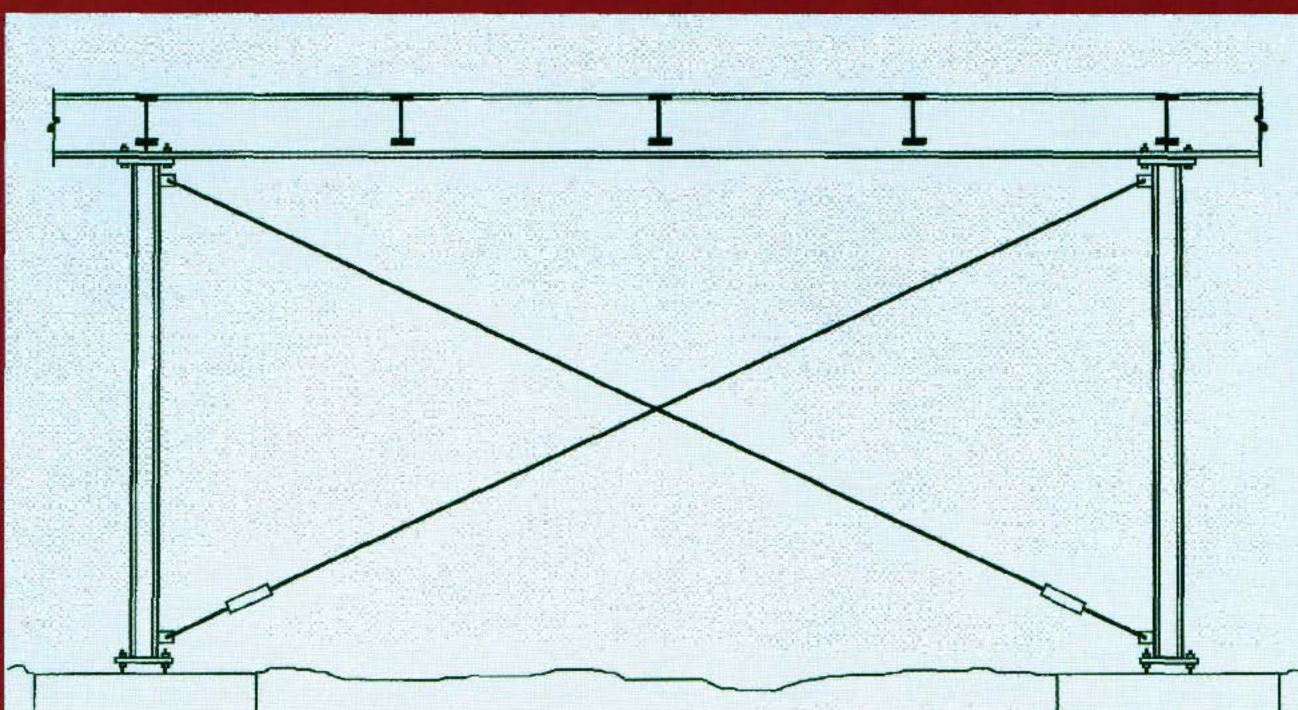
Steel Design Guide Series

10

Erection Bracing
of Low-Rise Structural Steel Buildings



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Erection Bracing
of Low-Rise Structural Steel Buildings

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1. INTRODUCTION

This guide is written to provide useful information and design examples relative to the design of temporary lateral support systems and components for low-rise buildings. For the purpose of this presentation, low-rise buildings are taken to have the following characteristics:

(1) Function: general purpose structures for such uses as light manufacturing, crane buildings, warehousing, offices, and other commercial and institutional buildings.

(2) Proportions:

(a) height: 60 feet tall or less.

(b) stories: a maximum of two stories.

Temporary support systems are required whenever an element or assembly is not or has not reached a state of completion so that it is stable and/or of adequate strength to support its self-weight and imposed loads. The need for temporary supports is identified in Paragraph M4.2 of the AISC Specification for Structural Steel Buildings and in Section 7 of the AISC Code of Standard Practice for Steel Buildings and Bridges.

To a great extent the need for this guide on temporary supports was created by the nature and practice of design and construction of low-rise buildings. In many instances, for example, the lateral bracing systems for low-rise buildings contain elements which are not in the scope of the steel erector's work. For this reason the Code of Standard Practice makes a distinction between Self-Supporting and Non-Self-Supporting framework as will be discussed later. Other temporary supports such as shoring and cribbing for vertical loads are not included in the scope of this guide.

1.1 Types of Systems

Lateral bracing systems for low-rise buildings can be differentiated as follows:

Braced construction: In this type of system, truss-like bays are formed in vertical and horizontal planes by adding diagonals in vertical bays bounded by columns and struts or in horizontal bays bounded by beams and girders. In general, braced construction would be characterized as self-supporting, however, the frames may contain elements such as a roof deck diaphragm which would change the frame to a non-self-supporting type.

Rigid Frame Construction: This system uses moment resisting joints between horizontal and vertical framing members to resist lateral loads by frame action. In many buildings the rigid frames are discretely located within the construction to minimize the number of more costly moment resisting connections. The remainder of the frame would have simple connections and the frame would be designed to transfer the lateral load to the rigid frames. Rigid frame construction would also be characterized as self-supporting, however in the case of braced construction the framework may contain non-structural elements in the system which would make it a non-self-supporting frame.

Diaphragm Construction: This system uses horizontal and/or vertical diaphragms to resist lateral loads. As stated above horizontal diaphragms may be used with other bracing systems. Horizontal diaphragms are usually fluted steel deck or a concrete slab cast on steel deck. Vertical diaphragms are called shear walls and may be constructed of cast-in-place concrete, tilt-up concrete panels, precast concrete panels or masonry. Vertical diaphragms have also been built using steel plate or fluted wall panel. In most instances, the elements of diaphragm construction would be identified as non-self-supporting frames.

Cantilever Construction: Also called Flag Pole Construction, this system achieves lateral load resistance by means of moment resisting base connections to the foundations. This system would likely be characterized as self-supporting unless the base design required post erection grouting to achieve its design strength. Since grouting is usually outside the erector's scope, a design requiring grout would be non-self-supporting.

Each of the four bracing systems poses different issues for their erection and temporary support, but they share one thing in common. All as presented in the project Construction Documents are designed as complete systems and thus all, with the possible exception of Cantilever Construction, will likely require some sort of temporary support during erection. Non-self-supporting structures will require temporary support of the erection by definition.

1.2 Current State of the Art

In high-rise construction and bridge construction the need for predetermined erection procedures and temporary support systems has long been established in the industry. Low-rise construction does not command a comparable respect or attention because of the low heights and relatively simple framing involved. Also the structures are relatively lightly loaded and the framing members are relatively light. This has lead to a number of common fallacies which are supported by anecdotal evidence.

1.3 Common Fallacies

1. Low-Rise frames do not need bracing. In fact, steel frames need bracing. This fallacy is probably a carryover from the era when steel frames were primarily used in heavy framing which was connected in substantial ways such as riveted connections.

2. Once the deck is in place the structure is stable. In fact, the steel deck diaphragm is only one component of a complete system. This fallacy obviously is the result of a misunderstanding of the function of horizontal diaphragms versus vertical bracing and may have resulted in the usefulness of diaphragms being oversold.

3. Anchor rods and footings are adequate for erection loads without evaluation. In fact, there are many cases in which the loads on anchor rods and footings may be greater during erection than the loads imposed by the completed structure.

4. Bracing can be removed at any time. In fact, the temporary supports are an integral part of the framework until it is completed and self-supporting. This condition may not even occur until some time after the erection work is complete as in the case of non-self-supporting structures.

5. The beams and tie joists are adequate as struts without evaluation. In fact, during erection strut forces are applied to many members which are laterally braced flexural members in the completed construction. Their axially loaded, unbraced condition must be evaluated independently.

6. Plumbing up cables are adequate as bracing cables. In fact, such cables may be used as part of temporary lateral supports. However, as this guide demonstrates additional temporary support cables will likely be needed in most situations. Plumbing a structure is as much an art as a science. It involves continual adjustment commonly done using diagonal cables. The size and number of cables for each purpose are determined by different means. For example, the lateral support cables would likely have a symmetrical pattern whereas the plumbing up cables may all go in one direction to draw the frame back to plumb.

7. Welding joist bottom chord extensions produces full bracing. In fact, the joist bottom chords may be a component of a bracing system and thus welding them would be appropriate. However, other components may be lacking and thus temporary supports would be needed to complete the system. If the joists have not been designed in anticipation of continuity, then the bottom chords must not be welded.

8. Column bases may be grouted at any convenient time in the construction process. In fact, until the column bases are grouted, the weight of the framework and any loads upon it must be borne by the anchor rods and leveling nuts or shims. These elements have a finite strength. The timing of grouting of bases must be coordinated between the erector and the general contractor.

1.4 Use of This Guide

This guide can be used to determine the requirements for temporary supports to resist lateral forces, i.e. stability, wind and seismic. The guide is divided into two parts. Part 1 presents a method by which the temporary supports may be determined by calculation of loads and calculation of resistance. Part 2 presents a series of prescriptive requirements for the structure and the temporary supports, which if met, eliminate the need to prepare calculations. The prescriptive requirements of Part 2 are based on calculations prepared using the principles presented in Part 1.

PART1

DETERMINATION OF BRACING REQUIREMENTS BY CALCULATION METHOD

2. INTRODUCTION TO PART 1

Part 1 consists of three sections. The first deals with design loads which would be applicable to the conditions in which the steel framework exists during the construction period and specifically during the period from the initiation of the steel erection to the removal of the temporary supports. Sections 4 and 5 deal with the determination of resistances, both of permanent structure as it is being erected and of any additional temporary supports which may be needed to complete the temporary support system. An appendix is also presented which provides tabulated resistances to various components of the permanent structure. This appendix follows the reference section at the end of the guide.

3. CONSTRUCTION PHASE LOADS FOR TEMPORARY SUPPORTS

The design loads for temporary supports can be grouped as follows:

Gravity loads
- Dead loads on the structure itself
- Superimposed dead loads
- Live loads and other loads from construction operations

Environmental loads
Wind
Seismic

Stability loads

Erection operation
Loads from erection apparatus
Impact loads caused by erection equipment and pieces being raised within the structure

3.1 Gravity Loads

Gravity loads for the design of temporary supports consist of the self-weight of the structure itself, the self-weight of any materials supported by the structure and the loads from workers and their equipment. Self-weights of materials are characterized as dead loads. Superimposed loads from workers and tools would be characterized as live loads. Gravity loads can be distributed or concentrated. Distributed loads can be linear, such as the weight of steel framing members, non-uniform such as concrete slabs of varying thicknesses or uniform such as a concrete slab of constant thickness.

Dead loads can be determined using the unit density and unit weights provided in the AISC Manual of Steel Construction, (LRFD Part 7, ASD Part 6) and ASCE 7-93, Tables Cl and C2. Dead loads can also be obtained from manufacturers and suppliers.

Live loads due to workers and their equipment should be considered in the strength evaluation of partially completed work such as connections or beams which are unbraced. The live load used should reflect the actual intensity of activity and weight of equipment. In general, live loads on the order of 20 psf to 50 psf will cover most conditions.

3.2 Environmental Loads

The two principal environmental loads affecting the design of temporary supports are wind and seismic loads. Other environmental loads such as accumulated snow or rain water may influence the evaluation of partially completed construction but these considerations are beyond the scope of this guide.

3.2.1 Wind Loads

Wind loads on a structure are the result of the passage of air flow around a fixed construction. The load is treated as a static surface pressure on the projected area of the structure or structural element under consideration. Wind pressure is a function of wind velocity and the aerodynamic shape of the structure element. Various codes and standards treat the determination of design and wind pressures slightly differently, however the basic concept is common to all methods. What follows is a discussion of the procedure provided in ASCE 7-93 (1) which will illustrate the basic concept.

In ASCE 7-93 the basic design pressure equation for the main force resisting system for a building is

$$p = qG_hC_p - qh(GC_{pi}) \qquad \text{Eq.3-1}$$

where

$$q - 0.00256K(IV)^2 \qquad \text{Eq. 3-2}$$

K = velocity pressure coefficient varying with height and exposure

Exposure classes vary from A (city center) to D (coastal areas) and account for the terrain around the proposed structure.

I = an importance factor which varies with the use of the building, for design of temporary supports I may be taken as 1.0 without regard to the end use of the structure

V = the basic wind speed for the area taken from weather data, usually a 50 year recurrence interval map

G_h = a factor accounting for gust response varying with horizontal exposure

C_p = a factor accounting for the shape of the structure

q_h = q taken at height, h

GCpi = a factor accounting for internal pressure

This method or one like it would have been used to determine the wind forces for the design of the lateral force resisting system for a structure for which temporary lateral supports are to be designed.

To address the AISC Code of Standard Practice requirement that "comparable" wind load be used, the same basic wind speed and exposure classification used in the building design should be used in the design of the temporary supports.

The design of temporary supports for lateral wind load must address the fact that the erected structure is an open framework and as such presents different surfaces to the wind.

In ASCE 7-93 the appropriate equation for open structures is:

$$p = q_zG_hC_f \qquad \text{Eq. 3-3}$$

where

q_z = q evaluated at height z

G_h = gust response factor G evaluated at height, h, the height of the structure

C_f = a force coefficient accounting for the height and aerodynamic geometry of the structure or element

The current draft of the ASCE Standard "Design Loads on Structures During Construction" provides a reduction factor to be applied to the basic wind speed. This factor varies between 1.0 for an exposure period more than 25 years and 0.75 for an exposure period of less than six weeks. The factor for an exposure period from 6 weeks to one year is 0.8.

To determine a wind design force, the design pressure, p, is multiplied by an appropriate projected area. In the case of open structures, the projected area is an accumulated area from multiple parallel elements. The accumulated area should account for shielding of leeward elements by windward elements. Various standards have provided methods to simplify what is a rather complex aerodynamic problem. The elements of the multiple frame lines can be solid web or open web members. Thus, the determination of wind forces requires an evaluation to determine the correct drag coefficient and the correct degree of shielding on multiple parallel members. It also requires the correct evaluation of the effects of wind on open web members.

This topic has been treated in the following documents:

1. Part A4.3.3 of the "Low Rise Building Systems Manual" (12) published by the Metal Building Manufacturers Association.

2. "Wind forces on Structures" (18), Paper No. 3269, ASCE Transactions, published by the American Society of Civil Engineers.

3. "Standards for Load Assumptions, Acceptance and Inspection of Structures" (16), No. 160, published by the Swiss Association of Engineers and Architects.

4. "Design Loads for Buildings" (5), German Industrial Standard (DIN) 1055, published by the German Institute for Standards.

Perhaps the most direct method is that given in the current draft of the ASCE Standard for Design Loads on Structures During Construction which states:

> "6.1.2. Frameworks without Cladding
> Structures shall resist the effect of wind acting upon successive unenclosed components.
>
> Staging, shoring, and falsework with regular rectangular plan dimensions may be treated as trussed towers in accordance with ASCE 7. Unless detailed analyses are performed to show that lower loads may be used, no allowance shall be given for shielding of successive rows or towers.
>
> For unenclosed frames and structural elements, wind loads shall be calculated for each element. Unless detailed analyses are performed, load reductions due to shielding of elements in such structures with repetitive patterns of elements shall be as follows:
>
> 1. The loads on the first three rows of elements along the direction parallel to the wind shall not be reduced for shielding.
>
> 2. The loads on the fourth and subsequent rows shall be permitted to be reduced by 15 percent.
>
> Wind load allowances shall be calculated for all exposed interior partitions, walls, temporary enclosures, signs, construction materials, and equipment on or supported by the structure. These loads shall be added to the loads on structural elements.
>
> Calculations shall be performed for each primary axis of the structure. For each calculation, 50% of the wind load calculated for the perpendicular direction shall be assumed to act simultaneously."

In this procedure one would use the projected area of solid web members and an equivalent projected area for open web members. This effective area is a function of the drag coefficient for the open web member which is a function of the solidity ratio. For the types of open web members used in low-rise construction an effective area (solidity ratio, (p) equal to 30 percent of the projected solid area can be used.

Shielding of multiple parallel elements can be determined using the following equation taken from DIN 1055. See Figures 3.1 and 3.2.

$$A = (1+\eta+(n-2)\eta^2)A_1 \qquad \text{Eq. 3-4}$$

where

A = total factored area

η = a stacking factor taken from Figure 3.2.

n = the total number of parallel elements

A_1 = the projected area of one element

The stacking factor, η, is a function of the element spacing to the element depth and a solidity ratio, φ.

3.2.2 *Seismic Loads*

As indicated in the AISC Code of Standard Practice, seismic forces are a load consideration in the design of temporary supports. In general, seismic forces are addressed in building design by the use of an equivalent pseudo-static design force. This force is a function of:

1. an assessment of the site specific seismic likelihood and intensity,

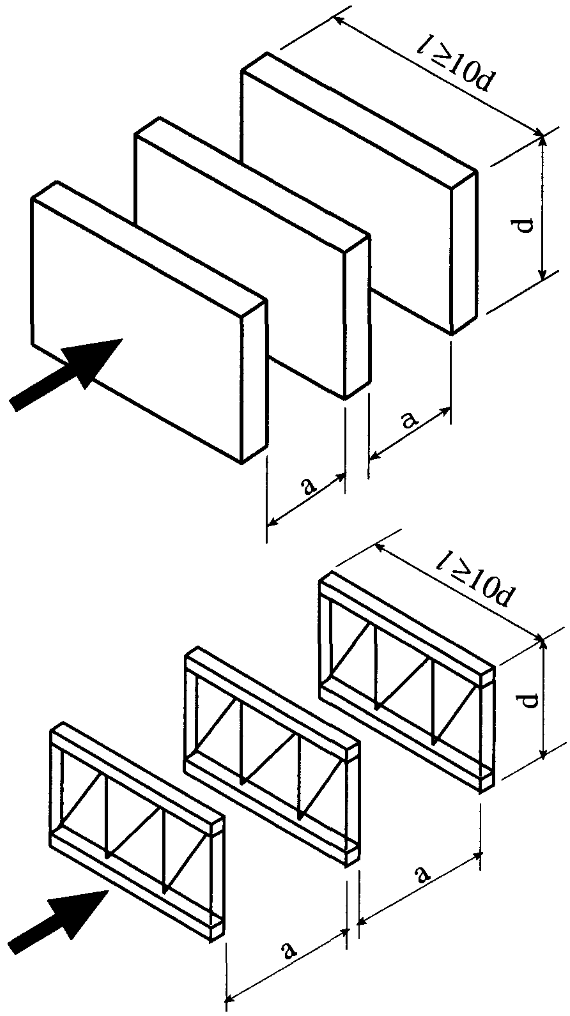


Fig. 3.1 Parameters for Use with Fig. 3.2

2. the use of the structure,

3. the geometry and framing system type of the structure,

4. the geological nature of the building site, and

5. the mass, i.e. self-weight of the structure.

Although codes and standards have differing approaches to seismic design, they are conceptually similar. The general approach can be seen in the description of the approach used in ASCE 7-93 which follows.

The general equation for seismic base shear, V, is:

$$V = C_s W \qquad \text{Eq.3-5}$$

where

C_s = the seismic design coefficient

W = the total dead load and applicable portions of other loads

For the structures within the scope of this guide it is unlikely that W would include any loads other than dead load.

The seismic design coefficient, C_s, is to be determined using the following equation:

$$C_s = \frac{1.2A_vS}{RT^{2/3}} \qquad \text{Eq. 3-6}$$

where

A_v = a coefficient representing the peak velocity related acceleration taken from a contour map supplied

S = a coefficient for site soil profile characteristics ranging from 1.0 to 2.0

R = a response modification factor, ranging from 1.5 to 8.0 depending on the structural system and the seismic resisting system used

T = the fundamental period of the structure which can be determined using equations provided

ASCE 7-93 states that the seismic design coefficient, C_s, need not exceed the value given by the following equation:

$$C_s = \frac{2.5A_a}{R} \qquad \text{Eq. 3–7}$$

where

A_a = a coefficient representing the effective peak acceleration taken from a contour map supplied

R = the response modification factor described above

For the structures within the scope of this guide the response modification factor, R, would be 5.0. This value for R_w is taken from ASCE 7, Table 9.3-2 and is the value given for "Concentrically-braced frames". Likewise for the majority of regular structures there is not significant penalty in using the simpler equation given above to determine C_s. The range of values in the contour map provided in ASCE 7-93 are 0.05 through 0.40. Thus, the range of values for C_s is 0.025 to 0.20. In general wind will govern the design of temporary supports in areas of low seismic activity such as the mid-west. Seismic forces will likely govern the design on the west coast. The value of A_a would be the same value used in the design of the completed structure. Although this discussion of the determination of C_s would apply to most structures in the scope of this guide, it is incumbent on the designer of the temporary support system to be aware of the requirements for seismic design to confirm that the general comments of this section apply to the specific structure at hand.

