



Steel Design Guide Series

Industrial Buildings

Roofs to Column Anchorage



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Steel Design Guide Series



Industrial Buildings

Roofs to Column Anchorage

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Computerized Structural Design, Inc.

Milwaukee, WI

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PREFACE

This booklet was prepared under the direction of the Committee on Research of the American Institute of Steel Construction, Inc. as part of a series of publications on special topics related to fabricated structural steel. Its purpose is to serve as a supplemental reference to the AISC Manual of Steel Construction to assist practicing engineers engaged in building design.

The design guidelines suggested by the author that are outside the scope of the AISC Specifications or Code do not represent an official position of the Institute and are not intended to exclude other design methods and procedures. It is recognized that the design of structures is within the scope of expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

The sponsorship of this publication by the American Iron and Steel Institute is gratefully acknowledged.

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1. INTRODUCTION

Part 1

INDUSTRIAL BUILDINGS - GENERAL

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.

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 however, 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 which 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 which often 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 which 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 based on a 50 year recurrence interval - maximum 5 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, and
 - 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, and
- 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 which 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, and
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 Low-Rise Buildings which is part of the AISC Steel Design Guide Series⁽¹⁷⁾ 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, of 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 (e.g., 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 authors have encountered the requirement of design for a uniform load and the general lack of appreciation of the inadequacy of such a 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⁽³³⁾.

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 bolts 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 masonry or concrete, and metal panels are used above.)
2. Installation of fender posts (bullards) 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 guard rails 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. Formal notification should be made to the owner of 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 (e.g. food processing plants) dust control is essential. Ideally there should be no horizontal surfaces on which dust can accumulate. Tube sections 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 (i.e. tubes or box sections may be preferable to beams 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 which guides 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 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 1-1/2 in. deep, but deeper units are also available. The Steel Deck Institute⁽⁹⁾ has identified three standard profiles for 1-1/2 in. steel deck, (narrow rib, intermediate rib and wide rib) and has published load tables for each profile for thicknesses varying from .0299 to .0478 inches. 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 which results in the least weight per square foot 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 US, 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.

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⁽³⁾ (Table 4.2).

Factory Mutual in its Loss Prevention Guide (LPG)I-28 Insulated Steel Deck⁽²³⁾ provides a standard for attachment of insulation to steel deck. LPG 1-29 Loose Laid Ballasted Roof Coverings⁽²³⁾ 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 inches (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 twelve inches 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⁽³⁸⁾ 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 through use of Ref. 11.

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 square foot 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. Special treatment for acoustical reasons must be specified by the owner.

4.2 Metal Roofs

Standing seam roof systems were first introduced in the late 1960's, 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

Recommended Maximum Spans for Construction and Maintenance Loads Standard 1-1/2 Inch and 3 Inch 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'-6"
	3DR22	2 or more	13'-0"	
	3DR20	1	12'-6"	4'-0"
	3DR20	2 or more	14'-8"	
	3DR18	1	15'-0"	4'-10"
	3DR18	2 or more	17'-8"	

NOTE: SEE SDI LOAD TABLES FOR ACTUAL DECK CAPACITIES.

Table 4.1 Steel Deck Institute Recommended Spans⁽³⁸⁾

Types 1.5A, 1.5F, 1.5B and 1.5BI Deck. Nominal 1-1/2 in. (38 mm) depth. No stiffening grooves.			
	22g.	20g.	18g.
Type 1.5A Narrow Rib	4'10" (1.5m)	5'3" (1.6m)	6'0" (1.9m)
Type 1.5F Intermediate Rib	4'11" (1.5m)	5'5" (1.7m)	6'3" (2.0m)
Type 1.5B, BI Wide Rib	6'0" (1.8m)	6'6" (2.0m)	7'5" (2.3m)

Table 4.2 Factory Mutual Data⁽³⁾

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 as-

sume 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 are 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.⁽¹⁴⁾

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. The report gives modifying factors which 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.

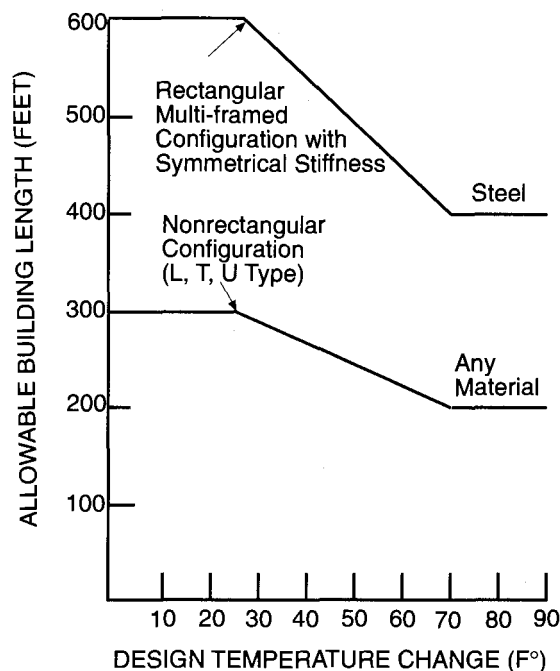


Fig. 4.4.1 Expansion Joint Spacing Graph

[Taken from F.C.C. Tech. Report No. 65, Expansion Joints in Buildings]

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 prevail 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 such as Fig. 4.4.2 are employed, low friction sliding elements are generally used. Slip connections may induce some level of inherent restraint to movement due to binding or debris build-up.

Very often buildings may be required to have fire walls in specific locations. Fire walls may be required to extend above the roof or it may be allowed to terminate at the underside of the roof. Such fire walls 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.

