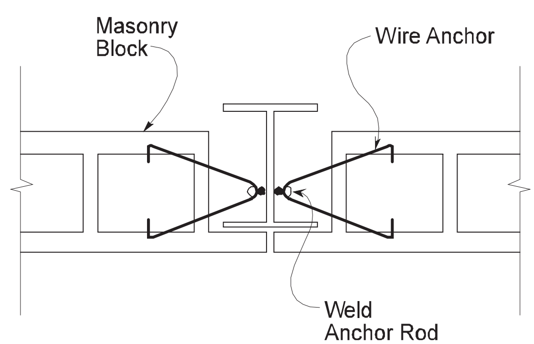


Fig. 15.1 Masonry Wall Anchorage

16. FRAMING SYSTEMS

The same general comments given previously for industrial buildings without cranes apply to crane buildings as well. However, the crane normally dictates the most economical framing schemes. Optimum bays are usually smaller for crane buildings than buildings without cranes and usually fall into the 25- to 30-ft range. This bay size permits the use of rolled shapes as crane runways for lower load crane sizes. Fifty- to 60-ft main bays, with wind columns, are generally more economical when deep foundations and heavy cranes are specified.

The design of framing in crane buildings must include certain serviceability considerations that are used to control relative and absolute lateral movements of the runways by controlling the frame and bracing stiffness. The source producing lateral movement is either an external lateral load (wind or earthquake) or the lateral load induced by the operation of the crane. The criteria are different for pendant operated versus cab-operated cranes since the operator rides with the crane in the latter case. In crane bays with gabled roofs, vertical roof load can cause spreading of the eaves and thus spreading of the crane runways. Conversely, eccentrically bracketed runways on building columns can result in inward tilting of the columns due to the crane loading. This would cause an inward movement of the runways toward each other. Lastly, the crane tractive force can cause longitudinal movement of the runway either by torsion in the supporting columns where brackets are used or flexing of the frame if rigid frame bents are used for the runway columns. Longitudinal runway movement is rarely a problem where braced frames are used. Recommended serviceability limits for frames supporting cranes include the following:

1. Pendant operated cranes: Frame drift to be less than the height to the runway elevation over 100, based on 10-year winds or the crane lateral loads on the bare frame. While this limit has produced satisfactory behavior, the range of movements should be presented to the building owner for review because they may be perceived as too large in the completed building.

2. Cab operated cranes: Frame drift to be less than the height to the runway elevation over 240 and less than 2 in., based on 10-year wind or the crane lateral loads on the bare frame.

3. All top running cranes: Relative inward movement of runways toward each other to be less than a ½-in. shortening of the runway to runway dimension. This displacement would be due to crane vertical static load.

4. All top running cranes: Relative outward movement of runways away from each other to be not more than an increase of 1 in. in the dimension between crane runways. The loading inducing this displacement would vary depending on the building location. In areas of roof snow load less than 13 psf, no snow load need be taken for this serviceability check. In areas of roof snow load between 13 psf and 31 psf, 50 percent of the roof snow load should be used. Lastly, in areas of where the snow loads exceed 31 psf, 75 percent of the roof snow load should be used.

The discussion of serviceability limits is also presented in more detail in AISC Steel Design Guide 3 (Fisher, 2003).

In addition to the above mentioned serviceability criteria, it is recommended that office areas associated with crane buildings should be isolated from the crane building, so that vibration and noise from the cranes is minimized in the office areas.

17. BRACING SYSTEMS

17.1 Roof Bracing

Roof bracing is very important in the design of crane buildings. The roof bracing allows the lateral crane forces to be shared by adjacent bents. This sharing of lateral load reduces the column moments in the loaded bents. This is true for all framing schemes (in other words, rigid frames of shapes, plates, trusses, or braced frames). It should be noted, however, that in the case of rigid frame structures the moments in the frame cannot be reduced to less than the wind induced moments.

Figures 17.1.1, 17.1.2 and 17.1.3 graphically illustrate the concept of using roof bracing to induce sharing of lateral crane loads in the columns. For wind loading all frames and columns are displaced uniformly as shown in Figure 17.1.1. For a crane building without roof bracing the lateral crane loads are transmitted to one frame line (Figure 17.1.2)

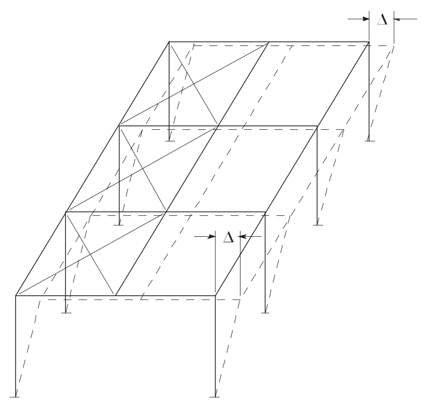


Fig. 17.1.1 Uniform Displacement Due to Wind

causing significant differential displacement between frames. The addition of roof bracing will create load sharing. Columns adjacent to the loaded frame will share in the load thus reducing differential and overall displacement. (Figure 17.1.3).

Angles or tees will normally provide the required stiffness for this system.

Additional information on load sharing is contained in Section 20.1.

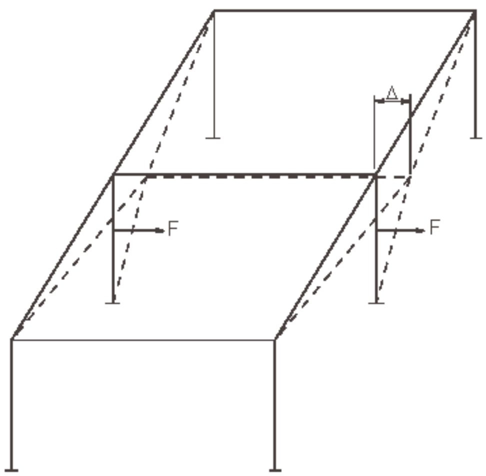


Fig. 17.1.2 Displacement of Unbraced Frames Due to Crane Lateral Load

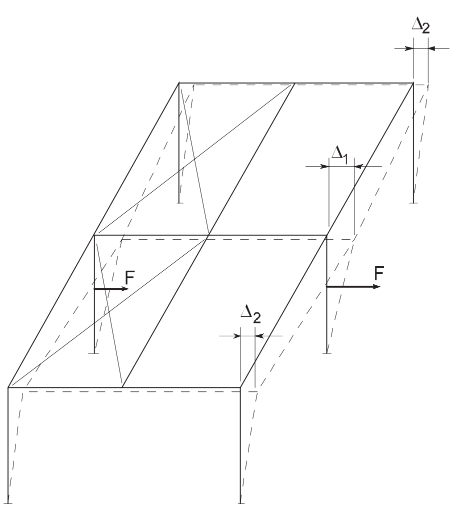


Fig. 17.1.3 Displacement of Braced Frames Due to Crane Lateral Load

17.2 Wall Bracing

It is important to trace the longitudinal crane forces through the structure in order to insure proper wall and crane bracing (wall bracing for wind and crane bracing may in fact be the same braces).

For lightly loaded cranes, wind bracing in the plane of the wall may be adequate for resisting longitudinal crane forces. (See Figure 17.2.1.) While for very large longitudinal forces, the bracing will most likely be required to be located in the plane of the crane rails. (See Figure 17.2.2.)

For the bracing arrangement shown in Figure 17.2.1, the crane longitudinal force line is eccentric to the plane of the X-bracing. The crane column may tend to twist if the horizontal truss is not provided. Such twisting will induce additional stresses in the column. The designer should calculate the stresses due to the effects of the twisting and add these stresses to the column axial and flexural stresses. A torsional analysis can be made to determine the stresses caused by twist, or as a conservative approximation the stresses can be determined by assuming that the twist is resolved into a force couple in the column flanges as shown in Figure 17.2.3. The bending stresses in the flanges can be calculated from the flange forces. In order to transfer the twist, P_e, into the two flanges, stiffeners may be required at the location of the force P.

The following criteria will normally define the longitudinal crane force transfer:

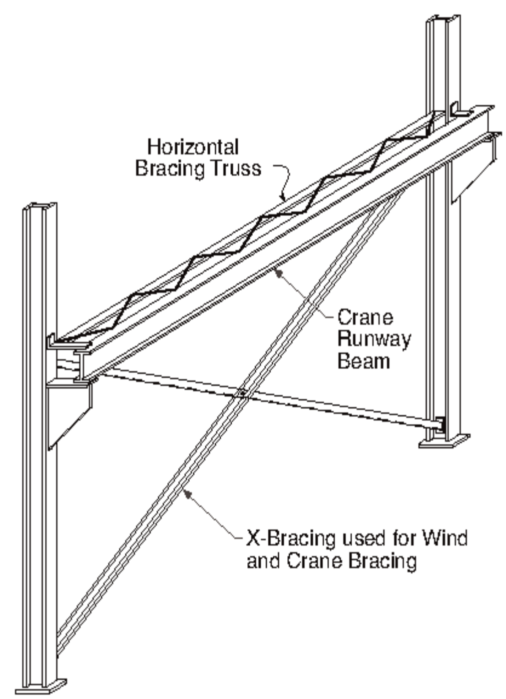


Fig. 17.2.1 Wall Bracing for Cranes

1. For small longitudinal loads (up to 4 kips) use of wind bracing is generally efficient, where columns are designed for the induced eccentric load.

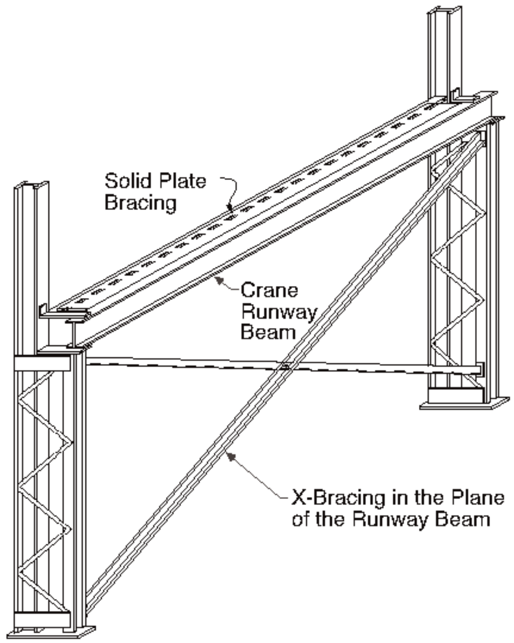


Fig. 17.2.2 Vertical Bracing for Heavy Cranes

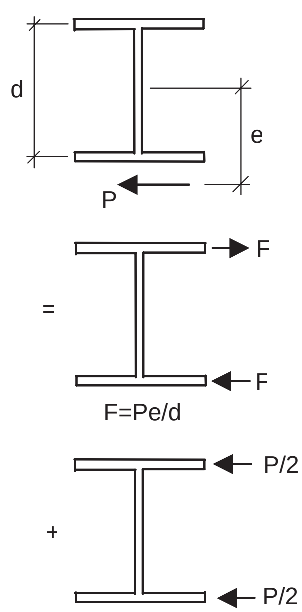


Fig. 17.2.3 Eccentric Column Forces

2. For medium longitudinal loads (4 kips-8 kips) a horizontal truss is usually required to transfer the force to the plane of X-bracing.

3. For large longitudinal loads (more than 8 kips) bracing in the plane of the longitudinal force is generally the most effective method of bracing. Separate wind X-bracing on braced frames may be required due to eccentricities.

Normally the X-bracing schemes resisting these horizontal crane forces are best provided by angles or tees rather than rods. In cases where aisles must remain open, portal type bracing may be required in lieu of designing the column for weak axis bending. (See Figure 17.2.4.)

It should be noted that portal bracing will necessitate a special design for the horizontal (girder) member, and that the diagonals will take a large percentage of the vertical crane forces. This system should only be used for lightly loaded, low fatigue situations. The system shown in Figure 17.2.5 could be used as an alternate to 17.2.4.

18. CRANE RUNWAY DESIGN

Strength considerations for crane girder design are primarily controlled by fatigue for CMAA 70 Class E and F

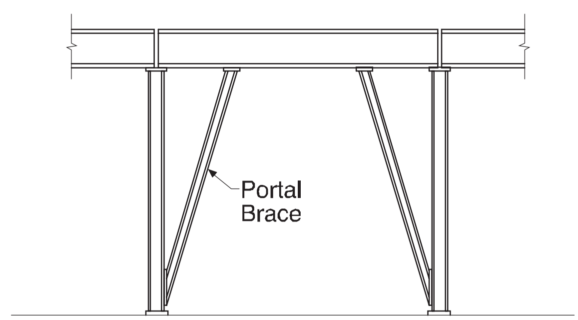


Fig. 17.2.4 Portal Crane Runway Bracing

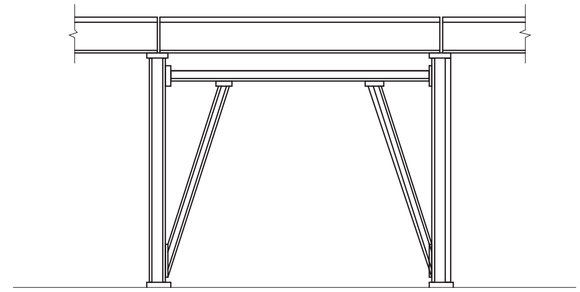


Fig. 17.2.5 Modified Portal Crane Runway Bracing

cranes, and to some degree Class D cranes. Wheel loads, their spacing, and girder span are required for the design of crane girders. The expense of crane girder construction normally increases when built-up shapes are required. Fatigue restrictions are more severe for built-up shapes. The difference between a rolled shape vs. a built-up member using continuous fillet welds is a reduction in the allowable fatigue stress.

The following summary of crane girder selection criteria may prove helpful.

1. Light cranes and short spans—use a wide flange beam.

2. Medium cranes and moderate spans use a wide flange beam, and if required reinforce the top flange with a channel.

3. Heavy cranes and longer spans—use a plate girder, with a horizontal truss or solid plate at the top flange.

4. Limit deflections under crane loads as follows:

   Vertical Deflections of the Crane Beam due to wheel loads (no impact):

   L/600 Light and Medium Cranes—CMAA 70 Classes A, B, and C.

   L/800 Light and Medium Cranes—CMAA 70 Class D (Fisher 2003).

   L/1000 Mill Cranes—CMAA 70 Classes E and F.

   Lateral Deflection of the Crane Beam due to crane lateral loads:

   L/400 All Cranes.

18.1 Crane Runway Beam Design Procedure (ASD)

As previously explained, crane runway beams are subjected to both vertical and horizontal forces from the supported crane system. Consequently, crane runway beams must be designed for combined bending about both the X and Y axis.

Salmon (1997) and Gaylord (1992) point out that the equations presented in the AISC specifications for lateral-torsional buckling strength are based upon the load being applied at the elevation of the neutral axis of the beam. If the load is applied above the neutral axis (for example, at the top flange of the beam as is the case with crane runway beams), lateral torsional buckling resistance is reduced. In addition, the lateral loads from the crane system are applied at the top flange level, generating a twisting moment on the beam. When vertical and lateral loads are applied simultaneously, these two effects are cumulative. To compensate for this, it is common practice to assume the lateral loads, due to the twisting moment, are resisted by only the top flange. With this assumption, Salmon and Gaylord both suggest that the lateral stability of a beam of this type subject to biaxial bending is otherwise typically not affected by the weak axis bending moment (M_y). Consequently they indicate that the appropriate allowable bending stress (F_b) for combined bending is based on a yield criterion and is equal to $0.6F_y$ for the unbraced section. Examples provided by Salmon and Gaylord are for relatively short beam lengths. In the earlier edition of this design guide, the procedure recommended by Salmon and Gaylord was used; however, the author is not aware of any significant research on this procedure for runway girders with varieties of shapes and spans, and thus recommends that the AISC axial load and bending moment interaction equations be used.

Another criterion related to crane runway beam design is referred to in the AISC specifications as “sidesway web buckling” (Section K1.5). This criterion is included to prevent buckling in the tension flange of a beam where flanges are not restrained by bracing or stiffeners and are subject to concentrated loads. This failure mode may predominate when the compression flange is braced at closer intervals than the tension flange or when a monosymmetric section is used with the compression flange larger than the tension flange (for example, wide flange beam with a cap channel). A maximum allowable concentrated load is used as the limiting criterion for this buckling mode.

This criterion does not currently address beams subjected to simultaneously applied multiple wheel loads.

The author is not aware of any reported problems with runway beams that are designed using these criteria with a single wheel load.

For crane runway beams the following ASD design procedure is recommended as both safe and reasonable where fatigue is not a factor. LRFD design procedures are similar in nature. See Examples 18.1.1 and 18.1.2 for ASD and LRFD procedures.

1. Compute the required moments of inertia (I_x and I_y) to satisfy deflection control criterion. L/600 to L/1000 for Vertical Deflection. L/400 for Lateral Deflection.

2. Position the crane to produce worst loading conditions. This can be accomplished using the equations found in the beam section of the AISC Manual for cranes with two wheel end trucks on simple spans. For other wheel arrangements the maximum moment can be obtained by locating the wheels so that the center of the span is midway between the resultant of the loads and the nearest wheel to the resultant. The maximum moment will occur at the wheel nearest to the

centerline of the span. For continuous spans the maximum moment determination is a trial and error procedure. Use of a computer for this process is recommended.

3. Calculate Bending Moments (M_x and M_y) including effects of impact. Many engineers determine M_y by applying the lateral crane forces to the top flange of the runway beam. AISE *Technical Report 13* requires that the lateral force be increased due to the fact that the force is applied to the top of the rail. This eccentricity of lateral load increases the magnitude of the lateral force to the top flange.

4. For sections without channel caps, select a trial section ignoring lateral load (M_y) effects from:

$$S_x = \frac{M_x}{F_{bx}}$$

where F_{bx} is obtained from AISC Equations in Chapter F. To account for the weak axis effects select a section a few sizes larger than provided by the equation, and select sections from groupings with wide flanges. For sections with channel caps Appendices 1 and 2 are of assistance. If A36 channel caps are used on A992 steel beams then lateral torsional buckling requirements must be based on the A36 material. Also the weak axis strength must be based on the channel cap material.

5. Check this section by using:

$$\frac{M_x/S_x}{F_{bx}} + \frac{M_y/S_t}{F_{by}} \le 1.0$$

S_t = Section modulus of top half of section about y-axis. For rolled beams without channel caps, S_t should be taken as ½ of the total S_y of the shape, since the design assumption is that only the top flange resists the lateral crane loads. For sections with channel caps, S_t is the section modulus of the channel and top flange area. Values of this parameter are provided in Appendix A, Table 1, for various W and C combinations. Table 1 also lists values for I_x, S_1, S_2 and y_1. S_1 and S_2 refer to bottom and top flange section moduli respectively. y_1 is the distance from the bottom flange to the section centroid. Table 1 also gives the moment of inertia I^t of the "top flange" of the combined W and C sections.

6. Check the section with respect to sidesway web buckling as described in Section K1.5 of the AISC specifications.

The above checks do not incorporate the stress in the runway beam due to the longitudinal tractive force. ASCE 7 does not provide load combinations specifically for the runway force combinations; however, as noted above AISE *Technical Report 13* does include two load cases which include the longitudinal tractive force. The author normally checks the longitudinal stress in the runway beam based on the full cross sectional area of the beam. In the majority of cases the stress level is so low the stress can be neglected.

In selecting a trial rolled shape section, it may be helpful to recognize that the following ratios exist for various W shapes without channel caps:

W Shape	**S_x/S_y**
W8 through W16	3 to 8
W16 through W24	5 to 10
W24 through W36	7 to 12

Table 2 in Appendix A provides the radius of gyration r_T and d/A_f for commonly used channel and wide flange combinations. In addition, for these combinations, the maximum span (unbraced length) for which the allowable bending stress can be taken as 0.6 F_y is listed.

Where fatigue is a consideration, the above procedure should be altered so that the live load stress range for the critical case does not exceed fatigue allowable stresses as per AISC Appendix K.

EXAMPLE 18.1.1:

Crane Runway Girder Design (ASD)

Crane Capacity = 20 Tons (40 kips)

Bridge Span = 70 ft

Type of Control—Cab Operated

Bridge Weight = 57.2 kips

Combined Trolley and Hoist Weight = 10.6 kips

Maximum Wheel Load (without impact) = 38.1 kips

Wheel Spacing = 12' - 0"

Runway Girder Span = 30" - 0"

Assume no reduction in allowable stress due to fatigue.

Use AISC criteria and A992 steel for the beam section and A572 Grade 50 channel cap.

The critical wheel locations with regard to bending moment are shown in Figure 18.1.1.

$$M_x = \frac{P}{2(30)}\left(30 - \frac{12}{2}\right)^2 = 9.60(P) \text{ kip-ft}$$

The critical wheel locations with regard to deflection are shown in Figure 18.1.2.

$$\Delta_{max} = \frac{P(9)}{24(29000)(I)}\left[3(30)^2 - 4(9)^2\right](1728)$$

$$= \frac{53.1(P)}{I}$$

At nominal loads,

Max. Vertical Load/Wheel = 38.1 kips

Max. Horizontal Load/Wheel = 0.20(40+10.6)/4

= 2.53 kips

Using the vertical deflection criterion of L/600,

$(\Delta)_{allow.}$ = 30(12)/600 = 0.60 in.

$I_{x\text{-}x\ req'd.}$ = 53.1(38.1)/0.60 = 3372 in.^{4}

Using horizontal deflection criterion of L/400,

$(\Delta)_{allow.}$ = 30(12)/400 = 0.9 in.

$I_{y\text{-}y\ req'd}$. (for top flange) = 53.1(2.53)/0.90

= 149 in.^{4}

<u>Calculate M_x and M_y:</u>

Assume the girder and rail weight = 148 plf

M_x (including impact) = 473.85 kip-ft

= 9.60(38.1)(1.25) + 0.148(30)^2/8

M_y = 9.60(2.53) = 24.29 kip-ft

For the tension flange: F_{bx} = 30 ksi, (0.60 F_y)

$(S_1)_{req'd.}$ = 472.26(12)/30 = 188.90 in.^{3}

For the compression flange:

Using Table 1 from Appendix A, try a W27×94 with a C15×33.9 cap channel.