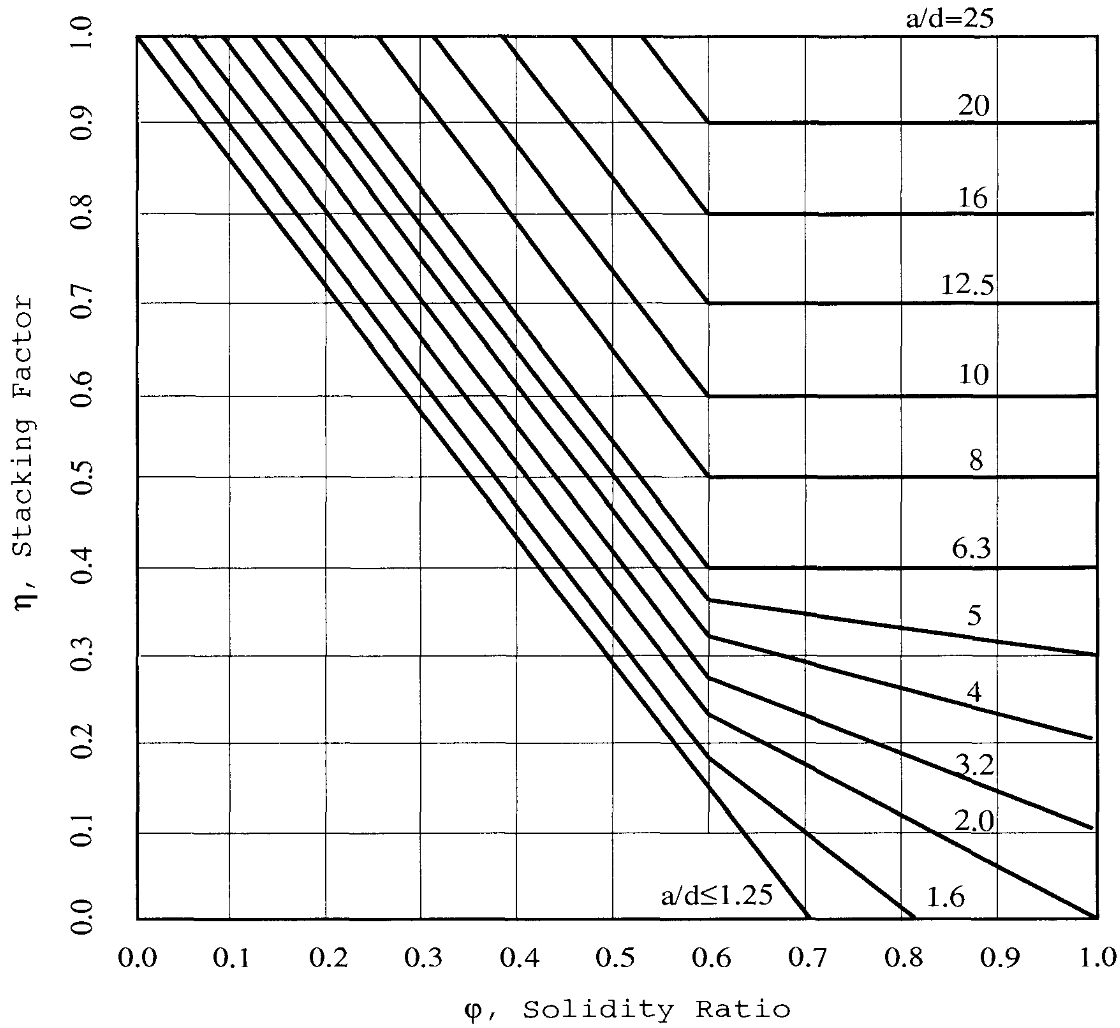


Fig. 3.2 Stacking Factor vs. Solidity Ratio

Based on the foregoing in general terms the pseudo-static force for seismic design is:

$$V = 0.05W \text{ to } 0.40\,W$$

depending on the structure's geographical location. It should be noted that in this method the seismic base shear, V, is a strength level value not an allowable stress value. For single story buildings this force would be applied at the roof level. For multi-story buildings, a procedure is given to distribute the force at each story. In many instances the distribution will be linear, however in certain conditions of structure location and height the distribution will be non-linear with the distribution skewed to the upper stories. Non-linear distribution will be required when the period of the structure exceeds 5 seconds. The period of the structure can be determined from equations given in ASCE-7.

For example, a 60-foot-tall structure located where A_v equals 0.4 would have a period T of 0.517 seconds. Whereas a 60-foot-tall structure located where A_v equals 0.05 would have a period T of 0.733 seconds.

A 40-foot-tall structure in the two locations would have periods of 0.382 seconds and 0.540 respectively. The higher periods in the low end of the A_v range will likely be of no consequence since the seismic force will not likely be the governing force. The reader is referred to ASCE 7-93 for the detailed presentation of vertical distribution of seismic forces.

The horizontal distribution of seismic force is an important consideration when seismic force is resisted by elements in plan connected by longitudinal diaphragms or other horizontal systems. In the design of temporary supports for lateral loads, each frame line will generally have its own temporary supports so the horizontal distribution would consist of applying the dead load, W, which is tributary to each frame.

3.3 Stability Loads

Columns supplied within standard mill practice and erected within tolerance will have an eccentricity between the line of action of the applied load/column and the line of action of the supporting resistance. This eccentricity produces a force couple or tipping moment which must be resisted by a righting force, which can be provided by base fixity, frame action or diagonal braces.

A common approach used in the design of bracing for stability loads is to apply a horizontal load at each level or story equal to 2 percent of the supported load. A righting force of 2 percent is associated with a top of column displacement of one-fiftieth of the column height. Since the maximum deviation from plumb per the AISC Code of Standard Practice is one-five hundredth of the column height, it can be seen that the 2 percent strength criteria also accounts for second order forces due to displacement in the bracing under load.

The 2 percent stability load was recommended by the authors in a previous paper on the subject (11). It has also been included in the Draft of the ASCE Standard for Design Loads on Structures During Construction (6).

3.4 Erection Operation Loads

Loads are applied to the steel frame work as a consequence of erection operations. Loads resulting from hoists, jibs or derricks must be addressed in the bracing design and in a check of the structure for the specific reactions from these devices. These calculations must include the magnitude of lifted loads and the reactions on the framework.

Raising and securing individual pieces results in incidental loads on the surrounding pieces. These small loads are resisted by the minimum connections provided. If significant prying, pulling or jacking is required, this should be evaluated prior to initiating these operations. To account for incidental erection operation lateral loading on the temporary supports, it is recommended that a lateral load of 100 pounds per foot be applied to the perimeter of the framework. This was recommended by the authors in a previous paper (11) and is included in the draft of the ASCE Standard, Design Loads on Structures During Construction.

Lastly, the Steel Erection Negotiated Rulemaking Advisory Committee (SENRAC) has recommended that: "Column and anchor rod assemblies, including the welding of the column to the base plate shall be designed to resist a 300 pound (136.2 kg) eccentric load located 18 inches (.46 m) from the column face in each direction at the top of the column shaft.".

Extraordinary loads such as those due to collisions cannot be anticipated in the design and are excluded by the AISC Code of Standard Practice.

3.5 Load Combinations

Per paragraph A.4.1. of the LRFD Specification the load combinations to be investigated in design are:

$$1.4D$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5L \text{ or } 0.8W)$$

$$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$1.2D \pm 1.0E + 0.5L + 0.2S$$

$$0.9D \pm (1.3W \text{ or } 1.0E)$$

The nominal loads to be considered are:

D: dead load due to the weight of the structural elements and the permanent features on the structure

L: live load due to occupancy and moveable equipment

L_r: roof live load

W: wind load

S: snow load

E: earthquake load determined in accordance with Part I of the AISC *Seismic Provisions for Structural Steel Buildings(15)*

R: load due to initial rainwater or ice exclusive of ponding contribution

Earlier in this guide, the procedure for calculation of a seismic design base shear and its vertical and horizontal distribution was discussed. Using the provisions of ASCE-7 which adopts the NEHRP provisions results in a base shear which is at a ".. .strength level, not an allowable stress level".

Provisions for seismic design in steel are given in "Seismic Provisions for Structural Steel Buildings" published by AISC. In Part II - Allowable Stress Design (ASD) Alternate, the "allowable stress" for members resisting seismic forces ".. .acting alone or in combination with dead and live loads shall be determined by multiplying 1.7 times the allowable stresses in [ASD Specification] Sect. D, E, F, G, J and K". Thus for both ASD and LRFD designs the load factors and combinations in the LRFD Specification part A4 are appropriate, i.e. Equations A4-5 and A4-6 which read:

$$1.2D \pm 1.0\,E + 0.5L + 0.2S$$

$$0.9D \pm (1.3W \text{ or } 1.0E)$$

These equations are the same as Equations 5 and 6 in ASCE 7, paragraph 2.4.2. It should be noted that E is not the exact effect of the seismic force due to the seismic base shear but must be modified by the following equations taken from ASCE 7, paragraph 9.3.7:

in Equation A4-5: $E = \pm Q_E + 0.5 A_v D$, and
in Equation A4-6: $E = \pm Q_E - 0.5 A_v D$

where

E = the effect of horizontal and vertical earthquake-induced forces

A_v = the coefficient representing effective peak velocity-related acceleration from ASCE 7

D = the effect of dead load, D

Q_E = the effect of horizontal seismic (earthquake-induced) forces

The term 0.5 $A_v D$ is a corrective term to reconcile the load factors used in the NEHRP requirements and the load factors used in the ASCE 7/LRFD requirements. This correction is described in detail in the Commentary to ASCE 7, which concludes that the correction is made separately "...so that the original simplicity of the load combination equations in Sec. 2 is maintained." It is also explained in this paragraph taken from the Commentary to the AISC Seismic Provisions:

> "The earthquake load and load effects E in ASCE 7-93 are composed of two parts. E is the sum of the seismic horizontal load effects and one half of A_v times the dead load effects. The second part adds an effect simulating vertical accelerations concurrent to the usual horizontal earthquake effects."

In forming combinations containing the effects of stability, the load factors for the load source (D or L) which induces the PΔ effect would be used for the load factor(s) on the effect of stability.

In the authors' earlier paper (11) on this topic the following ASD combinations were recommended:

a. Stability loading

b. 0.75 (stability loading plus wind loading)

These combinations reflected the current ASD Specification provision for one-third increases for stresses computed for combinations including wind loading, acting alone or in combination with dead and live load.

In this Guide the determination of load and resistance is based on the LRFD Specification. Allowable stress design is used only when LRFD procedures are not available or would be inappropriate.

4. RESISTANCE TO CONSTRUCTION PHASE LOADS BY THE PERMANENT STRUCTURE

The resistance to loads during construction on the steel framework is provided by a combination of the permanent work supplemented by temporary supports as needed. The resistance of the permanent structure develops as the work progresses. In a self-supporting structure the resistance is complete when the erector's work is complete. In a non-self-supporting structure resistance will be required after the completion of the erectors work and will be needed until the other non-structural-steel elements are in place. During the erection of both self-supporting and non-self-supporting frames, conditions will arise which require resistance to be supplied by the partially completed work. If the resistance of the partially completed work is not adequate, it must be supplemented by temporary supports.

Elements of the permanent structure which may be used to resist loads during construction are:

1. Columns
2. Column Bases
3. Beams and Joists
4. Diagonal Bracing
5. Connections
6. Diaphragms

Columns

In general columns will have the same unbraced length in the partially completed work as in the completed work so their axial design strength would be the same during erection as the completed work. The exceptions would be:

Columns which are free standing on their bases before other framing and bracing is installed.

Columns supported on leveling nuts or shims prior to grouting.

Columns which are to be laterally braced by girts or struts.

Columns which have additional axial load due to the temporary support system.

Column Bases

The column bases of the permanent structure are an essential element of both the permanent structure and the temporary support system. The column bases transfer vertical and lateral loads from the structural steel framework to the foundation and thence to the ground. The components of a column base are:

the base plate and its attachment to the column shaft
the anchor rods
the base plate grout
the supporting foundation.

Base Plate: Column base plates are square or rectangular plates which transfer loads from the column shaft to the foundation. In high-rise construction and in other cases of very high loading, large column bases are sometimes shipped and set separately from the column shafts. In the case of low-rise and one story buildings, the base plates are usually shipped attached the column shafts. The column base reaction is transferred to the column by bearing for compression forces and by the column to base plate weld for tension and shear.

Anchor Rods: Anchor rods have in the past been called anchor bolts. This Design Guide uses the term anchor rod which has been adopted by AISC in the 2nd edition of the LRFD Manual of Steel Construction to distinguish between bolts, which are generally available in lengths up to eight inches, and longer headed rods, such as threaded rods with a nut on the end, and hooked rods. In the completed construction (with the base plates grouted) anchor rods are designed to carry tension forces induced by net tension in the column, base bending moments and tension induced by shear friction resisting column base shears. During erection operations and prior to base plate grouting, anchor rods may also resist compression loads and shears depending on the condition of temporary support for the column and the temporary lateral support system. Anchor rods are embedded in the cast-in-place foundation and are terminated with either a hook or a headed end, such as a heavy hex nut with a tack weld to prevent turning.

Base Plate Grout: High strength, non-shrink grout is placed between the column base plate and the supporting foundation. Where base plates are shipped loose, the base plates are usually grouted after the plate has been aligned and leveled. When plates are shipped attached to the column, three methods of column support are:

1. The use of leveling nuts and, in some cases, washers on the anchor rods beneath the base plates.

2. The use of shim stacks between the base plate bottoms and top of concrete supports.

3. The use of 1/4" steel leveling plates which are set to elevation and grouted prior to the setting of columns.

Leveling nuts and shim stacks are used to transfer the column base reactions to the foundation prior to the installation of grout. When leveling nuts are used all components of the column base reaction are transferred to the foundation by the anchor rods. When shims are used the compressive components of the column base reaction are carried by the shims and the tension and shear components are carried by the anchor rods.

Leveling nuts bear the weight of the frame until grouting of the bases. Because the anchor rod, nut and washers have a finite design strength, grouting must be completed before this design strength would be exceeded by the accumulated weight of the frame. For example, the design strength of the leveling nuts may limit the height of frame to the first tier of framing prior to grouting. Also, it is likely that the column bases would have to be grouted prior to placing concrete on metal floor deck.

Properly installed shim stacks can support significant vertical load. There are two types of shims. Those which are placed on (washer) or around (horseshoe) the anchor rods and shim stacks which are independent of the anchor rods. Shims placed on or around the anchor rods will have a lesser tendency to become dislodged. Independent shims must have a reasonable aspect ratio to prevent instability of the stack. In some instances shim stacks are tack welded to maintain the integrity of the stacks. When shim stacks are used, care must be taken to insure that the stacks cannot topple, shift or become dislodged until grouting. Shims are sometimes supplemented with wedges along the base plate edges to provide additional support of the base plate.

Pregrouted leveling plates eliminate the need to provide temporary means for the vertical support for the column. The functional mechanisms of the base are the same in the temporary and permanent condition once the anchor rod nuts are installed.

The design of base plates and anchor rods is treated extensively in texts and AISC publications such as the Manual of Steel Construction and AISC Design Guides 1(7) and 7(10).

Foundations: Building foundations are cast-in-place concrete structures. The element which usually receives the anchor rods may be a footing, pile cap, grade beam, pier or wall. The design requirements for cast-in-place concrete are given in building codes which generally adopt the provisions of the American Concrete Institute standards such as ACI 318 "Building Code Requirements for Reinforced Concrete and Commentary"(3). The principal parameter in the design and evaluation of cast-in-place concrete is the 28-day cylinder compression stress, f'_c. Axial compressive strength, flexural strength, shear strength, reinforcing bar development and the development of anchor rods are a function of the concrete compressive strength, f'_c. Axial tension and flexural tension in concrete elements is carried by deformed reinforcing bars to which force is transferred by development of the bar which is a function of an average bond stress. Bar development is a function of concrete strength, reinforcement strength, bar size, bar spacing, bar cover and bar orientation.

Columns are sometimes supported on masonry piers rather than concrete piers. In this case the strength of the piers would be evaluated using ACI 530 "Building Code Requirements for Masonry Structures" (2) or another comparable code. Masonry is constructed as plain (unreinforced) or reinforced. Unreinforced masonry construction has very low tensile strength and thus unguyed cantilevered columns would be limited to conditions where relatively little base moment resistance is required. Reinforced masonry can develop strengths comparable to reinforced concrete. The masonry enclosing the grout and reinforcement must be made large enough to also accommodate and develop the anchor rods.

In some instances steel columns are erected on bases atop concrete or masonry walls. In these conditions the side cover on the anchor rods is often less than it would be in a pier and significantly less than it would be in the case of a footing. Although not specifically addressed in this guide, the design strength of the anchor rod can be determined based on the procedures provided in this Guide in conjunction with the requirements of ACI 318 or ACI 530 as appropriate. The wall itself should be properly braced to secure it against loads imposed during the erection of the steel framing.

The erection operation, sequence of the work, reactions from temporary supports and the timing of grouting may cause forces in the anchor rods and foundation which exceed those for which the structure in its completed state has been designed. This Guide provides procedures to evaluate the anchor rods and foundation for such forces.

One condition of loading of the column base and foundation occurs when a column shaft is set on the anchor rods and the nuts are installed and tightened. Unless there is guying provided, the column is a cantilever from the base and stability is provided by the development of a base moment in the column base. This condition is addressed in detail subsequently in this Guide.

Diagonal cables for temporary lateral support also induce tensions and shears in the column base which must be transferred from the column base, through the anchor rods to the foundation.

Lastly, the structural frame when decked may be subject to wind uplift which is not counterbalanced by the final dead load. A net uplift in the column base may induce forces in the base plates and welds, anchor rods, and foundation which exceed those for which the structure in its completed state was designed.

Beams and Joists

Framing members on the column center lines act as tie members and struts during erection. As such they are subject to axial forces as well as gravity load bending. In most cases the axial compression strength of tie members and struts will be limited by their unbraced length in the absence of the flange bracing. The resistance of strut and tie members must be evaluated with the lateral bracing in place at the time of load application.

Diagonal Bracing

Permanent horizontal and vertical bracing systems can function as temporary bracing when they are initially installed. When a bracing member is raised, each end may only be connected with the minimum one bolt, although the design strength may be limited by the hole type and tightening achieved. The bracing design strength may also be limited by other related conditions such as the strength of the strut elements or the base connection condition. For example, the strut element may have a minimum of two bolts in each end connection, but it may be unbraced, limiting its strength.

Connections

Structural steel frames are held together by a multitude of connections which transfer axial force, shear and moment from component to component. During erection connections may likely be subjected to forces of a different type or magnitude than that for which they were intended in the completed structure. Also, connections may have only some of the connectors installed initially with the remainder to be installed later. Using procedures presented in texts and the AISC Manual of Steel Construction the partially complete connections can be evaluated for adequacy during erection.

Diaphragms

Roof deck and floor deck (slab) diaphragms are frequently used to transfer lateral loads to rigid/braced framing and shear walls. Diaphragm strength is a function of the deck profile and gage, attachments to supports, side lap fastening and the diaphragm's anchorage to supporting elements, i.e., frames and shear walls. Partially completed diaphragms may be partially effective depending on the diaphragm geometry, extent of attachment and the relation of the partially completed section to the supporting frames or walls. Partially completed diaphragms may be useful in resisting erection forces and stabilizing strut members, but the degree of effectiveness must be verified in the design of the temporary support system analysis and design.

4.1 Columns

Exceptions were listed earlier wherein the columns may not have the same length as they would in the completed structure. Before using the permanent columns in the temporary support system the erector must evaluate whether the columns have the required strength in the partially completed structure.

Specific guidelines for this evaluation are not presented here, because of the many variables that can occur. Basic structural engineering principles must be applied to each situation.

4.2 Column Bases

Probably the most vulnerable time for collapse in the life of a steel frame occurs during the erection sequence when the first series of columns is erected. After the crane hook is released from a column and before it is otherwise braced, its resistance to overturning is dependent on the strength (moment resistance) of the column base and the overturning resistance of the foundation system. Once the column is braced by tie members and bracing cables it is considerably more stable.

It is essential to evaluate the overturning resistance of the cantilevered columns. Cantilevered columns should never be left in the free standing position unless it has been determined that they have the required stability to resist imposed erection and wind loads.

In order to evaluate the overturning resistance one must be familiar with the modes of failure which could occur. The most likely modes of failure are listed below. It is not the intent of this design guide to develop structural engineering equations and theories for each of these failure theories, but rather to provide a general overview of each failure mode and to apply existing equations and theories. Equations are provided to obtain the design strength for each mode based on structural engineering principles and the AISC LRFD Specification.

Modes of Failure:

1. Fracture of the fillet weld that connects the column to the base plate.
2. Bending failure of the base plate.
3. Tension rupture of the anchor rods.
4. Buckling of the anchor rods.
5. Anchor rod nut pulling or pushing through the base plate hole.
6. Anchor rod "pull out" from the concrete pier or footing.
7. Anchor rod straightening.
8. Anchor rod "push out" of the bottom of the footing.
9. Pier spalling.
10. Pier bending failure.
11. Footing overturning.

For a quick determination of the resistance for each of the failure modes, tables are presented in the Appendix.

Figures 4.1 through 4.11 shown below represent each of the failure modes.