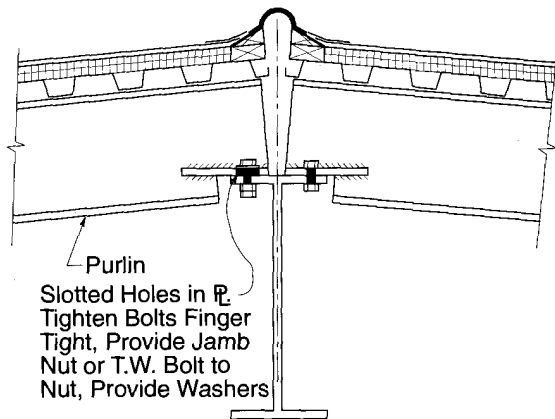


Fig. 4.4.2 Beam Expansion Joint

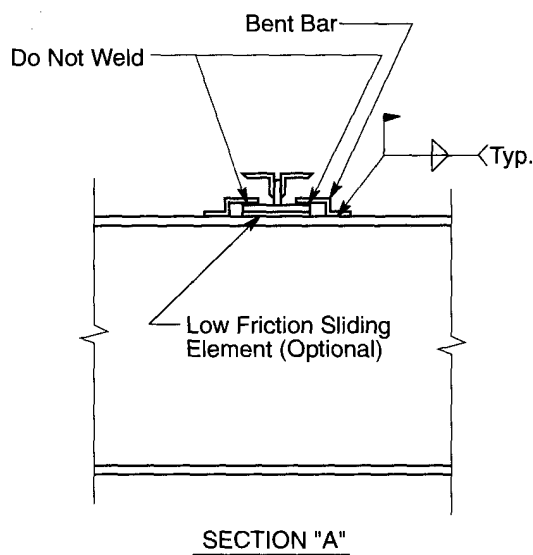
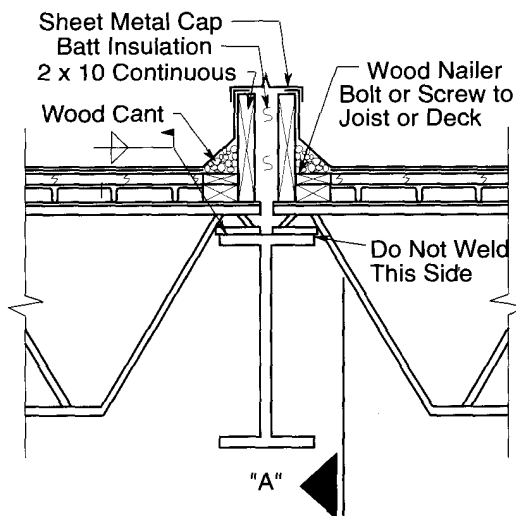


Fig. 4.4.3 Joist Expansion Joint

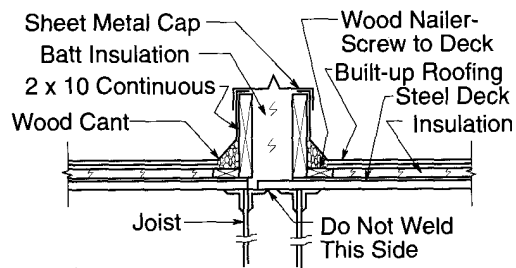


Fig. 4.4.4 Joist Expansion Joint

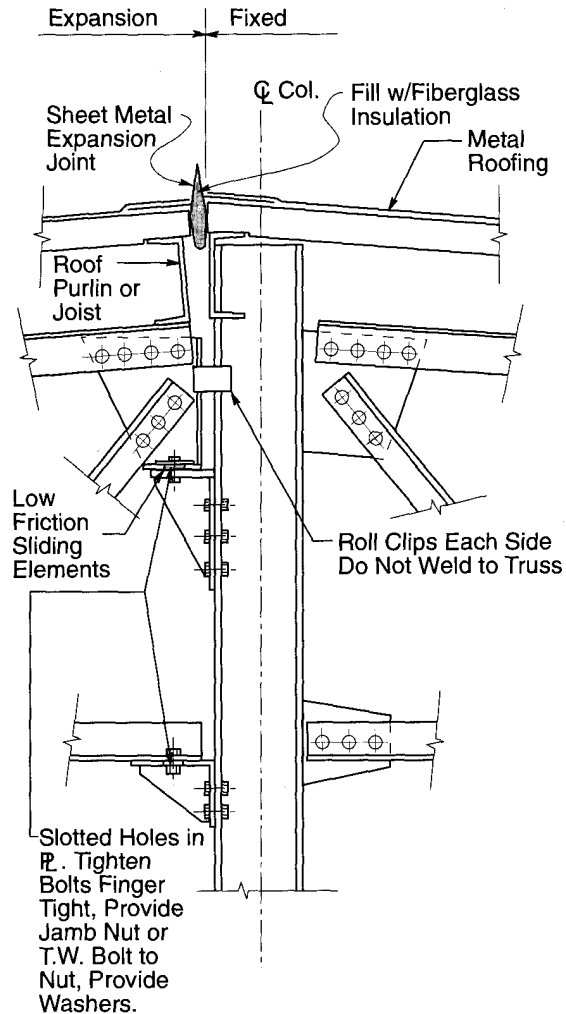


Fig. 4.4.5 Truss Expansion Joint

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 150-250 feet for adhered membranes, at somewhat greater intervals for ballasted membranes, and 100-200 feet 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 stormwater 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. Problems (e.g., ponding, roofing deterioration, leaking) which may result from poor drainage lead to the recommendation that a roof slope of at least 1/4 in. per ft. be provided for all building roof systems. 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 rain storm. Ponding of this nature is addressed in roof design by positive roof drainage and control of the deflections of roof framing members. Ponding an issue in structural engineering is a load/deflection situation in which there is incremental accumulation of rain water in the deflecting structure. The purpose of a ponding check is to insure that an 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.

The AISC Specifications for both LRFD⁽²²⁾ and ASD⁽⁴²⁾ 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 Chap. 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 Deck Institute⁽⁴¹⁾ gives some guidance as to the amount of snow load which 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 which 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 non-uniformly distributed roof loads, 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.

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.