$I_{x\text{-}x}$ = 4530 in.^{4} > 3372 in.^{4}

$I_{y\text{-}y}$ for top flange and cap channel = 377 in.^{4} > 149 in.^{4}

S_1 = 268.0 in.^{3}

S_2 = 435.0 in.^{3}

S_t = 50.25 in.^{3}

<u>Check bending about the x-axis:</u>

$$(f_{bx})_{tension} = \frac{M_x}{S_1} = \frac{473.85(12)}{268.0} = 21.2 \text{ ksi}$$

$$(F_{bx})_{tension} = 0.6F_y = 30 \text{ ksi} > 21.2 \text{ ksi}$$

$$(f_{bx})_{compression} = \frac{M_x}{S_2} = \frac{473.85(12)}{435.0} = 13.1 \text{ ksi}$$

From Table 2 of Appendix A, it can be seen that F_{bx} is not equal to 0.60 F_y, thus the lateral torsional allowable stress must be calculated.

From Table 2, r_T = 4.465, and d/A_f = 1.57. Therefore, L/r_T = 360/4.46 = 80.6. Based on this L/r, Equation F1-6 is the appropriate equation to calculate F_b, along with Equation F1-8. From Equation F1-6, F_b = 22.7, and from Equation F1-8 F_b = 21.2, thus F_b = 22.7.

<u>Check biaxial bending in the top flange.</u>

f_{by} = M_y/S_t = 24.29(12)/50.25 = 5.80 ksi

Maximum combined bending stress:

$$\frac{M_x/S_x}{F_{bx}} + \frac{M_y/S_t}{F_{by}} \le 1.0$$

$$\frac{13.07}{22.71} + \frac{5.80}{(0.75)50} = 0.58 + 0.15 = 0.73 \text{ o.k.}$$

<u>Check web sidesway buckling.</u>

Using Equation K1-7 from the AISC *ASD Specification*,

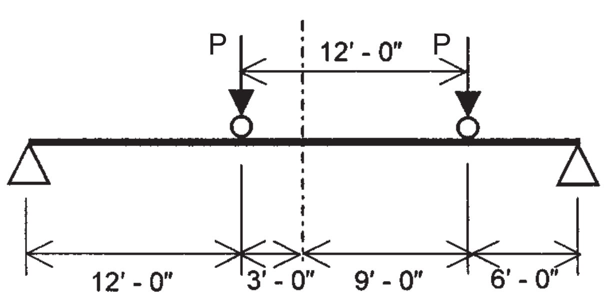


Fig. 18.1.1 Wheel Load Location for Bending

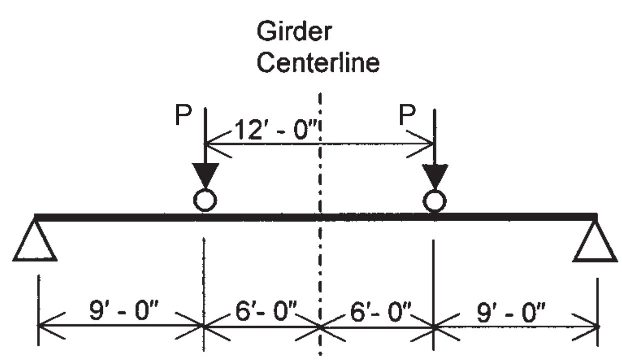


Fig. 18.1.2 Wheel Load Location for Deflection

$$R_{allow.} = \frac{6800 t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{L/b_f}\right)^3\right]$$

$$R_{allow.} = \frac{6800(0.49)^3}{25.41}\left[0.4\left(\frac{24.2/0.49}{360/9.99}\right)^3\right]$$

= 32.5 kips.

Maximum wheel load with impact = 38.1 (1.25)
= 46.9 kips

46.9 ≥ 32.5 kips

Calculations show a W30×99 w/MC18×42.7 or a W27×146 beam to be adequate. See the comments on web sidesway buckling at the end of Example 18.1.2.

Use a W30×99 w/MC18×42.7 or a W27×146. The W27×146 is obviously the more economical choice.

As a matter of interest the longitudinal tractive force for this example is 7.62 kips. Based on the area of the W27×146 the stress level in the beam due to the tractive force is 0.176 ksi.

EXAMPLE 18.1.2:

Crane Runway Girder Design (LRFD)

Same criteria as used in Example 18.1.1 but design check is per the AISC *LRFD Specification*.

Calculate factored loads from ASCE7-02.

The load factors that are currently proposed for crane loads are as follows:

Bridge weight: Load Factor = 1.2

Trolley, hoist weight and lifted load: Load Factor = 1.6

For the crane used in this example, the factored wheel loads are calculated as follows:

$P_{factored}$ = P_{bridge} (1.2) + $P_{trolley\ +\ lifted\ load}$ (1.6)

P_{bridge} = 57.2/4 = 14.3 kips/wheel (38.1 kips included bridge weight)

$P_{trolley\ +\ lifted\ load}$ = 38.1-14.3 = 23.8 kips.

For vertical loads,

$P_{factored}$ = 14.3(1.2)+23.8(1.6)
= 55.2 kips/wheel.

For horizontal loads,

$P_{factored}$ = (10.6+40)(1.6)(0.20)/4
= 4.05 kips/wheel.

The deflection criteria is based on working loads and therefore is the same as calculated for Example 18.1.1.

Calculate factored M_x and M_y

Assuming the girder weight to be 148 plf, the factored moments including impact are calculated as:

$(M_x)_{factored}$ = 9.60(55.2)(1.25)+0.148(30)^2(1.2)/8
= 682.9 kip-ft

$(M_y)_{factored}$ = 9.60(4.05) = 38.9 kip-ft

Investigate the W27×94 w/C15×33.9 section reviewed in the ASD solution.

Check bending about the x-axis.

For $L_b \le L_p$, $M_n = M_p = F_y Z$

For $L_p < L_b \le L_r$,

$$M_n = C_b\left[M_p - \left(M_p - M_r\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

For $L_b > L_r$, $M_n = M_{cr}$

$$L_p = 1.76\sqrt{\frac{E}{F_{yf}}}r_y$$

Use r_y of the compression flange as calculated below:

$(I_{y\text{-}y})_{comp.\ flg.}$ = 377 in.^{4}

$A_{comp.\ flg.}$ = 9.99(0.745)+9.96 = 17.4 in.^{2}

$$(r_{y\text{-}y})_{comp.flg.} = \sqrt{\frac{377}{17.40}} = 4.65 \text{ in.}$$

Therefore, $L_p = 1.76\sqrt{\dfrac{29000}{50}}4.65 = 197.0$ in.

L_b = 30(12) = 360 in. > L_p

Therefore, $M_n < M_p$.

The value of L_b that equates M_{cr} from the AISC *LRFD Specification* (Table A-F1.1) to $F_L S_{xc}$ can be found iteratively. The author tried several iterations and found that L_b equals 440 in. The last iteration is shown below.

Since the channel cap is welded to top flange, use

F_r = 16.5 ksi.

$F_L S_{xc}$ = 435.6(50-16.5) = 14591 in.-kips.

For this shape, the pertinent geometric properties are as follows:

I_y = 439 in.^{4}

J = 4.03+1.02 = 5.05 in.^{4}

I_{yc} = 377 in.^{4}

$h \quad \cong \quad 24$ in.

$C_b \quad = \quad 1.0$

$$B_1 = 2.25\left[2\left(I_{yc} / I_y\right) - 1\right]\left(\frac{h}{L_b}\right)\sqrt{\frac{I_y}{J}}$$

$$B_1 = 2.25\left[2\left(377 / 439\right) - 1\right]\left(\frac{24}{440}\right)\sqrt{\frac{439}{5.05}} = 0.821$$

$$B_2 = 25\left(1 - \frac{I_{yc}}{I_y}\right)\left(\frac{I_{yc}}{J}\right)\left(\frac{h}{L_b}\right)^2$$

$$B_2 = 25\left(1 - \frac{377}{439}\right)\left(\frac{377}{5.05}\right)\left(\frac{24}{440}\right)^2 = 0.784$$

$$M_{cr} = \frac{2EC_b}{L_b}\sqrt{I_y J}\left(B_1 + \sqrt{1 + B_2 + B_1^2}\right) \leq M_p$$

$$= \frac{58000}{440}\sqrt{439(5.05)}\left(0.821 + \sqrt{1 + 0.784 + (0.821)^2}\right)$$

$= 14826$ in.– kips ≈ 14591 in.– kips.

Thus $L_r = 440$ in.

M_r equals the minimum of $F_L S_{xc} = 14591$ in. kips, and $F_y\, S_{xt} = 50(267.8) = 13391$ in.–kips. Thus $M_r = 13391$ in.–kips.

$$M_n = C_b\left[M_p - \left(M_p - M_r\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$$

From table values in the *LRFD Manual of Steel Construction*, $Z_x = 357$ in.^{3}

$M_p \quad = \quad F_y\, Z_x = 50(357) = 17850$ in.–kips

$M_r \quad = \quad 13391$ in.–kips, $L_b = 360$ in., $L_r = 440$ in.

$$L_p = 1.76\sqrt{\frac{E}{F_y}}r_y = 1.76\sqrt{\frac{29000}{50}}(4.65) = 197.0 \text{ in.}$$

$$M_n = 1.0\left[17850 - (17850 - 13391)\left(\frac{360 - 197}{440 - 197}\right)\right]$$

$= \quad 14859$ in.–kips

$\phi M_{nx} = \quad 0.90(14859) = 13373$ in.–kips

$M_{ux} \quad = \quad 679.28(12) = 8151$ in.–kips < 13373 kip–in.

Check biaxial bending in the top flange.

Equation H1-1b of the Specification provides the appropriate check for this load case.
Equation H1-1b:

$$\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$$

$\phi = 0.90$

M_{ny} is determined from the plastic moment capacity of the channel cap plus the top flange of the W shape.

$M_p = M_{p\ channel} + M_{p\ top\ flange}$

$M_p = 50.8(50) + 38.8/2(50) = 3510$ in. – kips.

Continuously braced, thus $M_{ny} = M_p$.

$\phi M_n = 0.90(14859) = 13373$ in. – kips.

$$\frac{8148}{13373} + \frac{38.88(12)}{3159} = 0.757 \ < \ 1.0 \text{ o.k.}$$

Check web sidesway buckling.

Using Equation K1-7 from the AISC *LRFD Steel Specification*, for the compression flange not restrained against rotation:

$$\frac{h/t_w}{L / b_f} = 1.37 < 1.7$$

Thus,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{L/b_f}\right)^3\right]$$

$M_{ux} = \quad 8151$ in. – kips $< M_y = S_{xt}F_y = 13391$ in. – kips

Thus,

$C_r \quad = \quad 960000$ ksi

$t^w \quad = \quad 0.490$ in.

$t_f \quad = \quad 0.745$ in.

$h \quad = \quad 24.2$ in.

$$R_n = \frac{960000(0.490)^3(0.745)}{(24.2)^2}\left[0.4(1.36)^3\right] = 148 \text{ kips}$$

$\phi \quad = \quad 0.85$, Therefore,

$\phi R_n = \quad 0.85(148) = 125.9$ kips.

Maximum factored wheel load with impact equals $55.2(1.25) = 69$ kips < 125.9 kips.

$69.0 < 94.8$

The W27×94 w/C15×33.9 is adequate based on LRFD check.

Calculations also show a W24×131 beam to be adequate using an LRFD design.

Use a W30×94 w/ C15×33.9 or a W24×131. The W24×131 plain beam would be the most economical solution.

It should be noted that the *ASD Specification* currently does not have a comparable increase in allowable concentrated load for the web sidesway buckling check when flexural stress in the web is less than 0.6 F_y. Therefore, there is an inconsistency in the two specifications (ASD and LRFD) with the *ASD Specification* providing more conservative criteria. Although it is generally not recommended that ASD and LRFD design criteria be mixed, since web sidesway buckling is an independent limit state, it seems reasonable that crane runways designed using ASD procedures can be checked using LRFD equations for web sidesway buckling. For the examples presented, the W27×94 w/C15×33.9 would work for the ASD design if the LRFD web sidesway buckling equations were used.

18.2 Plate Girders

Plate girder runways can be designed in the same manner as rolled sections, but the following items become more important to the design.

1. Plate girder runways are normally used in mill buildings where many cycles of load occur. Since they are built-up sections, fatigue considerations are extremely important.

2. Welding stiffeners to the girder webs may produce a fatigue condition that would require reduction in stress range (Reemsynder, 1978). Thickening the girder web so that stiffeners are not required (except for the bearing stiffeners which are located at points of low flexural stress) may provide a more economical solution. However, in recent years, numerous cases of fatigue cracks at the junction of the top flange of the girder and the web have been noted. These cracks have been due to:

    a. The rotation of the top flange when the crane rail was not directly centered over the web. (See Figure 18.2.1.)

    b. The presence of residual stresses from the welding of the flange and stiffeners to the web.

    c. Localized stresses under the concentrated wheel loads.

The presence or absence of stiffeners affects problems a. and c. If intermediate stiffeners are eliminated or reduced, the problem of eccentric crane rail location becomes more severe. If intermediate stiffeners are provided, full penetration welds should be used to connect the top of the stiffener to the underside of the top flange. At the tension flange the stiffeners should be terminated not closer than 4 times nor more than 6 times the web thickness from the toe of the web-to-flange weld.

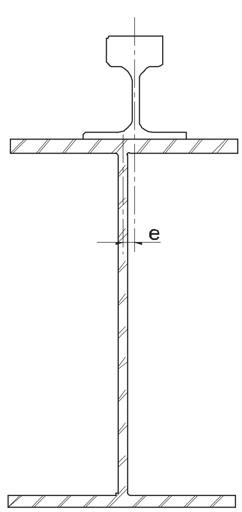


Fig. 18.2.1 Rail Misalignment

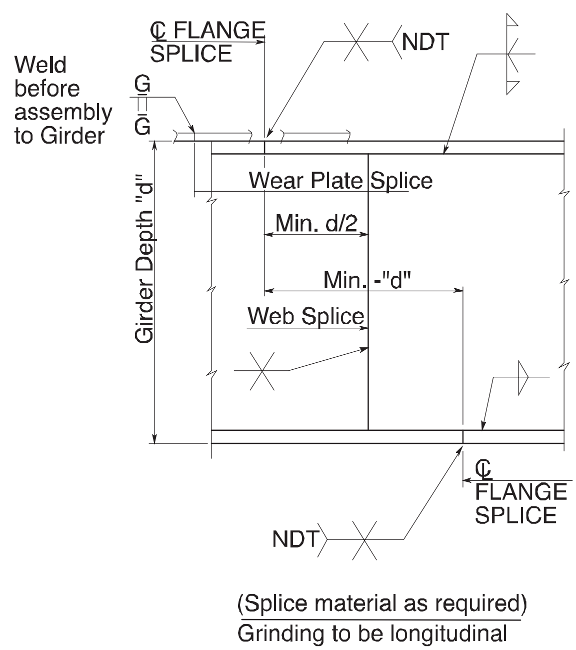


Fig. 18.2.2 Girder Splice

Shown in Figures 18.2.2 through 18.2.7 are details which pertain to heavy crane runway installations. The tension rod shown in Figure 18.2.7 provides an additional load (other than the bolts in combined tension and shear) path for the stop forces and may be a good detail to use with high-speed cranes.

The difference in weld and stiffener detailing between older AISC publications and the stiffeners shown here are generally the result of revised detailing techniques for fatigue conditions.

18.3 Simple Span vs. Continuous Runways

The decision as to whether simple span or continuous crane girders should be used has been debated for years. Following is a brief list of advantages of each system. It is clear that each can have an application.

1. Advantages of Simple Span:

   a. Much easier to design for various combinations of loads.

   b. Generally unaffected by differential settlement of the supports.

   c. More easily replaced if damaged.

   d. More easily reinforced if the crane capacity is increased.

2. Advantages of Continuous Girders:

   a. Continuity reduces deflections that quite often control.

   b. End rotations and movements are reduced.

   c. Result in lighter weight shapes and a savings in steel cost when fatigue considerations are not a determining factor.

Continuous girders should not be used if differential settlement of the supports is of the magnitude that could cause damage to the continuous members. Also, when continuous

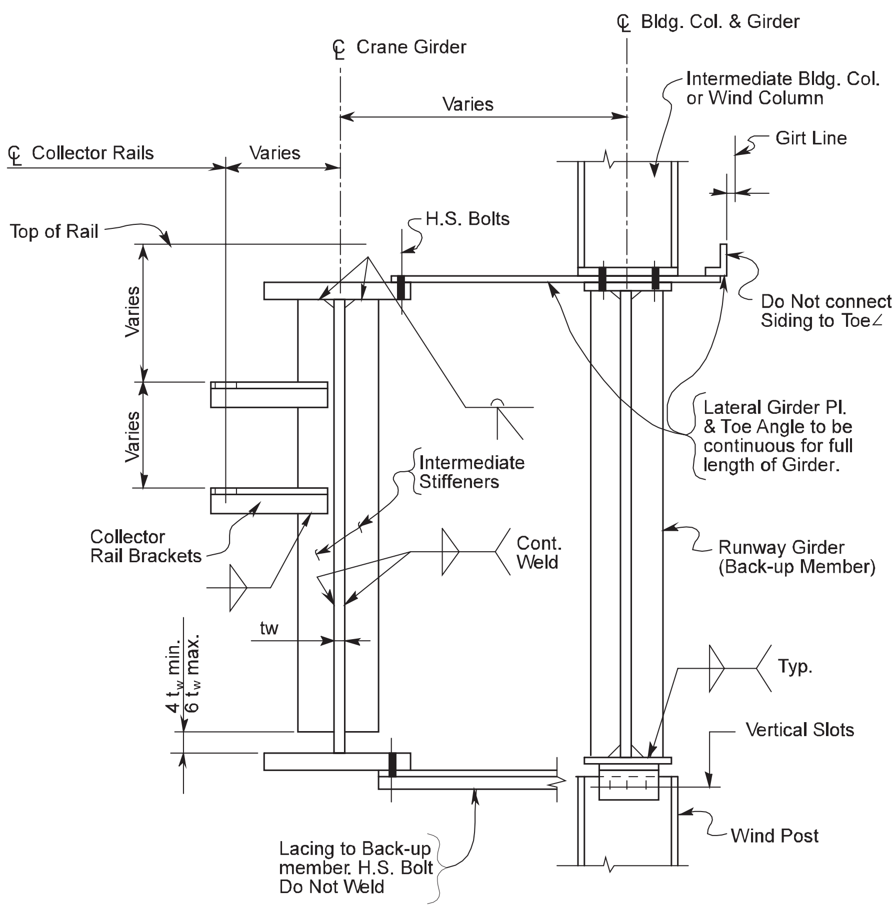


Fig. 18.2.3 Crane Runway Girder Detail

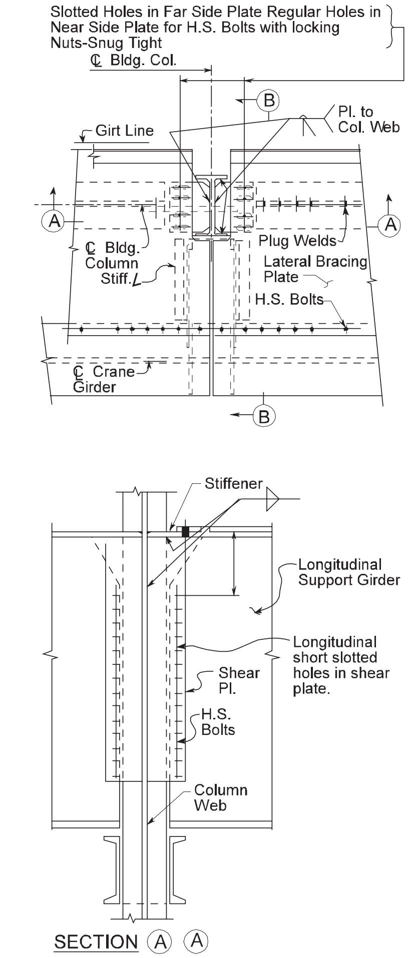


Fig. 18.2.4 Detail at Ends of Crane Girders

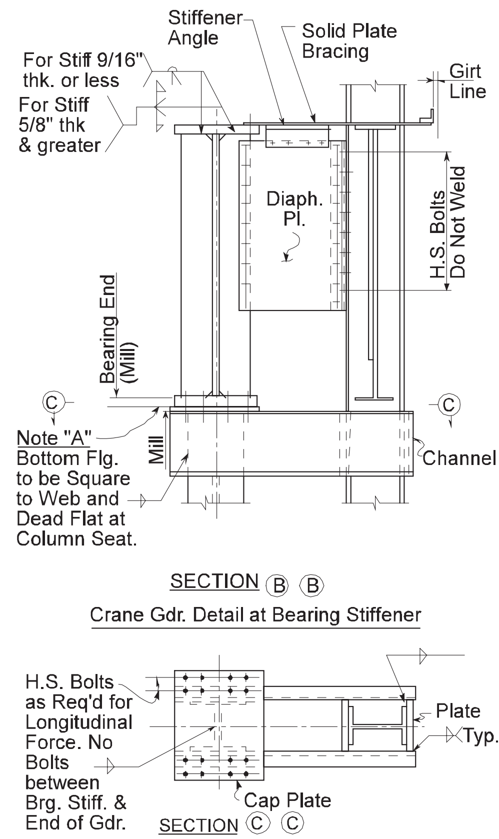


Fig. 18.2.5 Sections A and C of Fig. 18.2.4

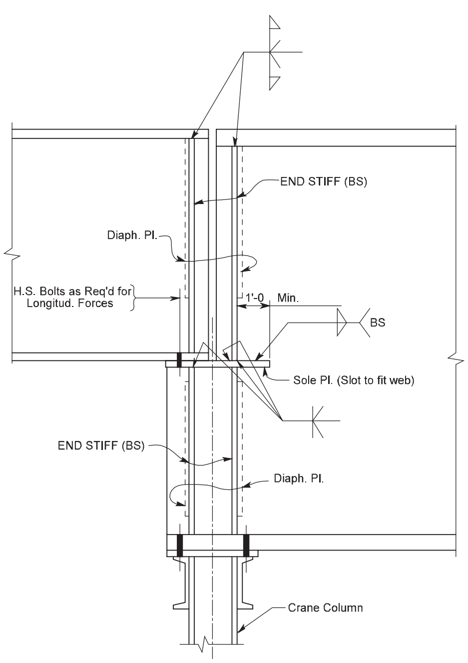


Fig. 18.2.6 Section at Different Depth Crane Girders

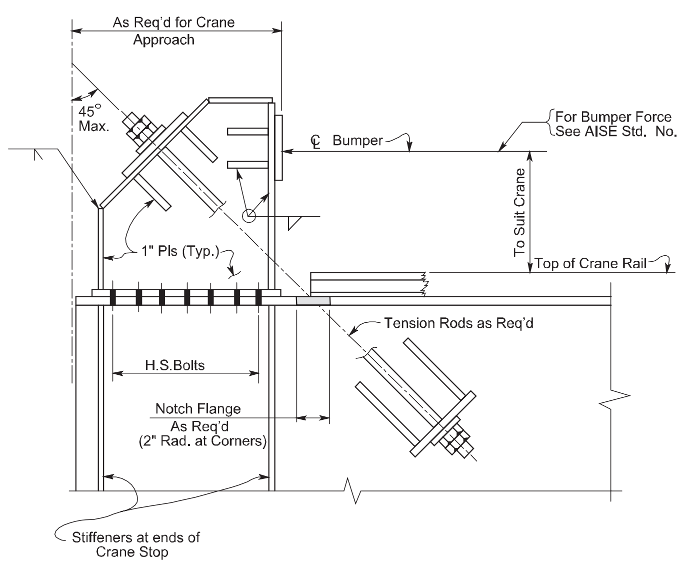


Fig. 18.2.7 Heavy Duty Crane Stop

girders are subjected to fatigue loading and have welded attachments on the top flange (rail clips) the allowable stress range is reduced considerably. Any advantage therefore may be eliminated.