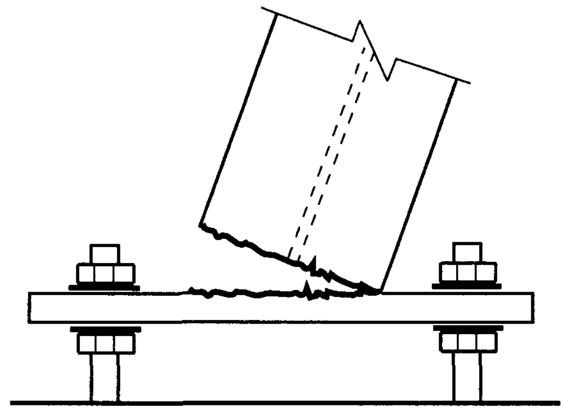


Fig. 4.1 Fracture of Weld

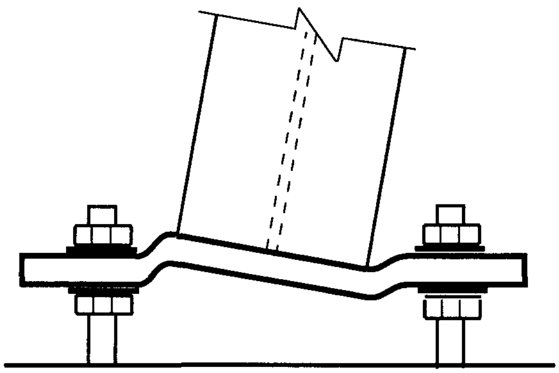


Fig. 4.2 Bending Failure of Base Plate

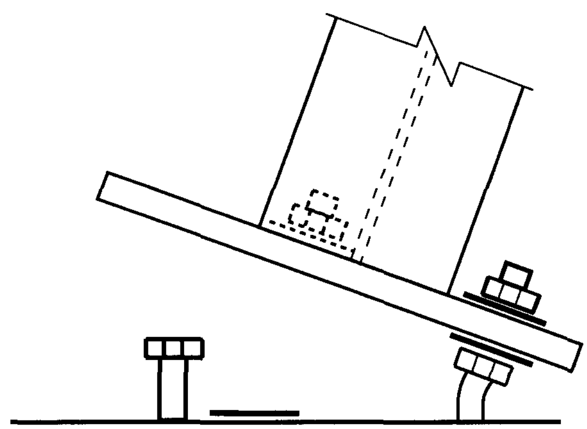


Fig. 4.3 Rupture of Anchor Rods

4.2.1 Fracture of the Fillet Weld Connecting the Column to the Base Plate.

Cantilevered columns are subjected to lateral erection and wind forces acting about the strong and/or the weak axis of the column. Weld fractures between the column base and the base plate are often found after an erection collapse. In the majority of cases the fractures

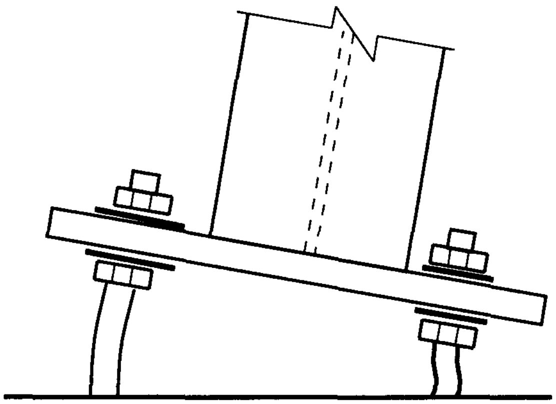


Fig. 4.4 Anchor Rod Buckling

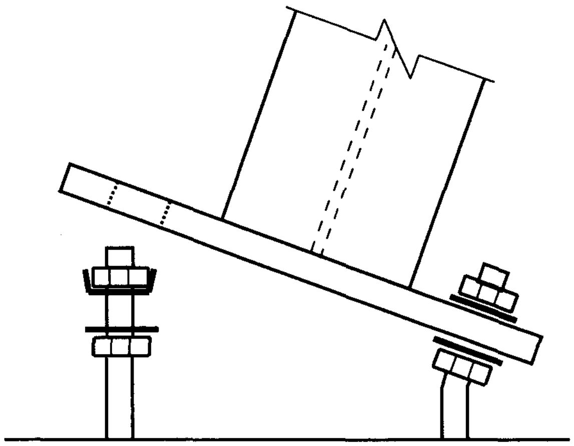


Fig. 4.5 Anchor Rod Pull Through

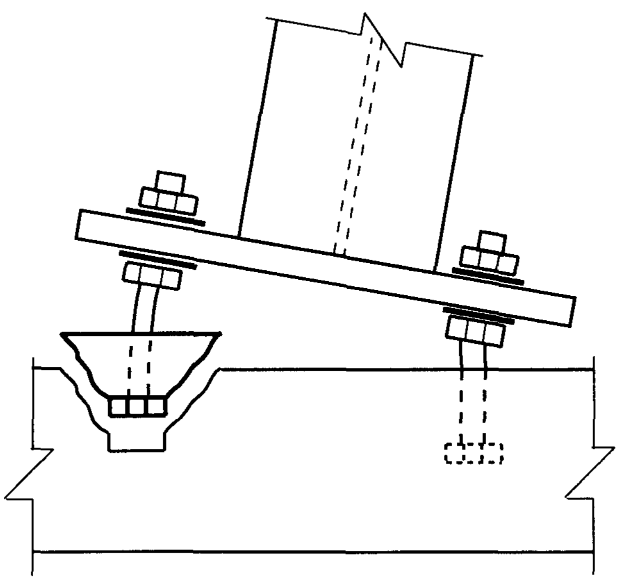


Fig. 4.6 Anchor Rod Pull Out

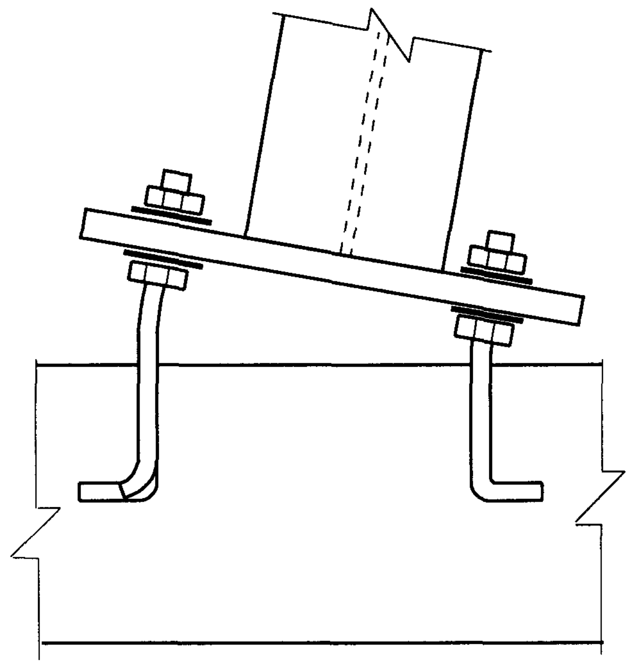


Fig. 4.7 Anchor Rod Straightening

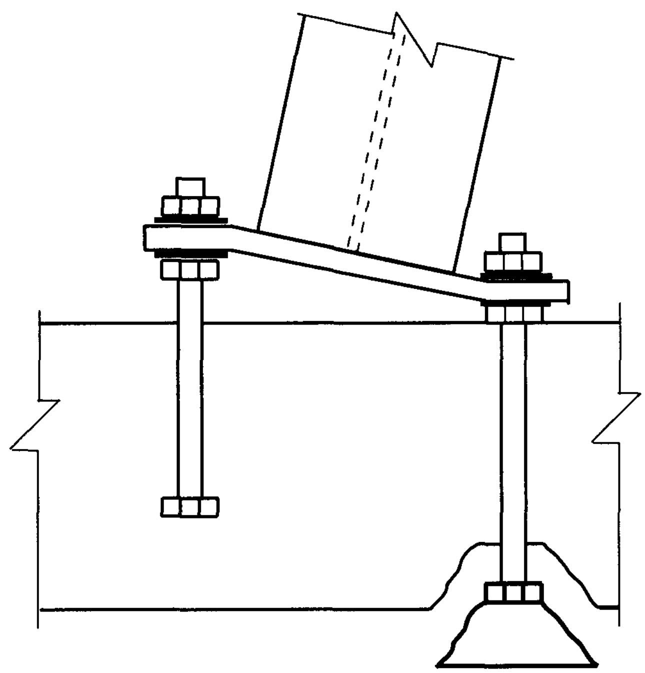


Fig. 4.8 Anchor Rod Push Out

are secondary, i.e. some other mode of failure initiated the collapse, and weld failure occurred after the initial failure. Fracture occurs when the weld design strength is exceeded. This normally occurs for forces acting about the weak axis of the column, because the strength of the weld group is weaker about the weak axis, and because the wind forces are greater when acting against the weak axis, as explained earlier.

The design strength of the weld between the column and the base plate can be determined by calculating the bending design strength of the weld group. Applied

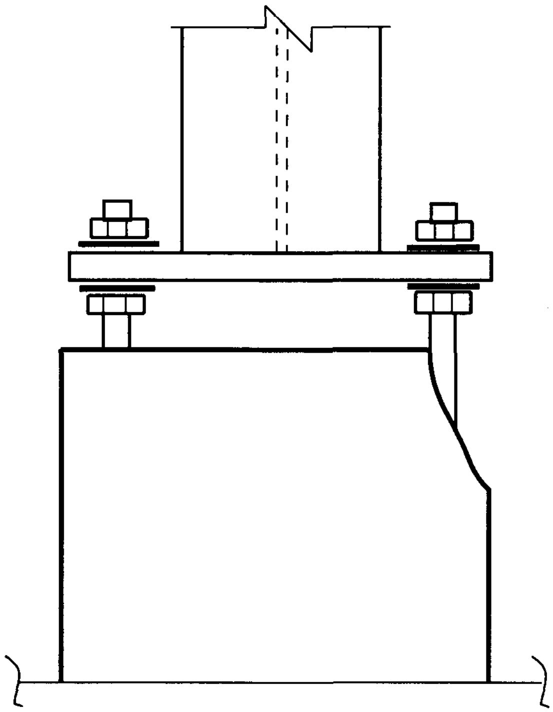


Fig. 4.9 Pier Spalling

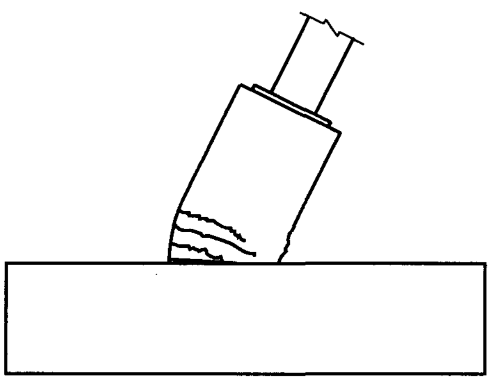


Fig. 4.10 Pier Bending Failure

shear forces on the weld are small and can be neglected in these calculations.

For bending about the column strong axis the design strength of the weld group is:

$$\phi M_n = \phi F_w S_x \qquad \text{Eq. 4-1}$$

For bending about the column weak axis the design strength of the weld group is:

$$\phi M_n = \phi F_w S_y \qquad \text{Eq. 4-2}$$

where

ϕ = 0.75

F_w = the nominal weld stress, ksi

= $0.60\, F_{EXX}\,(1.0 + 0.50 \sin^{1.5}\theta)$, ksi

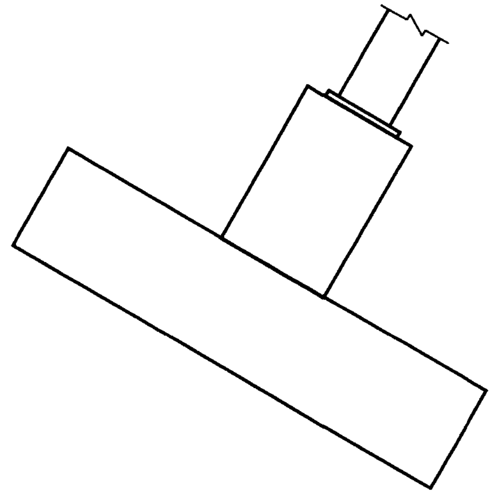


Fig. 4.11 Footing Overturning

= $1.5(0.60)\, F_{EXX}$, ksi (for 90° loading)

F_{EXX} = electrode classification number, i.e. minimum specified strength, ksi

S_x = the section modulus of the weld group about its strong axis, in.^{3}

S_y = the section modulus of the weld group about its weak axis, in.^{3}

4.2.2 Bending Failure of the Base Plate.

Ordinarily a bending failure is unlikely to occur. Experience has shown that one of the other modes of failure is more likely to govern. A bending failure results in permanent bending distortion (yielding) of the base plate around one or more of the anchor rods. The distortion allows the column to displace laterally, resulting in an increased moment at the column base, and eventual collapse. The design strength of the base plate is dependent on several variables, but it primarily depends on the base plate thickness, the support points for the base plate, and the location of the anchor rods.

The design strength of the base plate can be conservatively determined using basic principles of strength of materials.

Case A: Inset Anchor Rods - Wide Flange Columns.

Yield line theories can be used to calculate the bending design strength of the base plate for moments about the x and y axes. The lowest bound for all possible yield lines must be determined. The approach used here is a simplification of yield line theory and is conservative.

The design strength of the base plate is determined using two yield lines. Shown in Figure 4.12 are the two yield line lengths used, b_1 and b_2. The length b_1 is taken as two times d_1, the distance of the anchor rod to the cen-

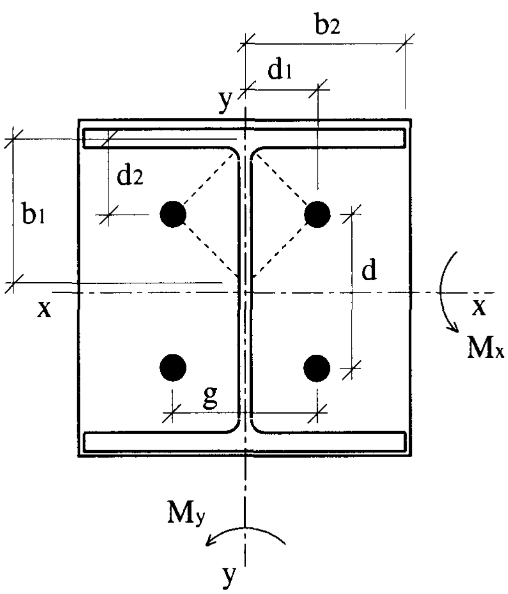


Fig. 4.12 Base Plate Dimensions

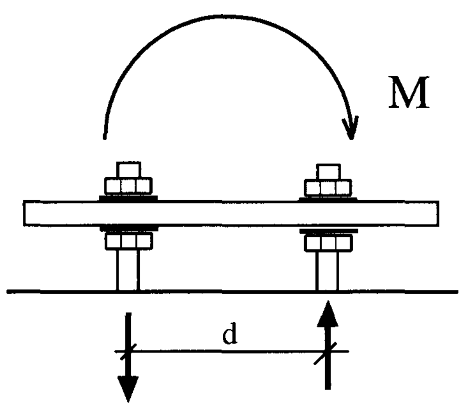


Fig. 4.13 Base Plate with Leveling Nuts

ter of the column web. The length b_2 is taken as the flange width divided by two. The yield line b_2 occurs as a horizontal line through the bolt Centerline.

Using the dimensions shown in Figure 4.12, the design strength for a single anchor rod is:

$$\phi P_n = \phi M^1_n / d_1 + \phi M^2_n / d_2 \qquad \text{Eq. 4-3}$$

where

ϕP_n = the anchor rod force which causes the base plate to reach its design strength, kips

M^1_n = the plastic moment resistance based on b_1 in.-kips

M^2_n = the plastic moment resistance based on b_2, in.-kips

ϕ = 0.90

Eq. 4-3 is based on d_1 and d_2 being approximately equal.

After determining ϕP, the design strength of the base plate is determined by multiplying $2\phi P$ by the appropriate lever arm, d or g (ϕP is multiplied by two if the base condition consists of two anchor rods in tension).

$$\phi M_n = 2\phi Pd \text{ or } 2\phi Pg \qquad \text{Eq.4-4}$$

If leveling nuts are used under the base plate the lever arm (d) is the distance between the anchor rods. See Figure 4.13. If shim stacks are used then the lever arm (d) is the distance from the anchor rods to the center of the shim stack. See Figure 4.14. See discussion of the use of shims at the beginning of this section.

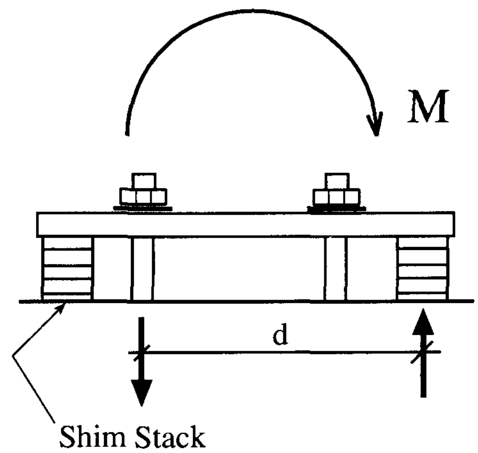


Fig. 4.14 Base Plate with Shim Stacks

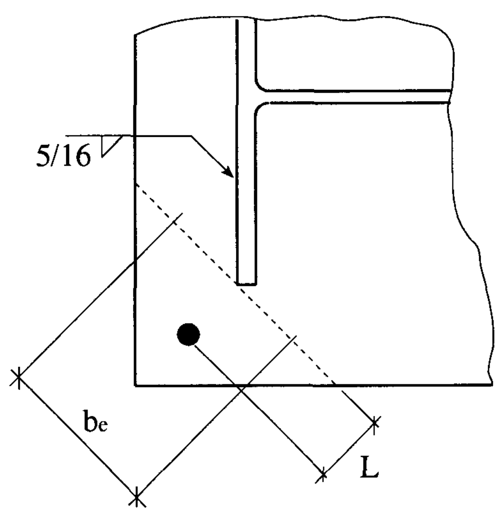


Fig. 4.15 Effective Width

Currently the AISC standard detail illustrates weld only along the flanges, unless shown otherwise on the contract drawings. The addition of a fillet weld along one side of the web adds considerable strength to the connection. Without the web weld only the length b_2 would be used in the strength calculations.

Case B: Outset Rods - Wide Flange Columns

The authors are unaware of any published solutions to determine base plate thickness or weld design strength for the base plate - anchor rod condition shown in Figure 4.15. By examining Figure 4.15 it is obvious that the weld at the flange tip is subjected to a concentration of load because of the location of the anchor rod. The authors have conducted elastic finite element analysis in order to establish a conservative design procedure to determine the required base plate thickness and weld design strength for this condition. The following conclusions are based on the finite element studies:

1. The effective width of the base plate, b_e, should be taken as 2L.
2. The maximum effective width to be used is five inches.
3. A maximum weld length of two inches can be used to transmit load between the base plate and the column section. If weld is placed on both sides of the flange then four inches of weld can be used.
4. The base plate thickness is a function of the flange thickness so as not to over strain the welds.

In equation format the design strength for a single anchor rod can be expressed as follows:

Based on the plate effective width:

$$\phi P_n = \phi_p b_e t_p{}^2 F_y/(4L) \qquad \text{Eq. 4-5}$$

Based on weld strength:

$$\phi P_n = \phi_w F_w t(2) \qquad \text{Eq. 4-6}$$

Based on weld strain:

$$\phi P_n = \phi_p (50) t\, t_p^{1.5} \qquad \text{Eq. 4-7}$$

where

ϕ_p = 0.90

ϕ_w = 0.75

b_e = the effective plate width, in.

$2L \leq 5$ in.

L = the distance of the anchor rod to the flange tip, in.

t = the throat width of the weld, in.

t_p = the base plate thickness, in.

F_y = the specified yield strength for the base plate, ksi

F_w = the nominal weld stress, ksi

= 0.9 FEXX, ksi (90° loading)

FEXX = electrode classification number, ksi

Using the controlling value for ϕP_n and d:

$$\phi M_n = 2\phi P_n d \qquad \text{Eq. 4-8}$$

Case C Outset Rods with hollow structural section (HSS) columns.

When hollow structural section (HSS) columns are used, Eq. 4-5 and Eq. 4-7 can be used to calculate ϕP; however, if fillet welds exist on all four sides of the column, then four inches of weld length at the corner of the HSS can be used for the calculation of ϕP in Eq. 4-6. Thus:

$$\phi P_n = \phi F_w t\,(4'') \qquad \text{Eq. 4-9}$$

4.2.3 Rupture of Anchor Rods

A tension rupture of the anchor rods is often observed after an erection collapse. This failure occurs when the overturning forces exceed the design strength of the anchor rods. Fracture usually occurs in the root of the anchor rod threads, at or flush with, the face of the lower or upper nut. Anchor rod rupture may be precipitated by one of the other failure modes. It is generally observed along with anchor rods pulling out of the concrete pier, or footing. Shown in Figure 4.3 is an anchor rod tension failure. The tension rupture strength for rods is easily determined in accordance with the AISC specification.

$$\phi P_n = \phi F_n A_b \qquad \text{Eq. 4-10}$$

where

ϕ = 0.75 (Table J3.2)

ϕP_n = the tension rod design strength, kips

F_n = nominal tensile strength of the rod F_t, ksi

F_t = $0.75F_U$ (Table J3.2)

F_u = specified minimum tensile strength, ksi

A_b = nominal unthreaded body area of the anchor rod, in.^{2}

For two anchor rods in tension the bending design strength can again be determined as:

$$\phi M_n = 2\phi P_n d \qquad \text{Eq. 4-11}$$

4.2.4 Buckling of the Anchor Rods

The buckling strength of the anchor rods can be calculated using the AISC LRFD Specification (Chapter

E). For base plates set using leveling nuts a reasonable value for the unbraced length of the anchor rods is the distance from the bottom of the leveling nut to the top of the concrete pier or footing. When shim stacks are used the anchor rods will not buckle and this failure mode does not apply. It is suggested that the effective length factor, K, be taken as 1.0, and that the nominal area (A_b) be used for the cross sectional area.

For anchor rod diameters greater than 3/4 inches used in conjunction with grout thickness not exceeding 8 inches, the authors have determined that buckling strength of the anchor rods will always exceed the design tensile strength of the rods. Thus this failure mode need not be checked for most situations.

4.2.5 *Anchor Rod Pull or Push Through*

The nuts on the anchor rods can pull through the base plate holes, or when leveling nuts are used and the column is not grouted, the base plate can be pushed through the leveling nuts. Both failures occur when a washer of insufficient size (diameter, thickness) is used to cover the base plate holes. No formal treatise is presented herein regarding the proper sizing of the washers; however, as a rule of thumb, it is suggested that the thickness of the washers be a minimum of one third the diameter of the anchor rod, and that the length and width of the washers equal the base plate hole diameter plus one inch.

Special consideration must be given to base plate holes which have been enlarged to accommodate misplaced anchor rods.

4.2.6 *Anchor Rod Pull Out*

Shown in Figure 4.6 is a representation of anchor rod pull out.

This failure mode occurs when an anchor rod (a hooked rod or a nutted rod) is not embedded sufficiently in the concrete to develop the tension strength of the rod.

The failure occurs in the concrete when the tensile stresses along the surface of a stress cone surrounding the anchor rod exceed the tensile strength of the concrete. The extent of the stress cone is a function of the embedment depth, the thickness of the concrete, the spacing between the adjacent anchors, and the location of free edges of in the concrete. This failure mode is presented in detail in Appendix B of ACI 349-90(4). 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 cones. By examining the geometry, it is evident that the pull out strength of a 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, or for the case of hooked rods the projected area of the hook.

The dotted lines in Figure 4.16 represent the failure cone profile. Note that for the rods in tension the cones will be pulled out of the footing or pier top, whereas the cones beneath the rods in compression will be pushed out the footing bottom. This latter failure mode will be discussed in the next section.

Depending on the spacing of the anchor rods and the depth of embedment of the rods in the concrete, the failure cones may overlap. The overlapping of the failure cones makes the calculation of A_e more complex.

Based on AISC's Design Guide 7 the following equation is provided for the calculation of A_e which covers the case of the two cones overlapping.

$$A_e = \pi(L_d+c/2)^2+2(L_d+c/2)(s+c)-\pi(c)^2/2 \quad \text{Eq. 4–12}$$

where

L_d = the embedment depth, in.

c = the rod diameter for hooked rods, in., and 1.7 times the rod diameter for nutted rods (the 1.7 factor accounts for the diameter of the nut)

s = the rod spacing, in.

Thus, the design strength of two anchor rods in tension is:

$$\phi T_n = 4\phi\sqrt{f'_c}A_e \quad \text{Eq. 4-13}$$

where

ϕ - 0.85

f'_c = the specified concrete strength, psi

When the anchor rods are set in a concrete pier, the cross sectional area of the pier must also be checked. Conservatively, if the pier area is less than A_e then the pier area must be used for A_e in the calculation of ϕT_n (Eq.4-13).

Also when anchor rods are placed in a pier the projected area of the cone may extend beyond the face of the pier. When this occurs A_e must be reduced. The pullout strength can also be reduced by lateral bursting forces. The failure mode shown in Figure 4.9 is representative of these failure modes. These failure modes are also discussed in AISC's Design Guide 7. Conservatively A_e can be multiplied by 0.5 if the edge distance is 2 to 3 inches.