On the use of steel joists or purlins, experience again shows that each case must be studied. Standard steel joist specifications (See Ref. 40) are based upon distributed loads only. Modifications for concentrated loads should be done in accordance with the SJI Code Of Standard Practice. Significant concentrated loads should be supported by hot rolled framing members. 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 (AISI) Specification for the Design of Cold-Formed Steel Structural Members⁽⁴³⁾ or the AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members⁽²¹⁾ 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 (bolts) 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 may normally be 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 30 to 40 feet steel beams provide an economical solution, particularly if a multitude of hanging loads are present. For spans greater than 40 but less than 80 feet steel joist girders are often used to support roof loads. Fabricated steel roof trusses are often used for spans greater than 80 feet. 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 about 14 feet. 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 feet are generally shippable and 100 feet is often possible. Since maximum available mill length is approximately 70 feet, the distance between splice points is normally set at 70 feet. 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 which 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 above) usually provides the use of more efficient truss members.
 6. Illustrated in Figs. 5.1.1 and 5.1.2 are web arrangements which 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 $4 \times 3 \times 1/4$ and an angle $4 \times 3 \times 5/16$ in the same truss.

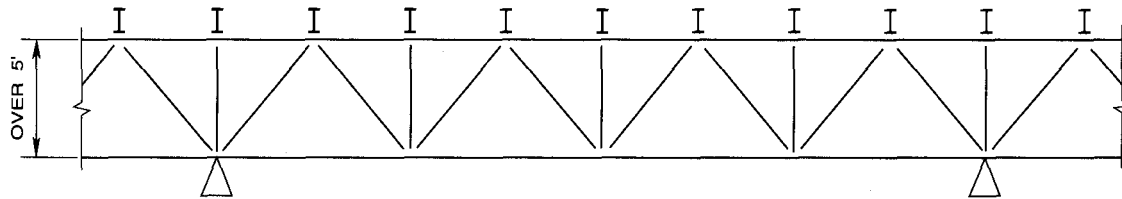


Fig. 5.1.1 Economical Truss Web Arrangement

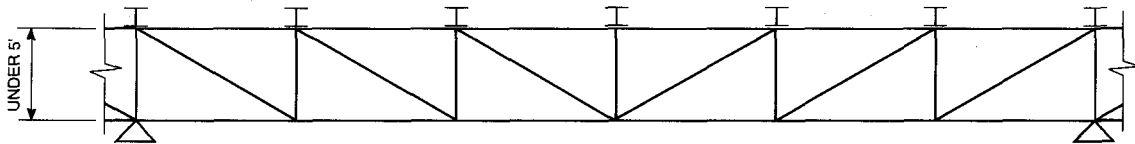


Fig. 5.1.2 Economical Truss Web Arrangement

8. Tube, 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⁽²²⁾ 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 bending moments will show up in the analysis. The reader is referred to Reference 27 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, e.g. 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. AISC Engineering for Steel Construction⁽¹³⁾ 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.⁽⁴⁰⁾

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 under Column Design; Double Angles.
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 over-size 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 $.004P$. 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 which 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 $.025P$ force requirement for bracing. More refined bracing equations are contained in Reference 20. Requirements for truss bottom chord bracing are discussed in Reference 15. 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. For columns, the International Standards Code of steel structure design (ISO) has postulated the equation:

$$F_n = F_1 (.2n + .8\sqrt{n})$$

where F_n is the assumed brace force for n columns, F_1 equals $.02N$ where N is the axial force in the column. This equation thus suggests that $(.2n + .8\sqrt{n})$ trusses should be considered in the bracing design. Thus, if ten trusses were to be braced, bracing forces could be based on five trusses. 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 Reference 28. 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 is comprised of axially loaded members arranged as shown in Fig. 5.3.1, the bracing can be designed for 0.6 percent of the truss chord axial load. Since two truss

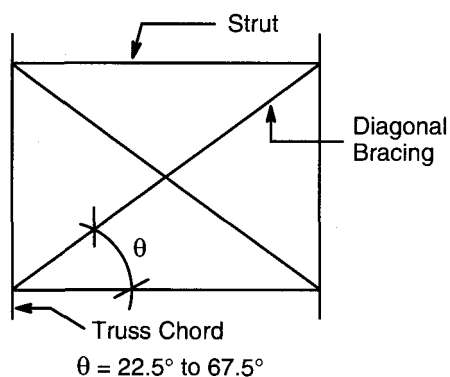


Fig. 5.3.1 Horizontal X-Bracing Arrangement

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 Fig. 5.3.2 determine the brace forces in the horizontal bracing system. Use the procedure discussed in Reference 28.