Shown in Figure 18.3.1 are the results of several runway designs for spans from 20 ft to 30 ft. A36 and 50 ksi steel designs were made for a 4-wheel, 10T crane, with a 70¢ bridge for continuous (two span) vs. simple span conditions. In these examples, deflection did not control. Fatigue was not considered.

The curves represent (in general) the trends for heavier cranes as well. In general, two span continuous crane girders could save about 18 percent in weight over simply supported girders.

18.4 Channel Caps

Use of channel caps is normally required to control lateral deflections and to control the stresses due to lateral loads. For light duty-lightweight cranes (less than 5T) channel caps may not be required. Studies have found that a steel savings of approximately 25 lb/ft is required to justify the cost of welding a cap to a structural shape.

18.5 Runway Bracing Concepts

An excellent paper on the subject of bracing of crane girders is that of Mueller (Mueller, 1965). Several significant (and common) considerations that need to be emphasized are:

1. As illustrated in Figure 2 in the Mueller paper (repeated here as Figure 18.5.1), improper detailing at the end bearing condition could lead to a web tear in the end of the crane girder. The detail shown in Figure 18.5.2 has been used to eliminate this problem for light crane systems. The details shown in Figures 18.2.3 and 18.2.4 would represent a similar detail for heavy cranes. Use of this detail allows the end rota-

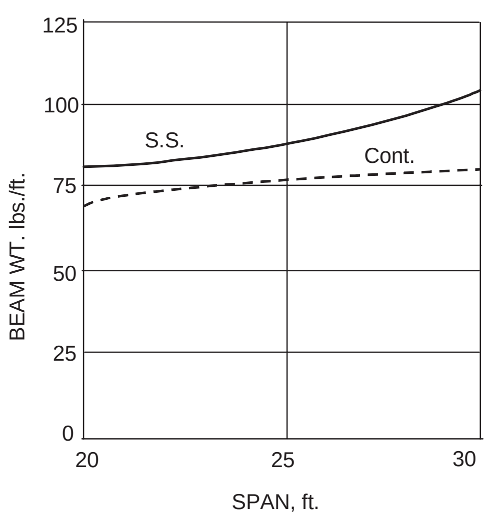


Fig. 18.3.1 Runway Designs

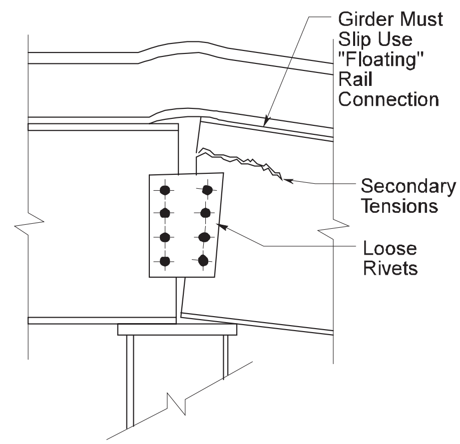


Fig. 18.5.1 Improper Girder Connection Detail

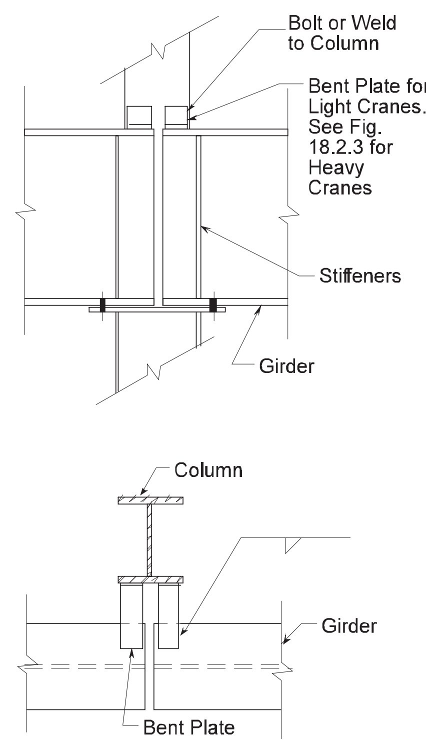


Fig. 18.5.2 Proper Tie Back Detail

tion and yet properly transfers the required lateral forces into the column.

2. A common method of bracing the crane girder is to provide a horizontal truss (lacing) or a horizontal plate to connect the crane girder top flange to an adjacent structural member as previously illustrated in Figures 17.2.1 and 17.2.2.

   A critical consideration in the use of this system is to have the lacing flexible in the vertical direction, enabling the crane girder to freely deflect relative to the structural member to which it is attached. If the lacing is not flexible, stresses will be produced which could cause a fatigue failure of the lacing system, thereby losing the lateral support for the girder.

3. AISE *Technical Report No. 13* requires that girders more than 36 ft in length have the bottom flange braced by a horizontal truss system. Where compliance with AISE *Technical Report 13* is not required, many engineers have used a bottom flange channel to brace the flange on long spans (perhaps 40 ft or more). The origin of this requirement is not obvious; however, it appears that compliance with the AISC sidesway web buckling equations may analytically satisfy this requirement.

4. Occasionally two parallel crane girders are connected by a top plate to "mutually brace each other." This, of course, results in a very stiff girder in terms of lateral load. Also, the plate can be used as a walkway for maintenance purposes. When tied together the loading and unloading of parallel girders can cause a fatigue failure of the bracing plate unless it is properly detailed. The interconnecting plate must be flexible to allow differential deflections between two girders.

18.6 Crane Stops

The end section of a crane runway must be designed for a longitudinal force applied to the crane stops. For spring type bumper blocks the longitudinal crane stop force may be calculated from the following formula.

$$F = \frac{WV^2}{ge_t}$$

where

W = total weight of crane exclusive of lifted load.
V = specified crane velocity at moment of impact, *ft/s* (required by AISE Technical Report No. 6 to be 50 percent of full load rated speed.
e_x = stroke of spring at point where the crane stopping energy is fully absorbed, ft.
F = total longitudinal inertia force acting at the elevation of the center of mass of the bridge and the trolley. The force on each runway stop is the maximum bumper reaction from the inertia force acting at such locations.
g = acceleration of gravity, 32.2 ft/s^2.

For bumper blocks of wood or rubber (commonly found in older cranes) the above equation is not directly applicable. Manufacturer's literature or experience must be used for such installations. In the absence of specific data, it is recommended that the designer assume the bumper force to be the greater of:

1. Twice the tractive force, or

2. Ten percent of the entire crane weight.

For calculations relative to bumper forces for hydraulic crane bumpers, the reader should refer to AISE *Technical Report No. 13*, (AISE, 2003).

18.7 Crane Rail Attachments

There are four general types of anchoring devices used to attach crane rails to crane runway beams. These types are hook bolts, rail clips, rail clamps and patented clips. Details of hook bolts and rail clamps are shown in the AISC Manual.

18.7.1 Hook Bolts

Hook bolts provide an adequate means of attachment for light rails (40 lb - 60 lb) and light duty cranes (CMAA 70 Classes A, B and C). The advantages of hook bolts are: 1. They are relatively inexpensive, 2. There is no need to provide holes in the runway beam flange and 3. It is easy to install and align the rail. They are not recommended for use with heavy-duty cycle cranes (CMAA 70 Classes D, E and F) or with heavy cranes (greater than 20 ton lifting capacity), because hook bolts are known to loosen and/or elongate. It is generally recommended that hook bolts should not be used in runway systems that are longer than 500 ft because the bolts do not allow for longitudinal movement of the rail. Because hook bolts are known to loosen in certain applications, a program of periodic inspection and tightening should be instituted for runway systems using hook bolts. Designers of hook bolt attachments should be aware that some manufacturers supply hook bolts of smaller than specified diameter by the use of upset threads.

18.7.2 Rail Clips

Rail clips are forged or cast devices that are shaped to match specific rail profiles. They are usually bolted to the runway girder flange with one bolt or are sometimes welded. Rail clips have been used satisfactorily with all classes of cranes. However, one drawback is that when a single bolt is used the clip can rotate in response to rail longitudinal movement. This clip rotation can cause a camming action, thus forcing the rail out of alignment. Because of this limitation rail clips should only be used in crane systems subject to infrequent use, and for runway systems less than 500 ft in length.

18.7.3 Rail Clamps

Rail clamps are a common method of attachment for heavy-duty cycle cranes. Rail clamps are detailed as one of two types: tight or floating. Each clamp consists of two plates: an upper clamp plate and a lower filler plate.

The lower plate is flat and roughly matches the height of the toe of the rail flange. The upper plate covers the lower plate and extends over the top of the lower rail flange. In the tight clamp the upper plate is detailed to fit tight to the lower rail flange top, thus "clamping" it tightly in place when the fasteners are tightened. In the past, the tight clamp had been illustrated with the filler plates fitted tightly against the rail flange toe. This tight fit-up was rarely achieved in practice and is not considered to be necessary to achieve a tight type clamp. In the floating type clamp, the pieces are detailed to provide a clearance both alongside the rail flange toe and below the upper plate. The floating type does not in reality clamp the rail but merely holds the rail within the limits of the clamp clearances. High-strength bolts are recommended for both clamp types.

Tight clamps are generally preferred and recommended by crane manufacturers because they feel that the transverse rail movement allowed in the floating type causes accelerated wear on crane wheels and bearings.

Floating rail clamps may be required by crane runway and building designers to allow for longitudinal movement of the rail thus preventing (or at least reducing) thermal forces in the rail and runway system.

Because tight clamps prevent longitudinal rail movement, they should not be used in runways greater than 500 ft in length. Since floating rail clamps are frequently needed and crane manufacturers' concerns about transverse movement are valid, a modified floating clamp is required. In such a clamp it is necessary to detail the lower plate to a closer tolerance with respect to the rail flange toe. The gap between lower plate edge and flange toe can vary between snug and a gap of ⅛ in. The ⅛-in. clearance allows a maximum of ¼ in. float for the system. This should not be objectionable to crane manufacturers since this amount of float is within normal CMAA 70 tolerances for crane spans in the range of 50 ft-100 ft, in other words, spans usually encountered in general construction. In order to provide field adjustment for variations in the rail width, runway beam alignment, beam sweep and runway bolt hole location, the lower plate can be punched with its holes off center so that the plate can be flipped to provide the best fit. An alternative would be to use short slotted or oversize holes. In this case one must rely on bolt tightening to clamp the connection so as to prevent filler plate movement.

Rail clamps are generally provided with two bolts per clamp. Two bolts are desirable in that they prevent the camming action described in the section on forged or cast rail clips. A two-bolted design is especially recommended if clamps of the longitudinal expansion type described above are used. Rails are sometimes installed with pads between the rail and the runway beam. When this is done the lateral float of the rail should not exceed $1/32$ in. to reduce the possibility of the pads being worked out from under the rail.

18.7.4 Patented Rail Clips

This fourth type of anchoring device covers various patented devices for crane rail attachment. Each manufacturer's literature presents in detail the desirable aspects of the various designs. In general they are easier to install due to their greater range of adjustment while providing the proper limitations of lateral movement and allowance for longitudinal movement. Patented rail clips should be considered as a viable alternative to conventional hook bolts, clips or clamps. Because of their desirable characteristics patented rail clips can be used without restriction except as limited by the specific manufacturer's recommendations. Installations using patented rail clips sometimes incorporate pads beneath the rail. When this is done the lateral float of the rail should be limited as in the case of rail clamps.

18.7.5 Design of Rail Attachments

The design of rail attachments is largely empirical. The selection of the size and spacing of attachments is related to rail size. This relation is reasonable in that rail size is related to load.

With regard to spacing and arrangement of the attachment the following recommendations are given. Hooked bolts should be installed in opposing pairs with three to four inches between the bolts. The hook bolt pairs should not be spaced farther than two feet apart. Rail clips and clamps should be installed in opposing pairs. They should be spaced three feet apart or less.

In addition to crane rail attachment, other attachments in the form of clips, brackets, stiffeners, etc. are often attached to the crane girder. Plant engineering personnel often add these attachments. Welding should only be done after a careful engineering evaluation of its effects. Welding (including tack welding) can significantly shorten the fatigue life. Therefore:

1. Never weld crane rail to girder.

2. Clamp, screw or bolt all attachments to crane girders to avoid fatigue problems.

3. All modifications and repair work must be submitted to engineering for review and approval before work is done.

18.8 Crane Rails and Crane Rail Joints

The selection of rail relates to crane considerations (basically crane weight) and is generally made by the crane manufacturer. Once this decision is made, the principal consideration is how the rail sections are to be joined. Several methods to join rails exist but two currently dominate.

The bolted butt joint is the most commonly used rail joint. Butt joint alignment is created with bolted splice plates. These plates must be properly maintained (bolts kept tight). If splice bars become loose and misaligned joints occur, a number of potentially serious problems can result, including chipping of the rail, bolt fatigue, damage to crane wheels, and as a result of impact loading, increased stresses in the girders. Girder web failures have been observed as a consequence of this problem.

The welded butt joint, when properly fabricated to produce full strength, provides an excellent and potentially maintenance free joint. However, if repairs are necessary to the rails, the repair procedure and consequently the down time of plant operations is generally longer than if bolted splices had been used. The metallurgy of the rails must be checked to assure the use of proper welding techniques, but if this is accomplished the advantages can be significant. Principal among these is the elimination of joint impact stresses, existent in non-welded rail construction, resulting in reduced crane wheel bearing wear.

Rail joints should be staggered so that the joints do not line up on opposite sides of the runway. The amount of stagger should not equal the spacing of the crane wheels and in no case should the stagger be less than one foot.

Rail misalignment is the single most critical aspect of the development of high impact and lateral stresses in crane girders. Proper use and maintenance of rail attachments is critical in this regard. Rail attachments must be completely adjustable and yet be capable of holding the rail in alignment. Because the rails may become misaligned regular maintenance is essential to correct the problem.

One aspect of crane rail design is the use of crane rail pads. These are generally preformed fabric pads that work best with welded rail joints. The resilient pads will reduce fatigue, vibration and noise problems. Reductions in concentrated compression stresses in the web have been achieved with the use of these pads. Significant reductions in wear to the top of the girder flange have also been noted. With the exception of a few patented systems, the pads are generally not compatible with floating rail installations since they can work their way out from under the rail. Also prior to using a pad system careful consideration to the cost benefits of the system should be evaluated.

19. CRANE RUNWAY FABRICATION AND ERECTION TOLERANCES

Crane runway fabrication and erection tolerances should be addressed in the project specifications because standard tolerances used in steel frameworks for buildings are not tight enough for buildings with cranes. Also, some of the required tolerances are not addressed in standard specifications.

Tolerances for structural shapes and plates are given in the Standard Mill Practice section of the *Manual of Steel Construction* published by AISC. These tolerances cover the permissible variations in geometrical properties and are taken from ASTM Specifications, AISI Steel Product Manuals and Producer's Catalogs. In addition to these Standards, the following should be applied to crane runways.

a. Sweep: not to exceed ¼ in. in a 50-ft. beam length.

b. Camber: not to vary from the camber given on the drawing by plus or minus ¼ in. in a 50-ft. beam length.

c. Squareness: within 18 in. of each girder end the flange shall be free of curvature and normal to the girder web.

Columns, base plates and foundations should adhere to the following tolerances.

a. Column anchor bolts shall not deviate from their theoretical location by 0.4 times the difference between bolt diameter and hole diameter through which the bolt passes.

b. Column base plates: Individual column base plates shall be within ± 1⁄16 in. of theoretical elevation and be level within ± 0.01 in. across the plate length or width. Paired base plates serving as a base for double columns shall be at the same level and not vary in height from one to another by 1⁄16 in.

Crane runway girders and crane rails shall be fabricated and erected for the following tolerances.

a. Crane rails shall be centered on the centerline of the runway girders. The maximum eccentricity of center of rail to centerline of girder shall be three-quarters of the girder web thickness.

b. Crane rails and runway girders shall be installed to maintain the following tolerances.

1. The horizontal distance between crane rails shall not exceed the theoretical dimension by ± ¼ in. measured at 68 °F.

2. The longitudinal horizontal misalignment from straight of rails shall not exceed ± ¼ inch in 50 ft with a maximum of ± ½ in. total deviation in the length of the runway.

3. The vertical longitudinal misalignment of crane rails from straight shall not exceed ± ¼ in. in 50 measured at the column centerlines with a maximum of ± ½ in. total deviation in the length of the runway.

The foregoing tolerances are from the AISE *Technical Report No. 13*. The Table shown in Figure 19.1 is taken from MBMA's *Low Rise Building Systems Manual* and gives alternate tolerances.

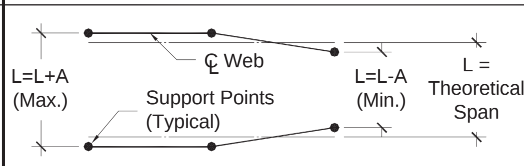
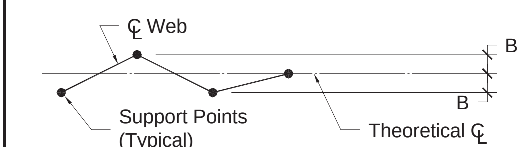
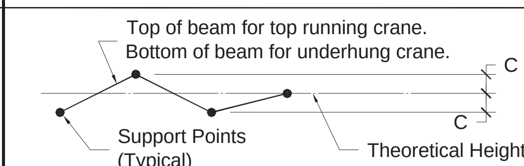
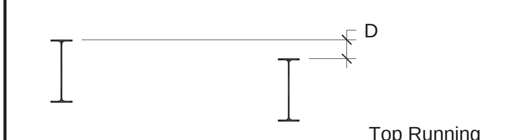
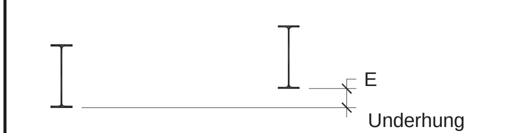
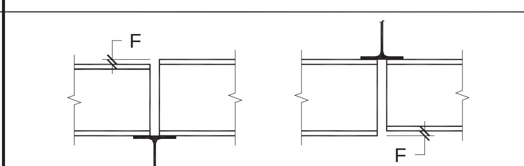
Item		Tolerance	Maximum Rate of Change
Span		A = 3/8"	1/4" / 20'
Straightness		B = 3/8"	1/4" / 20'
Elevation		C = 3/8"	1/4" / 20'
Beam to Beam Top Running		D = 3/8"	1/4" / 20'
Beam to Beam Underhung		E = 3/8"	1/4" / 20'
Adjacent Beams		F = 1/8"	N/A

Fig. 19.1 Summary of Crane Runway Tolerances

20. COLUMN DESIGN

No attempt will be made to give complete coverage of the design of steel columns. The reader is referred to a number of excellent texts on this subject (Gaylord, 1992), (Salmon, 1997).

This section of the guide includes a discussion of the manner in which a crane column can be analyzed, how the detailing and construction of the building will affect the loads the crane column receives, and how shears and moments will be distributed along its length. The guide also includes a detailed example of a crane column to illustrate certain aspects of the design.

In most crane buildings, the crane columns are statically indeterminate. Normally the column is "restrained" at the bottom by some degree of base fixity. The degree of restraint is to a large extent under the control of a designer, who may require either a fixed base or a pinned base.

It is essential to understand that the proper design of crane columns can only be achieved when column moments are realistically determined. This determination requires a complete frame analysis in order to obtain reliable results. Even if a complete computer frame analysis is employed, certain assumptions must still be made about the degree of restraint at the bottom of a column and the distribution of lateral loads in the structure. Furthermore, in many cases a preliminary design of these crane columns must be performed either to obtain approximate sizes for input into a computer analysis or for preliminary cost and related feasibility studies. Simplifying assumptions are essential to accomplish these objectives.

20.1 Base Fixity and Load Sharing

Crane columns are constructed as bracketed, stepped, laced, or battened. (See Figure 20.1.1.) In each case, the eccentric crane loads and lateral loads produce moments in the column. The distribution of column moments is one principal consideration.

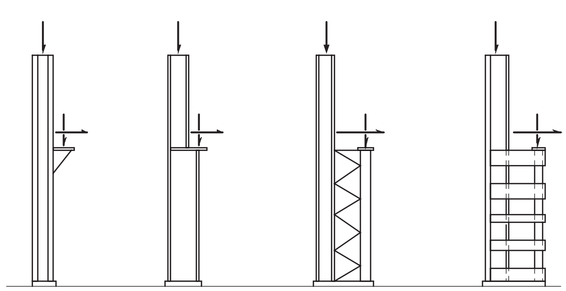


Fig. 20.1.1 Column Types

For a given loading condition, the moments in a crane column are dependent on many parameters. Most parameters (for example, geometry, nonprismatic conditions) are readily accommodated in the design process using standard procedures. However, two parameters that have a marked effect on column moments are:

1. Base fixity.

2. Amount of load sharing with adjacent bents.

As an example, refer to Figure 20.1.2. The loading consists of 100T crane (vertical crane load = 310 kips, lateral crane load to each side = 23 kips). A stepped column is used, but the same general principles apply to the other column types.

1. Base Fixity: The effect of base fixity on column moments was determined by a computer analysis for the frame for fixed and pinned base conditions. The results of the analysis shown in Figure 20.1.3 demonstrate that a simple base will result in extremely large moments in the upper portion of the column and the structure will be much more flexible as compared to a fixed base column.

   For fixed base columns the largest moment is carried to the base section of the column where it can, in the case of the stepped column, be more easily carried by the larger section.