It is recommended that plate washers not be used above the anchor rod nuts. Only heavy hex nuts should be used. Plate washers can cause cracks to form in the concrete at the plate edges, thus reducing the pull out resistance of the anchor rods. The heavy hex nuts should

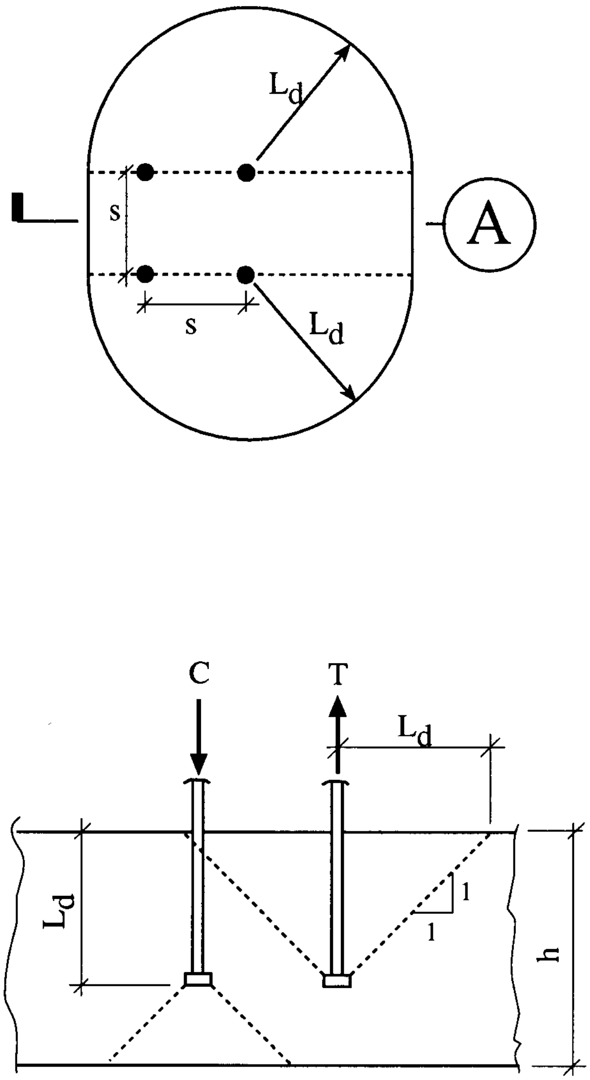


Fig. 4.16 Failure Cones

be tack welded to the anchor rods to prevent the rod from turning during tightening operations.

For hooked anchor rods an additional check must be made, because hooked rods can fail by straightening and pulling out of the concrete. When this occurs, the rods appear almost perfectly straight after failure. To prevent this failure mode from occurring the hook must be of sufficient length. The hook pullout resistance can be determined from the following equation:

$$\phi T_n = (0.7)(2)(0.85)f'_c d_{hook} L_{hook} \qquad \text{Eq.4-14}$$

where

ϕT_n = Hook Bearing Design Strength, kips

f'_c = the concrete compressive strength, psi

d_{hook} = the diameter of the anchor rod, in.

L_{hook} = the length of the hook, in.

Per ACI 318, (0.70) is the ϕ factor for bearing on concrete, and the value (2) represents the strength increase due to confinement.

The design strength obtained from Eq. 4-14 must be compared to the strength obtained from the failure cones, Eq. 4-13. The lower value provides the ultimate strength of the hooked rod to be used in the calculation for the bending moment design strength associated with rod pull out.

$$\phi M_n = 2\phi T_n d \qquad \text{Eq. 4-15}$$

4.2.7 *Anchor Rod "Push Out" of the Bottom of the Footing*

Anchor rod push out can occur when the rod is loaded to the point where a cone of concrete below the anchor rod is broken away from the footing. This failure mode is identical to anchor rod pull out but is due to a compressive force in the rod rather than a tension force. This failure mode does not occur when shim stacks are used, when piers are present or when an additional nut is placed on the anchor rods just below the top of the footing as shown in Figure 4.17.

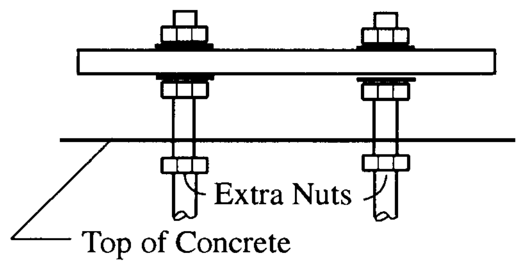
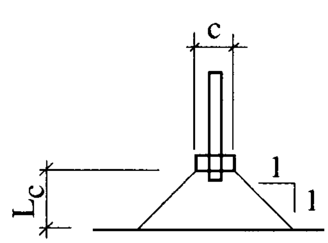


Fig. 4.17 Prevention of Push Out

Shown in Figure 4.18 is the individual failure cone for a nutted anchor rod, and the equation for A_e. The design strength for this mode of failure is:



$$A_e = \pi[L_c^2 + L_c(c)]$$

Fig. 4.18 Push Out Cones

$$\phi P_n = 4\phi\sqrt{f'_c}A_e \qquad \text{Eq. 4-16}$$

where

ϕ = .75

f'_c = the concrete compressive strength, psi

The push out design strength for hooked anchor rods is assumed to equal that of the nutted rod.

4.2.8 Pier Bending Failure

The design strength of a reinforced concrete pier in bending is calculated using reinforced concrete principles. The required procedure is as follows:

Determine the depth of the compression area.

$C = T$

$0.85f'_c ba = F_y A_s$

$a = F_y A_s / .85f'_c b$

$C - 0.85f'_c ab$

d = the effective depth of the tension reinforcing

= pier depth - cover - 1/2 of the bar diameter

$$\phi M_n = C(d\text{-}a/2) \qquad \text{Eq. 4-17}$$

In addition, to insure that the reinforcing steel can develop the moment, the vertical reinforcement must be fully developed. Based on ACI 318-95 (12.2.2.), the required development length can be determined from the equations below. These equations presume that ACI column ties, concrete cover, and minimum spacing criterion are satisfied.

For the hooked bar in the footing:

$$l_{dh} = 840d_b/\sqrt{f'_c} \geq 8d_b \text{ or } 6 \text{ in.} \qquad \text{Eq. 4-18}$$

For straight bars (#6 bars and smaller) in the pier:

$$l_d = 2400d_b/\sqrt{f'_c} \geq 12 \text{ in.} \qquad \text{Eq. 4-19}$$

For straight bars (#7 bars and greater) in the pier:

$$l_d = 3000d_b/\sqrt{f'_c} \geq 12 \text{ in.} \qquad \text{Eq. 4-20}$$

where

l_{dh} = the development length of standard hook in tension, measured from critical section to out-side end of hook, in. (See Figure 4.19)

l_d = development length, in.

f'_c = specified concrete strength, psi

d_b = the bar diameter, in.

If the actual bar embedment length is less than the value obtained from these equations then the strength requires further investigation. See ACI 318, Chapter 12.

4.2.9 Footing Over Turning

The resistance of a column footing to overturning is dependent on the weight of the footing and pier, if any, the weight of soil overburden, if any, and the length of

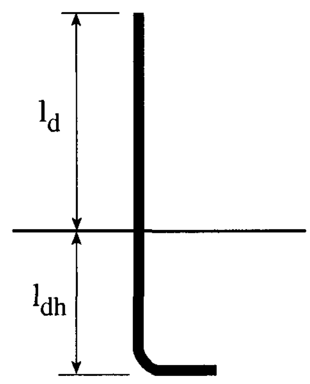


Fig. 4.19 Development Lengths

the footing in the direction of overturning. During construction the overburden, backfill, is often not present and thus is not included in this overturning calculation.

Shown in Figure 4.11 is a footing subjected to an overturning moment.

The overturning resistance equals the weight, W times the length, L divided by two, i.e.:

$$\phi M_n = \phi W(L/2) \qquad \text{Eq. 4-21}$$

where

$\phi = 0.9$

$W = P1 + P2 + P3$

P1 = the weight of any superimposed loads, kips

P2 = the weight of the pier, if any, kips

P3 = the weight of the footing, kips

After determining each of the individual design strengths, the lowest bending moment strength can be compared to the required bending moment to determine the cantilevered column's suitability.

Example 4-1:

Determine the overturning resistance of a W12X65, free standing cantilever column. Foundation details are shown in Figure 4.20, and base plate details are shown in Figure 4.21.

Given:

Leveling Nuts and Washers

4-3/4" ASTM A36 Hooked Anchor Rods with 12" Embedment and 4" Hook

Pier 1'-4" x 1'-4" with 4 - #6 Vert, and #3 Ties @ 12" o/c

Footing 6'-0" x 6'-0" x 1'-3"

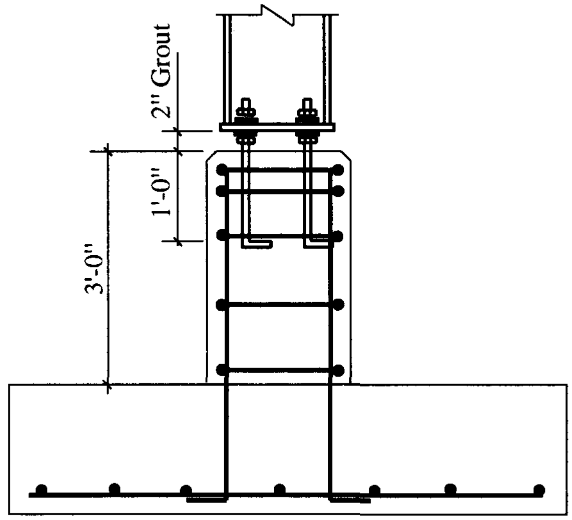


Fig. 4.20 Foundation Detail

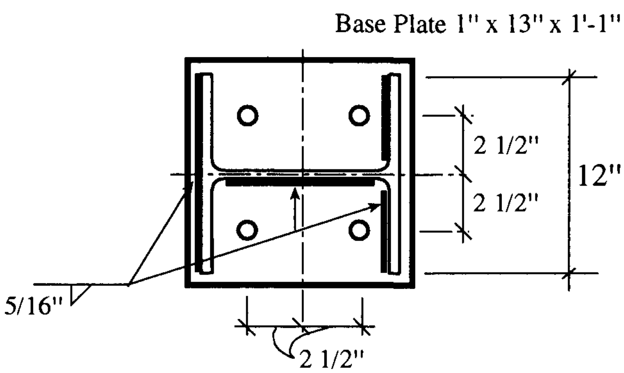


Fig. 4.21 Base Plate Detail

No Overburden

Material Strengths:

Plates: 36 ksi
Weld Metal: 70 ksi
Reinforcing Bars: 60 ksi
Concrete: 3 ksi

Solution:

Failure Mode 1: Weld Design Strength

$\phi M_n = \phi F_w S_y$ (Eq. 4–2)

Compute S_y (Neglecting Web Weld):

$I_y = 2\,(1/12)(.707)(5/16)(12)^3 = 63.63$

$S_y = 63.63/6 = 10.6$

$\phi = 0.75$

$\phi M_n = .75(1.5)(0.6)(70)(10.6) = 501$ in.–kips

$= 41.7$ ft.–kips

Failure Mode 2: Base Plate Failure

Case B: Inset Anchor Rods - Weak Axis Capacity.

Based on the weld pattern and the geometry provided:

(See Figure 4.12)

$d_2 = 12.12/2–2.5–0.605/2 = 3.26$ in.

$d_1 = 2.5$ in.

$b_2 = 12.12/2 = 6.06$ in.

$b_1 = 2d_1 = 5.0$ in.

$Z_1 = b_1t^2/4 = 5(1.0)^2/4 = 1.25$ in.^{3}

$\phi M_n^1 = 0.9(1.25)(36) = 40.5$ in.–kips

$Z_2 = b_2t^2/4 = 6(1.0)^2/4 = 1.52$ in.^{3}

$\phi M_n^2 = 0.9(1.52)(36) = 49.3$

$\phi P_n = \phi M_n^1/d_1 + \phi M_n^2/d_2$ (Eq. 4–3)

$= 40.5/2.5 + 49.3/3.26$

$= 31.22$ kips

$\phi M_n = 2\phi P_n d$ (Eq. 4–4)

where

$d = 5$ in.

$= 2(31.22)(5)$

$= 312.2$ in.–kips

$= 26$ ft.–kips

Failure Mode 3: Rupture of Anchor Rods

$\phi P_n = 0.75 F_n A_b$ (Eq. 4–10)

where

$F_n = 0.75F_u$, ksi

$F_u = 58$ ksi

$A_b = 0.4418$, in.^{2}

$\phi P_n = 0.75(0.75)(58)(0.4418) = 14.4$ kips/rod

$\phi M_n = 2\phi P_t d$ (Eq. 4–11)

$= 2(14.4)5 = 144$ in.–kips

$= 12$ ft. – kips

Failure Mode 4: Anchor Rod Buckling (Does not govern). (See Section 4.2.4.)

Failure Mode 5: Anchor Rod Nut Pull Through (Use proper washers to eliminate this failure mode.)

Failure Mode 6: Anchor Rod Pullout

$\phi T_n = 4\phi\sqrt{f'_c}\,A_e$ (Eq. 4–13)

$A_e = \pi(L_d+c/2)^2 + 2(L_d+c/2)(s+c) - \pi(c)^2/2$ (Eq. 4–12)

$= [3.14(12+0.75/2)^2] + 2(12+0.75/2)(5+0.75) + 3.14(.75)^2/2$

$= 628\text{ in.}^2$

Check Pier Area:

$A_e = 16(16) = 256\text{ in.}^2$ (Controls)

Note that edge distance will not control.

$\phi T_n = 4(0.75)\left(\sqrt{3000}\right)(256)/1000 = 42.1\text{ kips}$

Check Hook Bearing Strength:

$\phi T_n = 2(0.7)(0.85)f'_c d_{hook} L_{hook}$ (Eq. 4-14)

$= 2(0.7)(0.85)(3000)(0.75)(4)$

$= 10.7$ kips

$= 21.4$ kips for two rods (Controls)

$\phi M_n = \phi T_n d = (21.4)(5) = 107$ in.–kips (Eq. 4-15)

$= 8.9$ft.-kips

Failure Mode 7 : Anchor Rod Push Out (Does not occur with pier.)

Failure Mode 8 : Pier Bending Resistance

Determine the depth of the compression area:

$a = F_y A_s/.85 f'_c b$

$= 60{,}000(2)(0.44)/0.85(3000)(16)$

$= 1.294$ in.

$C = 0.85 f'_c a$

$= 0.85(3000)(16)(1.294)71000$

$= 52.8$ kips

$\phi M_n = C(d-a/2) = 692$ in.- kips (Eq. 4-17)

$= 52.8(13.75-1.294/2)$

$= 58$ ft.-kips

Check Reinforcing Development length:

Req'd length in footing:

$l_{dh} = 840 d_b/\sqrt{f'_c} \geq 8d_b$ or 6 in. (Eq. 4–18)

$= 840(.75)/\sqrt{3000} = 11.5\text{ in.} < 12\text{ in. o.k.}$

For the straight bars (#6 bars and smaller) in the pier:

$l_d = 2400 d_b/\sqrt{f'_c}$ (Eq. 4–19)

$= 2400(.75)/\sqrt{3000} = 32.9\text{ in.} < 33\text{ in. o.k.}$

Failure Mode 9: Footing Overturning

$\phi M_n = \phi W(L/2)$ (Eq.4-21)

where

$\phi = 0.9$

$W = P1+P2+P3$

$P1 = 65(40)7\ 1000 = 2.6$ kips (Column)

$P2 = 0.15(1.33)1.33(3) = 0.8$ kips (Pier)

$P3 = 0.15(1.25)6(6) = 6.75$ kips (Footing)

$W = 10.15$ kips, $L = 6$ft.

$\phi M_n = 0.9(10.15)(6/2) = 27.4$ ft. - kips

Comparing the above failure modes, the design moment strength is 8.9 ft.-kips. The governing failure mode would be anchor rod pull out.

Example 4-2:

Repeat Example 4-1 using outset anchor rods with embedded nuts.

Increase the pier size to 24" x 24" to accommodate the base plate. Increase the vertical reinforcement to be 8—#6 bars. The distance from the anchor rod to the flange tip, L equals 2.83 in.

BasePlate 1" x 20" x 1'-8"

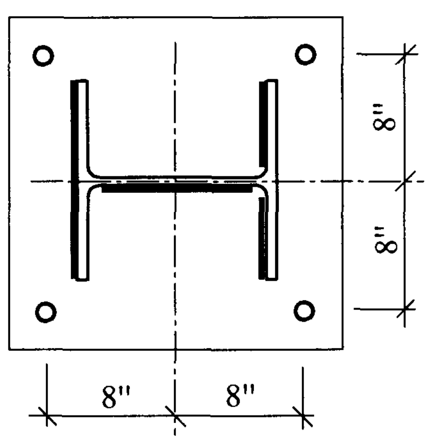


Fig. 4.23 Base Plate Detail

Solution:

Failure Mode 1: Weld Design Strength

$\phi M_n = \phi(F_w S_y) = 41.7$ ft. kips (Same as Example 4-1)

Failure Mode 2: Base Plate Failure

$b_e = 2L = 5.66\text{ in.} > 5.0\text{ in.}$

$\phi P_n = \phi_p b_e t_p^2 F_y/4L$ (Eq. 4-5)

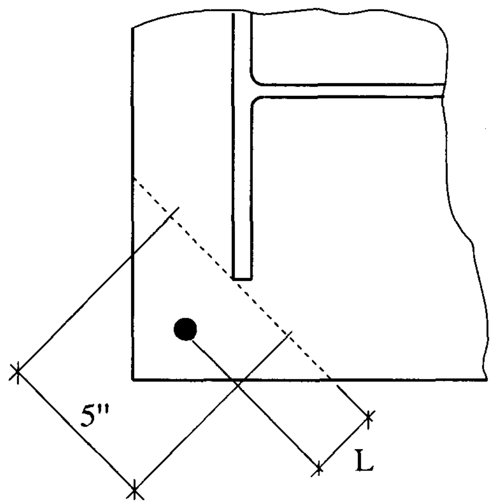


Fig. 4.24 Base Plate Yield Line

$= (0.9)(5)(1)^2(36)/[(4)(5)]$

$= 16.2$ kips

$\phi P_n = \phi_w F_w t(2)$ (Eq. 4-6)

$= (0.75)(0.9)(70)(.707)(5/16)(2)$

$= 20.9$ kips

$\phi P_n = \phi_p 50t\ t_p^{1.5}$ (Eq. 4-7)

$= (0.9)(50)(.221)(1)^{1.5}$

- 9.94 kips (Controls)

$\phi M_n = 2\phi P_n d$ (Eq. 4-8)

$= 2(9.94)(16) = 318$ in.-kips

$= 26.5$ft.-kips

Failure Mode 3: Rupture of Anchor Rods

$\phi P_n =$ 14.4 kips/rod (Same as Example 1)

$\phi M_n = 2\phi P_n d$ (Eq.4-11)

$= 2(14.4)(16) = 461$ in.-kips

$= 38.4$ ft.-kips

Failure Mode 4: Anchor Rod Buckling (Does not govern)

Failure Mode 5: Anchor Rod Nut Pull Over (Use proper washers)

Failure Mode 6: Anchor Rod Pull Out

$\phi T_n = 4\phi\sqrt{f'_c}A_e$ (Eq. 4-13)

$A_e = \pi(L_d+c/2)^2 + 2(L_d+c/2)(s+c) - \pi(c)^2/2$ (Eq. 4-12)

By inspection the pier area will control.

Check Pier Area:

$A_e = 20(20) = 400$ in.^{2}

$\phi T_n = 4(0.75)(\sqrt{3000})(400)/1000 = 65.7$ kips

$\phi M_n = 2\phi T_n d = 2(65.7)(16)$

$= 2102$ in.-kips (Eq. 4-15)

$= 175$ ft.-kips

Failure Mode 7: Anchor rod "push through" (Does not occur due to pier)

Failure Mode 8: Pier Bending Resistance

Determine the depth of the compression area:

$a = F_y A_s/.85f'_c b$

$= 60{,}000(2)(0.44)/0.85(3000)(24)$

$= 0.863$ in.

$C = 0.85f_c ab$

$= 0.85(3000)(0.863)(24)/1000$

$= 52.8$ kips

$\phi M_n = C(d-a/2)$ (Eq.4-17)

$= 52.8(21.75-0.863/2)$

$= 1126$ in.-kips

$= 94$ ft.-kips

Check Reinforcing Development length: (Same as Ex. 4-1)

Failure Mode 9: Footing Overturning:

$\phi M_n = \phi W(L/2)$ (Eq.4-21)

where

$\phi = 0.9$

$W = P1 + P2 + P3$

$P1 = 65(40) / 1000 = 2.6$ kips (Column)

$P2 = 0.15(2)(2)(3) = 1.8$ kips (Pier)

$P3 = 0.15(1.25)(6)(6) = 6.75$ kips (Footing)

$W = 11.15$ kips

$\phi M_n = 0.9(11.15)(3) = 30.2$ ft.-kips

Comparing the above failure modes, the design moment strength is 26.5 ft.-kips. The governing failure mode would be base plate failure.

Example 4-3:

Repeat Example 4-1, using the Tables provided in the Appendix.

Solution:

Failure Mode 1: Weld Design Strength

From Table 1, for a W12x65

$\phi M_n = 41.8$ ft. – kips

Failure Mode 2: Base Plate Failure

From Table 2, for a W12x65 with an anchor rod spacing of 5"x5", and abase plate 1"x13"x13"

$\phi M_n = 25.9$ ft.–kips

Failure Mode 3: Rupture of Anchor Rods

From Table 5, for a 3/4" A36 anchor rod the tension capacity, ϕP_n equals 14.4 kips, thus from:

$\phi M_n = 2\phi P_n d$

where

d = 5"

$\phi M_n = 2(14.4)(5) = 144$ in.-kips

$= 12$ ft.-kips

Failure Mode 4: Anchor Rod Buckling

(Does not govern.)

Failure Mode 5: Anchor Rod Nut Pull Over

To prevent pull over it is suggested that 3/16"x1-1/2"x1-1/2" plate washers be used.

Failure Mode 6: Anchor Rod Pull Out

From Table 10 the concrete pullout design strength for the 3/4 in. anchor rods spaced 5 inches apart and embedded 12 inches is 57.7 kips/rod. Thus, the total pullout design strength for the two rods is 115.4 kips.

Check the design strength based on pier area.

$A_e = 16(16) = 256$ in.^{2}

$\phi T_n = 4\phi\sqrt{f'_c}A_e$

$= 4(.75)\left(\sqrt{3000}\right)(256)/1000 = 42.1$ kips

$\phi M_n = 2\phi T_n d$

$= 2(42.1)(5) = 421$ in.–kips

$= 35.1$ ft.–kips

Since hooked rods are used the additional check for hook straightening must be made.