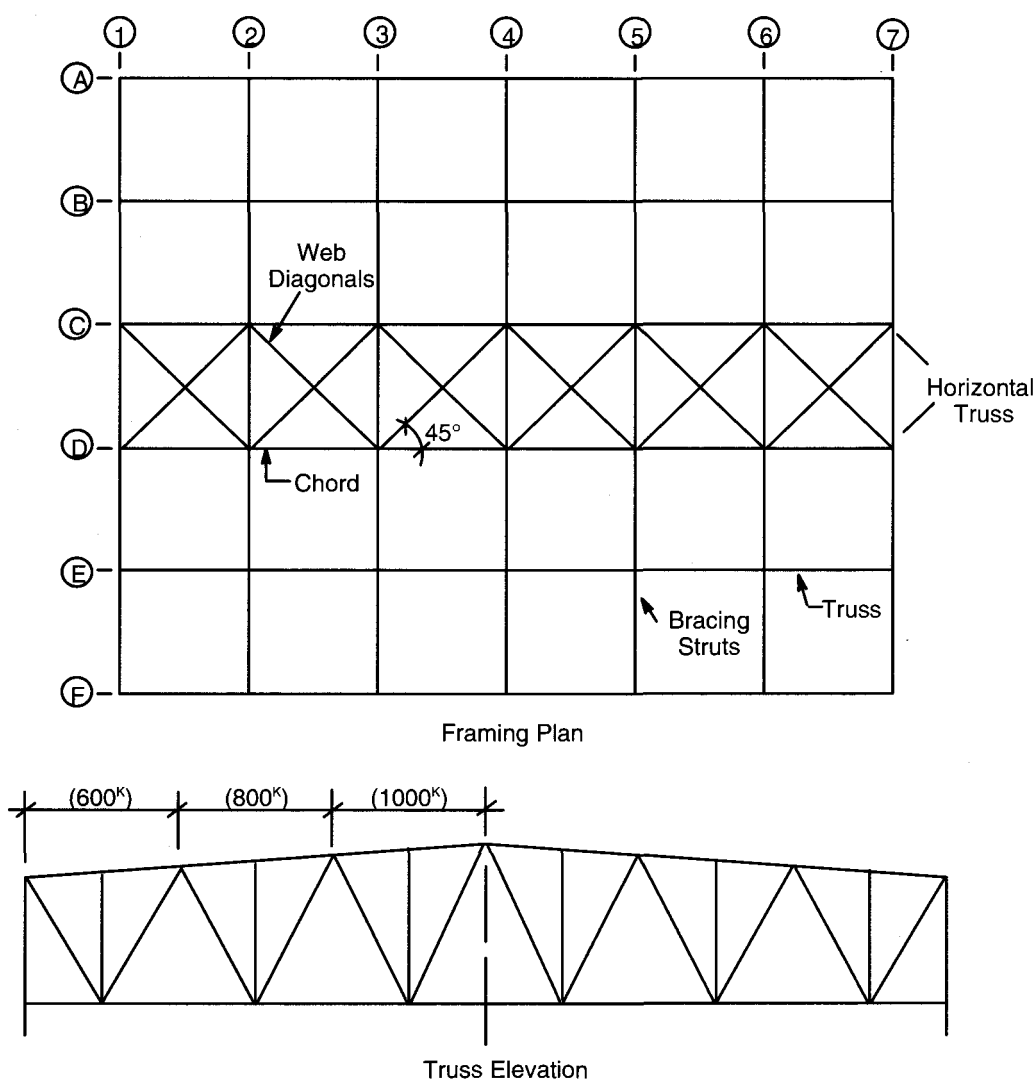


Fig. 5.3.2 Horizontal Bracing System

Solution:

Since the diagonal bracing layout as shown in Fig. 5.3.2 forms an angle of 45 degrees with the trusses, the

solution used in Reference 28 is suitable. The bracing force thus equals .6% of the chord axial load. Member forces are summarized below.

DESIGN FORCES (KIPS)		
HORIZONTAL TRUSS WEB MEMBER FORCES		
MEMBER	PANEL SHEAR	FORCE = (1.414) (PANEL SHEAR)
C1-D2 D1-C2	.006(6x600) = 21.6	30.5
C2-D3 D2-C3	.006(6x800) = 28.8	40.7
C3-D4 D3-C4	.006(6x1000) = 36	50.9
HORIZONTAL TRUSS CHORD FORCES		
MEMBER	MEMBER FORCE	
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	8.4	
B2-C2, D2-E2	16.8	
C2-D2	25.2	
A1-B1, E1-F1	3.6	
B1-C1, D1-E1	7.2	
C1-D1	10.8	

Note: Forces not shown are symmetrical

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. 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 Fig. 5.4.1). If top chord seats are 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. These concepts are shown in Fig.5.4.1.

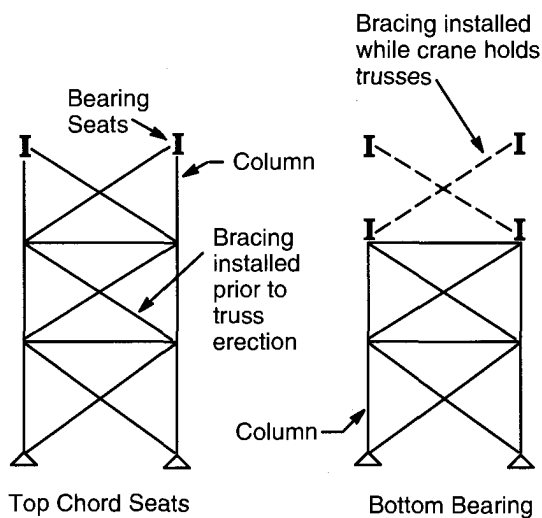


Fig. 5.4.1 Wall Bracing Erection Sequence

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 side wall 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, economy can generally be recovered due to a savings in bracing and erection time.
3. Wide flange beams, tubes 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 not 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.
4. Wind forces on the trusses during erection can be considerable. The AISC Code of Standard Practice states that "temporary supports will secure the steel framing, or any partly assembled steel framing, against loads comparable in intensity to those for which the structure was designed, resulting from wind, seismic forces and 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 Fig. 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.