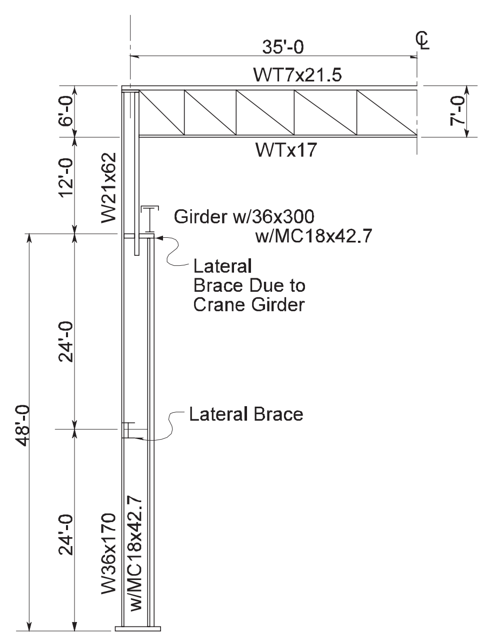


Fig. 20.1.2 Example Frame

It is frequently argued that taking advantage of full fixity cannot be achieved in any practical detail. However, the crane induced lateral loads on the crane column are of short duration, and for such short term loading an "essentially fixed" condition can normally be achieved through proper design. The reduced column moments (22 percent in the previous example) due to the fixed base condition provide good economy without sacrificing stiffness.

There will be cases where subsoil conditions, existing construction restrictions, property line limitations, etc., will preclude the development of base fixity and the hinged base must be used in the analysis. Although the fixed base concept as stated is deemed appropriate due to short term nature of crane loadings, for other long duration building loads the assumption of full fixity may be inappropriate. The reader is referred to an excellent article by Galambos (Galambos, 1960) that deals with the effects of base fixity on the buckling strength of frames.

2. Load Sharing To Adjacent Bents: If a stiff system of bracing is used (in other words, a horizontal bracing truss as shown in Figure 20.1.4) then the lateral crane forces and shears can be distributed to adjacent bents thereby reducing column moments. Note that such bracing does not reduce column moments induced by wind, seismic or roof loads but only the singular effects of crane loads. Figure 20.1.5 depicts the moment diagram in the column from a frame analysis based on lateral crane loads being shared by the two adjacent frames (in other words, two-thirds of the lateral sway force is distributed to other frames). The significant reductions in moment are obvious when compared to Figure 20.1.3. (Note the "two-thirds" is an arbitrary distribution used at this point only to illus-

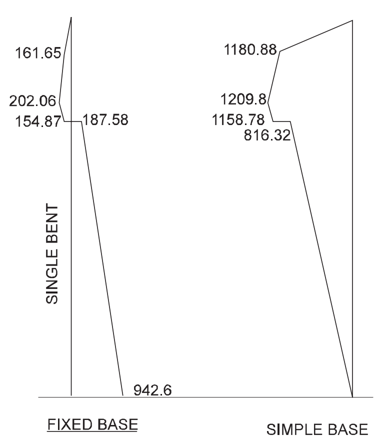


Fig. 20.1.3 Analysis Results

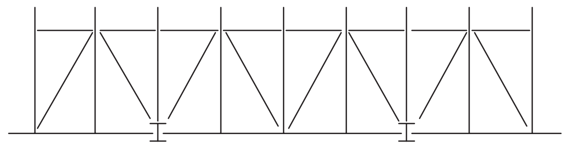


Fig. 20.1.4 Horizontal Bracing

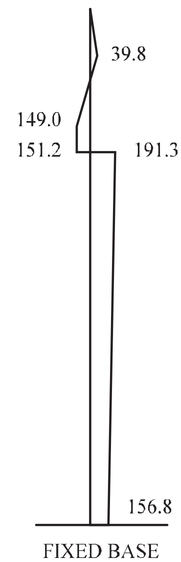


Fig. 20.1.5 Moment Diagram with Load Sharing

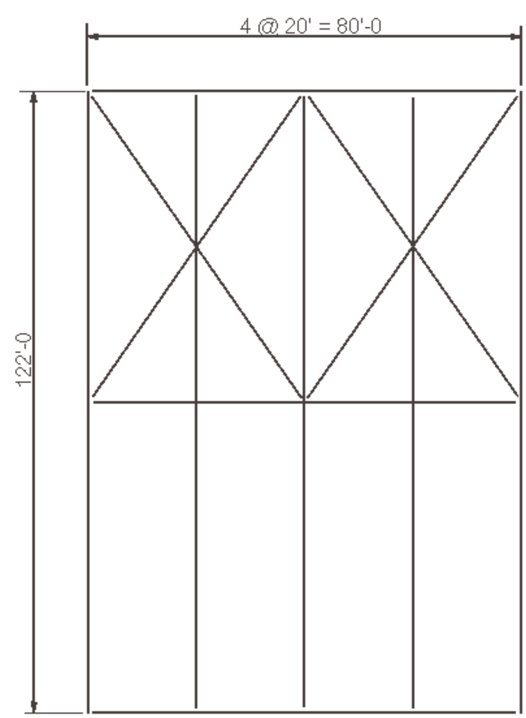


Fig. 20.1.6 Roof Portion

trate the concept and the significant advantage to be gained. The following paragraphs describe in detail how load sharing actually occurs and how it can be evaluated.

Consider a portion of a roof system consisting of five frames braced as shown in Figure 20.1.6. The lateral crane force will result in a reactive force at the level of the lower chord of the roof truss. (Figure 20.1.7.) The distribution of this reactive force to the adjacent frames can be obtained by stiffness methods. This is accomplished by analyzing the horizontal bracing system as a truss on a series of elastic supports. The supports are provided by the building frames and have linear elastic spring constants equal to the reciprocal of the displacement of individual frames due to a unit lateral load (Figure 20.1.8). The model is depicted in Figure 20.1.9. The springs are imaginary members that provide the same deflection resistance as the frames.

This procedure has been programmed and analyzed for many typical buildings. It is obvious that the degree of load sharing varies, and is dependent upon the relative stiffness of the bracing to the frames; however, it was found that for usual horizontal bracing systems a lateral load applied to a single interior frame will be shared almost equally by at least five frames. This is logical because bracing of reasonable proportions made up of axially loaded members is many times as stiff as the moment frames which are dependent upon the bending stiffness of their components.

A building supporting a 100-ton crane is used to illustrate the effect of load sharing. A roof system consisting of five frames X-braced as shown in Figure 20.1.6 was analyzed to determine the force in each frame due to a 20 kip force applied to the center frame. This 20 kips represents the reactive force at the elevation of the bottom chord bracing due to a horizontal crane thrust at the top of the crane girder as illustrated in Figure 20.1.7. The final distribution is shown in Figure 20.1.10.

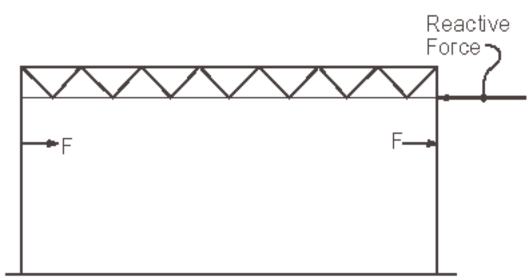


Fig. 20.1.7 Reactive Force

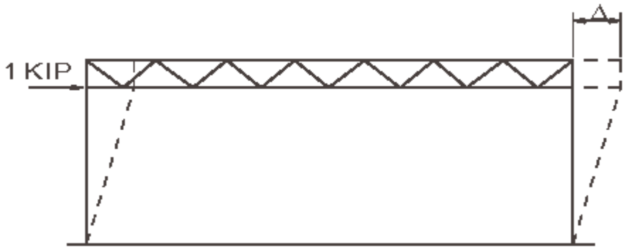


Fig. 20.1.8 Unit Lateral Load

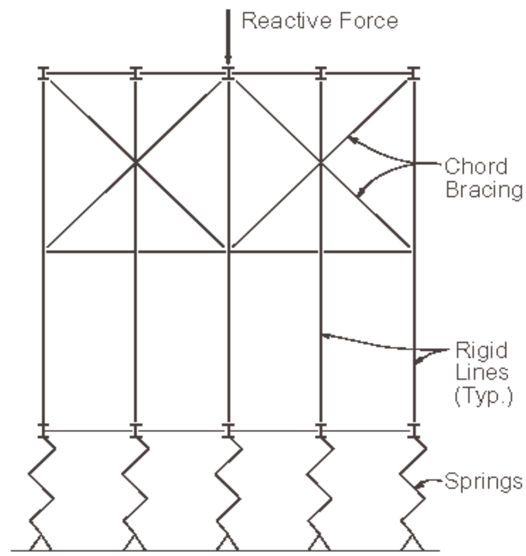
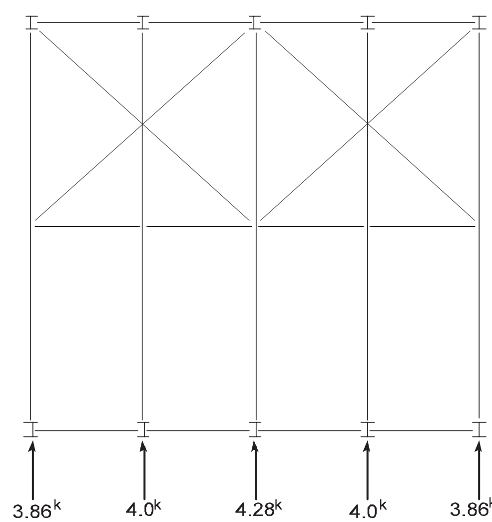


Fig. 20.1.9 Computer Model



20.1.10 Final Force Distribution

Even though reasonable truss type bracing will distribute a concentrated lateral force to at least 5 frames, it is recommended that load sharing be limited to 3 frames (the loaded frame plus the frame to either side). The reason for this conservative recommendation is that unless pretensioned the horizontal bracing truss members may tend to sag even though "draw" is provided. Thus, a certain amount of movement may occur before the truss "takes up" and becomes fully effective in distributing the load to adjacent frames.

Some designers may assume that if load sharing occurs a simple method to handle the analysis is to design a given column for one-third the lateral load, but such an assumption is wrong and unsafe! Each individual crane column must be designed for the full lateral force of the crane. It is only the reactive force applied at the level of the bracing that is distributed to the adjacent frames. *The results of this analysis must be added to or compared with the results of other analyses that are unaffected by the load sharing, in other words, gravity, wind, and seismic loadings.*

To summarize, the most economical designs will result when the following "assumptions" are designed into the structure:

1. Fixed base columns.

2. Horizontal bracing truss (unless wind loads control) such that lateral crane loads can be distributed to adjacent columns.

3. When the roof frames are fabricated trusses the most economical bracing truss location is at the elevation of the bottom chord where they are generally easier to erect. The bottom chord bracing system that is required for uplift and slenderness ratio control may also be adequate for distributing concentrated lateral forces.

20.2 Preliminary Design Methods

Preliminary design procedures for crane columns are especially helpful due to the complexity of design of these members. Even with the widespread availability of computers a good preliminary design can result in substantial gains in overall efficiency. The preceding sections of this guide have pointed out the fact that in order to obtain meaningful column moments a frame analysis is required. A reliable hand calculation method for preliminary design is not only helpful but also essential in order to reduce final design calculation time.

A frame analysis to obtain an "exact" solution will contain the following:

1. It accounts for sidesway.

2. It properly handles the restraint at the top and at the base of the column.

3. It accounts for non-prismatic member geometry.

A preliminary design procedure requires a method of analysis that will provide suitable column stiffness estimates so that an "exact" indeterminate frame analysis procedure need be conducted only once. The model given in Figure 20.2.1 has been found to give remarkably good results for crane loadings, providing horizontal bracing is used in the final design. It is a "no-sway" model, consisting of a fixed base, and supports introduced at the two points where the truss chords intersect the column.

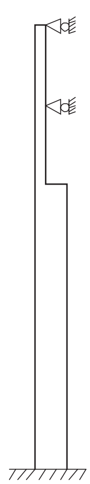


Fig. 20.2.1 No-Sway Computer Model

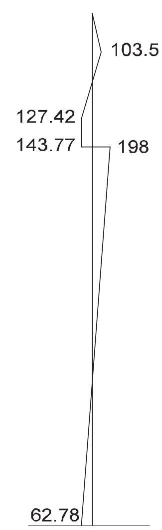


Fig. 20.2.2 Results of No-Sway Model

A moment diagram obtained from the "no-sway" model for the 100-ton crane column previously shown in Figure 20.1.2 is shown in Figure 20.2.2.

Comparing Figure 20.2.2 to Figure 20.1.5 it can be seen that the general moment configuration is similar, and the magnitudes of moments are almost identical. For preliminary design purposes the two-support "no-sway" model is adequate. The two-support no-sway model is statically indeterminate to the second degree. Thus, even a preliminary design requires a complex analysis and certain other assumptions.

The preliminary design procedure for wind or seismic loadings can usually be made by assuming an inflection point and selecting preliminary column size to control sway under wind loads. An appropriate procedure is shown in the bracketed crane column design example in the next section.

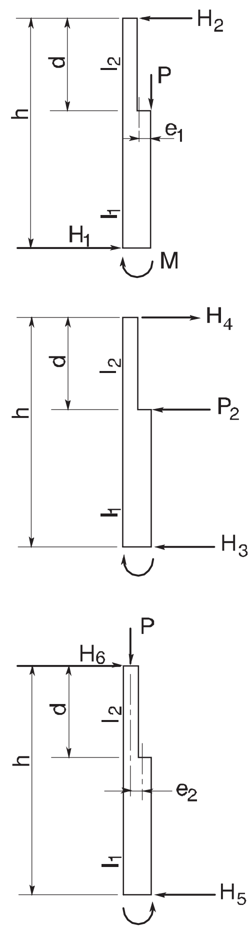
The sizes of bracketed columns are often controlled by wind; therefore, the design should first be made for wind and subsequently checked for wind plus crane.

AISE *Technical Report 13* recommends that bracket vertical loads should be limited to 50 kips.

Stepped and laced or battened columns are another matter. To obtain accurate values for moments, the effects of the nonuniform column properties must be included in the analysis. In doing a preliminary analysis of a stepped column another assumption is practical. The assumption involves the substitution of a single top hinge support to replace the two supports in the two-support no-sway mode. The single hinged support is located at the intersection of the bottom chord and the column.

The simplified structure is depicted in Figure 20.2.3. Equations for the analysis of this member are given in Figure 20.2.4.

In each case, the equation for the top shear force is given. For the single support assumption, the indeterminacy is eliminated once this shear force is known. The moment diagram for the single hinge, no-sway column evaluated using the equations is provided in Figure 20.2.5.



$$H_2 = \frac{3Pe_1\,(h^2 - d^2)}{2I_1\left(\frac{h^3}{I_1} - \frac{d^3}{I_1} + \frac{d^3}{I_2}\right)}$$

$$H_4 = \frac{P_2\,(2h^3 - 3dh^2 + d^3)}{2I_1\left(\frac{h^3}{I_1} - \frac{d^3}{I_1} + \frac{d^3}{I_2}\right)}$$

$$H_6 = \frac{3Pe_2\,(h^2 - d^2)}{2I_1\left(\frac{h^3}{I_1} - \frac{d^3}{I_1} + \frac{d^3}{I_2}\right)}$$

Fig. 20.2.4 Equations for Simplified Structure

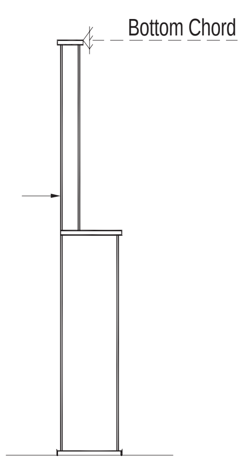


Fig. 20.2.3 Simplified Structure

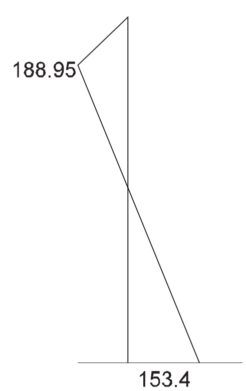


Fig. 20.2.5 Column Moments Using Fig. 20.2.4 Equations

While the variation in moment along the length is not in good agreement with that of the "exact" solution given in Figure 20.1.6, the values and signs of the moments at critical sections agree quite well.

There is one aspect of preliminary design that has not been discussed but is essential in the handling of the stepped and double column conditions. The non-prismatic nature of these column types requires input of the moment of inertia of the upper and lower segments of the column, which, of course, are not known initially. Therefore, some guidelines and/or methods are required to obtain reasonable values for I1 and I2.

20.2.1 Obtaining Trial Moments of Inertia for Stepped Columns:

The upper segment of the stepped column may be sized by choosing a column section based on the axial load acting on the upper column portion. Use the appropriate unsupported length of the column in its weak direction and determine a suitable column from the column tables contained in the AISC Manual. Select a column about three sizes (by weight) larger to account for the bending in the upper shaft.

The size of the lower segment of the stepped column may be obtained by assuming that the gravity load from the crane is a concentric load applied to one flange (or flange-channel combination). The preliminary selection may be made by choosing a member such that $P/A \cong 0.45F_y$ where A is the area of one flange or flange plus channel combination. The depth of the lower shaft is normally determined by the crane clearance requirements (see Figure 20.2.6).

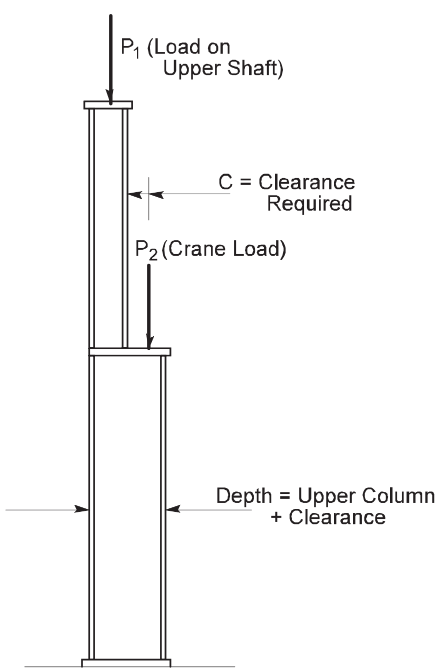


Fig. 20.2.6 Column Clearance Requirement

20.2.2 Obtaining Trial Moments of Inertia for Double Columns:

The building column portion of a double column can again best be selected based on the axial load in the building column. Select the size of the crane column based on the crane gravity load applied to the "separate" crane column. The allowable stress of this portion will normally be based on the major axis of the column assuming that the column is laced or battened to the building column to provide support about the weak axis. The actual sizes of the columns should be increased slightly to account for the bending moments. The moment of inertia of the combined sections can be calculated using standard formulas for geometrical properties of built-up cross sections. If the moment of inertia of the combined sections is obtained by assuming composite behavior, the lacing or batten plates connecting the two column sections must be designed and detailed accordingly.

20.3 Final Design Procedures (Using ASD)

After obtaining the final forces and moments in the crane column, it can be designed. The design of a crane column is unique in that the column has both a varying axial load and a "concentrated" moment at the location of the bracket or "step" in the column.

The best design approach for prismatic bracketed columns is to design the upper and lower portions of the columns as individual segments with the top portion designed for P_1 and the associated upper column moments, and the lower portion designed for $P_1 + P_2$, and the lower column moments (Figure 20.3.1). The column can normally be considered as laterally braced about the Y axis at the crane girder elevation. When considering the X axis, F_a, F'_{ex} and K should be calculated based on the entire length of the column and the properties of the cross section. C_m can be assumed to be 0.85 since each column segment is free to sway. A formal theoretical treatment of this procedure can be found in *The Design of Steel Beam-Columns*, by Peter F. Adams, published by the Canadian Steel Industries Construction Council (Adams, 1974). The best reference for the design of crane columns is contained in the AISE Technical Report No. 13, *Guide for the Design and Construction of Mill Buildings* (AISE, 2003). The AISE Report 13 procedure suggests that two equations be checked.

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left[1 - \left(f_a / F'_{ex}\right)\right] F_{bx}} + \frac{C_{my} f_{by}}{\left[1 - \left(f'_a / F'_{ey}\right)\right] F_{by}} \le 1.0$$

$$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$$

These equations are nearly identical with equations H1-1 and H1-2 of the AISC *ASD Specification* for members subjected to both axial compression and bending stresses, except for the introduction of f'_a as different from f_a. In addition, the other terms are in some cases evaluated in a manner adapted especially to the stepped-column problem.

The terms in these equations are defined as follows:

f_a = In the lower shaft $f_a = (P_1 + P_2)/A$ where A is the area of the lower shaft. In the upper shaft. $f_a = P_1/A$, with A the area of the upper (building) shaft.

f'_a = In checking the lower shaft for bending about the Y-Y axis, it is conservatively assumed that the crane support segment resists all of the bending introduced by eccentricity of the crane girder reactions. The amplifications of f_{by} as a result of deflection are dependent on the average axial stress (f'_a) in the crane segment alone. The stress f'_a is determined by adding (or subtracting) the average stress due to moment about the X axis, calculated at the centroid of the crane segment, to (or from) the average stress f_a of the entire lower shaft.