From Table 6, the tension design strength for a 3/4 in. rod with a 4 in. hook is 10.7 kips. Therefore the moment resistance is controlled by straightening of the hooked rods. The moment resistance:

$\phi M_n = 2\phi T_n d$

$= 2(10.7)(5) = 107$ in.-kips

$= 8.9$ ft.-kips (controls)

Failure Mode 7: Anchor Rod "Push Out" (Does not occur due to pier.)

Failure Mode 8: Pier Bending Resistance

The reinforcement ratio for the 16"x16" pier with 4-#6 bars equals $4(0.44)(100)/(16)^2 = 0.69\%$.

From Table 18 the bending design strength for a pier with 0.5% reinforcing equals 51.4 ft.-kips.

The development length of the reinforcing must also be checked. From Table 20, for #6 hooked bars the development length is 12 inches. Therefore o.k. For the straight bar the development length is 33 inches, therefore o.k.

Failure Mode 9: Footing overturning

From Table 19, the overturning resistance for the 6'-0"x6'-0"x1'-3" can be conservatively (not including the weight of the column and pier) based on the table value for a 6'-0"x6'-0"x 1-2' footing.

$\phi M_n = 18.9$ft.-kips

Based on the above calculation the overturning resistance is 8.9 ft.-kips and is based on anchor rod pullout.

It should be noted that concrete punch out of the anchor rods is not a failure mode because of the existence of the concrete pier. To illustrate the use of the tables relative to punch out, determine the overturning resistance with no pier. The anchor rods have a 3 inch clearance from the bottom of the footing.

From Table 14, for the 3/4 in. anchor rods on a 5 in. by 5 in. grid $\phi P_n = 6.5$ kips per rod.

Determine the design strength:

$\phi M_n = 2(\phi P_n)(5)/12$

$= 2(6.5)(5)/12 = 5.4$ ft.-kips

This illustrates the importance of providing sufficient clear cover or adding the nut as shown in Figure 4.17.

Example 4-4:

Repeat Example 4-2, using the Tables provided in the Appendix.

Solution:

Failure Mode 1: Weld Design Strength

Same as Example 3.

$\phi M_n =$ 41.7ft.-kips

Failure Mode 2: Base Plate Failure

From Table 3, $\phi M_n = 26.5$ ft.-kips

Failure Mode 3: Rupture of Anchor Rods

From Table 5, $\phi P_n = 14.4$ kips

$\phi M_n = 2\phi P_n d$

$= 2(14.4)(16) = 461$ in.-kips

$=$ 38.4 ft.-kips

Failure Modes: 4 and 5

Same as Example 3.

Failure Mode 6: Anchor Rod Pull Out

From Table 10, for the 3/4 in. anchor rods spaced 16" o.c. with nutted ends, embedded 12 inches:

$\phi P_n =$ 82.3 kips/rod

$\phi M_n = 2\phi P_n d$

$= 2(82.3)(16) = 2{,}634$ in.-kips

$=$ 219 ft.-kips

Check the design strength based on pier area.

$A_e = 20(20) = 400$ in.^{2}

$\phi T_n = 4\phi \sqrt{f'c}\, A_e$

$= 4(.75)\left(\sqrt{3000}\right)(400) = 65.7$ kips

$\phi M_n = 2\phi T_n d$

$= 2(65.7)(16) = 2{,}102$ in.-kips

$=$ 175 ft.-kips (controls)

Failure Mode 7: Anchor Rod "push through" (Does not occur because of pier.)

Failure Mode 8: Pier Bending Resistance

The reinforcement ratio for the 24"x24" pier with 8-#6 bars equals:

$8(0.44)(100)/(24)^2 = 0.6\%$

From Table 18, the bending design strength for the pier is 147.4 ft.-kips. (Based on a 0.5% reinforcement ratio.)

The development length calculations are the same as in Example 4-3.

Failure Mode 9: Footing overturning

Same as Example 4-3,

$\phi M_n =$ 18.9 ft.-kips

Based on the above calculations the overturning resistance equals 18.9 ft.-kips and is controlled by footing overturning.

Since the controlling failure mode was based on conservative values taken from Table 19, and which do not include the pier or column weight, a more exact calculation could be performed as in Example 4-1.

Example 4-5

For the column/footing detail provided in Example 4-1, determine if a 25 foot and a 40 foot tall column could safely resist the overturning moment from a 60 mph wind. Use exposure B conditions.

The reduction factor of 0.75 is not applied to the wind velocity because this check is for an actual expected velocity.

From Example 4-1, the overturning design strength equals 8.9 ft.-kips.

Wind Calculations:

$F = q_z G_h C_f A_f$

where

q_z = evaluated at height Z above ground

G_h = given in ASCE 7 Table 8

C_f = given in ASCE 7 Tables 11-16

A_f = projected area normal to wind

q_z - $0.00256 K_Z (IV)^2$

K_z = ASCE 7 Table 6, Velocity Exposure Coefficient

I = ASCE 7 Table 5, Importance Factor

V = Basic wind speed per ASCE 7 para. 6.5.2.

25 foot column calculations:

$q_z = 0.00256(0.46)[(1.0)(60)]^2 = 4.24$ psf

$F = (4.24)(1.54)(1.5)Af = 9.8A_f$ psf

A_f = 12 in. (column width) = 1.0 ft.

$F = 9.8(1.0) = 9.8$ psf

$F_u = (1.3)(9.8) = 12.74$ psf

$M_u = F_u h^2/2 = (12.74)(25)^2/2 = 3.981$ ft.-lbs.

$=$ 3.98 ft.-kips

$3.98 < 8.9$ o.k.

40 foot column calculations:

q_z = $0.00256(0.57)[(1.0)(60)]^2 = 5.25$ psf

F = $(5.25)(1.46)(1.5)A_f = 11.5$ psf

A_f = 1.0 ft.

F = $11.5(1.0) = 11.5$ psf

F_u = $(1.3)(11.5) = 14.95$ psf

M_u = $F_u h^2/2 = (14.95)(40)^2/2 = 11{,}960$ ft.–lbs.

= 11.96 ft.–kips

$11.96 > 8.9$ n.g.

Example 4-6

Would the columns described in Example 4-5 safely support a 300 pound load located 18 inches off of the column face?

Factored load:

P_u = $1.6(300) = 480$ lbs.

M_u = $480(24)/12 = 960$ ft.–lbs.

$= 0.96$ ft.–kips

$0.96 << \phi M_n$

From Example 4-1, the overturning design strength equals 8.9 ft.-kips.

$0.96 << 8.9$ ft.–kips o.k.

4.3 Tie Members

During the erection process the members connecting the tops of columns are referred to as tie members. As the name implies, tie members, tie (connect) the erected columns together. Tie members can serve to transfer lateral loads from one bay to the next. Their function is to transfer loads acting on the partially erected frame to the vertical bracing in a given bay. Tie members also transfer erection loads from column to column during plumbing operations. Typical tie members are wide flange beams, steel joists and joist girders.

Since tie members are required to transfer loads, their design strength must be evaluated. Strength evaluation can be divided into three categories:

A. Tensile Strength
B. Compressive Strength
C. Connection Strength

4.3.1 Wide Flange Beams

Tensile Design Strength

The tension design strength of any wide flange beam acting as a tie member will typically not require detailed evaluation. The design strength in tension will almost always be larger than the strength of the connection between the tie member and the column. Thus, the tie member will not control the design of the tie. If the tensile design strength of a tie member must be determined, it can be determined as the lesser value of the following:

For yielding in the gross section:

ϕ_t = 0.90

P_n = $F_y A_g$

For fracture in the net section:

ϕ_t = 0.75

P_n = $F_u A_e$

where

A_e = effective net area, in.^{2}

A_g = gross area of member, in.^{2}

F_y = specified minimum yield stress, ksi

F_u = specified minimum tensile strength, ksi

P_n = nominal axial strength, kips

Compression Design Strength

For compression loading wide flange tie beams can buckle since they are not laterally supported. Shown in Table 4.1 are buckling design strengths for the lightest wide flange shapes for the depths and spans shown in the Table. These values cannot exceed the connection value for the type of connection used.

Span (ft.)	Depth (in.)	Compression Design Strength (kips)
20	14	20
25	16	20
30	18	25
35	21	25
40	24	25
45	27	60
50	30	65

Table 4.1 Wide Flange Design Buckling Strengths

The compression design strengths for specific wide flange beams can be determined from the column equations contained in Chapter E of the AISC Specifications and the design aids of the LRFD Manual Part 3.

Connection Design Strength

Common connections consist of:

1. Beams resting on column tops.
2. Framing angle connections.
3. Single-Plate Shear Connections.
4. Seat angles.

Presented in Table 4.2 are connection design strengths for these connections. These strengths are based on the installation of two 3/4" diameter A325 bolts snug tight in each connection. The controlling element is also shown.

Connection Type	Design Strength (kips)	Controlling Element
Beams on Columns	30	Bolts
1/4 in. Framing Angles	10	Framing Angles
5/16 in. Framing Angles	15	Framing Angles
3/8 in. Framing Angles	22	Framing Angles
1/4 in. Single-Plate Shear Connections	30	Bolts
3/8 in. Seat	30	Bolts

Table 4.2 WF Connection Strengths

4.3.2 Steel Joists

Tensile Strength

As for the case of wide flange beams the tensile design strength for a tie joist will generally not require evaluation. The connection of the tie joist to the column is almost always weaker than the tensile design strength for the joist. If one wants to evaluate the tensile design strength, it can again be determined from the equation:

$$\phi T_n = \phi F_y A_g \text{ or } F_u A_e$$

It is suggested that only the top chord area be used for A in the calculation. The area can be determined by contacting the joist supplier or by physically measuring the size of the top chord. The yield strength of K and LH series joists top chords is 50 ksi.

Compressive Strength

Because the compressive design strength of an unbridged K-series joist is low, unbridged K-series joists should not be relied upon to transfer compression forces from one bay to the next. The unbridged strength is generally in the 700 to 800 pound range. Once the joists are bridged they have considerably greater compressive strength. Approximate compressive design strengths (LRFD) are shown in Table 4.3a for several spans with the joist sizes as shown. Provided in Table 4.3b are the service load (ASD) values.

Span (ft.)	Joist Desig-nation	Rows of Bridging	Design Strength (kips)
20	10K1	2	11.0
25	14K1	2	7.0
30	18K3	3	7.0
35	20K4	3	6.0
40	20K5	4	7.0
45	26K5	4	7.0
50	28K7	4	7.0

Table 4.3a Joist Compression Design Strength

Span (ft.)	Joist Desig-nation	Rows of Bridging	Allowable Load (kips)
20	10K1	2	6.0
25	14K1	2	4.0
30	18K3	3	4.0
35	20K4	3	3.5
40	20K5	4	4.0
45	26K5	4	4.0
50	28K7	4	4.0

Table 4.3b Joist Compression Allowable Load

Compressive design strengths for other spans and joist sizes can be obtained from the joist supplier.

Connection Strength

Tie joists are typically connected to column tops using two ½-inch A307 bolts. Many erectors also weld the joists to their supports using the Steel Joist Institute's minimum weld requirements (two ⅛-inch fillet welds one inch long). Since most joist manufacturers supply long slotted holes in the joist seats the welding is required to hold the joists in place. The design shear strength for the two ⅛-inch fillet welds is 7.4 kips, based on using E70 electrodes.

It should be remembered that if the connections are not welded a considerable displacement may occur before the bolts bear at the end of the slot.

The design shear strength for other weld sizes can be determined from the AISC LRFD Specification. For E70 electrodes the design shear strength per inch of weld length can be calculated by multiplying the fillet weld size in sixteenths by 1.392.

4.3.3 Joist Girders

Tensile Strength

The same comments apply to joist girders as do for joists acting as tension ties. Connection strengths will again typically control the design.

Compressive Strength

The design compressive strength of joist girders can be determined from the AISC LRFD Specification column equations. Joist girders should be considered as laterally unbraced until the roof or floor deck has been secured to the joists. Joists which are not decked may supply some lateral bracing to the joist girder but the amount of support cannot be readily determined.

Shown in Table 4.4a are design compressive strength (LRFD) values for joist girders with the top chord angles shown. Provided in Table 4.4b are the service load (ASD) values. In all cases the minimum available thicknesses of the angles has been assumed in calculating the values provided in the table.

Connection Strength

Tie joist girders are typically connected to column tops using two $^3/_4$-inch A325 bolts. The minimum size SJI welds consist of two ¼-inch fillet welds 2 inches long. Long slotted holes are generally provided in the joist girder seats as in the case of joists. The design shear strength for the two ¼-inch fillet welds is 29.6 kips.

Span ft.	Top Chord Angle Leg Length, (in.)					
	21/2	3	31/2	4	5	6
30	3	6	12	18	43	74
35	2	4	9	13	32	55
40	2	3	7	10	24	42
45	1	2	5	8	19	33
50	1	2	4	6	16	27
55	–	2	4	5	13	22
60	–	–	3	4	11	19

Table 4.4a Joist Girder Design Buckling Strengths (kips)

4.4 Use of Permanent Bracing

The design procedure for temporary bracing can be applied to permanent bracing used as part of the temporary bracing scheme. It involves the determination of a design lateral force (wind, seismic, stability) and confirmation of adequate resistance. The design procedure is illustrated is the following example.

Span ft.	Top Chord Angle Leg Length, (in.)					
	2½	3	3½	4	5	6
30	1.8	3.5	7.1	10.6	25.3	43.5
35	1.2	2.5	5.3	7.6	18.8	32.4
40	1.2	1.8	4.1	5.9	14.1	24.7
45	0.6	1.2	2.9	4.7	11.2	19.4
50	0.6	1.2	2.5	3.5	9.4	15.9
55	-	1.2	2.5	2.9	7.6	12.9
60	–	–	1.8	2.5	6.5	11.2

Table 4.4b Joist Girder Service Load Buckling Strengths (kips)

Example 4-7: (Service Load Design)

This example is done with service loads for easy comparison to Example 5-1.

Given: One frame line braced with permanent bracing.
Bays: 6 bays at 40'-0"
Transverse bay: 40'-0" to one side of frame
Have height: 25'-0"
Tie beams: W18X35
Girders: W24X55
Joists: 22K9 @5'-0" o.c.
Columns: W8X31
Permanent bracing: 2(2) < 3 X 3½ X¼ w/(4) ¾" dia. A325N Bolts
Permanent brace force: 38 kips
Wind speed: 75 mph
Exposure: B

Determination of wind load:

From ASCE 7 Table 4:

$F = q_z G_h C_f A_f$ Eq.5-5

where

q_z = evaluated at height Z above ground

G_h = given in ASCE 7 Table 8

C_f = given in ASCE 7 Tables 11-16

A_f = projected area normal to wind

$q_z = 0.00256K_z(IV)^2$ Eq. 3-2

K_z = ASCE 7 Table 6, Velocity Exposure Coefficient

I = ASCE 7 Table 5, Importance Factor

V = Basic wind speed per ASCE 7 para. 6.5.2.

Per the proposed ASCE Standard "V" can be reduced using the 0.75 factor for an exposure period of less than 6 weeks.

Calculating:

$$q_z = 0.00256(0.46)(1.0(0.75)75)^2 = 3.73 \text{ psf}$$

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$$F = 3.73(1.54)\boxed{(1.5)}(A_f) = 8.61(A_f)$$

The area of the frame (A_f) is computed as follows:

First frame $= (40)\ 0.5\ (18/12) + 25\ (0.5)\ (8/12)$

$= 30 + 8.33 = 38.33 \text{ ft.}^2$

Thus the total frame area is:

$3(38.33) + 4\ (38.33)\ (1.0 - 0.15) = 245.3 \text{ ft.}^2$

The net area of joists is computed as:

$(22/12)20(6)7(0.3)(0.7) = 323.4 \text{ ft.}^2$

Thus,

$$A_f = 245.3 + 323.4 = 568.7 \text{ ft.}^2$$

F at the level of the roof strut is:

$$F = 8.61\ (568.7) = 4{,}896.6 \text{ lbs.}$$

$$F = 4.9 \text{ kips}$$

Force in diagonal = 4.9 kips (47.2/40) = 5.8 kips

This force is less than the bracing force of 38 kips for which the permanent bracing is designed.

One bolt in each angle is adequate to resist the temporary bracing force in the diagonal. The permanent bracing connections are adequate by inspection.

The roof strut itself is a W24X55 spanning 40 feet. The strut force is 4.8 kips. Per Tables 4.1 and 4.2, it can be seen that this member is adequate to carry the strut force.

A check of PA effects is not necessary for permanent diagonal bracing used as part of the temporary bracing scheme.

Lastly, the column on the compression side of the diagonally braced bay must be checked.

The column itself is adequate by inspection for the vertical component of the temporary bracing force. This vertical component is 5.8 (25/47.2) = 3.1 kips which is far less than the column axial capacity.

4.5 Beam to Column Connections

In the typical erection process, the beam to column connections are erected using only the minimum number of bolts required by OSHA regulations. This is done to expedite the process of "raising" the steel in order to minimize the use of cranes. Final bolting is not done until the structure is plumbed.

In addition to the connection design strength using the minimum fasteners, additional design strength can be obtained by installing more fasteners up to the full design strength. This additional design strength can be incorporated in the temporary bracing scheme. Because of the complexity of integrating final connections in the temporary supports this topic is not developed in this guide, however the principles are fully developed in current literature such as LRFD Manual of Steel Construction, Volume II (14) and [ASD] Manual of Steel Construction, "Volume II – Connections" (13).

4.6 Diaphragms

Roof or floor deck can be used during the erection process to transfer loads horizontally to vertical bracing locations. The ability of the deck system to transfer loads is dependent on the number and type of attachments made to the supporting structure and the type and frequency of the deck sidelap connections. Because of the number of variables that can occur with deck diaphragms in practice, no general guidelines are presented here. The designer of the temporary bracing system is simply cautioned not to use a partially completed diaphragm system for load transfer until a complete analysis is made relative to the partially completed diaphragm strength and stiffness. Evaluation of diaphragm strength can be performed using the methods presented in the Steel Deck Institute's "Diaphragm Design Manual" (8).

5. RESISTANCE TO DESIGN LOADS — TEMPORARY SUPPORTS

The purpose of the temporary support system is to adequately transfer loads to the ground from their source in the frame. Temporary support systems transfer lateral loads (erection forces and wind loads) to the ground. The principal mechanism used to do this is temporary diagonal bracing, such as cables or struts, the use of the permanent bracing or a combination thereof. Temporary diagonal struts which carry both tension and compression or just compression are rarely used. Cable braces are often used. In cases when the building is framed with multiple bays in each direction, diaphragms are used in the completed construction to transfer lateral loads to rigid frames or braced bays. Before the diaphragm is installed temporary supports are required in the frame lines between the frames with permanent bracing.

The use of cables to provide temporary lateral bracing in a frame line requires that the following conditions be met:

1. Functional strut elements (beams, joists, girders) to transfer the lateral load to the cable braced bay.
2. Functional transfer of the lateral load into the bracing tension cable and compression column pair.
3. Functional resistance of the anchorage of the cable and the column to their respective bases and to the ground.

The development of the beams or joists as functional strut elements requires a check of their design strength as unbraced compression elements, since their stabilizing element, the deck, will not likely be present when the strength of the struts is required. The strut connections must also be checked since the connections will likely only be minimally bolted at the initial stage of loading. The evaluation of strut members is discussed in detail elsewhere in this Design Guide.

The development of the cable is accomplished by its attachment to the top of the compression column and to the point of anchorage at the bottom end. In multi-tier construction the bottom end would be attached to the adjacent column. In the lowest story of a multi story frame or a one story frame, the lower end of the cable would be attached to the base of the adjacent column or to the foundation itself.

5.1 Wire Rope Diagonal Bracing

Bracing cables are composed of wire rope and anchorage accessories. Wire rope consists of three components: (a) individual wires forming strands, (b) a core and (c) multi-wire strands laid helically around the core. The wires which form the strands are available in grades, such as "plow steel", "improved plow steel" and "extra improved plow steel". Cores are made of fiber, synthetic material, wire or a strand. The core provides little of the rope strength but rather forms the center about which the strands are "laid". Laying is done in four patterns: regular, left and right and Lang, left and right. The left and right refer to counter-clockwise and clockwise laying. Regular lay has the wires in the strands laid opposite to the lay of the strands. Lang lay has the wires in the strands laid in the same direction as the lay of the strands. Most wire rope is right lay, regular lay. Wire rope is designated by the number of strands, the number of wires per strands, the strand pattern (construction), the type of core, type of steel and the wire finish. The diameter of a wire rope is taken at its greatest diameter. The wire rope classification is designated by the number of strands and by the number of wires per strand.

The strength of wire rope is established by the individual manufacturers who publish tables of "Nominal Breaking Strength" for the rope designation and diameter produced. The safe working load for wire rope is established by dividing the Normal Breaking Strength by a factor of safety. This factor of safety ranges between 6 and 2 depending on how the wire rope is used. The information presented on wire rope in this guide is taken from two references: the "Wire Rope Users Manual" published by the Wire Rope Technical Board (19) and the "Falsework Manual" published by the State of California Department of Transportation (Caltrans) (9). The Wire Rope Technical Board does not set a factor of safety for wire rope used as temporary lateral supports. However, the Users Manual does state that "a 'common' design factor is 5". This design factor is used for slings and other rigging, but it is unnecessarily conservative for the diagonal bracing covered in this guide. The authors recommend the use of a factor of safety of 3 for ASD and the use of $\phi = 0.5$ for LRFD. The Caltrans Falsework Manual uses a factor of safety of 2.0 but it applies to the breaking strength reduced by a connection efficiency factor. Caltrans assigns the following connection efficiencies:

Sockets-Zinc Type	100%
Wedge Sockets	70%
Clips-Crosby Type	80%
Knot and Clip (Contractor's Knot)	50%
Plate Clamp-Three Bolt Type	80%
Spliced eye and thimble	
3/8 inch to 3/4 inch	95%
7/8 inch to 1 inch	88%

Wire rope connections using U-bolt clips (Crosby type) are formed by doubling the rope back upon itself and securing the loose or "dead" end with a two part clip consisting off a U-bolt and a forged clip. Table 5.1 is taken from OSHA 1926.251. It gives the minimum number and spacing of clips for various wire sizes. The spacing is generally six times the wire diameter. Clip manufacturers give minimum installation torques for the nuts in their literature. When installing the clips, the U-bolt is set on the dead (loose) end. The clip is placed against the live (loaded) side. "Never saddle a dead horse," as the saying goes.