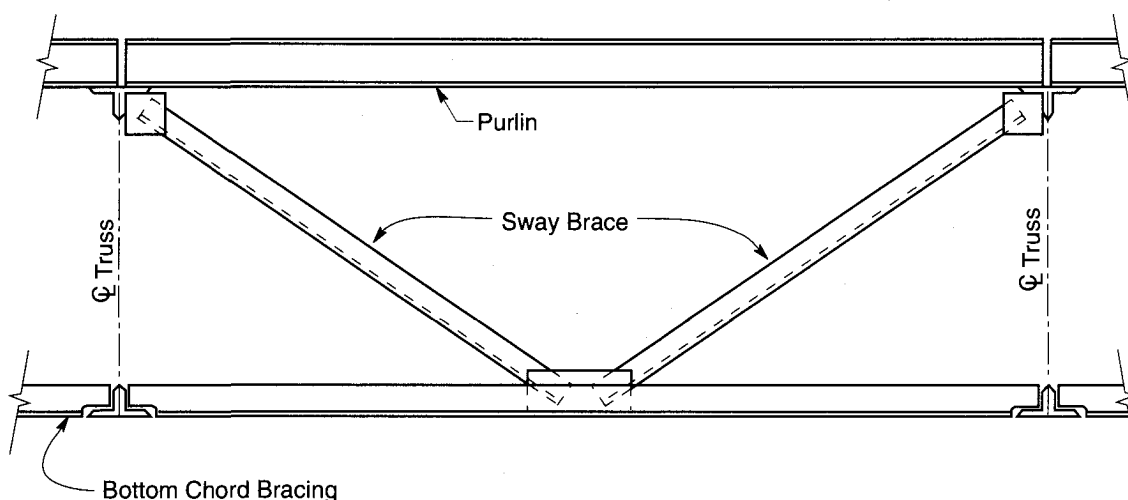


Fig. 5.4.2 Sway Frame

5.5 Other Considerations

1. Camber: Large clear span trusses are generally cambered to accommodate dead load deflections. This is accomplished by the fabricator, 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 which can accommodate temperature changes are generally necessary. Long span trusses which are fabricated at one temperature and erected at a significantly different temperature can grow or shrink significantly.
3. Roof deck diaphragm strength and stiffness is 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.

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 a 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,
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 which has been used successfully is shown in Fig. 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 ASHRAE90.1.⁽¹²⁾

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.

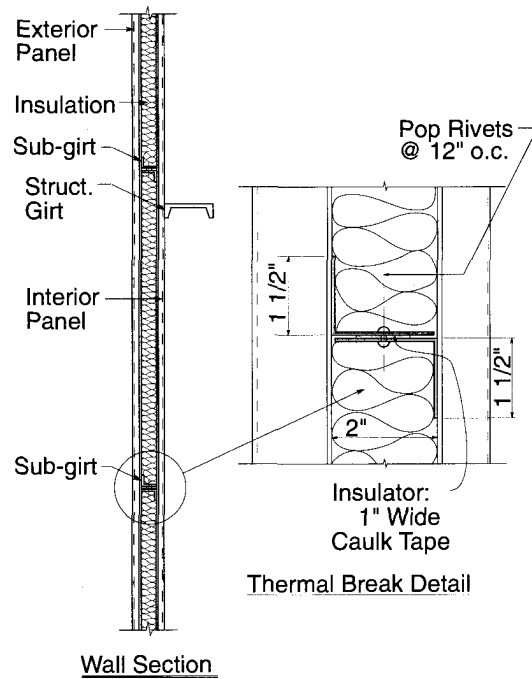


Fig. 6.1.1 Wall Thermal Break Detail

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 non-composite. 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 which may be a problem in high humidity situations.

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. They 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 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. The 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 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, or
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.⁽³⁴⁾

For the design of hot-rolled channels, the beneficial lateral support provided by cladding attached to the tension flange has generally been ignored. 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. Lateral stability based on this assumption is checked using Chap. 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 Chap. 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.

Cold-formed girts should be designed in accordance with the provisions of the American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Structural Members,⁽⁴³⁾ or the AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members.⁽²¹⁾ Many manufacturers of cold-formed girts have provided this design 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 sheathing (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.⁽³⁴⁾ 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:

$$M_n = R S_e F_y$$

where

- R = 0.40 for simple span C sections.
= 0.50 for simple span Z sections.
= 0.60 for continuous span C sections.
= 0.70 for continuous span Z sections.
- S_e = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y .
- F_y = Design yield stress as determined in Section A5.2.1 of the Specification.

The procedure does not apply to a continuous girt system between inflection points adjacent to a support.

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

6.6 Wind Columns

When bay spacings exceed 30 feet 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 which 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" (i.e. "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 (i.e. Fig. 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 Fig. 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 D1.3,⁽⁴⁴⁾ or from the AISI Specifications.⁽²¹⁾⁽⁴³⁾ 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 Ref. 11.

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 owners 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

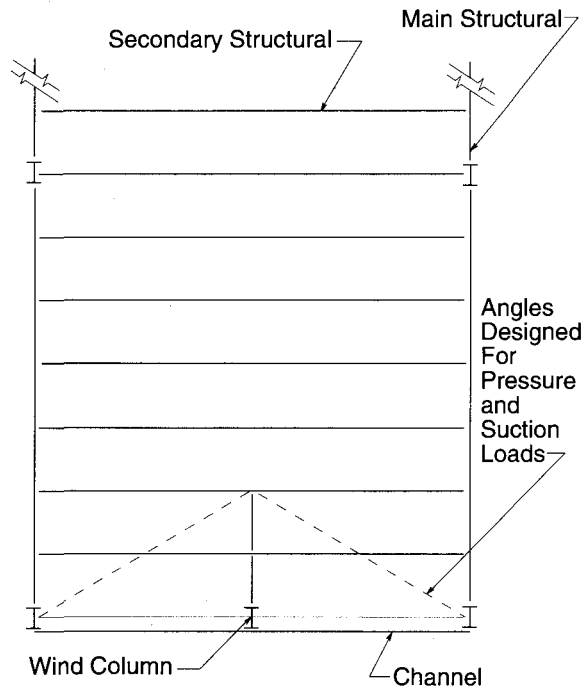


Fig. 6.6.1 Wind Column Reaction Load Transfer

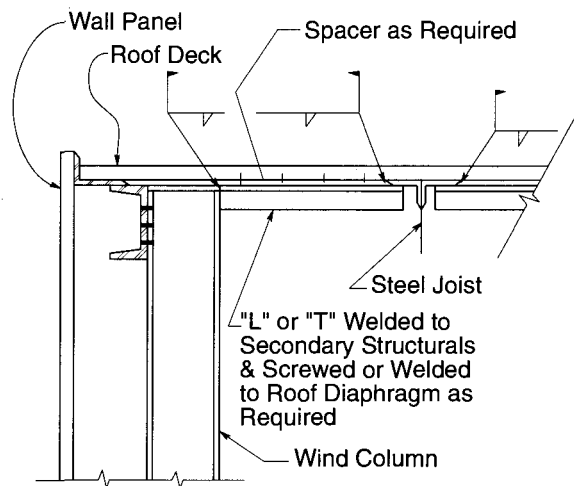


Fig. 6.6.2 Wind Column Reaction Load Transfer

with expansion. Extra dollars invested at the outset reduce potential future costs.