F_a = The allowable axial stress under axial load. It may be determined for buckling of the entire stepped column about the X-X axis, based on the equivalent length KL/r_x, or by buckling about the Y-Y axis for whatever column length is unsupported, in either the upper or lower shaft. It is to be taken as the minimum of the two values in each of the two sets pertinent to the upper and lower shafts, respectively. Exterior wall girts are often assumed not to provide longitudinal (lateral) support to the columns in mill buildings because building alterations may result in their removal. If support in the x direction is provided only at locations A, B and C (Figure 20.3.2) the equivalent length KL for buckling about the Y-Y axis should be taken as the full unsupported length AB in checking the upper shaft. In checking the lower shaft for the Y-Y axis, the equivalent length KL should be taken as

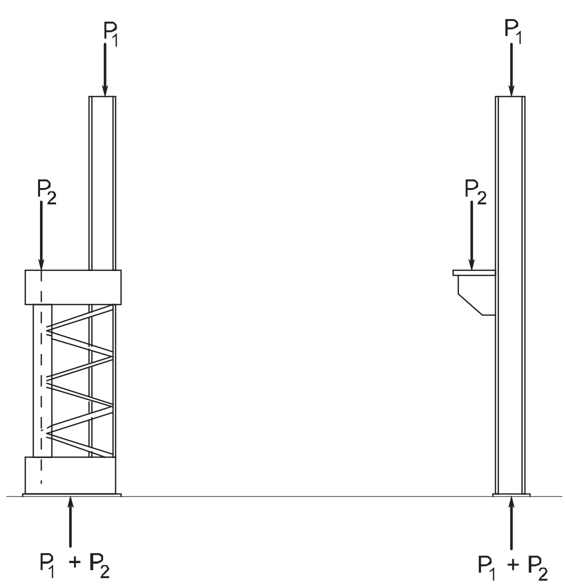


Fig. 20.3.1 Column Loads

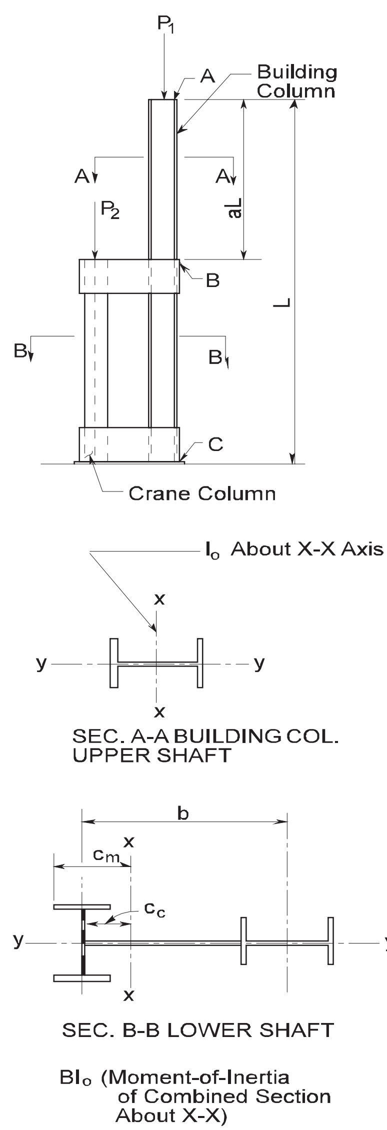


Fig. 20.3.2 Typical Column

0.8 of length BC if the base is assumed to be fully fixed, or as length BC if base fixity against rotation cannot be assured.

C_{mx} = For bending about the X-X axis, use a value of 0.85 when all bents are under simultaneous wind load and sidesway is assumed to take place. When one bent is being considered, under maximum crane loading, without wind (AISE Report 13 Case 2 loading) assume a value of 0.95 for C_{mx}.

C_{my} = Since the crane segment of the lower shaft is assumed to resist all of the bending about the Y-Y axis, this term is applied to the lower shaft (f_{by} is assumed zero in the upper shaft). Assuming fixity at the base but no interaction with the building column, half of the moment introduced at B as a result of unequal reactions from adjacent girders will be carried down to the base, in which C_{my} = 0.4 (AISC *Specification* Section H1). If base fixity cannot be assumed, take C_{my} = 0.6 (hinged condition at base), or, in intermediate situations, interpolate between 0.4 and 0.6.

f_{bx} = Maximum stress due to bending about the x-x axis, assuming an integral action of crane and building column segments in the lower shaft, and the building column alone in the upper shaft.

f_{by} = Maximum stress due to bending about the Y-Y axis in the crane column segment of the lower shaft; usually zero in upper shaft.

F_{bx} = For compression on the crane column side of the lower shaft, F_{bx} is the permissible extreme fiber stress due to bending about the X-X axis, reduced if necessary below $0.6F_y$ because of lack of lateral support.

The reduced allowable stress may be based on the permissible axial stress in the crane column segment for buckling about the Y-Y axis as shown in Figure 20.3.2. (The Y-Y axis in this sketch would correspond to the X-X axis of the individual wide flange segment in the AISC Manual.) The permissible column stress, so determined, should be multiplied by the ratio C_m/C_c, as defined by Section B-B in Figure 20.3.2. In no case is the allowable stress to be greater than $0.6F_y$.

F_{by} = Since this component of bending is about the weak axis of the combined crane and building columns, no reduction in permissible stress need be made for lateral buckling. Also, because the bending resistance is assumed to be provided solely by the crane segment of the lower shaft, the allowable stress for a compact section may be used if the provisions of Section F2 of the AISC *ASD Specification* are met.

F'_{ex} = Since this stress is used as a basis for the determination of the amplification of column deflection in the plane of bending, it should be based on the equivalent length of the completed stepped column, as in the case of F_a, for bending about the X-X axis.

F'_{ey} = If the base may be assumed as fixed let K = 0.8 for the crane column segment alone; otherwise assume K = 1.0. The length in the determination of KL is that of the column segment BC.

Example 20.3.1 will illustrate the procedure.

Contained in the AISE Report 13 publication are effective length values for stepped columns, in terms of three parameters: the ratio of the length of the reduced section to the total length of the column; B, the ratio of the maximum moment-of-inertia of the combined column cross section to that of the *reduced* section; and P_1/P_2, the ratio of the axial force in the upper segment to the crane force in the lower segment. (See Figure 20.3.2.)

The AISE Report 13 tables do not address column end conditions other than fixed or hinged and often times the ratios of P_1/P_2 fall outside the scope of the tables. Contained in Appendix B are tables which address seven different column end conditions, these include:

a. Pinned–Pinned

b. Fixed–Free

c. Fixed–Pinned

d. Fixed–Slider

e. Fixed–Fixed

f. Pinned–Fixed

g. Pinned–Slider

In addition, these tables include prismatic and nonprismatic columns, and virtually all combinations of P_1 and P_2 load ratios.

EXAMPLE 20.3.1:

Bracketed Crane Column Design (ASD)

Design the column shown in Figure 20.3.3:

Use AISE Report 13 provisions and A992 steel.

1. Load Cases:

A frame analysis was performed using the AISE Report 13 load combinations. The critical moment diagrams were obtained from Load Cases 2 and 3.

Case 2 = DL+LL+Crane (Lateral and Vertical)

Case 3 = (DL+Crane Vertical + Wind) 0.75

Case 2 produces the most critical moment condition in the lower portion of the column, and Case 3 in the upper portion.

2. Preliminary Design:

Since this structure is quite tall it is very likely that lateral sway movement could control the column size. Thus, it is recommended that the preliminary design of the column be based on deflection considerations.

Base the allowable sway on:

$$\frac{H}{240} = \frac{45(12)}{240} = 2.25 \text{ in.} \qquad \text{Use 2.0 in.}$$

For a fixed-fixed column wit a WL = 20 psf:

$$\Delta = \frac{P_W H^3}{12EI} \qquad \text{See Figure 20.3.5.}$$

P_w = (WL)(BAY SPACING)(H/2)
P_w = 20(20)(45/2) = 9.0 kips

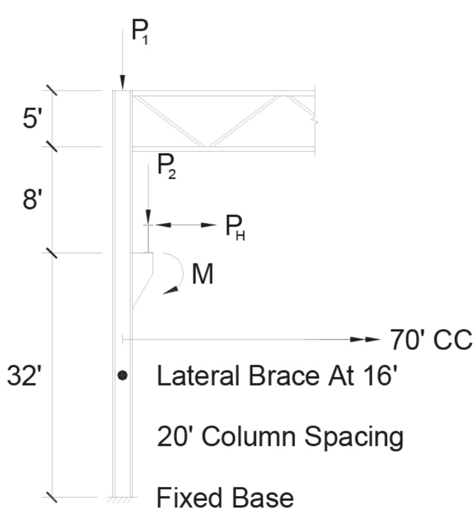


Fig. 20.3.3 Example

Assuming P_w is divided equally between both columns.

$$I_x = \frac{4.5(45)^3(1728)}{24(29000)} = 1018 \text{ in.}^4$$

Try a W16×77.

I_x = 1120 in.^{4}

3. Stress Check:

The properties of the W16×77 are:

I_x = 1110 in.^{4}, r_x = 7.0 in. r_T = 2.77 in.
S_x = 134 in.^{3}, r_y = 2.47 in.
A = 22.6 in.^{2}, d/A_f = 2.11

Lower Column Check:

From Case 2, P_1 = 31 kips, P_2 = 50 kips

Use the effective length charts contained in the Appendix B to determine K_x. Assume the column base is fixed and the column top is a fixed roller.

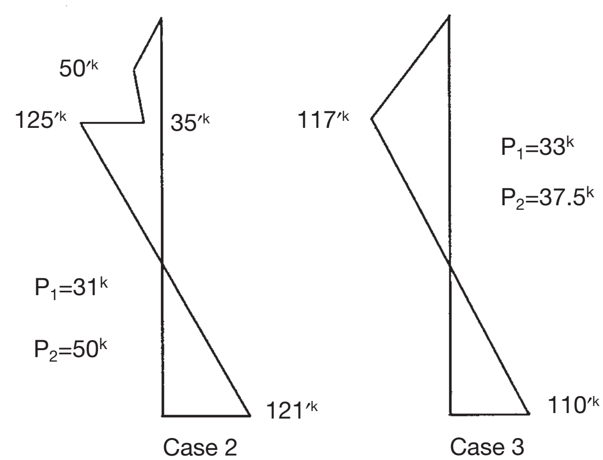


Fig. 20.3.4 Critical Moment Diagrams

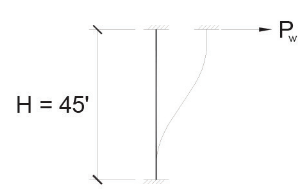


Fig. 20.3.5 Sway Calculation

$I_1/I_2 \;=\; 1.0$

$L_2/L_T \;=\; 32/42.5 = 0.75$

$P_2/P_T \;=\; 50/81 = 0.62$

Note that L_T is based on the midheight of the roof truss.

By interpolation from the tables: $K2 = 0.97$

$KL_x \;=\; (0.97)(42.5) = 41.2$ ft

$KL_y \;=\; (1)(16) = 16.0$ ft

Checking the AISC interaction equations:

$f_a \;=\; P_T/A = 81/22.6 = 3.58$ ksi

$KL_x/r_x \;=\; (41.2)(12)/7 = 70.7$

$KL_y/r_y \;=\; (16)(12)/2.47 = 77.7$

$\therefore F_a \;=\; 19.5$ ksi, $\quad F'_e = 29.9$ ksi

$f_b \;=\; M/S_x = (125)(12)/134 = 11.2$ ksi

$C_m \;=\; 0.85$, $F_b = 30$ ksi (braced at runway and at 16 ft above the base)

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} \le 1.0$$

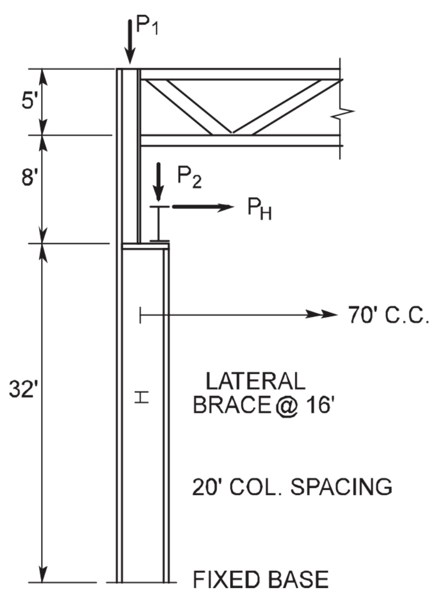


Fig. 20.3.6 Example

$$\frac{3.58}{19.47} + \frac{0.85(11.19)}{\left(1 - \dfrac{3.58}{29.9}\right)30} = 0.54 \le 1.0$$

$$\frac{f_a}{0.60F_y} + \frac{f_b}{F_b} = \frac{3.58}{30} + \frac{11.19}{30} = 0.49 \le 1.0$$

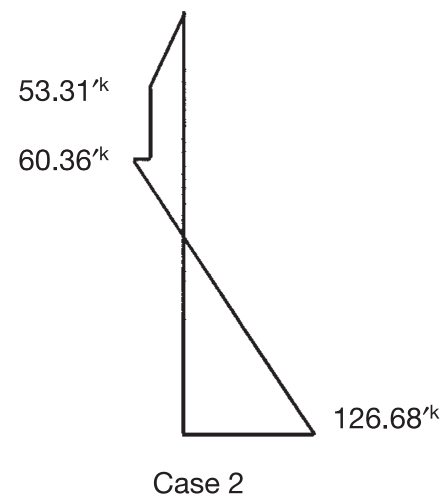


Fig. 20.3.7 Critical Moment Diagram

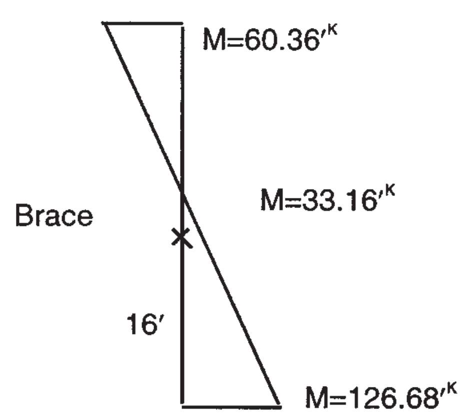


Fig. 20.3.8 Critical Moment Diagram

Upper Column Check:

From Case 3, P_1 = 33 kips, P_2 = 37.5 kips.

Use the effective length charts contained in the Appendix B to determine K_x.

I_1/I_2 = 1.0 and L_2/L_T = 0.75

P_2/P_T = 33/70.5 = 0.47

By interpolation from the tables:

K_1 = 1.36

KL_x = (1.36)(42.5) = 57.8 ft

KL_y = 8 ft

Checking the AISC interaction equations:

f_a = 33/22.6 = 1.46 ksi

KL_x/r_x = (57.8)(12)/7 = 99.1

KL_y/r_y = (8)(12)/2.47 = 38.9

$\therefore F_a$ = 14.9 ksi, F_e' = 15.3 ksi

f_b = M/S_x = (117)(12)/134 = 10.5 ksi

f_a/F_a = 1.46/14.9 = 0.10 < 0.15

Check:

$$\frac{f_a}{F_a}+\frac{f_b}{F_b}\le 1.0$$

$$\frac{1.46}{14.9}+\frac{10.5}{30}=0.45\le 1.0$$

Use a W16×77

Deflection Controls.

EXAMPLE 20.3.2:

Stepped Crane Column Design

Design the column shown in Figure 20.3.6 using ASD.

Use AISE Report 13 provisions and A992 steel.

1. Load Cases:

A frame analysis was performed using the AISE Report 13 loading combinations. The critical moment diagram was obtained from Case 2. See Figure 20.3.7.

Case 2 = DL + LL + Crane (Lateral and Vertical)

2. Preliminary Design

Use the strength preliminary design procedures discussed in this guide.

For the upper shaft, $P = 31^K$.

Based on the AISC Manual column tables, try a W12×35 section. For the lower shaft the crane load equals 50 kips.

Estimate the flange area.

$0.45\,F_y$ = 16.2 ksi
A_{flange} = 50/16.2 = 3.09 in.^{2}

A W24 section is required for crane clearance.

Try a W24×62, Aflange = 4.15 in.^{2}

As an approximation to the moment of inertia for the stepped column, use a weighted average of the moment of inertia for the upper and lower shafts.

$$\frac{(32)(1550)+(10.5)(285)}{42.5}=1237\text{ in.}^4$$

Since this average is greater than 984 in.^{4} from the previous example, the column should satisfy the L/240 deflection requirement. After a final stress check the deflection check can be verified by computer analysis.

3. Stress Check:

The properties of the W12×35 are:

A = 10.3 in.^{2}, I_x = 285 in.^{4}
S_x = 45.6 in.^{3}, r_x = 5.25 in.
r_y = 1.54 in., r_T = 1.74 in.

For the W24×62:

A = 18.2 in.^{2}, I_x = 1550 in.^{4}
S_x = 131 in.^{3}, r_x = 9.23 in.
r_y = 1.38 in., r_T = 1.71 in.
d/A_f = 5.71

Use the effective length charts contained in the Appendix B to determine K_x values. Assume the column base is fixed and the column top is a fixed roller.

$I_1/I_2 = 285/1550 = 0.18$
$L_2/L_T = 32/42.5 = 0.75$
$P_2/P_T = 50/81 = 0.62$

By interpolation from the tables:

$K_1 = 0.89, \quad K_2 = 1.29$

Lower Column Check:

$KL_x/r_x = (1.29)(42.5)(12)/9.23 = 71.3$
$KL_y/r = (1)(16)(12)/1.38 = 139$
$\therefore F_a = 7.73 \text{ ksi}, F_e' = 29.4 \text{ ksi}$
$f_a = P_T/A = 81/18.2 = 4.45 \text{ ksi}$
$f_{bx} = M/S_x = (126.7)(12)/131 = 11.6 \text{ ksi}$

Determine F_b:

From the moment diagram for the lower shaft (Figure 20.3.8):

$C_b = 1.75+1.05(M_1/M_2)+0.3(M_1/M_2)^2 \leq 2.3$
$M_1/M_2 = -33.2/126.7 = -0.26$
$C_b = 1.75+1.05(-0.26)+0.3(-0.26)2=1.5$
$L/r_T = (16)(12)/1.71 = 112$

AISC Eq. F1-7 and F1-8 apply.

$$F_b = \frac{170\text{x}10^3 C_b}{\left(L/r_T\right)^2} \leq 0.60F_y$$

$$F_b = \frac{170x10^3\left(1.5\right)}{\left(112\right)^2} = 20.33 \leq 0.60F_y$$

$$F_b = \frac{12x10^3 C_b}{ld/A_f} \leq 0.60F_y$$

$$F_b = \frac{12x10^3\left(1.5\right)}{\left(16\right)\left(12\right)\left(5.71\right)} = 16.42 \leq 0.60F_y$$

$\therefore F_b = 20.3 \text{ ksi.}$

Checking the AISE Report 13 interaction equations:

$$\frac{f_a}{F_a} + \frac{C_m f_{bx}}{\left(1-\frac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_m f_{by}}{\left(1-\frac{f'_a}{F'_{ey}}\right)F_{by}} \leq 1.0$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$\frac{4.45}{7.73} + \frac{\left(0.95\right)11.6}{\left(1-\frac{4.45}{29.38}\right)20.33} = 1.21 \quad \text{n.g.}$$

Repeating the above calculations for a W24×68 section yields that a W24×68 is o.k.

Upper Column Check:

P = 31 kips, M = 53.3 kip-ft

The change in I_1/I_2 (change due to W24×68) causes a slight change in the effective length of the upper shaft. Iterating from the effective length tables yields $K1$ = 0.85

$KL_x/r_x = (0.85)(42.5)(12)/9.55 = 45.4$
$KL_y/r_y = (1)(8)(12)/1.54 = 62$

Therefore:
$F_a = 22.4 \text{ ksi}, F_e' = 72.5 \text{ ksi}$
$f_a = P/A = 31/10.3 = 3.01 \text{ ksi}$
$f_b = M/S_x = (53.3)(12)/45.6 = 14.0 \text{ ksi}$
$F_b = 30 \text{ ksi} \quad (L_u < 8 \text{ ft})$

$$\frac{3.01}{22.37} + \frac{0.95\left(14.03\right)}{\left(1-\frac{3.01}{72.47}\right)30} = 0.60 \leq 1.0$$

$$\frac{3.01}{22.37} + \frac{0.95\left(14.03\right)}{\left(1-\frac{3.01}{72.47}\right)30} = 0.60 \leq 1.0$$

$$\frac{f_a}{0.60F_y} + \frac{f_b}{F_b} = \frac{3.01}{30} + \frac{14.03}{30} = 0.57 \leq 1.0$$

Use a W12×35 with a W24×68.

20.4 Economic Considerations

Although it is not possible to provide a clear-cut rule of thumb as to the most economical application of the various crane columns, in other words, bracketed, stepped, or separate crane column, due to differences in shop techniques; it is possible however, to generalize to some degree.

1. The stepped column will be economical if "clean." In fact, for many jobs a "clean" stepped column can prove economical as compared to the bracketed column even for light loads. By "clean" is meant that the column is fabricated without a face channel or extra welded attachments. (See Figure 20.4.1.) For example,

the cap plate could be made thick enough to eliminate the need for a stiffener under the upper shaft's interior flange.

2. Separate crane columns are economical for heavy cranes. Fabricators favor tying the crane column to the building column with short W shapes acting as a diaphragm as opposed to a lacing system using angles. (See Figure 20.4.2.)

   Lacing systems are economical as compared to the diaphragm system if the miscellaneous framing pieces are not required. For example, if the building column flange width is equal to the crane column depth, the columns can be laced economically using facing angles. (See Figure 20.4.3.)

3. Bracketed columns are generally most efficient up to bracket loads of 25 kips. Crane reactions between 25 kips and 50 kips may best be handled by either a bracket column or a stepped column.

4. If the area of one flange of a stepped column multiplied by $0.5F_y$ is less than the crane load on the column, a separate crane column should definitely be considered.

21. OUTSIDE CRANES

Outside cranes are common in many factories for scrap handling, parts handling and numerous other operations. There are several important aspects of outside crane usage that are unique to that type of crane.

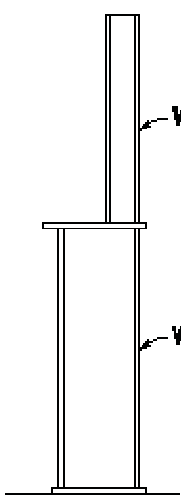


Fig. 20.4.1 "Clean" Column

1. The exterior exposure in many climates requires that extra attention be given to painting and general maintenance, material thickness, and the elimination of pockets, which would collect moisture.

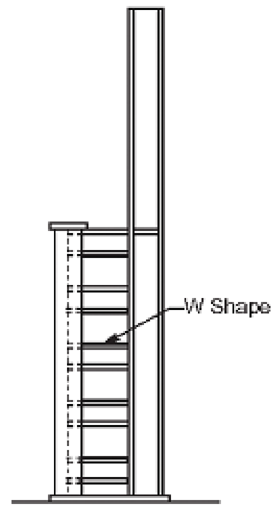


Fig. 20.4.2 Connections with W Shapes

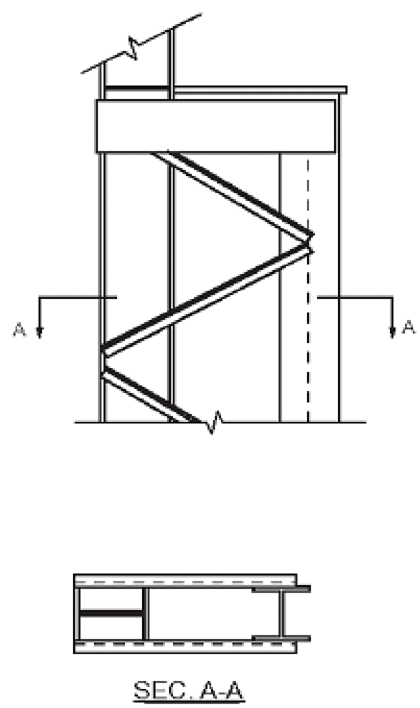


Fig. 20.4.3 Laced Column

2. Due to drive aisles, railways and other similar restrictions, exterior cranes often require longer spans than interior cranes. The outside crane has no building columns from which to derive lateral support. Therefore, long, unbraced spans are more common to these installations. Horizontal bracing trusses, wide truss columns or other bracing elements must often be employed to achieve stability.