OSHA CFA 1926.251

TABLE H-20 - NUMBER AND SPACING OF U-BOLT WIRE ROPE CLIPS

Improved plow steel, rope diameter (inches)	Number of clips		Minimum spacing (inches)
	Drop forged	Other material	
$^1/_2$	3	4	3
$^5/_8$	3	4	$3^3/_4$
$^3/_4$	4	5	$4^1/_2$
$^7/_8$	4	5	$5^1/_4$
1	5	6	6
$1^1/_8$	6	6	$6^3/_4$
$1^1/_4$	6	7	$7^1/_2$
$1^3/_8$	7	7	$8^1/_4$
$1^1/_2$	7	8	9

Table 5.1 U-Bolt Wire Rope Clips

The use of wire rope (cables) in diagonal temporary bracing also requires an assessment of the stiffness of the braced panel which is primarily a function of the elongation of the cable under load. This elongation has two sources: elastic stretch (roughly (PL)/(AE)) and constructional stretch, which is caused by the strands compacting against one another under load. Wire rope can be pre-stretched to remove some constructional elongation.

Elastic stretch in cable is not a linear function as with true elastic materials. The modulus of elasticity (E) for wire rope varies with load. When the load is less than or equal to 20 percent of the breaking strength a reduced E equal to 0.9E is used in industry practice. When the cable load exceeds 20 percent of the breaking strength the elastic stretch is the sum of Δ_1 and Δ_2 as defined below.

$$\Delta_1 = \frac{0.2(NBS - P)L}{A(0.9)E} \qquad \text{Eq. 5-1}$$

$$\Delta_2 = \frac{(CDF - 0.2(NBS))(L + \Delta_1)}{A(E)} \qquad \text{Eq.5-2}$$

where

Δ_1 and Δ_2 = cable stretch, ft.

NBS = Nominal Breaking Strength, lbs.

P = Cable Preload, lbs.

CDF = Cable Design Force, lbs.

L = cable length, ft.

A = net metallic area of cable, in.^{2}

E = nominal modulus of elasticity, psi

Constructional stretch is given by the following formula:

$$\Delta_{cs} = \left(\frac{\text{Applied Load}}{0.65(NBS)}\right)(CS\%)(L) \qquad \text{Eq. 5-3}$$

where

CS% is the constructional stretch percentage supplied by the manufacturer (usually between 0.75% and 1.0%).

Δ_{cs} = constructional stretch, ft.

L = cable length, ft.

The load and cable strength are in pounds.

In order for wire rope cables to perform properly it is necessary to provide an initial preload by drawing them up to a maximum initial drape. The Caltrans Falsework Manual provides the following maximum drapes for these cable sizes:

Cable Size	Maximum Drape (A)
3/8	1 inches
1/2	2 inches
3/4	2-3/4 inches

The cable drape (A) is a vertical distance measured at mid-bay between the two cable end points.

Drawing up the cable to the maximum allowed drape induces a force in the cable which can be calculated from the following equation presented in the Falsework Manual.

$$P = qx^2/8A\cos\psi. \qquad \text{Eq.5-4}$$

where

P = cable preload value, lbs.

q = cable weight, pounds per ft.

x = horizontal distance between connection points, ft.

A = cable drape, ft.

ψ = angle between horizontal and cable (if straight), degrees

The Caltrans Falsework Manual also recommends a minimum preload of 500 pounds.

It should be noted that the installers should be cautioned not to overdraw the cable as this may pull the frame out of plumb or may overload components of the frame.

The following eight tables (Tables 5.2 through 5.8) present wire rope data taken from the "Wire Rope Users Manual" for various classifications, core types and steel grades. The values for weight and metallic area are labeled approximate since the actual values are different for each manufacturer. The value given for area is that appropriate to the particular construction identified (S, Seale; FW, Filler Wire; W, Warington). The Nominal Breaking Strength given is the industry consensus value. Galvanized wire is rated at 10 percent less than the values given for Bright (uncoated) wire. Data for a specific wire rope (diameter, classification, construction, core and steel) should be obtained from the manufacturer.

6x7 Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 13,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs/ft.	$in.^2$	lbs.
3/8	0.21	0.054	11,720
7/16	0.29	0.074	15,860
1/2	0.38	0.096	20,600
9/16	0.48	0.122	26,000
5/8	0.59	0.150	31,800
3/4	0.84	0.216	45,400
7/8	1.15	0.294	61,400
1	1.50	0.384	79,400

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.2 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 11,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	$in.^2$	lbs.
3/8	0.24	0.060	12,200
7/16	0.32	0.082	16,540
1/2	0.42	0.107	21,400
9/16	0.53	0.135	27,000
5/8	0.66	0.167	33,400
3/4	0.95	0.240	47,600
7/8	1.29	0.327	64,400
1	1.68	0.427	83,600

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.4 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 12,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	$in.^2$	lbs.
3/8	0.24	0.057	12,200
7/16	0.32	0.077	16,540
1/2	0.42	0.101	21,400
9/16	0.53	0.128	27,000
5/8	0.66	0.158	33,400
3/4	0.95	0.227	47,600
7/8	1.29	0.354	64,400
1	1.68	0.404	83,600

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.3 Nominal Breaking Strength of Wire Rope

8x19 (W) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 9,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	$in.^2$	lbs.
3/8	0.22	0.051	10,480
7/16	0.30	0.070	14,180
1/2	0.39	0.092	18,460
9/16	0.50	0.116	23,200
5/8	0.61	0.143	28,600
3/4	0.88	0.206	41,000
7/8	1.20	0.280	55,400
1	1.57	0.366	72,000

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.5 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.066	13,120
7/16	0.35	0.090	17,780
1/2	0.46	0.118	23,000
9/16	0.59	0.149	29,000
5/8	0.72	0.184	35,400
3/4	1.04	0.264	51,200
7/8	1.42	0.360	69,200
1	1.85	0.470	89,800

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.6 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 14,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.069	13,120
7/16	0.35	0.094	17,780
1/2	0.46	0.123	23,000
9/16	0.59	0.156	29,000
5/8	0.72	0.193	35,400
3/4	1.04	0.277	51,200
7/8	1.42	0.377	69,200
1	1.85	0.493	89,800

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.8 Nominal Breaking Strength of Wire Rope

6x19 (S) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.066	15,100
7/16	0.35	0.090	20,400
1/2	0.46	0.118	26,600
9/16	0.59	0.149	33,600
5/8	0.72	0.184	41,200
3/4	1.04	0.264	58,800
7/8	1.42	0.360	79,600
1	1.85	0.470	103,400

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.7 Nominal Breaking Strength of Wire Rope

6x37 (FW) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 14,000,000 psi			
Nominal Diameter	Approximate Weight	Approximate Metallic Area	Nominal Breaking Strength[1]
inches	lbs./ft.	in.^{2}	lbs.
3/8	0.26	0.069	15,100
7/16	0.35	0.094	20,400
1/2	0.46	0.123	26,600
9/16	0.59	0.156	33,600
5/8	0.72	0.193	41,200
3/4	1.04	0.277	58,800
7/8	1.42	0.377	79,600
1	1.85	0.493	103,400

[1] $\phi = 0.5$ for LRFD, F.S. = 3 for ASD

Table 5.9 Nominal Breaking Strength of Wire Rope

Because of the relative flexibility of wire rope due to its construction, forces can be induced in the bracing due to the frame's initial lateral displacement. This second order effect is commonly referred to as a PA effect. In the case of a cable diagonal in a braced bay the bracing must resist gravity load instability such as might be induced by out of plumb columns and more importantly must resist the induced forces when the upper end of the column is displaced by a lateral force (wind) to a position that is not aligned over the column base.

Gravity load stability is usually addressed with a strength design of the bracing for an appropriate equivalent lateral static force, commonly 2 percent of the supported gravity load. Other sources have recommended that a 100 pound per foot lateral load be applied to the perimeter of the structure to be braced. This stability check would not normally govern the design of temporary bracing.

The forces induced by lateral load displacements are more significant however. Since each increment of load induces a corresponding increment of displacement, the design of a diagonal cable brace would theoretically require an analysis to demonstrate that the incremental process closes and that the system is stable. If the incremental load/displacement relationship does not converge, the system is unstable. In general, the cables braces within the scope of this guide would converge and one cycle of load/displacement would account for 90% of the PA induced force. In the example which follows, the induced force is approximately 20% of the initial wind induced force. Using a factor of safety of 3, a design which resists the induced wind force plus one cycle of PA load-displacement should be deemed adequate.

The design procedure for the design of temporary diagonal cable bracing is illustrated in the following example.

Example 5-1: (Service Load Design)

Given: One frame line braced with cables.
Bays: 6 bays of 40'-0"
Transverse bays: 40'-0" each side of frame
Have height: 25'-Q"
Tie beams: W18X35
Girders: W24X68
Joists: 22K9 @ 5'-0" o.c.
Columns: W8X40
Wind speed: 75 mph
Exposure: B
Seismic coefficients: $A_a = 0.10$, $A_v = 0.10$

Wind pressure and seismic base shear per ASCE 7-93 and Proposed ASCE Standard "Design Loads on Structures During Construction."

Determination of wind load:

From ASCE 7 Table 4:

$$F = q_z G_h C_f A_f \qquad \text{(Eq.5-5)}$$

where

q_z = evaluated at height Z above ground

G_h = given in ASCE 7 Table 8

C_f = given in ASCE 7 Tables 11-16

A_f = projected area normal to wind

$$q_z = 0.00256K_z\,(IV)^2 \qquad \text{(Eq. 3-2)}$$

K_z = ASCE 7 Table 6, Velocity Exposure Coefficient

I = ASCE 7 Table 5, Importance Factor

V = Basic wind speed per ASCE 7 para. 6.5.2.

Per the proposed ASCE Standard V can be reduced using the 0.75 factor for an exposure period of less than 6 weeks.

Calculating:

$$q_z = 0.00256(0.46)[1.0(0.75)75]^2 = 3.73 \text{ psf}$$

$$F = 3.73(1.54)(1.5)(Af) = 8.61(A_f)\text{lbs.}$$

Determination of Af:

The frame in this example has the following surface area to the wind. There are seven transverse bays. The frame area for the first frame is equal to the tributary beam area plus the tributary column area.

First frame: 2(40)(0.5)(18/12) + 25(0.5)(8/12)
= 60.0 + 8.33 = 68.33 sq. ft.

The second through seventh frame have the same area. The total frame area, including the 0.15 reduction is thus:

= 3(68.33)+ 4(68.33)(1.0-0.15)

= 437.3 sq.ft.

The net effective area of the joists can be computed as follows. There are seven joists per bay in six bays. The gross area is:

(22/12)x40x7x6 = 3080 sq. ft.

The effective solid area would be gross projected area times 0.3 for net area. The shielding reduction is $(1+\eta+(n-2)\eta^2)/n = 0.66$, use 0.7.

where

η = 0.8 (a/d = 2.5, φ = 0.2)

n = 7x6 = 42

Thus the total effective area of the joists is:

3080x0.3x0.7 = 647.8 sq. ft.

The total frame area, A_f, is

A_f = 437.3+ 646.8 =1084 sq.ft.

F at the level of the roof struts is:

F = 8.61(1084) = 9333 lbs.

Determination of stability loading:

"Design Loads on Structures During Construction", proposed ASCE Standard would require a 100 pound per foot along the 40 foot perimeter or 2 percent of the total dead load applied horizontally along the structure edge.

Total vertical supported dead load:

7 columns: 7(40)25	=	7,000 lbs.
7 beams: 7(35)40	-	9,800 lbs.
6 girders: 6 X (68)40	=	16,320 lbs.
Roof framing*: 6(40)40(5)	=	48.000 lbs.
Total		81,120 lbs.

*Joists and bundled deck.

In this example the two stability design values would be:

(100)(40) = 4000 lbs.
or
(81,120)(0.02)=1622 lbs.

In this example neither of these forces would govern as both are less than the wind design force of 9,333 lbs.

Determination of seismic base shear:

$V = C_s W$ (Eq . 3-5)

Determine C_s

$$= \frac{2.5A_a}{R} = \frac{2.5(0.10)}{5} = 0.050 \quad \text{(Eq. 3-7)}$$

where

A_a = 0.10 (ASCE 7 Figure 9.1 (Building located in Kansas City))

R = 5.0 (ASCE 7 Table 9.3-2)

Determine W

W = 81,120 lbs. per calculation above.

V = 0.050 (81,120) = 4056 lbs.

Seismic loading does not govern the design.

Design of diagonal cable:

The geometry of the cable for the purposes of this calculation is:
25 feet vertical (column height)
40 feet horizontal (bay width)

Using the Pythagorean theorem, the diagonal length (L) is 47.2 feet.

The strut force at the brace = 9333 lbs.

The column force component =9333(25/40)=5833 lbs.

The diagonal cable force = 9333 (47.2/40) = 11,013 lbs.

Using a factor of safety of 3.0, the minimum nominal breaking strength required is:

(11,013)(3) =33,039 lbs.

Based on Table 5.2 data a 3/4 inch diameter wire rope has the following properties:

Designation: 6x7 FC-IPS
(Fibercore - improved plow steel)

Area: 0.216, $in.^2$

Wt. per foot: 0.84 lbs. per ft.

Modulus of elasticity: 13,000 ksi (nominal)

CS% = 0.75%

Nominal breaking strength = 45,400 lbs.

Calculation of cable pre-loading to remove drape:

Per Caltrans the maximum cable drape (A) should be 2.375 inches.

The preload required for this maximum drape (A) is

$P = q(x)^2/[8(A)\cos\psi]$ (Eq 5-4)

In this example, cosy - (40/47.2) = 0.847

q = 0.84 lbs. per foot, cable weight

x = 40 feet, horizontal distance between cable connections points

p = $(0.84)(40)^2/8(2.375/12)(0.847)$

= 1002 lbs.

The horizontal and vertical components of the preload force are 849 pounds and 531 pounds respectively.

Calculation of elastic and constructional stretch:

Elastic stretch:

20% of breaking strength is

0.2(45,400) = 9080 lbs.

which is less than the cable design force.

$$\Delta = \Delta_1 + \Delta_2$$

$$\Delta_1 = \frac{[0.2(45,400) - 1002](47.2)}{(0.216)0.9(13,000,000)} \qquad \text{(Eq. 5–1)}$$

$$= 0.15 \text{ ft.}$$

$$\Delta_2 = \frac{[11,013 - 0.2(45,400)](47.2 + 0.15)}{(0.216)(13,000,000)} \qquad \text{(Eq. 5–2)}$$

$$= 0.03 \text{ ft.}$$

$\Delta_1 + \Delta_2 = 0.15 + 0.03 = 0.18$ ft.

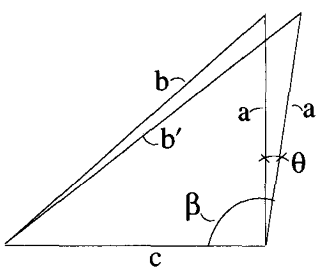
Constructional Stretch:

$$\Delta_{cs} = \left(\frac{11,013}{0.65(45,400)}\right)\frac{0.75}{100}(47.2) \qquad \text{(Eq. 5-3)}$$

$$= 0.13 \text{ ft.}$$

Total elongation = 0.18 + 0.13 = 0.31 ft.

Top of column movement:



$b' = 47.2 + 0.31 = 47.51$ft.

From the law of cosines:

$$\beta = \cos^{-1}\left[\frac{40^2 + 25^2 - 47.51^2}{2(40)25}\right] = 90.9°$$

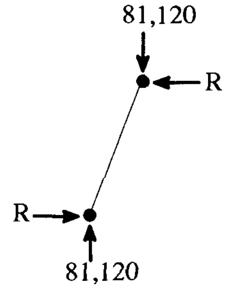
$$\theta = 90.9 - 90 = 0.9°$$

Determine lateral movement of column top:

$\sin\theta\,(a) = \sin 0.9\,(25) = 0.016$ ft.

Determination of force induced by PA:

P = 81,120 lbs. as determined previously.



$$R = \frac{81,120(0.016)}{25}$$

$$= 52 \text{ lbs.}$$

52 (47.51/40) = 62 lbs.

Cable force including $P\Delta$ effects:

11,013+ 62=11,075 lbs.

Cable force: 11,075 lbs.

Allowable cable force = 45,400/3 = 15,133 > 11,075 lbs.

Therefore, use a 3/4" diameter cable.

5.2 Wire Rope Connections

Wire rope connections can be made in a variety of ways. If a projecting plate with a hole in it is provided, then a Spelter Socket, Wedge Socket or Clevis End fitting can be used. Cables are also secured to columns by wrapping the column, either with a section of wire rope to which a hook end turnbuckle is attached or with the end of the diagonal cable itself which is secured by cable clamps. If cables are wrapped around an element, such as a column, a positive mechanism should be provided to prevent the cable from slipping along the column or beam. Also when cables are terminated by wrapping, care should be taken to avoid damage to the wire rope by kinking or crushing. Cables can also be terminated at the column base by attachment to a plate or angle attached to the anchor rods above the base plate. The plate or angle must be designed for the eccentric force induced by the diagonal cable force. Cables are tensioned and adjusted by the use of turnbuckles which can have a variety of ends (round eye, oval eye, hook and jaw). The capacities of turnbuckles and clevises are provided in manufacturer's literature and the AISC Manual of Steel Construction. Cable and rope pullers (come-a-longs) are also used.

5.2.7 *Projecting Plate (Type A)*

The design of a projecting plate from the face of a column is illustrated in the following example. Design strengths for various conditions of cable size, type and angle of cable can be determined from the accompanying tables. The location of the hole can be set at the upper corner. This would allow a reuse after the plate had been flame cut from a column.

Example 5-2

Design a projecting plate attachment (Type A) for the cable force determined in Design Example 5-1.

Design of weld to column:

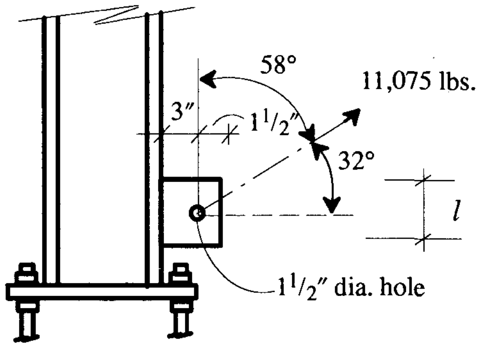


Fig. 5.2.1

Using $^3/_{16}$″ weld fillets along each side of the wing plate, calculate l_{min} per LRFD, 2nd ed. Table 8.38.

$$l_{min} = \frac{P_u}{CC_1D}$$

P_u = 1.3 X 11.1 = 14.4 kips

C_1 = 1.0 for E70XX electrodes

D = 3

C is taken from Table 8.38 with:

k = 0

e_x = al = 3 in.

a = 0.75 (with a trial l = 4 in.)

C = 1.97

l_{min} = 14.4/1.97(1.0)(3) = 2.44 in.

Use 4 inches for l and $^3/_{16}$ in. x 4 in. fillet welds each side of plate.

Design of plate:

Check $^5/_{16}$″ plate.

Component bending the plate (vertical)

P_u = 14.4 (25/47.2) = 7.6 kips

M_u = 7.6 (3) = 22.9 in.–kip

Component tensioning the plate (horizontal)

P_u = 14.4 (40/47.2) = 12.2 kips

Check plate b/t (local buckling):

b/t = 3/.313 = 9.6

$b/t_{max} = 65/(F_y)^{1/2} = 95/(36)^{1/2} = 15.8$
per AISC Table B5.1

Plate is fully effective

Flexure in plate:

ϕM_n = 0.9 $(F_y)Z_x$

F_y = 36 ksi

Z_x = $bh^2/4 = (.313)(4)^2/4 = 1.252$ in.^{3}

ϕM_n = 0.9 (36) (1.252) = 40.5 in.–kip

Tension in plate:

ϕP_n = 0.9 $(F_y)A_g$

= 0.9 (36) (.313) 4 = 40.5 kips

ϕP_u = 0.75 $(F_u)A_e$

= 0.75 (58) (.313) 2.5 = 34.0 kips

Checking interaction:

$$\frac{22.9}{40.5} + \frac{12.2}{34.0} = 0.924 < 1.0$$

Check bearing strength at hole per J3.10 of the Specification.

ϕR_n = $\phi L_e t F_u \le \phi\ 2.4 d t F_u$

where

ϕ = 0.75

L_e = 1.76″, distance from hole centerline to plate edge

t = 5/16″, thickness of plate

F_u = 58 ksi, A36 material

d = 1.5 in. diameter bolt (hole)

ϕR_n = 0.75(1.776)0.3125(58) = 23.9 kips

but not greater than

0.75(2.4)1.5(0.3125)58 = 48.9 kips

Thus

ϕR_n = 23.9 kips

which is greater than the factored cable force of 14.4 kips

Use $^5/_{16}$″ x 4″ plate.

The plate and weld can also be found in Table 22 for the cable type and geometry given.

5.2.2 *Bent Attachment Plate (Type B)*

Another means of attachment of the diagonal cable to the column base is a bent plate on one of the column anchor rods as illustrated in Figure 5.2.2.

The use of this plate requires extra anchor rod length to accommodate it. If the plates are to be left in place, they

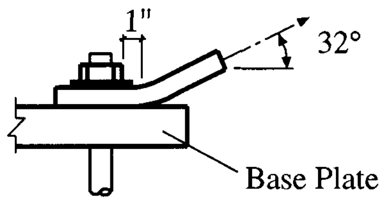


Fig. 5.2.2

must either be in a buried condition or approval must be obtained if exposed. If the plates are to be removed, the nut should not be loosened until this can be safely done, such as when the column and frame are made stable by other means than full development of all the anchor rods.

The design of a bent attachment plate (Type B) for cable attachment is illustrated in the following example. Design strength for various conditions of cable size, type and angle of cable can be read from the accompanying tables.

Example 5-3

Design a bent plate attachment (Type B) for the cable force determined in Design Example 5-1.

Design of bent plate:

Cable force: 11.1 kips at 32° from the horizontal.

As before the force bending the plate is P_u = 7.6 kips (vertical) and the force tensioning the plate is P_U = 12.2 kips.

M_u = 7.6 (e) = 7.6(1) = 7.6 in.-kip

where

e = the distance from the bend to the face of the nut

Check a ½ inch thick plate, 5 inches wide

$\phi M_n = \phi F_y Z_x = 0.9\,(36)\,(0.313) = 10.1$ in.–kip

$\phi = 0.9$

F_y = 36 ksi

$Z_x = (0.5)^2 5/4 = .313$ in.^{3}

$\phi P_n = \phi F_y A_g = 0.9\,(36)\,2.5 = 81.0$ kips

$\phi = 0.9$

Fy = 36 ksi

$A_g = 0.5\,(5) = 2.5$ in.^{2}

Combining flexure and tension:

$$\frac{7.6}{10.1} + \frac{12.2}{81.0} = 0.90 < 1.0 \text{ o.k.}$$

The strength of the plate at the anchor rod hole and cable attachment hole can be determined as in the previous example.