The economics of use 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 (e.g., 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.

AISC offers two manuals that may aid in the design of efficient framing details: Engineering for Steel Construction⁽¹³⁾ and Detailing for Steel Construction.⁽¹⁰⁾

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 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 feet by 300 feet with potential future expansion in the long direction may best be suited for rigid frames to minimize or eliminate bracing which would interfere with future changes.

Use of a metal building system requires a strong interaction between the designer and the metal building manufacturer, because of much of the detailing process concerning the 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.

7.2 Tube Columns vs. W Shapes

The design of columns in industrial buildings includes considerations which do not apply to other types of structures. Interior columns can normally be braced only at the top and bottom, thus square tube columns are desirable due to their equal stiffness about both principal axes. Difficult connections with tube members can be eliminated in single-story frames by placing the beams over the tops of the tubes. Thus a simple to fabricate cap plate detail with bearing stiffeners on the girder web may be designed. Other advantages of tube 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 tubes 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 tube section. Because tubes have no flanges, extra clip angles are required to connect girts.

7.3 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.

The development of the most economical framing scheme for a roof system is a worthy goal. 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'-0" x 240'-0" building with a 25'-0" 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 tubes with a yield strength of 46 ksi.

Variables in the analysis were:

1. Joist spans: 25, 30, 40, 50 and 60 feet.
2. Girder spans, W sections: 25, 30, 40, 48 and 60 feet.

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 feet and joists spanning 40 feet.

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' x 240' building being considered wind columns could add \$0.10 per square foot, of roof, to the cost. Interestingly, if the building were 120' x 120', the addition of intermedi-

		RELATIVE COSTS*				
		MAIN FRAMING MEMBERS - W SECTIONS				
JOIST DATA Depth (in.)	Span (ft.)	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.

ate wind columns would add \$0.20 per square foot 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 pattern 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 proper design can be provided by the joist manufacturer. The procedure must be used with conscious engineering judgment and full recognition that standard steel joists are designed as simple span members subject to dis-

tributed loads. (See Standard Specifications for Standard Steel Joists and Long Span Joists.)⁽⁴⁰⁾ 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 which 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 Roof Bracing 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.

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 designers or contractors preference. Thus the main design variable is the spacing of the fasteners. The designer calculates the maximum shear per foot of diaphragm and then selects the fastener spacing from the load tables. Load tables are most often based on the requirements set forth in References (11) and (37).

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 References (11) and (37). 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 Reference (11).

EXAMPLE 8.2.1: Diaphragm Design

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

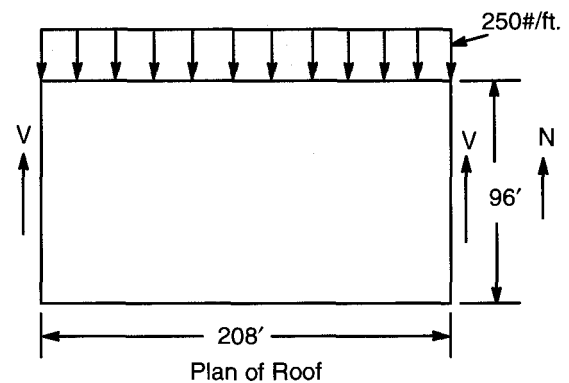


Fig. 8.2.1. Example

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 inch thick) intermediate rib deck spanning 5'-6" 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 sidelaps.

Solution:

1. Calculate the maximum diaphragm shear.

$$V = WL/2 = (250)(208)/2 \approx 26,000 \text{ lbs.}$$

$$v = V/96 = 26,000/96 \approx 271 \text{ plf.}$$
2. Obtain the shear capacity of the deck from the SDI Diaphragm Design Manual, Second Edition.

For a 20 gage (.0358" thickness) deck, spanning 5'-6" the allowable shear is:

- (a) 240 plf with a 36/4 weld pattern and one side lap screw.
- (b) 285 plf with a 36/4 weld pattern and two side lap screws.
- (c) 300 plf with a 36/5 weld pattern and one side lap screw.

Use patterns (a) and (b) as shown in Fig. 8.2.1.1:

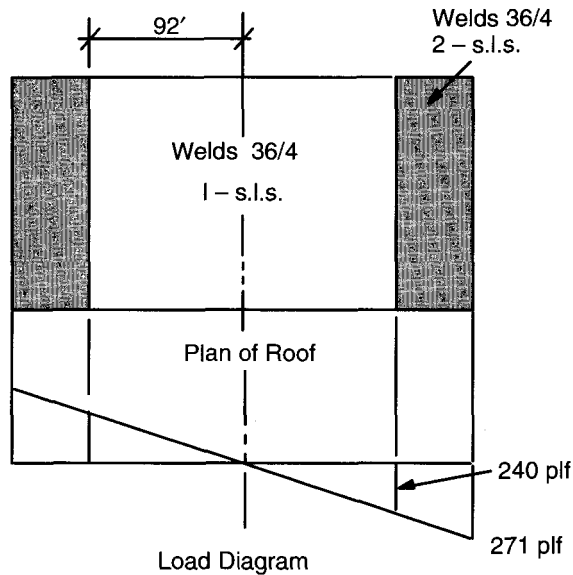


Fig. 8.2.1.1

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: $\Delta_b = \frac{5wL^4}{384EI}$

(b) For shear: $\Delta_s = \frac{wL^2}{8DG'}$

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

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

From the SDI tables:

$$K2 = 1056$$

$$D_{xx} = D_{ir} = 909 \text{ (intermediate rib)}$$

$$K1 = 0.561 \quad K1 = .509 \text{ corresponds to 22 ga. deck.}$$

$$\therefore G' = \frac{1056}{3.78 + .3(909) / 5.5 + 3(.561)5.5} = 16.9$$

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:

$$I = 2Ad^2 = (2)(3.0)(48 \times 12)^2 = 1.99 \times 10^6 \text{ in.}^4$$

The bending deflection equals:

$$\Delta_b = \frac{(5)(.25)(208)^4(1728)}{(384)(29000)(1.99 \times 10^6)} = 0.18 \text{ in.}$$

The shear deflection equals:

$$\Delta_s = \frac{(.25)(208)^2}{(8)(96)(16.9)} = 0.83 \text{ in.}$$

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, 5/8 in. dia. arc spot welds 9 inches 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 Fig. 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 Fig. 8.2.3 is a typical shear collector.