3. Long spans may dictate that trusses, rather than plate girders or rolled sections, be used for the runway beams. This can have certain advantages including improved stiffness. The disadvantages are clearly the increased depth plus joints that are highly susceptible to fatigue problems. Secondary stresses must be calculated and included in the fatigue analysis for trusses used as crane girders.

4. Another special girder that may be appropriate for use in these long span applications is the trussed girder. This "hybrid" involves the coupling of a girder (top flange) and a truss. The member can develop excellent stiffness characteristics and many times can temporarily support the crane weight even if a truss member is damaged. As with the basic truss, the overall greater depth is a disadvantage.

5. Still another solution to the long span problem may lie in the use of "box" or "semi-box" girders. An excellent reference on this subject was developed by Schlenker, (Schlenker, 1972). These girders have excellent lateral and torsional strength. In addition, the problem associated with off center crane rails is eliminated.

6. Brittle fracture should be considered for cranes operating in low temperature environments.

22. UNDERHUNG CRANES

Underhung cranes in industrial buildings are very common and quite often prove to be economical for special applications. One of the distinct operational advantages that underhung cranes possess is that they can be arranged to provide for trolley transfer from one runway or aisle to another. Proper provision in the design must be made for handling lateral and impact loads from underhung cranes. The concepts presented in this guide (for example, load transfer) are, in general, applicable to underhung crane systems. Because these cranes are generally supported by roof members load is not transferred directly to the columns and therefore the column design does not involve the moment distribution problems of the top running crane column. Pay particular attention to the method of hanging the cranes. Fatigue problems with these connections have existed in the past and proper provisions must be made with the hanging connection to guarantee adequate service life.

Hanger systems should provide for vertical adjustment in order to properly adjust the elevation of the runway beam. After the runways are positioned vertically, a lateral anti-sway brace should be attached at each hanger location. The sway brace prevents the hanger system from flexing perpendicular to the runway. Most hanger systems experience fatigue at a relatively low stress level if they are allowed to sway. In addition to the lateral anti-sway braces, longitudinal braces should be installed parallel to the runway beams to prevent sway along the length of the runway. These braces should be placed at approximately 100 ft intervals and at all turns in the runway.

Runway splices can be accomplished in many ways. The splice should allow for a smooth running crane as the wheels transfer from one beam to the next. A typical splice detail is shown in Figure 22.1.

Many crane suppliers prefer to supply the runway beams. The building designer must carefully coordinate hanger locations and hanger reactions with the crane supplier. Many times the structure must be designed prior to the selection of the crane system. The building designer must estimate hanger locations and reactions. Hanger reactions can be calculated from manufacturer's catalogs. Hangers should be provided at a 15-ft to 20-ft spacing if possible. The deflection limit for underhung crane runway beams due to wheel loads should be limited to span divided by 450.

In addition to the various AISC *Specification* checks that must be made for the design of underhung crane beams, a bottom flange localized combined stress check must be made to determine the effects of the wheel contact load on the bottom flange. The effect of the concentrated wheel load

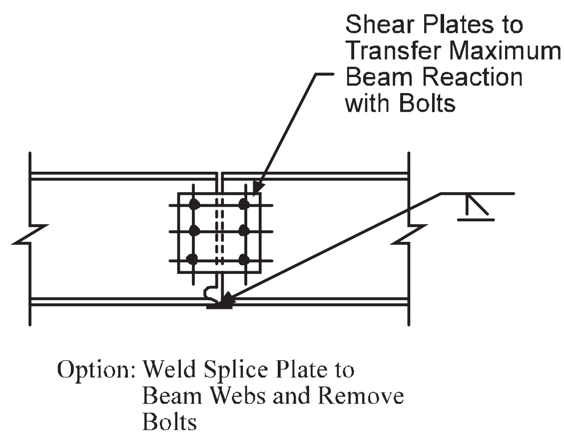


Fig. 22.1 Underhung Crane Beam Splice

can be to "cold work" the steel in the bottom flange which can in the long term result in autofrettage, cracking and break-off of portions of the bottom flange. Contained in the CMAA *Specifications for Top Running & Under Running Single Girder Electric Traveling Cranes Utilizing Under Running Trolley Hoist*, (CMAA, 2000) is a suggested design approach for the examination of the wheel contact stresses.

23. MAINTENANCE AND REPAIR

As stated earlier in this guide, crane buildings require an extra measure of maintenance. Crane rail alignment is especially critical. Wear on the crane and the rail, and potential fatigue problems can result if proper maintenance is not provided. Crane rails must also be inspected for uneven bearing, to minimize fatigue problems.

If fatigue cracks occur and must be repaired, the repair procedure may create additional problems if proper procedures are not taken. Simple welding of doubler plates, stiffeners or other reinforcement may create a "notch effect" which could be more serious than the original problem. Engineers should use common sense in detailing procedures for repair of fatigue cracks. In particular they should not create a worse fatigue problem with the repair. Referral to Appendix K of the AISC Specifications is essential.

24. SUMMARY AND DESIGN PROCEDURES

Many concepts have been presented in this guide relative to the design and analysis of structural frames for crane buildings. In an effort to optimize design time, the following procedural outline has been developed for the designer.

1. Determine the best geometrical layout for the building in question.

2. Design the crane girders and determine column and frame forces from the crane loadings.

3. Perform preliminary design of the crane columns.

4. Design the roof trusses or roof beams for dead loads and live loads.

5. Determine all loading conditions for which the entire frame must be analyzed.

6. Analyze the frame in question for dead, live, wind and seismic loadings. This analysis should be performed without load sharing from the adjacent frames. Also determine the lateral stiffness of the frame.

7. Analyze the frame (considering load sharing) for crane loadings.

8. Combine moments and forces from the two analyses for subsequent design.

9. Perform the final design of columns, trusses, braces and details.

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Appendix A

Table 1											
W-Shape	Channel	Total Wt.	Axis X-X					Axis Y-Y		
			I_x	S_1	S_2	y_1	Z_x	I_t	S_t	Z_t
W36x150	MC18x42.7	193	12000	553	831	21.8	738	689.4	76.6	108.9
	C15x33.9	184	11500	546	764	21.1	716	450.4	60.1	84.6
W33x141	MC18x42.7	184	10000	490	750	20.4	652	675.7	75.1	106.8
	C15x33.9	175	9580	484	689	19.8	635	436.7	58.2	82.5
W33x118	MC18x42.7	161	8280	400	656	20.7	544	647.8	72.0	99.6
	C15x33.9	152	7900	395	596	20.0	529	408.8	54.5	75.3
W30x116	MC18x42.7	159	6900	365	598	18.9	492	636.0	70.7	98.5
	C15x33.9	150	6590	360	544	18.3	480	397.0	52.9	74.2
W30x99	MC18x42.7	142	5830	304	533	19.2	412	618.6	68.7	93.6
	C15x33.9	133	5550	300	481	18.5	408	379.6	50.6	69.3
W27x94	C15x33.9	128	4530	268	435	16.9	357	376.9	50.3	69.4
W27x84	C15x33.9	118	4050	237	403	17.1	316	367.7	49.0	66.7
W24x84	C15x33.9	118	3340	217	367	15.4	286	362.1	48.3	66.5
	C12x20.7	105	3030	211	302	14.3	275	176.1	29.4	41.3
W24x68	C15x33.9	102	2710	173	321	15.7	232	350.2	46.7	62.6
	C12x20.7	88.7	2440	168	258	14.5	224	164.2	27.4	37.4
W21x68	C15x33.9	102	2180	156	287	13.9	207	347.3	46.3	62.5
	C12x20.7	88.7	1970	152	232	12.9	200	161.3	26.9	37.3
W21x62	C15x33.9	95.9	2000	142	272	14.1	189	343.7	45.8	61.2
	C12x20.7	82.7	1800	138	218	13.0	183	157.7	26.3	36.0
W18x50	C15x33.9	83.9	1250	100	211	12.5	133	335.0	44.7	58.8
	C12x20.7	70.7	1120	97.3	166	11.5	127	149.0	24.8	33.6
W16x36	C15x33.9	69.9	748	64.5	160	11.6	86.8	327.2	43.6	56.1
	C12x20.7	56.7	670	62.8	123	10.7	83.2	141.2	23.5	30.9
W14x30	C12x20.7	50.7	447	46.7	98.1	9.57	62	138.8	23.1	30.0
	C10x15.3	45.3	420	46	84.5	9.11	60.3	77.1	15.4	20.3
W12x26	C12x20.7	46.7	318	36.8	82.1	8.63	48.2	137.7	22.9	29.6
	C10x15.3	41.3	299	36.3	70.5	8.22	47	76.0	15.2	19.9

Table 2 (Based on ASD)					
W-Shape	Channel	COMPOSITE SECTION		MAXIMUM SPAN (FT) FOR $F_b = 0.6F_y$ Steel Yield is for the Channel Cap	
		r_T	d/A_f	F_y = 36 ksi	F_y = 50 ksi
W36x150	MC18x42.7	5.090	1.522	30.41	21.90
	C15x33.9	4.324	1.710	27.08	19.49
W33x141	MC18x42.7	5.094	1.428	32.43	23.35
	C15x33.9	4.309	1.606	28.84	20.76
W33x118	MC18x42.7	5.284	1.580	29.30	21.10
	C15x33.9	4.445	1.804	25.66	18.48
W30x116	MC18x42.7	5.208	1.415	32.73	23.56
	C15x33.9	4.357	1.611	28.74	20.70
W30x99	MC18x42.7	5.386	1.536	30.15	21.71
	C15x33.9	4.495	1.772	26.12	18.81
W27x94	C15x33.9	4.465	1.570	29.49	21.24
W27x84	C15x33.9	4.558	1.660	27.89	20.08
W24x84	C15x33.9	4.468	1.450	31.93	22.99
	C12x20.7	3.494	1.872	24.73	17.81
W24x68	C15x33.9	4.645	1.586	29.19	21.02
	C12x20.7	3.621	2.117	21.87	15.74
W21x68	C15x33.9	4.581	1.377	33.62	24.21
	C12x20.7	3.547	1.821	25.43	18.31
W21x62	C15x33.9	4.655	1.425	32.49	23.39
	C12x20.7	3.606	1.909	24.25	17.46
W18x50	C15x33.9	4.756	1.293	35.79	25.77
	C12x20.7	3.672	1.766	26.22	18.88
W16x36	C15x33.9	4.954	1.258	36.80	26.49
	C12x20.7	3.845	1.781	25.99	18.72
W14x30	C12x20.7	3.922	1.624	28.51	20.53
	C10x15.3	3.214	1.986	23.32	16.79
W12x26	C12x20.7	3.957	1.461	31.70	22.82
	C10x15.3	3.243	1.791	25.85	18.61

Appendix B

Appendix B
CALCULATION OF EFFECTIVE LENGTHS OF STEPPED COLUMNS

KRISHNA M. AGRAWAL AND ANDREW P. STAFIEJ

Travelling cranes are frequently used to move heavy loads in industrial buildings. To accomplish a general movement, the crane traverses a crane bridge, which in turn moves on rails along the length of the building, supported by the main building columns.

Designers frequently choose stepped columns, with the wider lower section serving a dual purpose: (a) to support the crane rail and (b) to provide the necessary strength to support the extra load from the crane. The design of the stepped columns is time-consuming and complicated. Effective lengths, which must be calculated for each segment, depend upon the following: the end fixity types at the two ends, the ratio of segment lengths (l_1 / l_2), the ratio of the segment inertias I_1 / I_2, and the ratio of the applied axial loads (P_1 / P_2) applied at the top of the column and at the stepped levels.

Various cases of end fixities are encountered in practice (Fig. 1). Anderson and Woodward[1] have presented equations for five end-fixity types to be used in calculating effective lengths. These types are: (1) Pin-Pin (2) Fix-Free, (3) Fix-Pin, (4) Fix-Slider, and (5) Fix-Fix. Two other cases which have not been dealt with previously are: (6) Pin-Fix and (7) Pin-Slider (Fig. 1). Industrial building frames are often designed as pinned at the bottom, supporting a deep roof truss at the top which provides for a fix or slider end condition.

The characteristic equation in Ref. 1 for case (5) when $P_2 = 0$ [Eq. (A-15) and FUNCTION FC5(x)] appears to be in error. This technical note is intended to correct the equation for the end-fixity case (5) and to extend the directory of characteristic equations for end-fixities to include two additional cases: (6) Pin-Fix and (7) Pin-Slider. The derivation of the equations is omitted in this paper, since the process has been adequately described in Ref. 1. The nomenclature of Ref. 1 is used throughout to maintain continuity.

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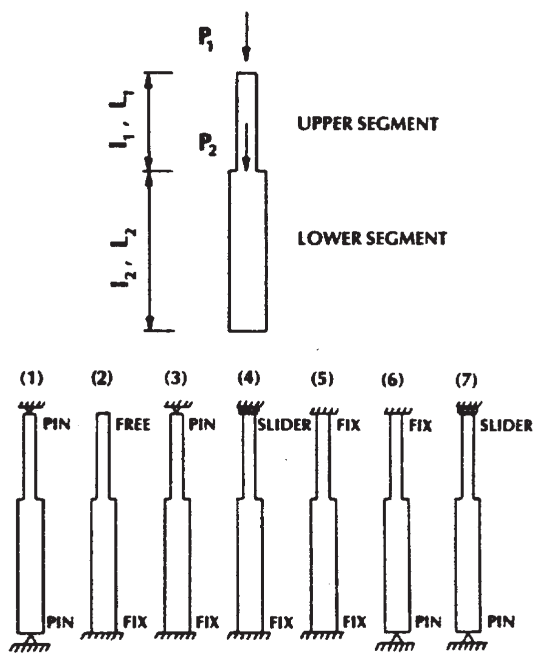

Fig. 1. End condition types.

CHARACTERISTIC EQUATIONS

The parameters in the equations have the following definitions:

$\mathrm{IR} = I_1 / I_2$; $\mathrm{LR} = l_1 / l_2$; $\mathrm{PR} = P_1 / P_2$

$\mathrm{Z} = \Upsilon_1 / l_1$; $\mathrm{BZ} = \Upsilon_2 / l_2$

$\beta = \mathrm{B} = (l_2 / l_1) \times \sqrt{(I_1 / I_2)[1 + (P_2 / P_1)]}$

$\Upsilon_1^2 = P_1 / EI_1$; $\Upsilon_2^2 = (P_1 + P_2) / EI_2$

Finding the lowest root of the characteristic equation Z = ZRT allows the calculation of buckling load

$$P_{1cr} = \left(\frac{\mathrm{ZRT}}{l_1}\right)^2 \times EI_1$$

$$(P_1 + P_2)_{cr} = \left(\frac{\mathrm{ZRT} \times \beta}{l_2}\right)^2 \times EI_2$$

Equating these critical loads to the Euler buckling formula $P_{cr} = \pi^2 EI / L^2$, one obtains:

$$l_{1(eff)} = \pi l_1 / \text{ZRT}$$

$$l_{2(eff)} = \pi l_2 / (\beta \text{Z})$$

These effective lengths are used in the AISC interaction formula for designing beam columns.

The variable definitions are:

I_1, I_2 Moments of inertia of upper and lower segments, respectively

P_1, P_2 Applied axial loads at the top and at step level

P_T Total column axial load = $(P_1 + P_2)$

l_1, l_2 Lengths of upper and lower segments, respectively

L_T Total column length = $(l_1 + l_2)$

$l_{1(eff)}, l_{2(eff)}$ Effective lengths of the upper and lower segments for Euler buckling formula, respectively

K_1, K_2 Effective length factors for upper and lower segments, respectively, with the following definition:

$$K_1 = l_{1(eff)} / (l_1 + l_2)$$

$$K_2 = l_{2(eff)} / (l_1 + l_2)$$

Case 5—Fix-Fix [corrected characteristic equation to replace Eq. (A-15) in Ref. 1]:

c. $P_2 = 0$

$$\begin{aligned}&[\text{COS(Z)} - \text{COS(BZ)}]\{\text{Z}[(1 + \text{LR}) / \text{LR}] \times \text{SIN (Z)} \\ &+ \text{COS(Z)} - \text{COS(BZ)}\} + [\text{LR} \times \text{SIN(Z)} \\ &- \text{Z}(1 + \text{LR})\text{COS(Z)} + \text{SIN(BZ)} / \text{B}][\text{SIN(Z)} / \\ &\text{LR} + \text{B} \times \sin(\text{BZ})] = 0\end{aligned} \quad \text{(A-8)}$$

Case 6—Pinned-Fixed:

a. General ($P_1 > 0$; $P_2 > 0$):

$$\begin{aligned}&\text{SIN(BZ)}\{2 / \text{PR} - \text{Z} \times \text{SIN(Z)} \times [(1 + \text{LR}) / \text{LR} \\ &+ 1 / \text{PR}] - \text{COS(Z)} \times [\text{PR} / (1 + \text{PR}) + 2 / \text{PR}]\} \\ &+ \text{COS(BZ)}\{-\text{B} \times \text{LR} \times \text{SIN(Z)} + \text{BZ} \times \\ &\text{COS(Z)}[1 + \text{LR} - 1 / (1 + \text{PR})]\} = 0\end{aligned} \quad \text{(A-16)}$$

b. $P_1 = 0$:

$$\begin{aligned}&\text{SIN(BZ)} \times \{(\text{BZ} \times \text{LR})^2 - 6\text{IR} / (\text{BZ} \times \text{LR})^2 - 6[1 + \\ &(1 / \text{LR})]\} + 2\text{BZ} \times \text{COS(BZ)} \times \{[3\text{IR} / (\text{BZ} \times \text{LR})^2] \\ &- \text{LR}\} = 0\end{aligned} \quad \text{(A-17)}$$

c. $P_2 = 0$:

$$\begin{aligned}&\text{B} \times \text{LR} \times \text{COS(BZ)} \times [\text{SIN(Z)} - \text{Z} \times \text{COS(Z)} \times (1 + \\ &\text{LR}) / \text{LR}] + \text{SIN(BZ)} \times [\text{COS(Z)} + \text{Z} \times \text{SIN(Z)} \times \\ &(1 + \text{LR}) / \text{LR}] = 0\end{aligned} \quad \text{(A-18)}$$

Case 7—Pinned-Slider:

a. General ($P_1 > 0$; $P_2 > 0$):

$$\begin{aligned}&[1 / (1 + \text{PR})] \times \text{Z} \times \text{SIN(Z)} \times \text{SIN(BZ)} - \text{LR} \times \text{BZ} \times \\ &\text{COS(Z)} \times \text{COS(BZ)} = 0\end{aligned} \quad \text{(A-19)}$$

b. $P_1 = 0$:

$$\text{LR} \times \text{BZ} \times \text{SIN(BZ)} - \text{IR} \times \text{COS(BZ)} = 0 \quad \text{(A-20)}$$

c. $P_2 = 0$:

$$\begin{aligned}&\text{Z} \times \text{SIN(Z)} \times \text{SIN(BZ)} - \text{BZ} \times \text{LR} \times \text{COS(Z)} \times \\ &\text{COS(BZ)} = 0\end{aligned} \quad \text{(A-21)}$$

Reference 4 outlines a computer program similar to the one described in Ref. 1. This program was developed to calculate the roots of the characteristic equations. The solution routine which serves to find the lowest root was modified to improve the speed of convergence to the root. Residual values were calculated by spacing points at equal intervals until a sign change in the residual was observed. At this point, instead of halving the incremental value of Z, a new value for Z was calculated by interpolating the two values of Z, which gave residuals of differing signs. The process was repeated retaining two values for Z, which produced the smallest residuals for further interpolation. The last step was repeated several times, producing a much faster convergence to the characteristic root.

The output from this program (Table 1) lists the slenderness ratios for all seven end-fixity types (Fig. 1) for a wide selection of segment inertia ratios, segment length ratios, and top- and step-level axial-load ratios. Any intermediate value can be easily interpolated from the values presented.

Note that the axial load ratio $P_2 / P_T = P_2 / (P_1 / P_2)$ varies from 0 to 1. A value of zero corresponds to $P_1 > 0$, $P_2 = 0$ and a value of 1 corresponds to $P_1 = 0$ and $P_2 > 0$. All other values of the ratio correspond to P_1 and P_2 both greater than zero.