Use plate ½" x 5".

The attachment plate can also be found in Table 24 for the cable type and geometry given.

5.2.3 Anchor Rods

The development of the cable force requires that the anchor rods be adequate to transfer the brace force into the footing and also that the footing be adequate to resist the brace force acting as a deadman. The adequacy of the anchor rods in tension is discussed in Part 4 of this Guide. The anchor rods are also subjected to shear loading. If the base plates are set on pregrouted leveling plates or are grouted when the cable force is applied then the procedures presented in AISC Design Guide 7 "Industrial Buildings" can be used. This method is a shear friction method in which a anchor rod tension is induced by the shear. If leveling nuts (or shims) are used and there is no grout at the time of cable force application, then another procedure must be used. Such a procedure is found in the 1994 edition of the Uniform Building Code (17), in Section 1925. This procedure is an ultimate strength design approach and checks both the anchor rod and the concrete failure modes. The formulas of this method are given in the design example which follows. When leveling nuts (or shims) are used the anchor rods are also subject to bending. In the design example a check for anchor rod bending is made. The calculation takes as the moment arm, one half of the anchor rod height since the base of the anchor rod is embedded in concrete and the top of the anchor rod has nuts above and below the base plate.

Design Example 5-4 illustrates the procedure for evaluating the strength of anchor rods with leveling nuts.

Example 5-4

Check the column anchor rods for the forces induced by the diagonal cable force determined in Design Example 5-1, using a Type A anchor.

Determine the design strength of four-1 inch diameter anchor rods with leveling nuts for resistance to the cable diagonal force.

Grout thickness: 3 in.

Cable diagonal force: 11.1 kips

Vertical component: 11.1 (25/47.2) = 5.9 kips

Horizontal component: 11.1 (40/47.2) = 9.4 kips

Determine net axial load on column:

As determined previously the weight of the frame tributary to one interior column is:

Column: 1(40)25 = 1,000 lbs.

Beams: 2(35)40(0.5) = 1,400 lbs.

Girders: 2(68)40(0.5) = 2,720 lbs.

Roof framing (40)40(5) = 8,000 lbs.
Total = 13,120 lbs. = 13.1 kips

Gravity load: 13.1 kips lbs.

Wind vertical component: 5.9 kips

Net compression on anchor rods: 7.2 kips

Using load factors per the AISC LRFD Specification:

P_u = 0.9D ± 1.3W=0.9 (13.1)-1.3 (5.9) = 4.1 kips (compression)

P_u = 1.2D-1.3W= 1.2 (13.1) -1.3 (5.9) = 8.1 kips (compression)

V_u = 1.3(W) = 1.3 (9.4) = 12.2 kips

Check resistance of (4) 1 in. diameter anchor rods. Grout thickness is 3 in. Anchor rods have heavy hex leveling nuts and 3/8 in. plate washers. Anchors are spaced at 10 in. centers and are embedded 12 in.

Anchor rods: ASTM A36

Concrete: f'_c = 3500 psi

Force to each anchor rod:

Axial: 8.1 ÷ 4 = 2.0 kips (compression)

Shear: 12.2 ÷ 4 = 3.1 kips

Using procedure from Section 4.2.4 for axial load:

k = 1.0

A_b = 0.7854in.^{2}

ℓ = 3-(0.375+1)= 1.625 in.

r = 0.25 (d) = 0.25(1) = 0.25 in.

kL/r = 1(1.625)70.25 = 6.5

$\phi_c F_{cr}$ = 30.53 ksi per LRFD Table 3-36

ϕP_n = $\phi_c F_{cr}(A_b)$ = (30.53) 0.7854 = 24.0 kips

Bending:

Moment arm = 0.5 (3 - (0.375 + 1)) = 0.81 in.

M_u = 3.1 (0.81) = 2478 in.-lb. = 2.5 in.-kip

ϕM_n = $\phi F_y Z_x$ = 0.9 (36) 0.167 = 5.4 in.-kip

where

Z_x = $d^3/6 = (1)^3/6$ = 0.167 in.^{3}

F_y = 36 ksi

ϕ = 0.9

Using LRFD *Eq.* H1-16($P_u/\phi P_n < 0.2$)

$$\frac{2.0}{2(24.0)} + \frac{2.5}{5.4} = 0.50 < 1.0 \text{ o.k.}$$

It should be noted that the anchor rods must be adequately developed to resist a punch through failure per Section 4.2.5.

Design strength in shear using the procedure and notation in UBC-94:

$V_{ss} = 0.75\ A_b f'_s$

ϕV_c = $\phi(800)A_b\lambda(f'_c)^{1/2}$

V_{ss} = 0.75(0.785)58 = 34.1 kips

ϕV_c = $0.85(800)(0.785)(1)(3500)^{1/2}$ (1/1000) =31.5 kips

V_u = 3.1 kips

3.1 <31.5 o.k.

In this example the loads, load factors and load combinations resulted in a net compressive force on the anchor rods. To illustrate the calculation procedure, using a net tension force the example continues using a P_u = 8.1 kips tension. All other design parameters remain unchanged.

Force to each anchor rod:

Axial: 8.1 ÷ 4 = 2.0 kips (tension)

Shear: 12.2 ÷ 4 = 3.1 kips

Using the procedure and notation in UBC-94

Design strength in tension:

ϕP_{ss} = $0.9\ (A_b)\ f'_s$

ϕP_c = $\phi\lambda(2.8A_s + 4A_t)\sqrt{f'_c}$

where

A_b = $\pi\ (.5)^2 = 0.785$ in.^{2}

f'_s = F_u = 58,000 psi = 58 kips

ϕP_{ss} = 0.9 (0.785) 58 = 40.9 kips

ϕ = 0.85

λ = 1.0 for normal weight concrete

($2.8\ A_s + 4A_t$) represents the surface of a truncated failure surface cone as presented elsewhere in this guide as:

A_e = $\pi\ (L_d + c/2)^2 + 4(L_d + c/s)\ (s+c) - \pi\ (c)^2$

where

L_d = the embedment depth, in.

c = 1.7 (rod diameter)

s = spacing, in.

$A_e = \pi\,(12+1.7/2)^2+4(12+1.7/2)(10+1.7)-\pi\,(1.7)^2$

$= 706.5 \text{ in.}^2$

$\phi P_c = 0.85\,(1)\,706.5\,(4)\,(3500)^{1/2}\,(1/1000)$

$\phi P_c = 142.1$ kips

$142.1 \div 4 = 35.5$ kips per rod

Design strength in shear:

$V_{ss} = 0.75\,A_b f'_s$

$\phi V_c = \phi 800\,A_b\,\lambda\,(f'_c)^{1/2}$

$V_{ss} = 0.75(0.785)58 = 34.1$ kips

$\phi V_c = 0.85\,(800)\,(0.785)\,(1)\,(3500)^{1/2}\,(1/1000)$
$= 31.5$ kips

Combining tension and shear per UBC-94, para. 1925.3.4

$$\left(\frac{2.0}{35.5}\right)^2 + \left(\frac{3.1}{31.5}\right)^2 = 0.013 < 1.0 \text{ o.k.}$$

This establishes the resistance based on the anchor rod strength and concrete strength at the level of the concrete. The rods must also be checked in bending.

Rod in bending and tension.

Moment arm = 0.5(3-1-0.375) = 0.81 in.

$M_{rod} = 3050 \times 0.81$ in. = 2478 in.-lb.

= 2.5 in.-kip

$\phi M_n = \phi F_y Z_x = 0.9(36)0.167 = 5.4$ in.-kip

where

$Z_x = d^3/6 = (1)^3/6 = 0.167 \text{ in.}^3$

F_y = 36,000 psi = 36 ksi

ϕ = 0.9

Axial tension is as calculated above.

Combining bending and tension per AISC:

$$\frac{2.5}{5.4} + \frac{2.0}{40.9} = 0.51 < 1.0 \text{ o.k.}$$

This result can also be found in Table 23 where an allowable cable force of 18,114 pounds is given for this geometry, anchor rod and grout combination. This value exceeds the actual cable force of 11,075 pounds.

Example 5-5

Check the column anchor rods for the forces induced by the diagonal cable force determined in Design Example 5-1, using a bent plate Type B attachment.

This check is the same as that of Example 5-4 except that the vertical force component is carried by only the anchor rod to which the bent plate anchor is secured. The design for bending and shear is the same.

Axial force: 8.1 kips (one anchor rod only.)

Using the procedure in UBC-94 and section 4.2.5. of this guide.

Design strength in tension.

P_{ss} = 40.9 kips as before

$\phi P_c = \phi\,\lambda\,(A_e)\,(f'_c)^{1/2}$

where

$\phi = 0.85$

$\lambda = 1.0$

$$A_e = \pi\left(L_d + \frac{c}{2}\right)^2 - \pi\left(\frac{c}{2}\right)^2$$

where

L_d = the embedment depth, in.

c = 1.7 (rod diameter)

$A_e = \pi(12+1.7/2)^2 - \pi\,(1.7/2)^2$

$= 516.5 \text{ in.}^2$

$\phi P_c = 0.85\,(1)\,516.5\,(4)\,(3500)^{1/2}\,(1/1000)$

$\phi P_c = 103.9$ kips

In this case the rod strength governs. The shear strength is as in Example 5-4 and thus the interaction per UBC-94 is as follows:

$$\left(\frac{8.1}{40.9}\right)^2 + \left(\frac{3.1}{31.5}\right)^2 = 0.049 < 1.0 \text{ o.k.}$$

Checking the rod in bending and tension, the bending is as before. The tension is carried by only one rod.

P_u = 8.1 kips

ϕP_n = 40.9 kips, as before

M_u = 2.5 in.-kips, as before

ϕM_n = 5.4 in.-kips, as before

Combining bending and tension per AISC:

$$\frac{2.5}{5.4} + \frac{8.1}{40.9} = 0.66 < 1.0 \text{ o.k.}$$

This result can also be found in Table 25 where an allowable cable force of 13,471 pounds is given for this geometry, anchor rod and grout combination. This value exceeds the actual cable force of 11,075 pounds.

The footing must also be evaluated to determine its resistance to the cable diagonal force. In this situation the footing can be evaluated using the procedure developed for deadmen, which follows.

5.3 Design of Deadmen

On occasion the erector must anchor cable bracing to a "deadman". A deadman may be constructed on top of the ground, near the ground surface, or at any depth within the soil. They may be short in length or continuous.

5.3.1 Surface Deadmen

The simplest form of a deadman is a mass of dead weight sitting on top of the ground surface. A block of concrete is generally used. The anchor resistance provided by such a deadman is dependent upon the angle that the bracing cable makes with the deadman and the location of the bracing cable attachment relative to the center of gravity of the deadman. As the angle of the bracing from the horizontal becomes greater, the resistance of the deadman to horizontal sliding reduces.

The resistance to sliding equals the total weight of the deadman less the upward force from the bracing cable, times the coefficient of friction between the deadman and the soil. A coefficient of friction of 0.5 is generally used. In equation format:

$$R_n = 0.5\,(W_d - P\sin\theta) \qquad \text{Eq.5-6}$$

where

R_n = the nominal horizontal resistance of the deadman

W_d = the weight of the deadman, lbs.

P = the required brace force, lbs.

0.5 = the coefficient of friction

Using a factor of safety of 1.5 for sliding the allowable resistance is thus:

$$R_{all} = 0.33\,(W_d - P\sin\theta) \qquad \text{Eq. 5-7}$$

In addition to satisfying Eq. 5-7 the overturning resistance of the deadman must be checked. This can be accomplished by taking moments about the top of the deadman. A factor of safety of 1.5 is commonly used for overturning.

5.3.2 Short Deadmen Near Ground Surface

On occasion a deadman may also be buried into the soil. The deadman must be designed to resist the vertical and horizontal force exerted by the bracing system. The vertical force is resisted by the weight of the deadman. The required weight equals:

$$W_d = 1.5\,(P\sin\theta) \qquad \text{Eq. 5-8}$$

where

W_d = the weight of the deadman, lbs.

P = the bracing force, lbs.

θ = the angle measured from the horizontal of the bracing cable, degrees

1.5 = the factor of safety used for uplift

The horizontal resistance varies depending upon the soil condition at the site.

Granular Soils

Based on soil mechanics principles the total resistance to sliding can be expressed as:

$$T_n = L(P_p - P_a) + 1/3\, K_o\, \gamma\, (\sqrt{K_p} + \sqrt{K_a})H^3 \tan\phi \qquad \text{Eq. 5-9}$$

where

T_n = the total nominal horizontal resistance, lbs.

L = length of the deadman, perpendicular to the force, ft.

P_p = total passive earth pressure, lbs. per lineal ft.

P_a = total active earth pressure, lbs. per lineal ft.

K_o = coefficient of earth pressure at rest

γ = unit density of the soil, pcf

K_p = coefficient of passive earth pressure

K_a = coefficient of active earth pressure

H = depth of the deadman in soil, ft.

ϕ = angle of internal friction for the soil, degrees

The following values may be used except in unusual situations:

$(P_p - P_a) = \gamma\ (2.67)H^2 = 320H^2$

K_o = 0.4

γ = 120 pcf

K_p = 3.0

K_a = 0.33

$\tan \phi = 0.6$

Thus,

$$T_n = 320LH^2 + 22H^3, \text{ lbs.} \qquad \text{Eq. 5-10}$$

Using a factor of safety of 1.5,

$$T_{all} = 213LH^2 + 15H^3 \qquad \text{Eq. 5-11}$$

where

T_{all} = the allowable resisting force.

Cohesive Soils

For cohesive soils the ultimate horizontal resistance provided by the deadman can be calculated from the following equation:

$$T_n = L(P_p - P_a) + q_u H^2 \qquad \text{Eq. 5-12}$$

where

L = the length of the deadman, ft.

P_p = total passive earth pressure, lbs. per lineal ft.

P_a = total active earth pressure, lbs. per lineal ft.

q_u = the unconfined compression strength of the soil, psf

H = depth of the deadman, ft.

The following values may be used in this equation:

q_u = 1500 psf (usually conservative)

$(P_p - P_a) = 2q_u H = 3000 H$

Thus,

$$T_n = 3000LH + 1500H^2 \qquad \text{Eq. 5-13}$$

Using a factor of safety of 1.5,

$$T_{all} = 2000LH + 1000H^2 \qquad \text{Eq. 5-14}$$

Example 5-6

Check footing as surface deadman.

Footing: 6'-0" x 6'-0" x l'-6"

Soil: Granular type

Calculate weight of footing:

W_d = 6 x 6 x 1.50 x 0.150 = 8.1 kips

Calculate weight of frame

Column: 25(40) = 1,000 lbs.

Beams: 40(35) = 1,400 lbs.

Girders: 40(68) = 2,720 lbs.

Framing: 40(40)5 = <u>8.000 lbs.</u>

Total 13,120 lbs. = 13.1 kips

$$R_n = 0.5\,(W_d - P \sin \theta) \qquad \text{(Eq. 5-6)}$$

W_d = 8.1 + 13.1=21.2 kips

From Example 5-1

P = 11.1 kips

θ = 32°

R_n = 0.5 (21.2 -(11.1 (sin 32°)) = 7.7 kips

Using a factor of safety of 1.5,

R_{all} = 0.67(R_n) = 0.67(7.7) = 5.1 kips

$P(\cos\theta)$ = 11.1 (cos 32°) = 9.4 kips

5.1 < 9.4 n.g.

Check footing as deadman in ground:

$$T_{all} = 213LH^2 + 15H^3 \qquad \text{(Eq.5-11)}$$

L = length of deadman, ft.

H = depth of deadman, ft.

T_{all} = 213 (6) 1.5^2 + 15 $(1.5)^3$ = 2909 lbs. - 2.9 kips

A thicker footing is required

$T_{req'd}$ = 9.4 kips

Solving for H

9400 = $213(6)x^2 + 15(x)^3$

x = 2.68ft.

Try a footing: 6'-0" x 6'-0" x 2'-9"

Check overturning. The anchor is attached to the footing top at the center of the footing:

Overturning moment:

(11.1 sin 32°)(3) + (11.1 cos 32°)(2.75) = 43.5 ft.-kips

Resisting moment:

(6)(6)(2.75)(0.150)(3) + 13.1(3) = 83.8 ft.-kips

Factor of Safety = 89.2/46.6 = 1.9 > 1.5 o.k.

In the foregoing example the size of the footing required to resist the diagonal cable force was substantially larger than would be common in the building described elsewhere in the examples. The example indicates that the footing resistance may often be the limiting factor. The schedule of a construction project may not allow redesign and rebidding to account for changes due to the erection bracing. In this event the footing and foundations must be taken as a limiting constraint to the erection bracing design. This condition will result in an increase in the number of diagonal bracing cables required.

PART 2

DETERMINATION OF BRACING REQUIREMENTS USING PRESCRIPTIVE REQUIREMENTS

6. INTRODUCTION TO PART 2

Part 2 presents a series of prescriptive requirements which if followed eliminates the need to use the calculation methods, thus simplifying the determination of the temporary bracing required. The prescriptive requirements are:

1. Requirements relating to the permanent construction, such as bay size, frame layout, anchor rod characteristics and foundation characteristics.
2. Requirements relating to the temporary bracing requirements and minimum requirements for the sequence of erection and installation of temporary bracing.

These prescriptive requirements are grouped by exposure category and by size. An illustrative example of an erection plan incorporating the prescriptive requirements is also presented.

7. PRESCRIPTIVE REQUIREMENTS

7.1 Prescriptive Requirements for the Permanent Construction

Tables 7.1 through 7.24 present prescriptive requirements which limit features of the permanent construction. The features which are critical are:

1. Bay size.
2. Column height.
3. Column size.
4. Base plate thickness.
5. Pier size.
6. Footing size.
7. Column setting type.
8. Anchor rod diameter.
9. Anchor rod pattern.
10. Anchor rod termination, hooked or nutted.
11. Anchor rod embedment.
12. Anchor rod cover below bottom end.

Three bay sizes are presented: 30-foot, 40-foot and 50-foot. The column heights presented are: 15-foot, 30-foot and 45-foot. Two types of settings are presented. The first type loads the anchor rods in compression. This type of base uses leveling nuts. The second type are those bases which do not transmit compression forces to the anchor rods, namely, pre-grouted setting plates, shims and anchor rods with an additional nut installed just below the top surface of the concrete, as illustrated in Figure 4.17.

If the conditions upon which these tables are based are present in the construction and the erector follows the requirements for erection sequence and cable bracing, then no separate analysis for the determination of temporary supports is required. Both single story and two story structures are addressed in the tables.

The tables are based on the following parameters:

1. Both wind exposure categories B and C are tabulated. The exposure category used is to be that for which the structure is designed.
2. The design wind pressures are those associated with an 80 mph basic wind speed. The tables are not be valid for greater speeds. The design wind speed has been reduced for a six week (or less) exposure duration as described in paragraph 3.2.1 of the text. Also a design wind speed of 35 mph has been used for elements which are exposed to the wind for a period of no more than twenty-four hours. This includes individual columns supported on their bases and individual beam/column pairs prior to the installation of tie members. A single row of beams and columns supported only by their bases would not meet the limitations of these tables. In the case of a two story column both the upper and lower beams may be erected following the limitations cited above for beam/column pairs.
3. In calculating wind forces on frames, 24 inch deep solid web members and 48 inch deep open web members were used. Member depths on the frame lines exceeding these maximums would invalidate the prescriptive requirements. Also, 12 inch deep columns were used. Greater depth columns would not be valid.
4. With regard to the footings and piers the following parameters are used. The concrete strength is 3000 psi. This strength is the 28-day cylinder strength which may be achieved in less than 28 days, but must be confirmed by test. The area of reinforcement in the piers must be at least one half of one percent of the area of the concrete pier. The factor of safety against overturning and sliding used is 1.5. In the determination of uplift and over-

turning resistance, a dead load equal to 4 psf over the column tributary area plus the footing weight is used.

5. The strength of the column to base plate weld is based on a fillet weld size of 5/16 inch. The weld must be made to both sides of each flange and each side of the web. Lesser weld sizes and/or extents would require calculations as presented in Part 1.

6. In several cases, hooked anchor rods may be used per the tables. It is permissible in these cases to substitute a headed anchor rod with the same embedment.

7. In the determination of column base moment strength for columns with setting plates, a moment arm equal to one half the bolt spacing plus one half the column flange width is used.

8. In the determination of the diagonal cable force to be resisted, the degree of base fixity provided by the column bases is considered. This has the effect of reducing the required cable force to be developed.

9. The tables require the placement of opposing pair diagonal cable braces in each frame line in both orthogonal directions. These braces must be placed in every fourth bay along the frame lines in Exposure B conditions and in every third bay in Exposure C conditions.

10. The diagonal cable brace required for the one story frames presented is a 1/2 inch diameter wire rope with a minimum nominal breaking strength of 21,000 pounds. For the two story frames, a 5/8 inch diameter wire rope with a minimum nominal breaking strength of 30,000 pounds is required.

11. The wire rope diagonals can be anchored to the columns with Type A or Type B anchors as illustrated in Figures 5.2.1 and 5.2.2.

    Anchor required for one story frames:

    Type A:
    Plate thickness = 3/8 in.
    L = 3 in.
    Weld = 3/16 fillets
    Grout thickness = 3 in., maximum

    TypeB:
    Plate thickness - 5/8 in.
    B = 4 in.
    Grout thickness = 2 in., maximum for 3/4 in. diameter anchor rods and 3 in., maximum for diameters greater than 3/4 in.

    Anchor required for two story frames:

    Type A:
    Plate thickness = 5/16 in.
    L = 4 in.
    Weld = 3/16 in, fillets
    Grout thickness = 2 in., maximum for 3/4 in. diameter anchor rods and 3 in., maximum for diameters greater than 3/4 in.

    TypeB:
    Plate thickness = 1/2 in.
    B = 5 in.
    Grout thickness = 2 in., maximum for 3/4 in. diameter anchor rods and 3 in., maximum for diameters greater than 3/4 in.

    Termination of wire rope can be made by wrapping, if the limitations presented in paragraph 5.2 are followed.

7.2 Prescriptive Requirements for Erection Sequence and Diagonal Bracing

In addition to the prescriptive requirements for the permanent structure, there are prescriptive requirements for erection sequence and diagonal bracing.

Figure 7.1 illustrates an erection plan with diagonal bracing in specific bays. It also identifies an initial box from which the erection is to commence. Figures 7.2 through 7.5 illustrate the build out from the initial box. The pattern of column, girder, column, girder, tie beam, x-brace is to be repeated as the erection proceeds. This limitation on sequence is established to restrict the surface of frame exposed to wind when that portion of the frame is supported solely by the anchor bolts. The sequence given above limits the exposure to one column and one-half of one beam. In a two story frame, the exposure is limited to one column and one -half each of the upper and lower beams. The number of braced bays, the size and strength of wire rope to be used and the anchorage required for this wire rope are given in Section 7.1

The erection plan in Figure 7.1 illustrates columns, girders, tie members and temporary x-braces. This plan is divided into four erection sequences. Figure 7.1 contains features which are solely illustrative and others which are prescriptive.