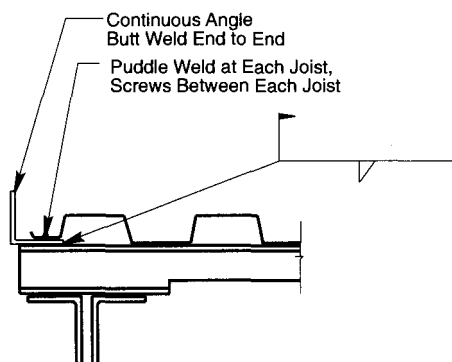


Fig. 8.2.2 Eave Angle

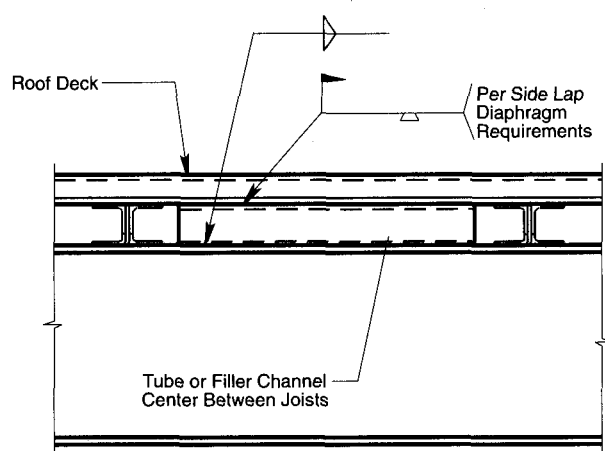


Fig. 8.2.3 Shear Collector

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 1/4 in. bar stock does not usually interfere with deck placement and facilitates erection.

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 by the AISC Specifi-

cation^(22,42) (Section M4.2) and by the Code of Standard Practice⁽⁶⁾ (Section 7.9).

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

First, the broad standard means that it is difficult to judge the adequacy of temporary bracing in a particular situation, nor is there a codified standard upon which to judge whether or not a minimum level of conformity is met. Secondly, it is not emphasized in the broad standard that the process of erection can induce forces and stresses into components and systems which are not part of the structural steel framework. Unless otherwise specified in the contract documents, it is generally the practice of architects and engineers to design the elements and systems in a building for the forces acting upon the completed structure only.

The lack of clear standards makes it 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, which is inconsistent with maintaining the bracing as the sole responsibility of the erector. The lack of emphasis on the need to check the effect of erection forces on other elements allows erection problems to be interpreted as being caused by other reasons. This is most obvious in the erection of steel columns. In order 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 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 they are braced by adequate guys or designed as a rigid frame 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 very large forces can be induced into anchor bolts, 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. Fig. 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.

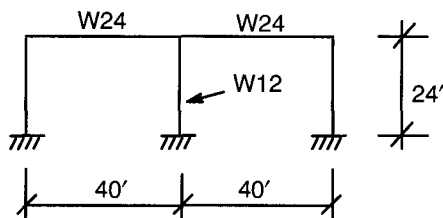


Fig. 8.3.1 Erection Bracing Example

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 foot-pounds occurs. If a 5 inch by 5 inch placement pattern were used with four anchor bolts and an ungrouted base plate, a tension force of approximately 21.6 kips would be applied to the two anchor bolts. The allowable force for a 3/4 in A36 anchor bolt is 8.4 kips. Even if the bolts were fully developed (and the 33% increase for wind was utilized) in the concrete, they would be severely overstressed and would likely fail. Four 1-1/8 inch anchor bolts would be required to resist the wind force. Of course not only the size of the anchor bolt is affected, but the design of the base plate and its attachment to the column, the spacing of the anchor bolts and the design of the pier and footing must also be checked.

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. This must also be checked by the erector. The placement of material such as decking on the incomplete structure can induce unanticipated loadings. This loading must also be considered explicitly. The foregoing points to the need for a well planned bracing scheme prepared and executed by the erector.

Erection bracing involves other issues as well. First, the Code of Standard Practice distinguishes between self-supporting and non-self-supporting structures. The distinction is drawn because the timing of the removal of bracing is affected. In a self-supporting structural steel frame, lateral stability is achieved in the design and detailing of the framework itself. Thus the bracing can be removed when the erector's work is complete. A non-self-supporting steel framework may rely on elements other than the structural steel to provide lateral stability. Such frames should be identified in the contract documents along with the necessary elements to be installed to provide the stability. The coordination of the installation of such elements is a matter which must be addressed between the erector and the owner's agents.

Temporary support beyond the requirements discussed above would be the responsibility of the owner according to the Code of Standard Practice.

The timing of column base grouting affects the performance of column bases during erection. Although the Code of Standard Practice assigns the responsibility for grouting to the owner, the erector should be involved in the coordination of this work.

All of the foregoing points to the need for care, attention and thoroughness on the part of the erector.

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 bolts. Shear forces are transferred to the foundation system through friction, shear friction, or bearing. 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 bolts have caused numerous structural problems in industrial buildings. These problems include:

1. inadequate sizing of the anchor bolts,
2. inadequate development of the anchor bolts for tension,
3. inadequate design or detailing of the foundation for forces from the anchor bolts,
4. inadequate base plate thickness,
5. inadequate design and/or detailing of the anchor bolt - base plate interface,
6. misalignment or misplacement of the anchor bolts during installation, and
7. fatigue.