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Table 1. Equivalent Length Factors for Various End Conditions

$$[K_1 = l_{1\,(eff)}/L_T ; K_2 = l_{2\,(eff)}/L_T]$$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4)FIX -SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1 / I2	ℓ2 / LT	P2 / PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.1	0.1	0.0	0.997	3.153	1.820	5.755	0.637	2.014	0.910	2.878	0.455	1.439	0.695	2.198	1.999	6.320
		0.2	1.020	2.886	1.820	5.148	0.637	1.802	0.910	2.574	0.455	1.287	0.719	2.033	2.047	5.791
		0.4	1.059	2.595	1.820	4.458	0.637	1.560	0.910	2.229	0.455	1.115	0.759	1.858	2.127	5.211
		0.6	1.137	2.275	1.820	3.640	0.637	1.274	0.910	1.820	0.455	0.910	0.836	1.672	2.282	4.565
		0.8	1.360	1.923	1.820	2.574	0.637	0.901	0.910	1.287	0.455	0.644	1.045	1.478	2.708	3.829
		1.0	0.000	1.560	0.000	0.200	0.000	0.197	0.000	0.199	0.000	0.196	0.000	1.283	0.000	2.997
0.1	0.3	0.0	0.941	2.975	1.461	4.621	0.523	1.654	0.732	2.314	0.370	1.170	0.629	1.989	1.964	6.212
		0.2	0.982	2.779	1.461	4.133	0.524	1.483	0.732	2.071	0.372	1.051	0.664	1.879	2.088	5.906
		0.4	1.050	2.572	1.461	3.580	0.526	1.289	0.733	1.795	0.375	0.918	0.720	1.764	2.283	5.593
		0.6	1.176	2.353	1.462	2.924	0.531	1.061	0.735	1.469	0.384	0.769	0.822	1.644	2.633	5.267
		0.8	1.500	2.121	1.464	2.071	0.552	0.780	0.742	1.049	0.439	0.621	1.074	1.519	3.487	4.932
		1.0	0.000	1.880	0.000	0.600	0.000	0.556	0.000	0.590	0.000	0.531	0.000	1.389	0.000	4.631
0.1	0.5	0.0	0.793	2.506	1.107	3.502	0.468	1.480	0.566	1.788	0.342	1.081	0.493	1.561	1.850	5.851
		0.2	0.826	2.336	1.109	3.135	0.479	1.356	0.572	1.616	0.356	1.006	0.518	1.464	2.008	5.679
		0.4	0.880	2.156	1.112	2.724	0.499	1.223	0.583	1.429	0.379	0.928	0.556	1.361	2.255	5.524
		0.6	0.980	1.960	1.120	2.239	0.540	1.080	0.614	1.229	0.422	0.845	0.626	1.252	2.691	5.381
		0.8	1.234	1.746	1.148	1.623	0.659	0.932	0.747	1.056	0.535	0.757	0.801	1.133	3.696	5.227
		1.0	0.000	1.513	0.000	1.000	0.000	0.786	0.000	0.962	0.000	0.665	0.000	1.004	0.000	5.074
0.1	0.7	0.0	0.561	1.775	0.785	2.482	0.436	1.378	0.482	1.524	0.279	0.883	0.331	1.045	1.588	5.022
		0.2	0.579	1.638	0.800	2.263	0.448	1.268	0.518	1.467	0.289	0.816	0.345	0.975	1.755	4.963
		0.4	0.608	1.489	0.828	2.029	0.469	1.148	0.577	1.414	0.303	0.743	0.369	0.903	2.006	4.914
		0.6	0.662	1.325	0.893	1.786	0.508	1.016	0.683	1.367	0.331	0.663	0.417	0.833	2.429	4.858
		0.8	0.807	1.141	1.103	1.560	0.612	0.865	0.937	1.325	0.405	0.573	0.546	0.772	3.395	4.802
		1.0	0.000	0.939	0.000	1.400	0.000	0.689	0.000	1.289	0.000	0.469	0.000	0.729	0.000	4.734
0.1	0.9	0.0	0.329	1.039	0.638	2.016	0.245	0.774	0.461	1.456	0.194	0.614	0.275	0.869	1.094	3.459
		0.2	0.356	1.007	0.697	1.971	0.257	0.727	0.512	1.448	0.214	0.607	0.305	0.863	1.219	3.446
		0.4	0.399	0.977	0.787	1.927	0.279	0.683	0.588	1.441	0.245	0.600	0.350	0.858	1.405	3.441
		0.6	0.476	0.951	0.942	1.884	0.324	0.648	0.716	1.433	0.297	0.594	0.426	0.853	1.718	3.437
		0.8	0.657	0.929	1.302	1.841	0.440	0.623	1.008	1.425	0.417	0.589	0.599	0.848	2.425	3.430
		1.0	0.000	0.909	0.000	1.800	0.000	0.606	0.000	1.418	0.000	0.000	0.000	0.843	0.000	3.417
0.2	0.1	0.0	0.998	2.230	1.840	4.115	0.645	1.441	0.920	2.057	0.460	1.029	0.696	1.555	1.999	4.470
		0.2	1.021	2.042	1.840	3.680	0.645	1.289	0.920	1.840	0.460	0.921	0.720	1.439	2.047	4.095
		0.4	1.060	1.836	1.840	3.187	0.645	1.117	0.920	1.594	0.461	0.798	0.760	1.316	2.128	3.685
		0.6	1.139	1.610	1.840	2.602	0.645	0.912	0.920	1.302	0.461	0.652	0.838	1.185	2.283	3.229
		0.8	1.363	1.363	1.840	1.840	0.646	0.646	0.921	0.921	0.462	0.462	1.049	1.049	2.710	2.710
		1.0	0.000	1.109	0.000	0.200	0.000	0.194	0.000	0.198	0.000	0.192	0.000	0.914	0.000	2.123
0.2	0.3	0.0	0.947	2.117	1.523	3.406	0.557	1.246	0.765	1.711	0.396	0.887	0.635	1.421	1.969	4.404
		0.2	0.990	1.979	1.523	3.047	0.561	1.121	0.767	1.534	0.403	0.805	0.672	1.344	2.093	4.187
		0.4	1.059	1.835	1.525	2.641	0.567	0.983	0.771	1.335	0.415	0.719	0.730	1.265	2.290	3.966
		0.6	1.190	1.683	1.528	2.160	0.584	0.826	0.778	1.100	0.446	0.630	0.836	1.183	2.646	3.743
		0.8	1.523	1.523	1.537	1.537	0.653	0.653	0.812	0.812	0.549	0.549	1.097	1.097	3.508	3.508
		1.0	0.000	1.358	0.000	0.600	0.000	0.522	0.000	0.581	0.000	0.484	0.000	1.010	0.000	3.297

Table 1. Equivalent Length Factors for Various End Conditions

$[K_1 = l_{1\,(eff)} / L_T ; K_2 = l_{2\,(eff)} / L_T]$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4)FIX -SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1 -- I2	ℓ2 -- LT	P2 -- PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.2	0.5	0.0	0.813	1.818	1.221	2.730	0.539	1.204	0.645	1.443	0.384	0.860	0.512	1.145	1.860	4.159
		0.2	0.848	1.696	1.227	2.453	0.555	1.110	0.664	1.328	0.402	0.804	0.539	1.077	2.030	4.060
		0.4	0.906	1.569	1.239	2.146	0.584	1.012	0.698	1.210	0.430	0.745	0.581	1.007	2.289	3.964
		0.6	1.015	1.435	1.265	1.788	0.641	0.906	0.775	1.096	0.482	0.682	0.659	0.932	2.738	3.872
		0.8	1.287	1.287	1.365	1.365	0.794	0.794	1.000	1.000	0.616	0.616	0.852	0.852	3.762	3.762
		1.0	0.000	1.126	0.000	1.000	0.000	0.678	0.000	0.930	0.000	0.545	0.000	0.767	0.000	3.640
0.2	0.7	0.0	0.609	1.361	0.983	2.197	0.478	1.069	0.618	1.382	0.304	0.680	0.387	0.864	1.640	3.667
		0.2	0.631	1.263	1.017	2.034	0.493	0.986	0.671	1.343	0.316	0.632	0.413	0.826	1.809	3.619
		0.4	0.670	1.160	1.078	1.868	0.517	0.896	0.754	1.306	0.335	0.580	0.456	0.790	2.075	3.594
		0.6	0.742	1.050	1.201	1.699	0.563	0.796	0.899	1.271	0.373	0.527	0.536	0.757	2.507	3.545
		0.8	0.935	0.935	1.538	1.538	0.684	0.684	1.238	1.238	0.481	0.481	0.730	0.730	3.513	3.513
		1.0	0.000	0.827	0.000	1.400	0.000	0.555	0.000	1.208	0.000	0.458	0.000	0.708	0.000	3.473
0.2	0.9	0.0	0.454	1.015	0.899	2.010	0.324	0.724	0.574	1.284	0.262	0.585	0.368	0.823	1.220	2.729
		0.2	0.495	0.991	0.983	1.965	0.347	0.694	0.639	1.278	0.290	0.580	0.409	0.818	1.361	2.723
		0.4	0.559	0.968	1.110	1.923	0.385	0.667	0.735	1.273	0.332	0.575	0.470	0.814	1.571	2.721
		0.6	0.670	0.947	1.332	1.883	0.455	0.643	0.897	1.269	0.403	0.571	0.573	0.811	1.923	2.719
		0.8	0.927	0.927	1.843	1.843	0.622	0.622	1.264	1.264	0.566	0.566	0.807	0.807	2.716	2.716
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.259	0.000	0.562	0.000	0.803	0.000	2.706
0.3	0.1	0.0	0.998	1.822	1.860	3.396	0.652	1.191	0.930	1.698	0.465	0.850	0.696	1.271	1.999	3.650
		0.2	1.021	1.668	1.860	3.038	0.652	1.065	0.930	1.519	0.466	0.761	0.720	1.176	2.048	3.344
		0.4	1.061	1.500	1.860	2.631	0.653	0.923	0.930	1.316	0.466	0.659	0.761	1.076	2.128	3.010
		0.6	1.140	1.316	1.860	2.148	0.653	0.754	0.931	1.075	0.467	0.539	0.840	0.970	2.284	2.637
		0.8	1.367	1.116	1.861	1.519	0.655	0.535	0.932	0.761	0.469	0.383	1.053	0.860	2.711	2.214
		1.0	0.000	0.910	0.000	0.200	0.000	0.191	0.000	0.197	0.000	0.188	0.000	0.751	0.000	1.736
0.3	0.3	0.0	0.953	1.740	1.587	2.897	0.587	1.072	0.800	1.460	0.420	0.768	0.642	1.172	1.974	3.604
		0.2	0.997	1.628	1.586	2.590	0.593	0.969	0.803	1.312	0.431	0.703	0.680	1.111	2.099	3.428
		0.4	1.070	1.513	1.590	2.248	0.605	0.856	0.811	1.146	0.450	0.636	0.741	1.048	2.300	3.252
		0.6	1.204	1.390	1.595	1.842	0.633	0.731	0.827	0.955	0.492	0.568	0.851	0.983	2.658	3.069
		0.8	1.547	1.263	1.616	1.319	0.734	0.600	0.899	0.734	0.619	0.505	1.122	0.916	3.529	2.881
		1.0	0.000	1.133	0.000	0.600	0.000	0.495	0.000	0.572	0.000	0.450	0.000	0.846	0.000	2.710
0.3	0.5	0.0	0.833	1.521	1.335	2.438	0.578	1.055	0.718	1.311	0.407	0.744	0.533	0.973	1.890	3.450
		0.2	0.872	1.424	1.345	2.197	0.598	0.977	0.746	1.219	0.427	0.697	0.563	0.919	2.048	3.344
		0.4	0.934	1.321	1.368	1.935	0.632	0.894	0.797	1.128	0.458	0.647	0.610	0.863	2.318	3.278
		0.6	1.051	1.214	1.416	1.635	0.697	0.805	0.901	1.040	0.516	0.595	0.697	0.805	2.772	3.201
		0.8	1.342	1.096	1.586	1.295	0.870	0.711	1.181	0.964	0.661	0.540	0.911	0.744	3.829	3.126
		1.0	0.000	0.971	0.000	1.000	0.000	0.613	0.000	0.903	0.000	0.481	0.000	0.681	0.000	3.014
0.3	0.7	0.0	0.660	1.205	1.156	2.111	0.509	0.929	0.708	1.292	0.331	0.604	0.444	0.810	1.696	3.097
		0.2	0.690	1.126	1.205	1.968	0.526	0.859	0.771	1.260	0.347	0.567	0.479	0.783	1.862	3.040
		0.4	0.739	1.045	1.289	1.824	0.553	0.783	0.869	1.229	0.375	0.530	0.535	0.757	2.141	3.027
		0.6	0.832	0.961	1.451	1.675	0.606	0.699	1.039	1.199	0.430	0.496	0.636	0.734	2.611	3.015
		0.8	1.074	0.877	1.876	1.532	0.744	0.607	1.436	1.172	0.576	0.470	0.874	0.714	3.658	2.987
		1.0	0.000	0.801	0.000	1.400	0.000	0.509	0.000	1.146	0.000	0.454	0.000	0.695	0.000	2.933

Table 1. Equivalent Length Factors for Various End Conditions

$[K_1 = l_{1\,(eff)}/L_T; K_2 = l_{2\,(eff)}/L_T]$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4)FIX -SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1/I2	L2/LT	P2/PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.3	0.9	0.0	0.553	1.009	1.101	2.010	0.390	0.712	0.651	1.189	0.309	0.564	0.433	0.790	1.342	2.450
		0.2	0.604	0.986	1.202	1.963	0.421	0.687	0.726	1.186	0.343	0.560	0.482	0.787	1.495	2.441
		0.4	0.683	0.966	1.362	1.926	0.469	0.663	0.836	1.183	0.393	0.556	0.554	0.784	1.729	2.445
		0.6	0.819	0.946	1.632	1.884	0.556	0.641	1.021	1.179	0.479	0.553	0.677	0.781	2.115	2.442
		0.8	1.135	0.927	2.255	1.841	0.762	0.622	1.441	1.176	0.673	0.549	0.953	0.779	2.988	2.440
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.172	0.000	0.546	0.000	0.775	0.000	2.430
0.4	0.1	0.0	0.998	1.578	1.880	2.973	0.660	1.043	0.940	1.487	0.471	0.744	0.696	1.101	1.999	3.161
		0.2	1.022	1.445	1.880	2.659	0.660	0.933	0.941	1.330	0.471	0.666	0.721	1.020	2.048	2.896
		0.4	1.061	1.300	1.880	2.303	0.660	0.809	0.941	1.152	0.472	0.578	0.762	0.933	2.128	2.607
		0.6	1.141	1.141	1.881	1.881	0.661	0.661	0.941	0.941	0.473	0.473	0.842	0.842	2.285	2.285
		0.8	1.370	0.969	1.882	1.330	0.664	0.469	0.943	0.667	0.478	0.338	1.058	0.748	2.713	1.919
		1.0	0.000	0.792	0.000	0.200	0.000	0.189	0.000	0.197	0.000	0.185	0.000	0.656	0.000	1.506
0.4	0.3	0.0	0.959	1.517	1.648	2.606	0.612	0.968	0.833	1.318	0.440	0.695	0.649	1.027	1.980	3.131
		0.2	1.005	1.421	1.649	2.332	0.621	0.878	0.840	1.187	0.453	0.640	0.689	0.975	2.105	2.977
		0.4	1.079	1.322	1.655	2.027	0.637	0.780	0.851	1.042	0.476	0.583	0.752	0.922	2.308	2.826
		0.6	1.218	1.218	1.665	1.665	0.674	0.674	0.878	0.878	0.526	0.526	0.867	0.867	2.666	2.666
		0.8	1.571	1.111	1.698	1.201	0.798	0.564	0.985	0.696	0.668	0.472	1.146	0.810	3.549	2.509
		1.0	0.000	1.002	0.000	0.600	0.000	0.472	0.000	0.563	0.000	0.424	0.000	0.753	0.000	2.362
0.4	0.5	0.0	0.857	1.355	1.451	2.294	0.605	0.957	0.779	1.232	0.424	0.670	0.556	0.879	1.908	3.017
		0.2	0.897	1.268	1.460	2.065	0.628	0.888	0.815	1.153	0.445	0.629	0.589	0.833	2.076	2.936
		0.4	0.964	1.181	1.494	1.830	0.665	0.815	0.878	1.075	0.478	0.586	0.642	0.787	2.352	2.881
		0.6	1.088	1.088	1.560	1.560	0.737	0.737	1.002	1.002	0.541	0.541	0.738	0.738	2.829	2.829
		0.8	1.401	0.991	1.789	1.265	0.925	0.654	1.320	0.933	0.697	0.493	0.974	0.688	3.903	2.760
		1.0	0.000	0.885	0.000	1.000	0.000	0.568	0.000	0.877	0.000	0.442	0.000	0.639	0.000	2.650
0.4	0.7	0.0	0.713	1.127	1.309	2.069	0.537	0.849	0.775	1.225	0.359	0.568	0.494	0.781	1.751	2.768
		0.2	0.749	1.060	1.370	1.938	0.556	0.787	0.845	1.195	0.380	0.538	0.537	0.759	1.919	2.714
		0.4	0.809	0.990	1.475	1.806	0.588	0.720	0.954	1.169	0.416	0.510	0.603	0.738	2.212	2.709
		0.6	0.921	0.921	1.670	1.670	0.648	0.648	1.143	1.143	0.484	0.484	0.719	0.719	2.700	2.700
		0.8	1.206	0.853	2.164	1.530	0.807	0.570	1.584	1.120	0.657	0.464	0.991	0.701	3.785	2.677
		1.0	0.000	0.791	0.000	1.400	0.000	0.494	0.000	1.097	0.000	0.450	0.000	0.685	0.000	2.622
0.4	0.9	0.0	0.637	1.006	1.272	2.011	0.447	0.707	0.715	1.130	0.347	0.548	0.484	0.766	1.460	2.309
		0.2	0.696	0.984	1.389	1.964	0.484	0.684	0.798	1.128	0.385	0.545	0.540	0.764	1.618	2.289
		0.4	0.788	0.965	1.570	1.922	0.540	0.662	0.920	1.127	0.442	0.542	0.622	0.761	1.877	2.298
		0.6	0.946	0.946	1.891	1.891	0.641	0.641	1.124	1.124	0.539	0.539	0.760	0.760	2.305	2.305
		0.8	1.312	0.928	2.613	1.847	0.880	0.622	1.586	1.122	0.758	0.536	1.071	0.758	3.257	2.303
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.118	0.000	0.533	0.000	0.754	0.000	2.282
0.5	0.1	0.0	0.998	1.412	1.901	2.688	0.667	0.943	0.951	1.344	0.476	0.673	0.697	0.985	2.000	2.828
		0.2	1.022	1.293	1.900	2.404	0.667	0.844	0.951	1.202	0.477	0.603	0.722	0.913	2.048	2.591
		0.4	1.062	1.164	1.901	2.082	0.668	0.731	0.951	1.042	0.477	0.523	0.763	0.836	2.129	2.332
		0.6	1.143	1.022	1.901	1.700	0.669	0.598	0.952	0.851	0.479	0.429	0.844	0.755	2.285	2.044
		0.8	1.373	0.868	1.902	1.203	0.673	0.426	0.955	0.604	0.487	0.308	1.062	0.672	2.715	1.717
		1.0	0.000	0.712	0.000	0.200	0.000	0.186	0.000	0.196	0.000	0.182	0.000	0.590	0.000	1.349

Table 1. Equivalent Length Factors for Various End Conditions

$[K_1 = l_{1\,(eff)}/L_T; K_2 = l_{2\,(eff)}/L_T]$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4)FIX -SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1/I2	ℓ2/LT	P2/PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.5	0.3	0.0	0.965	1.365	1.712	2.421	0.633	0.895	0.866	1.224	0.455	0.643	0.657	0.929	1.986	2.808
		0.2	1.013	1.281	1.712	2.166	0.644	0.814	0.874	1.106	0.470	0.595	0.698	0.883	2.111	2.670
		0.4	1.090	1.194	1.719	1.883	0.664	0.727	0.890	0.975	0.497	0.544	0.764	0.837	2.315	2.536
		0.6	1.233	1.103	1.733	1.550	0.707	0.632	0.927	0.829	0.553	0.494	0.883	0.790	2.682	2.399
		0.8	1.595	1.009	1.783	1.127	0.847	0.536	1.064	0.673	0.706	0.446	1.172	0.741	3.569	2.257
		1.0	0.000	0.913	0.000	0.600	0.000	0.454	0.000	0.556	0.000	0.403	0.000	0.691	0.000	2.126
0.5	0.5	0.0	0.878	1.242	1.562	2.208	0.626	0.886	0.832	1.177	0.437	0.618	0.580	0.820	1.949	2.757
		0.2	0.923	1.168	1.570	1.985	0.651	0.823	0.873	1.104	0.460	0.582	0.617	0.780	2.099	2.655
		0.4	0.995	1.090	1.612	1.765	0.691	0.757	0.945	1.035	0.496	0.544	0.675	0.740	2.385	2.612
		0.6	1.128	1.009	1.703	1.523	0.768	0.687	1.084	0.969	0.563	0.503	0.781	0.698	2.831	2.532
		0.8	1.460	0.923	1.974	1.248	0.969	0.613	1.434	0.907	0.729	0.461	1.038	0.656	3.908	2.472
		1.0	0.000	0.833	0.000	1.000	0.000	0.536	0.000	0.857	0.000	0.416	0.000	0.615	0.000	2.412
0.5	0.7	0.0	0.766	1.083	1.452	2.053	0.565	0.799	0.829	1.172	0.387	0.547	0.539	0.762	1.775	2.510
		0.2	0.808	1.022	1.520	1.923	0.587	0.742	0.905	1.144	0.413	0.522	0.586	0.742	1.969	2.491
		0.4	0.877	0.961	1.639	1.795	0.623	0.682	1.023	1.120	0.455	0.498	0.660	0.723	2.262	2.478
		0.6	1.006	0.899	1.855	1.659	0.691	0.618	1.229	1.099	0.533	0.477	0.789	0.706	2.754	2.464
		0.8	1.328	0.840	2.425	1.534	0.871	0.551	1.707	1.080	0.727	0.460	1.093	0.691	3.873	2.450
		1.0	0.000	0.785	0.000	1.400	0.000	0.488	0.000	1.057	0.000	0.446	0.000	0.675	0.000	2.422
0.5	0.9	0.0	0.710	1.004	1.416	2.002	0.498	0.704	0.772	1.092	0.379	0.536	0.529	0.748	1.572	2.223
		0.2	0.777	0.983	1.553	1.964	0.539	0.682	0.861	1.089	0.421	0.533	0.590	0.746	1.736	2.196
		0.4	0.879	0.964	1.763	1.931	0.603	0.661	0.993	1.088	0.484	0.530	0.680	0.745	2.019	2.211
		0.6	1.056	0.945	2.117	1.893	0.716	0.641	1.215	1.087	0.590	0.528	0.831	0.743	2.482	2.220
		0.8	1.467	0.928	2.933	1.855	0.984	0.622	1.716	1.085	0.831	0.525	1.173	0.742	3.509	2.219
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.081	0.000	0.523	0.000	0.739	0.000	2.190
0.6	0.1	0.0	0.999	1.289	1.921	2.480	0.674	0.870	0.961	1.240	0.481	0.621	0.697	0.900	2.000	2.581
		0.2	1.023	1.181	1.921	2.218	0.674	0.778	0.961	1.110	0.482	0.556	0.722	0.834	2.049	2.365
		0.4	1.063	1.063	1.921	1.921	0.675	0.675	0.961	0.961	0.483	0.483	0.764	0.764	2.129	2.129
		0.6	1.144	0.934	1.921	1.569	0.677	0.553	0.963	0.786	0.486	0.396	0.846	0.691	2.286	1.867
		0.8	1.376	0.795	1.923	1.110	0.683	0.394	0.966	0.558	0.497	0.287	1.066	0.616	2.717	1.569
		1.0	0.000	0.654	0.000	0.200	0.000	0.184	0.000	0.195	0.000	0.179	0.000	0.543	0.000	1.234
0.6	0.3	0.0	0.973	1.256	1.774	2.290	0.651	0.840	0.897	1.158	0.467	0.603	0.665	0.858	1.992	2.572
		0.2	1.021	1.179	1.774	2.048	0.663	0.766	0.907	1.048	0.484	0.559	0.708	0.817	2.115	2.442
		0.4	1.100	1.100	1.785	1.785	0.686	0.686	0.927	0.927	0.514	0.514	0.776	0.776	2.323	2.323
		0.6	1.248	1.019	1.803	1.472	0.735	0.600	0.973	0.794	0.574	0.468	0.899	0.734	2.691	2.197
		0.8	1.620	0.935	1.867	1.078	0.889	0.513	1.135	0.655	0.737	0.425	1.197	0.691	3.590	2.073
		1.0	0.000	0.850	0.000	0.600	0.000	0.438	0.000	0.549	0.000	0.386	0.000	0.648	0.000	1.953
0.6	0.5	0.0	0.903	1.166	1.654	2.135	0.644	0.831	0.877	1.132	0.450	0.581	0.604	0.780	1.967	2.539
		0.2	0.950	1.097	1.678	1.937	0.670	0.774	0.922	1.064	0.475	0.548	0.645	0.745	2.118	2.445
		0.4	1.027	1.027	1.726	1.726	0.713	0.713	1.001	1.001	0.513	0.513	0.709	0.709	2.411	2.411
		0.6	1.167	0.953	1.837	1.500	0.795	0.649	1.153	0.941	0.584	0.476	0.822	0.671	2.864	2.338
		0.8	1.522	0.879	2.151	1.242	1.007	0.581	1.531	0.884	0.759	0.438	1.100	0.635	3.958	2.285
		1.0	0.000	0.796	0.000	1.000	0.000	0.512	0.000	0.836	0.000	0.398	0.000	0.599	0.000	2.238