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

9.1 Resisting Tension Forces with Anchor Bolts

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

1. Determine the maximum net uplift for the column.
2. Select the anchor bolt material and number and size of anchor bolts to accommodate this uplift.
3. Determine the appropriate base plate size, thickness and welding to transfer the uplift

forces from the column to the anchor bolts. Refer to AISC Design Guide 1.⁽⁶⁾

4. Determine the method for developing the anchor bolt in the concrete (i.e. transferring the tension force from the anchor bolt 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 bolts are often specified to be ASTM A307 material. However, these bolts are frequently not available (or readily available) for the anchor bolt lengths required. In these instances a threaded rod with a hook or nut is usually used in lieu of an A307 bolt. In anticipation of this, the ASTM A307 specification states that "non-headed anchor bolts, either straight or bent, to be used for structural anchorage purposes, shall conform to the requirements of ASTM Specification A36". In other words, if anchor bolts are designated to be A307 material but are not available for the lengths required, then non-headed anchor bolts made from threaded rods of A36 material with hooks or nuts are to be used. The ASTM A36 specification states that the thread dimensions and nuts to be used with these threaded rods are to conform to the ASTM A307 specification. Consequently, since the ultimate tensile strengths of A36 material and A307 bolts are approximately equal, this substitution results in anchor bolts of approximately equal strength.

Anchor bolts of higher tensile strength than A307 bolts or A36 threaded rods may be used if desired. Table J3.2 in the AISC Specification may be consulted to obtain the allowable stress for the higher strength bolts or threaded rods.

The number of anchor bolts required is a function of the maximum net uplift on the column and the allowable tensile load per bolt for the anchor bolt material chosen. Prying forces in anchor bolts 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 bolts are positioned outside the column profile and the bolt 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 bolts 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 bolts 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 bolts to prevent fluctuation of the tensile stress in anchor bolts and, therefore, eliminate fatigue concerns. This is not recommended since creep in the supporting concrete foundation can eventually lead to relaxation of the pretensioning. Table 9.1.1 shows recommended allowable fatigue stresses for non-pretensioned steel bolts. These values are based on S-N 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 A36 or A307 anchor bolts are used. However, fatigue can govern the design of higher strength anchor bolts for this load case.

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.

Table 9.1.1 Allowable Bolt Fatigue Stress

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 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 bolt loads generate bending moments in the base plate consistent with cantilever action about the web or flanges of the column section (one-way bending). A more refined analysis for bolts 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 degree distribution from the centerline of the anchor bolt 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 - The development of anchor bolts in tension is usually accomplished by using a hook, nut, or steel plate at the embedded end of the anchor bolt. Although chemical bond developed between the anchor bolt and the surrounding concrete may also aid in developing tension in the anchor bolt, it is typically neglected because anchor bolts are often oiled during manufacturing. Appendix B of the ACI Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-85)⁽⁷⁾ recommends that the design for the development of the bolt in concrete accommodate the full tensile strength of the anchor bolt ($A_t \times F_{ut}$) to insure a ductile failure for the anchor. The authors suggest that, for non nuclear structures, the design for the development should accommodate 1.25 times the yield strength of the anchor bolt ($1.25 \times A_t \times F_y$). For an A307 or A36 anchor bolt, this is equivalent to approximately 3/4 of the full or ultimate tensile strength of the bolt. This is consistent with the provisions for development length, splices and mechanical connections listed in Chapter 12 of the ACI Building Code Requirements for Reinforced Concrete Structures (ACI 318-89)⁽⁴⁾. The designer may use his/her judgment in applying this criterion when analysis shows that tension in the anchor bolts is either nonexistent or minimal.

Hooked anchor bolts 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 this example, a hook is generally not capable of developing the recommended tensile capacity mentioned in the previous paragraph ($1.25 \times A_t \times F_y$). Therefore, hooks should only be used when tension in the anchor bolt is either nonexistent or minimal.

Tests have shown that a heavy bolt head, or a heavy hex nut on a threaded rod, will develop the full tensile capacity of even high strength anchor bolts when properly embedded and confined in concrete. Therefore, the design for development for headed anchor bolts (typically

threaded rods with heavy hex nuts) is a matter of determining the required embedment depths, edge distances and/or steel reinforcement to prevent failure in the concrete prior to the development of the recommended tensile capacity for the bolt.

As presented in Appendix B of ACI 349-85, failure occurs in the concrete when tensile stresses along the surface of a stress cone surrounding the anchor bolt 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 tensile strength of the concrete (in ultimate strength terms) is represented as a uniform tensile stress of $4\phi\sqrt{f'_c}$ over the surface area of these stress cones. By examining the geometry, it is evident that the ultimate pull-out strength of this cone is equal to $4\phi\sqrt{f'_c}$ times the projected area (A_e) of the cone at the surface of the concrete (excluding the area of the anchor head). Expressions for A_e and the ultimate pullout strength ($T_{ULT.}$) are included in Figures 9.1.1, 9.1.2 and 9.1.3.

The use of plate washers or bearing plates in lieu of or in conjunction with a heavy hex bolt head or nut can increase the surface area of the stress cone and the pullout strength of the concrete for a given embedment depth. However, this may not be the case where bolts are relatively close to the edges of the concrete structure and the extent of the stress cones surrounding the bolts is limited by these edges. In this case, the use of larger washers or bearing plates at the embedded end of the anchor bolts may actually reduce the pullout strength of the concrete (in effect, creating a weakened plane in the concrete). This is often the case with anchor bolts embedded in concrete piers. Appendix B of ACI 349-85 lists dimensional criteria that are to be maintained for washers or bearing plates used as anchor heads. These criteria were developed to be consistent with the dimensional characteristics of a standard heavy hex bolt head or nut.

The previously described stress cone checks rely upon the strength of plain concrete for developing the anchor bolts and would 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 bolts 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 develop the anchor bolts. 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 recommended tensile capacity of the anchor bolts