Table 1. Equivalent Length Factors for Various End Conditions

$$[K_1 = l_{1\,(eff)} / L_T ; K_2 = l_{2\,(eff)} / L_T]$$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4) FIX - SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1 / I2	ℓ2 / LT	P2 / PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.6	0.7	0.0	0.817	1.055	1.578	2.037	0.592	0.765	0.873	1.128	0.413	0.534	0.578	0.746	1.860	2.401
		0.2	0.864	0.998	1.656	1.913	0.617	0.712	0.954	1.101	0.443	0.511	0.630	0.728	2.026	2.339
		0.4	0.942	0.942	1.793	1.793	0.657	0.657	1.083	1.083	0.490	0.490	0.711	0.711	2.350	2.350
		0.6	1.087	0.887	2.027	1.656	0.734	0.599	1.302	1.063	0.577	0.471	0.851	0.695	2.887	2.358
		0.8	1.441	0.832	2.660	1.536	0.935	0.540	1.804	1.041	0.789	0.455	1.180	0.682	4.058	2.343
		1.0	0.000	0.782	0.000	1.400	0.000	0.486	0.000	1.024	0.000	0.442	0.000	0.667	0.000	2.278
0.6	0.9	0.0	0.776	1.002	1.560	2.014	0.544	0.702	0.825	1.065	0.407	0.525	0.568	0.734	1.663	2.147
		0.2	0.851	0.982	1.704	1.968	0.590	0.681	0.919	1.061	0.453	0.523	0.634	0.733	1.850	2.136
		0.4	0.963	0.963	1.934	1.934	0.660	0.660	1.062	1.062	0.521	0.521	0.731	0.731	2.148	2.148
		0.6	1.156	0.944	2.330	1.903	0.784	0.640	1.298	1.060	0.636	0.519	0.895	0.731	2.626	2.144
		0.8	1.605	0.926	3.227	1.863	1.077	0.622	1.833	1.059	0.896	0.517	1.263	0.729	3.711	2.143
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.055	0.000	0.515	0.000	0.726	0.000	2.127
0.7	0.1	0.0	0.999	1.194	1.941	2.320	0.680	0.813	0.971	1.160	0.486	0.581	0.698	0.834	2.000	2.390
		0.2	1.023	1.094	1.941	2.075	0.681	0.728	0.971	1.038	0.487	0.521	0.723	0.773	2.049	2.190
		0.4	1.064	0.985	1.941	1.797	0.682	0.632	0.972	0.900	0.488	0.452	0.765	0.709	2.130	1.972
		0.6	1.145	0.866	1.942	1.468	0.684	0.517	0.973	0.736	0.492	0.372	0.848	0.641	2.287	1.729
		0.8	1.380	0.737	1.944	1.039	0.692	0.370	0.978	0.523	0.507	0.271	1.071	0.572	2.719	1.453
		1.0	0.000	0.608	0.000	0.200	0.000	0.182	0.000	0.194	0.000	0.176	0.000	0.506	0.000	1.145
0.7	0.3	0.0	0.979	1.171	1.835	2.194	0.665	0.795	0.925	1.106	0.477	0.570	0.673	0.804	1.988	2.376
		0.2	1.029	1.100	1.834	1.961	0.679	0.726	0.938	1.003	0.496	0.530	0.718	0.767	2.122	2.268
		0.4	1.112	1.029	1.847	1.709	0.704	0.652	0.962	0.891	0.528	0.488	0.789	0.730	2.333	2.160
		0.6	1.264	0.956	1.872	1.415	0.758	0.573	1.016	0.768	0.592	0.447	0.916	0.692	2.703	2.043
		0.8	1.644	0.879	1.951	1.043	0.924	0.494	1.199	0.641	0.763	0.408	1.224	0.654	3.611	1.930
		1.0	0.000	0.803	0.000	0.600	0.000	0.424	0.000	0.541	0.000	0.371	0.000	0.615	0.000	1.820
0.7	0.5	0.0	0.926	1.107	1.743	2.083	0.659	0.788	0.911	1.089	0.463	0.553	0.629	0.752	1.956	2.338
		0.2	0.977	1.045	1.773	1.895	0.687	0.734	0.964	1.030	0.488	0.522	0.673	0.719	2.142	2.290
		0.4	1.059	0.980	1.836	1.700	0.733	0.678	1.049	0.971	0.529	0.490	0.741	0.686	2.428	2.248
		0.6	1.208	0.913	1.958	1.480	0.819	0.619	1.210	0.915	0.604	0.457	0.864	0.653	2.901	2.193
		0.8	1.584	0.846	2.312	1.236	1.042	0.557	1.615	0.863	0.789	0.422	1.159	0.620	4.018	2.148
		1.0	0.000	0.772	0.000	1.000	0.000	0.493	0.000	0.820	0.000	0.386	0.000	0.587	0.000	2.105
0.7	0.7	0.0	0.866	1.036	1.692	2.023	0.620	0.741	0.913	1.091	0.438	0.523	0.612	0.732	1.890	2.259
		0.2	0.918	0.982	1.785	1.908	0.647	0.692	0.998	1.066	0.470	0.502	0.669	0.715	2.086	2.230
		0.4	1.004	0.929	1.926	1.784	0.692	0.641	1.134	1.050	0.522	0.483	0.756	0.700	2.387	2.210
		0.6	1.161	0.878	2.205	1.667	0.776	0.587	1.363	1.030	0.617	0.466	0.907	0.685	2.949	2.229
		0.8	1.550	0.829	2.880	1.539	0.997	0.533	1.895	1.013	0.845	0.451	1.257	0.672	4.153	2.220
		1.0	0.000	0.780	0.000	1.400	0.000	0.484	0.000	0.996	0.000	0.439	0.000	0.660	0.000	2.169
0.7	0.9	0.0	0.839	1.003	1.677	2.005	0.587	0.701	0.873	1.044	0.433	0.517	0.604	0.723	1.779	2.127
		0.2	0.918	0.982	1.840	1.968	0.636	0.680	0.973	1.040	0.482	0.515	0.675	0.721	1.955	2.090
		0.4	1.040	0.963	2.100	1.944	0.712	0.660	1.125	1.042	0.555	0.513	0.778	0.720	2.267	2.099
		0.6	1.249	0.944	2.523	1.907	0.847	0.640	1.376	1.040	0.677	0.512	0.951	0.719	2.812	2.126
		0.8	1.740	0.930	3.475	1.857	1.164	0.622	1.944	1.039	0.954	0.510	1.343	0.718	3.976	2.125
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.035	0.000	0.509	0.000	0.716	0.000	2.081

Table 1. Equivalent Length Factors for Various End Conditions

$[K_1 = l_{1\,(eff)}/L_T ; K_2 = l_{2\,(eff)}/L_T]$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4) FIX - SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1 / I2	ℓ2 / LT	P2 / PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.8	0.1	0.0	0.999	1.117	1.961	2.192	0.687	0.768	0.980	1.096	0.491	0.549	0.698	0.781	2.000	2.236
		0.2	1.023	1.023	1.961	1.961	0.688	0.688	0.981	0.981	0.492	0.492	0.724	0.724	2.049	2.049
		0.4	1.064	0.922	1.961	1.699	0.689	0.597	0.982	0.850	0.494	0.428	0.766	0.664	2.130	1.845
		0.6	1.147	0.811	1.962	1.388	0.692	0.489	0.984	0.696	0.498	0.352	0.850	0.601	2.288	1.618
		0.8	1.383	0.692	1.965	0.982	0.702	0.351	0.990	0.495	0.518	0.259	1.075	0.538	2.721	1.360
		1.0	0.000	0.572	0.000	0.200	0.000	0.180	0.000	0.193	0.000	0.173	0.000	0.477	0.000	1.072
0.8	0.3	0.0	0.987	1.103	1.890	2.113	0.678	0.758	0.953	1.065	0.486	0.543	0.681	0.762	2.004	2.240
		0.2	1.038	1.038	1.896	1.896	0.693	0.693	0.967	0.967	0.506	0.506	0.728	0.728	2.128	2.128
		0.4	1.122	0.972	1.911	1.655	0.721	0.624	0.995	0.862	0.540	0.467	0.802	0.694	2.338	2.025
		0.6	1.279	0.904	1.940	1.372	0.779	0.551	1.056	0.747	0.607	0.429	0.933	0.660	2.721	1.924
		0.8	1.671	0.835	2.035	1.018	0.954	0.477	1.258	0.629	0.786	0.393	1.250	0.625	3.619	1.810
		1.0	0.000	0.765	0.000	0.600	0.000	0.412	0.000	0.536	0.000	0.359	0.000	0.589	0.000	1.713
0.8	0.5	0.0	0.950	1.062	1.836	2.053	0.673	0.752	0.947	1.059	0.475	0.531	0.653	0.730	1.966	2.199
		0.2	1.005	1.005	1.868	1.868	0.703	0.703	1.002	1.002	0.502	0.502	0.700	0.700	2.165	2.165
		0.4	1.092	0.945	1.933	1.674	0.751	0.650	1.092	0.946	0.546	0.473	0.773	0.669	2.442	2.115
		0.6	1.250	0.884	2.078	1.469	0.842	0.595	1.262	0.893	0.624	0.442	0.903	0.639	2.930	2.072
		0.8	1.645	0.822	2.457	1.228	1.075	0.538	1.689	0.845	0.819	0.410	1.219	0.609	4.083	2.042
		1.0	0.000	0.753	0.000	1.000	0.000	0.478	0.000	0.803	0.000	0.376	0.000	0.579	0.000	2.000
0.8	0.7	0.0	0.913	1.021	1.811	2.025	0.647	0.723	0.943	1.054	0.460	0.514	0.644	0.720	1.936	2.164
		0.2	0.970	0.970	1.895	1.895	0.677	0.677	1.035	1.035	0.495	0.495	0.705	0.705	2.133	2.133
		0.4	1.063	0.920	2.067	1.790	0.726	0.628	1.181	1.023	0.551	0.477	0.797	0.691	2.494	2.160
		0.6	1.234	0.872	2.364	1.671	0.818	0.578	1.424	1.007	0.652	0.461	0.957	0.677	2.992	2.115
		0.8	1.651	0.826	3.058	1.529	1.056	0.528	1.975	0.988	0.895	0.447	1.329	0.664	4.198	2.099
		1.0	0.000	0.778	0.000	1.400	0.000	0.483	0.000	0.973	0.000	0.436	0.000	0.653	0.000	2.084
0.8	0.9	0.0	0.895	1.001	1.804	2.017	0.626	0.700	0.917	1.026	0.456	0.510	0.638	0.713	1.839	2.056
		0.2	0.981	0.981	1.971	1.971	0.679	0.679	1.024	1.024	0.509	0.509	0.713	0.713	2.053	2.053
		0.4	1.111	0.962	2.220	1.923	0.761	0.659	1.186	1.027	0.586	0.507	0.821	0.711	2.406	2.084
		0.6	1.335	0.944	2.705	1.913	0.905	0.640	1.447	1.023	0.716	0.506	1.004	0.710	2.907	2.056
		0.8	1.862	0.931	3.751	1.876	1.244	0.622	2.045	1.023	1.010	0.505	1.422	0.711	4.110	2.055
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.019	0.000	0.503	0.000	0.708	0.000	2.047
0.9	0.1	0.0	1.000	1.054	1.980	2.087	0.693	0.731	0.990	1.044	0.495	0.522	0.699	0.737	2.001	2.109
		0.2	1.024	0.965	1.981	1.867	0.694	0.654	0.991	0.934	0.497	0.468	0.724	0.683	2.049	1.932
		0.4	1.065	0.870	1.981	1.618	0.696	0.568	0.992	0.810	0.499	0.408	0.767	0.627	2.131	1.740
		0.6	1.148	0.766	1.982	1.322	0.699	0.466	0.994	0.663	0.505	0.336	0.852	0.568	2.289	1.526
		0.8	1.386	0.654	1.986	0.936	0.711	0.335	1.003	0.473	0.528	0.249	1.080	0.509	2.722	1.283
		1.0	0.000	0.542	0.000	0.200	0.000	0.178	0.000	0.193	0.000	0.171	0.000	0.452	0.000	1.013
0.9	0.3	0.0	0.994	1.048	1.954	2.059	0.689	0.727	0.978	1.031	0.493	0.520	0.690	0.728	2.002	2.110
		0.2	1.047	0.987	1.954	1.842	0.706	0.665	0.994	0.937	0.514	0.485	0.739	0.697	2.132	2.010
		0.4	1.134	0.926	1.971	1.610	0.735	0.600	1.025	0.837	0.550	0.449	0.815	0.665	2.348	1.918
		0.6	1.294	0.863	2.007	1.338	0.797	0.531	1.093	0.729	0.621	0.414	0.951	0.634	2.726	1.817
		0.8	1.696	0.799	2.115	0.997	0.981	0.463	1.311	0.618	0.807	0.380	1.276	0.602	3.654	1.722
		1.0	0.000	0.735	0.000	0.600	0.000	0.402	0.000	0.529	0.000	0.349	0.000	0.569	0.000	1.625

Table 1. Equivalent Length Factors for Various End Conditions

$$[K_1 = l_{1\,(eff)}/L_T; K_2 = l_{2\,(eff)}/L_T]$$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4) FIX - SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
$\frac{I1}{I2}$	$\frac{\ell 2}{LT}$	$\frac{P2}{PT}$	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
0.9	0.5	0.0	0.976	1.028	1.922	2.026	0.686	0.724	0.974	1.027	0.488	0.514	0.677	0.713	1.991	2.098
		0.2	1.033	0.974	1.959	1.847	0.718	0.677	1.035	0.975	0.516	0.487	0.727	0.685	2.193	2.067
		0.4	1.125	0.918	2.037	1.663	0.769	0.627	1.130	0.922	0.562	0.459	0.804	0.656	2.471	2.018
		0.6	1.290	0.860	2.196	1.464	0.864	0.576	1.310	0.873	0.645	0.430	0.941	0.627	2.965	1.977
		0.8	1.699	0.801	2.618	1.234	1.107	0.522	1.756	0.828	0.849	0.400	1.271	0.599	4.110	1.938
		1.0	0.000	0.738	0.000	1.000	0.000	0.466	0.000	0.788	0.000	0.370	0.000	0.572	0.000	1.913
0.9	0.7	0.0	0.957	1.009	1.906	2.009	0.673	0.710	0.972	1.025	0.481	0.507	0.673	0.709	1.963	2.069
		0.2	1.020	0.961	2.012	1.897	0.706	0.665	1.069	1.008	0.518	0.489	0.737	0.695	2.194	2.069
		0.4	1.119	0.914	2.193	1.790	0.759	0.619	1.221	0.997	0.578	0.472	0.835	0.681	2.549	2.081
		0.6	1.300	0.867	2.514	1.676	0.858	0.572	1.469	0.979	0.686	0.457	1.003	0.669	3.060	2.040
		0.8	1.747	0.823	3.290	1.551	1.113	0.525	2.048	0.966	0.943	0.445	1.394	0.657	4.308	2.031
		1.0	0.000	0.777	0.000	1.400	0.000	0.482	0.000	0.953	0.000	0.433	0.000	0.646	0.000	2.015
0.9	0.9	0.0	0.950	1.001	1.902	2.005	0.664	0.700	0.967	1.019	0.479	0.505	0.669	0.706	1.983	2.091
		0.2	1.041	0.981	2.095	1.975	0.720	0.679	1.071	1.010	0.534	0.503	0.748	0.705	2.162	2.039
		0.4	1.179	0.962	2.401	1.960	0.807	0.659	1.242	1.015	0.615	0.502	0.862	0.704	2.522	2.059
		0.6	1.416	0.944	2.843	1.895	0.960	0.640	1.526	1.017	0.751	0.501	1.056	0.704	3.135	2.090
		0.8	1.965	0.926	3.909	1.843	1.319	0.622	2.157	1.017	1.060	0.500	1.492	0.703	4.433	2.090
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	1.007	0.000	0.498	0.000	0.701	0.000	2.020
1.0	0.1	0.0	1.000	1.000	2.001	2.001	0.699	0.699	1.000	1.000	0.500	0.500	0.699	0.699	2.001	2.001
		0.2	1.024	0.916	2.001	1.789	0.700	0.626	1.001	0.895	0.502	0.449	0.725	0.649	2.049	1.833
		0.4	1.066	0.826	2.001	1.550	0.702	0.544	1.002	0.776	0.504	0.391	0.769	0.595	2.131	1.651
		0.6	1.150	0.727	2.003	1.267	0.706	0.447	1.005	0.636	0.511	0.323	0.854	0.540	2.289	1.448
		0.8	1.390	0.622	2.007	0.897	0.720	0.322	1.015	0.454	0.538	0.241	1.084	0.485	2.724	1.218
		1.0	0.000	0.517	0.000	0.200	0.000	0.176	0.000	0.192	0.000	0.169	0.000	0.432	0.000	0.962
1.0	0.3	0.0	1.002	1.002	2.010	2.010	0.699	0.699	1.002	1.002	0.500	0.500	0.700	0.700	2.010	2.010
		0.2	1.056	0.944	2.011	1.799	0.717	0.641	1.020	0.912	0.522	0.467	0.750	0.671	2.138	1.913
		0.4	1.145	0.887	2.031	1.573	0.748	0.579	1.054	0.816	0.560	0.434	0.828	0.642	2.354	1.824
		0.6	1.310	0.828	2.069	1.309	0.813	0.514	1.127	0.713	0.634	0.401	0.967	0.612	2.744	1.735
		0.8	1.721	0.770	2.196	0.982	1.005	0.450	1.360	0.608	0.826	0.370	1.301	0.582	3.662	1.638
		1.0	0.000	0.711	0.000	0.600	0.000	0.393	0.000	0.522	0.000	0.340	0.000	0.552	0.000	1.551
1.0	0.5	0.0	1.000	1.000	2.013	2.013	0.699	0.699	1.000	1.000	0.500	0.500	0.699	0.699	2.013	2.013
		0.2	1.061	0.949	2.047	1.831	0.732	0.655	1.065	0.952	0.530	0.475	0.752	0.673	2.202	1.969
		0.4	1.157	0.896	2.125	1.646	0.785	0.608	1.164	0.901	0.578	0.448	0.833	0.645	2.520	1.952
		0.6	1.330	0.841	2.282	1.443	0.885	0.560	1.352	0.855	0.665	0.421	0.978	0.619	3.012	1.905
		0.8	1.765	0.789	2.735	1.223	1.138	0.509	1.816	0.812	0.879	0.393	1.323	0.591	4.180	1.869
		1.0	0.000	0.728	0.000	1.000	0.000	0.457	0.000	0.774	0.000	0.364	0.000	0.565	0.000	1.841
1.0	0.7	0.0	1.000	1.000	2.013	2.013	0.699	0.699	1.000	1.000	0.500	0.500	0.699	0.699	2.013	2.013
		0.2	1.067	0.955	2.110	1.888	0.734	0.657	1.103	0.987	0.540	0.483	0.767	0.686	2.228	1.993
		0.4	1.173	0.909	2.288	1.772	0.791	0.613	1.261	0.977	0.603	0.467	0.870	0.674	2.577	1.996
		0.6	1.364	0.863	2.609	1.650	0.897	0.567	1.517	0.959	0.716	0.453	1.046	0.661	3.146	1.990
		0.8	1.832	0.819	3.455	1.545	1.168	0.522	2.119	0.947	0.987	0.441	1.454	0.650	4.432	1.982
		1.0	0.000	0.776	0.000	1.400	0.000	0.481	0.000	0.936	0.000	0.430	0.000	0.640	0.000	1.960

Table 1. Equivalent Length Factors for Various End Conditions

$[K_1 = l_{1\,(eff)}/L_T; K_2 = l_{2\,(eff)}/L_T]$

END CONDITION (BOTTOM - TOP)			(1) PIN - PIN		(2) FIX - FREE		(3) FIX - PIN		(4)FIX -SLIDER		(5) FIX - FIX		(6) PIN - FIX		(7) PIN-SLIDER	
I1/I2	ℓ2/LT	P2/PT	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2	K1	K2
1.0	0.9	0.0	1.000	1.000	2.013	2.013	0.699	0.699	1.000	1.000	0.500	0.500	0.699	0.699	2.083	2.083
		0.2	1.097	0.981	2.198	1.966	0.759	0.679	1.117	0.999	0.558	0.499	0.781	0.699	2.236	2.000
		0.4	1.242	0.962	2.508	1.943	0.851	0.659	1.292	1.001	0.642	0.498	0.901	0.698	2.592	2.008
		0.6	1.493	0.944	3.089	1.954	1.012	0.640	1.579	0.998	0.785	0.496	1.104	0.698	3.294	2.083
		0.8	2.086	0.933	4.262	1.906	1.391	0.622	2.231	0.998	1.108	0.495	1.560	0.698	4.659	2.083
		1.0	0.000	0.909	0.000	1.800	0.000	0.605	0.000	0.997	0.000	0.495	0.000	0.695	0.000	1.998