The illustrative features are:

1. Proportion of bay: A square bay is shown and is required for use of the Tables. The dimension of the bays are the 30-foot, 40-foot, and 50-foot bays as presented in Tables 7.1 through 7.24. Rectangular bays induce a different set of loads, cable forces and angles and the prescriptive requirements are not valid. If the structure to be erected has rectangular bays, the calculation method must be used.

2. Number of bays: An arrangement of five bays by seven bays is shown. The number of bays in each direction is not limited.

3. Columns: A wide flange column is shown. Pipe and tube columns may also be used.

4. Column orientation: Any arrangement of column orientations is permitted.

5. Erection sequences: Four (I to IV) erection sequences are illustrated. The number and pattern of erection sequences is not limited.

6. Starting point of erection: Erection begins at the "initial box" in the upper left hand corner of the plan. The location of the starting point is not limited; however, at the starting point an initial box must be formed.

7. Progression from the initial box: The plan and the supplementary figures illustrate a progression from the initial box. This progression follows this sequence: bay 1-2, B-C, bay 1-2, C-D, bay 2-3, A-B, etc. The progression from the initial box can follow any order however it must follow a bay by bay development in which beam/column pairs are erected followed by the erection of the tie members followed by the installation of the temporary x-brace. This is illustrated in Figure 7.3, which shows an x-brace installed between columns C/l and C/2 before the erection proceeds to grid line D.

8. Location of x-braces: The plan shows x-braces in the exterior bay 1-2. It is not required that x-braces be located in exterior bays unless it is necessary to meet the prescriptive requirements. X-braces must be located per the prescriptive requirements, namely every third or fourth bay depending on the exposure category, on each frame line, on all four sides of the initial box and in the bays which proceed outward from the initial box (see Figures 7.2-7.5).

9. Use of x-braces: Each opposing cable pair is shown as an x-brace. The opposing cable pairs do not necessarily need to be installed as an "x" except when a single bay is to be braced such as the four sides of the initial box and the bays framed out from the initial box (see Figures 7.2 and 7.3).

10. Use of temporary bracing: Figures 7.1 through 7.5 show the use of only temporary bracing. Permanent bracing may be used; however, this requires evaluation by the calculation method (Part 1) to properly determine the interaction of permanent and temporary bracing.

Lastly, temporary bracing must remain in place until its removal is permitted as provided for in the AISC Code of Standard Practice.

Exposure Category	B
Bay Size, ft.	30
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.0X4.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.1 Prescriptive Requirements for Exposure B, 30 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	30
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.5X4.5X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.2 Prescriptive Requirements for Exposure B, 30 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	30
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.5X5.5X13
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	5X5
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	5X5
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.3 Prescriptive Requirements for Exposure B, 30 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	40
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.0X4.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.4 Prescriptive Requirements for Exposure B, 40 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	40
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.5 Prescriptive Requirements for Exposure B, 40 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	40
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.5X5.5X17
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	5X5
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	5X5
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.6 Prescriptive Requirements for Exposure B, 40 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	50
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.0X4.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.7 Prescriptive Requirements for Exposure B, 50 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	50
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	18X18
Footing Size, ft. x ft. x in.	5.0X5.0X13
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.8 Prescriptive Requirements for Exposure B, 50 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	50
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	22X22
Footing Size, ft. x ft. x in.	5.5X5.5X17
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.9 Prescriptive Requirements for Exposure B, 50 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	30
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.0X4.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.10 Prescriptive Requirements for Exposure C, 30 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	30
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.11 Prescriptive Requirements for Exposure C, 30 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	30
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	6.0X6.0X15
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.12 Prescriptive Requirements for Exposure C, 30 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	40
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.0X4.0X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.13 Prescriptive Requirements for Exposure C, 40 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	40
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.5X5.5X13
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	3 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.14 Prescriptive Requirements for Exposure C, 40 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	40
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.25
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	6.0X6.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	3 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	3 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.15 Prescriptive Requirements for Exposure C, 40 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	50
Column Height, ft.	15
Stories	1
Column Size	W8X24
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	4.5X4.5X12
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.16 Prescriptive Requirements for Exposure C, 50 ft. Bays, 15 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	50
Column Height, ft.	30
Stories	1
Column Size	W8X31
Base Plate, Thickness, in.	0.875
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.5X5.5X17
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.17 Prescriptive Requirements for Exposure C, 50 ft. Bays, 30 ft. Column Height, One Story Frame

Exposure Category	C
Bay Size, ft.	50
Column Height, ft.	45
Stories	1
Column Size	W12X65
Base Plate, Thickness, in.	1.25
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	6.5X6.5X16
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	15X15
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.18 Prescriptive Requirements for Exposure C, 50 ft. Bays, 45 ft. Column Height, One Story Frame

Exposure Category	B
Bay Size, ft.	30
Column Height, ft.	20
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.19 Prescriptive Requirements for Exposure B, 30 ft. Bays, 20 ft. Column Height, Two Story Frame

Exposure Category	B
Bay Size, ft.	40
Column Height, ft.	30
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.20 Prescriptive Requirements for Exposure B, 40 ft. Bays, 30 ft. Column Height, Two Story Frame

Exposure Category	B
Bay Size, ft.	50
Column Height, ft.	30
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.21 Prescriptive Requirements for Exposure B, 50 ft. Bays, 30 ft. Column Height, Two Story Frame

Exposure Category	C
Bay Size, ft.	30
Column Height, ft.	30
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	0.75
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	5 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.22 Prescriptive Requirements for Exposure C, 30 ft. Bays, 30 ft. Column Height, Two Story Frame

Exposure Category	C
Bay Size, ft.	40
Column Height, ft.	30
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	5.0X5.0X18
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.75
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	3 in. Hook
Embedment, in.	6
Cover Below Anchor, in.	6
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	1.0
Anchor Pattern, in. x in.	4X4
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.23 Prescriptive Requirements for Exposure C, 40 ft. Bays, 30 ft. Column Height, Two Story Frame

Exposure Category	C
Bay Size, ft.	50
Column Height, ft.	30
Stories	2
Column Size	W8X31
Base Plate, Thickness, in.	1.0
Pier Size, in. x in.	12X12
Footing Size, ft. x ft. x in.	6.0X6.0X14
Anchor Rods with Leveling Nuts	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	9
Anchor Rods, Base Plate Shimmed or Grouted	
Anchor Rod, Diameter, in.	0.875
Anchor Pattern, in. x in.	11X11
Hooked or Nutted	4 in. Hook
Embedment, in.	9
Cover Below Anchor, in.	3

Note: Footing thickness given is a minimum which must be increased to match embedment plus cover in some cases.

Note: Pier size given is the minimum size required for strength. A larger pier may be required to match the column provided.

Note: The anchor rod parameters given are minimums.

Table 7.24 Prescriptive Requirements for Exposure C, 50 ft. Bays, 30 ft. Column Height, Two Story Frame

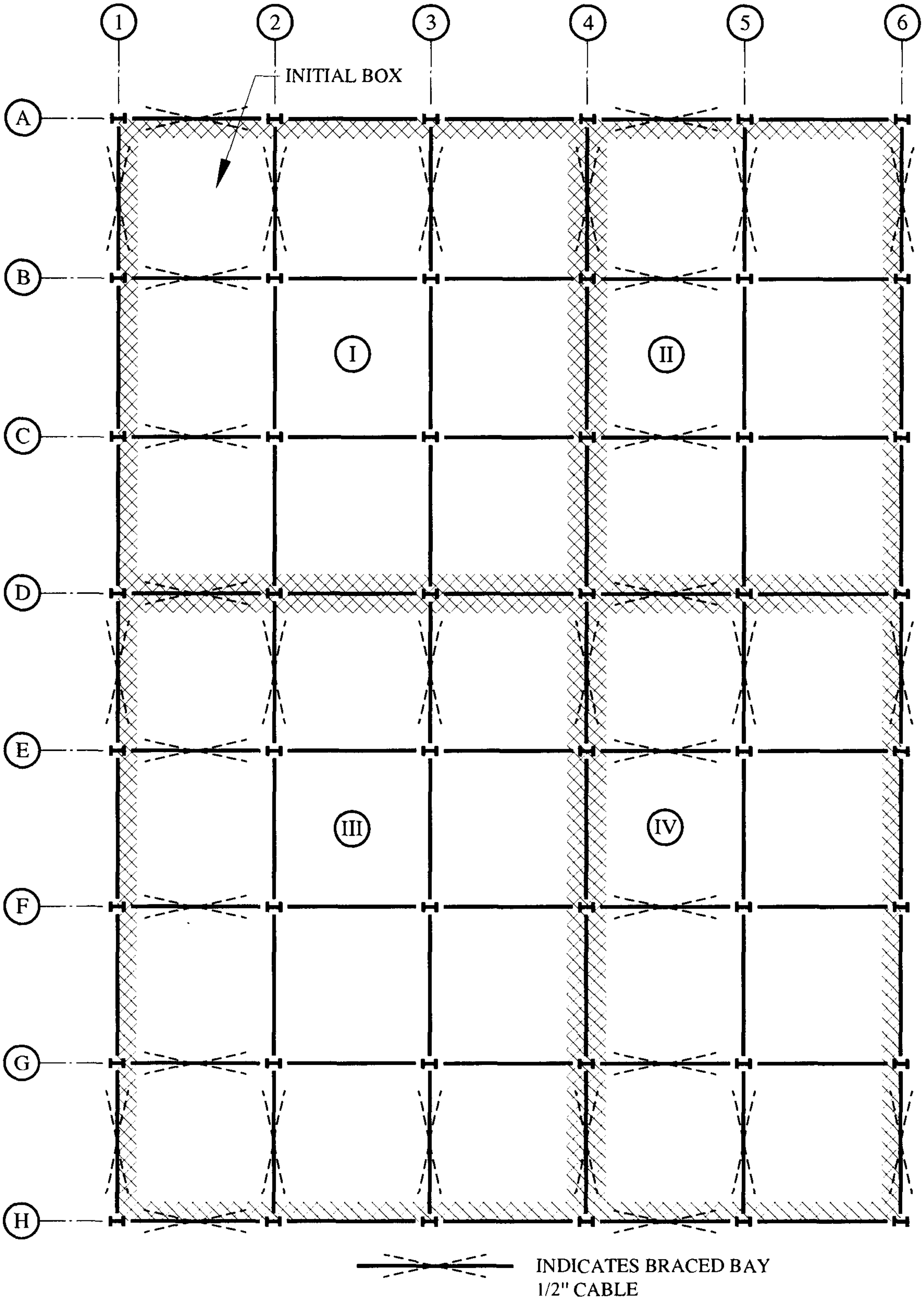


Fig. 7.1 Erection Plan

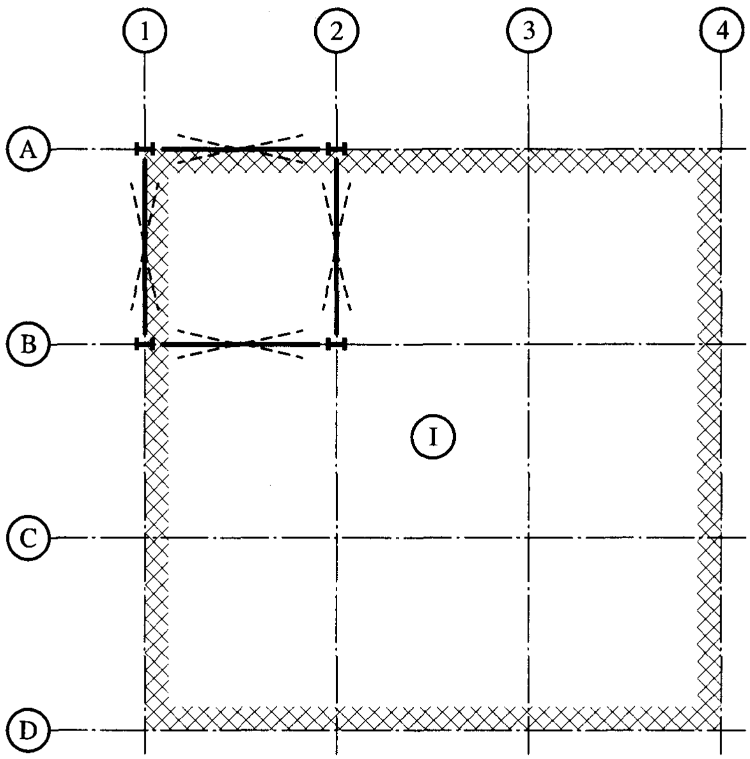


Fig. 7.2 Initial Braced Box

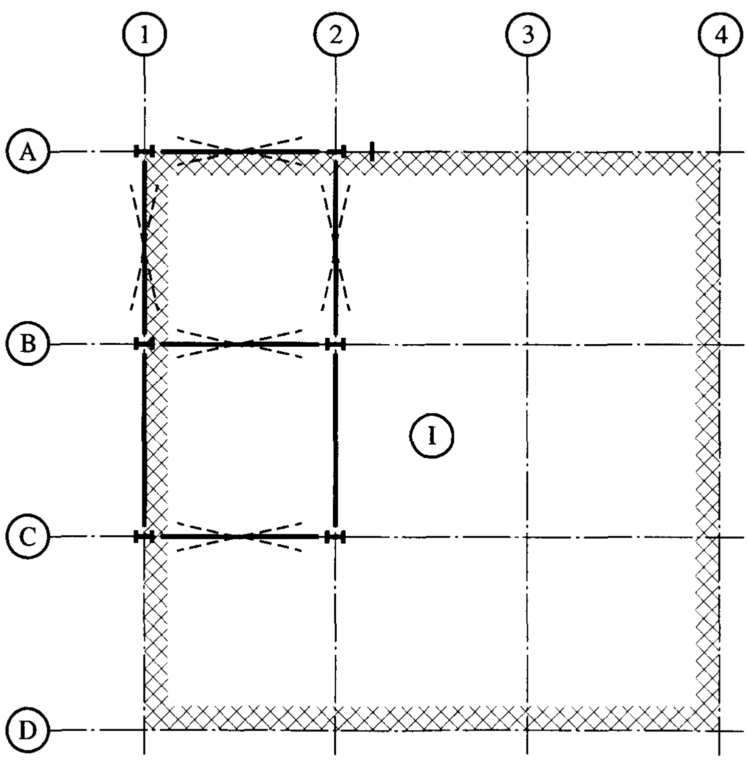


Fig. 7.3 Build Out from Initial Box

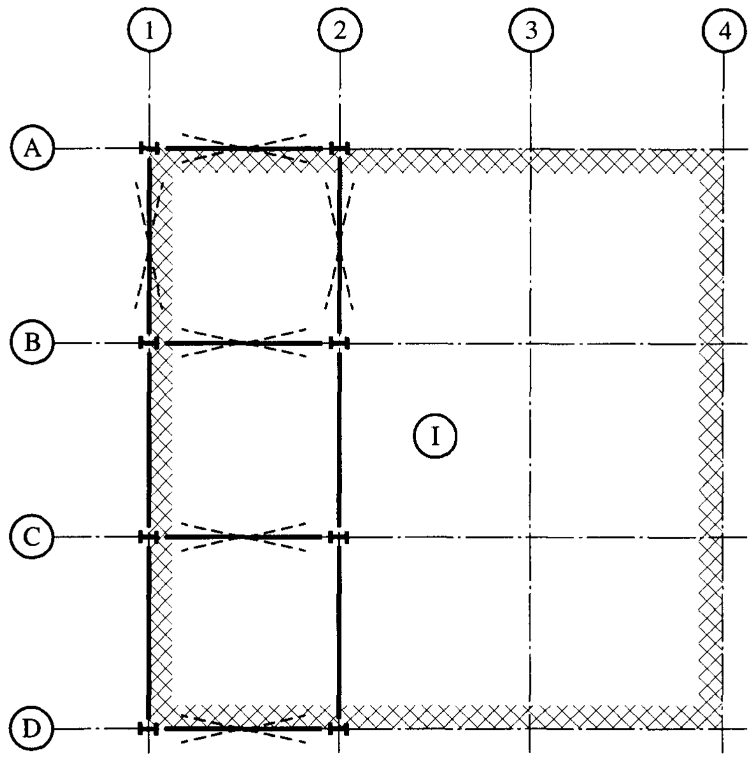


Fig. 7.4 Build Out from Initial Box, Continued

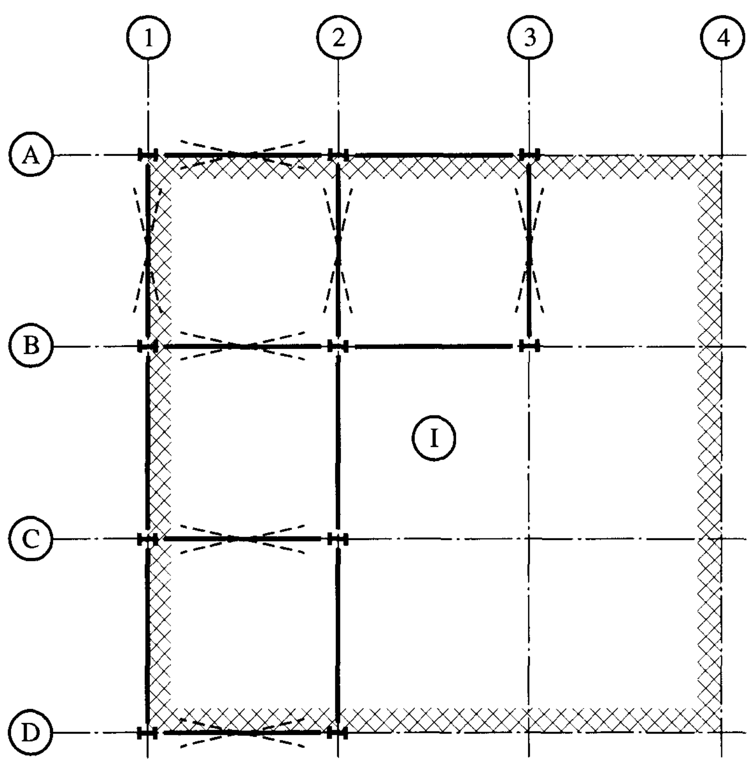


Fig. 7.5 Build Out from Initial Box, Continued

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APPENDIX

Annotated Table of Contents

Table A-l Moment Resistance of Base Plates with Inset Anchor Rods Based on Weld Strength

For the column sizes indicated the weak axis moment resistance for the column to base plate connection is provided. The failure mode is shown in Figure 4-1. The design strengths are based on Eqs. 4-1 and 4-2 using a 5/16 in. weld pattern as shown in Figure 4-21.

Table A-2 Moment Resistance of Base Plates with Inset Anchor Rods Based on Plate Strength

Design strengths are provided for the parameters shown in the table, based on Eqs. 4—3 and 4—4.

Table A-3 Moment Resistance of Base Plates with Outset Anchor Rods

For the column sizes indicated the design strengths for the column to base plate connection is provided for the condition where the anchor rods are outside the footprint of the column. Due to the configuration of the anchor rods, the design strengths are applicable to loads applied about either axis of the column. The design strength is based on Eq. 4-6 using 5/16 inch fillet weld two inches long, and the anchor rod offset from the flange tip by 2 in. in each direction.

Table A-4 Moment Resistance of Tube Column Base Plates

For the column sizes and anchor rod spacings shown the design strength of column base plate is provided. The tables are based on 4 in. long 5/16 inch welds and E70 electrodes and the anchor rod offset from the flange tip by 2 in. in each direction. The columns are assumed to be welded all around. The design strengths are based on Eq. 4-9.

Table A-5 Tension Resistance of Anchor Rods Based on Anchor Rod Strength

Provided in Table A-5 are the tension design strengths for the A36 anchor rods. The values provided in the Table are taken directly from the AISC Manual of Steel Construction. The failure mode associated with the Table values is that of anchor rod fracture as shown in Figure 4-3.

Table A-6 Tension Resistance of Hooked Anchor Rods Based on Hook Length, f'_c = 3000 psi

The values provided in Table A-6 are derived from Eq. 4-14. The values are somewhat conservative in that no allowance for strength is provided for any bond between the anchor rod and the concrete. It is the authors opinion that inclusion of the bond strength can be unconservative since anchor rods are often oily after the threads are cut. The values provided in Table A-6 are based on a concrete strength of 3000 psi.

Table A-7 Tension Resistance of Hooked Anchor Rods Based on Hook Length, f'_c = 4000 psi

Table A-7 is identical to Table A-6 except that a concrete strength of 4000 psi is used to determine the provided values.

Table A-8 Tension Resistance of Single Anchor Rods Based on Concrete Pull Out Capacity

Presented in Table A-8 are pull out resistance for the anchor rod sizes and embedment depths shown. The concrete strength used for the calculations is 3000 psi. The values are for single anchor rods, i.e. no group action. The values are based on the pull out strength of the concrete cone, which equals $4\phi\sqrt{f'c}$ times the projected area, A_e, of the cone at the surface of the concrete.

Tables A-9, A-10, A-ll and A-12 Tension Resistance of Two Anchor Rods Based on Spacing and Embedment

Pull out resistance for a group of two anchor rods are presented. The Tables are based on a concrete strength of 3000 psi, and embedment depths equal to 9, 12, 15 and 18 inches. Eq. 4—13 is used to determine the values.

Table A-13 Compression Resistance of Single Anchor Rods Based on Concrete Push Out

Push out values for single anchor rods are presented in Table A-13. Eq. 4-16 and the failure cone area shown in Figure 18 are used to calculate the table values. A concrete strength of 3000 psi is used. The table values can be used for both hooked rods and nutted rods.

Table A-14 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

Push out values for a group of two anchor rods are provided. Eq. 4-16 is used to determine the values shown. A concrete strength of 3000 psi is used. A clear cover of 3 inches under the nut or hook of the anchor rod is used to determine the push through values shown.

Tables A-15, A-16 and A-17 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

Tables A-15, A-16 and A-17 are identical to Table A-14 with the exception that clear covers of 6, 9 and 12 inches are used respectively.

Table A-18 Concrete Pier Bending Resistance

Bending design strengths are provided for the data shown in the Table. Eq. 4-17 is used with a concrete strength of 3000 psi to determine the listed values.

Table A-19 Concrete Footing Overturning Resistance

Overturning resistances are provided for the footing sizes shown in the Table. The values are based on Eq. 4-21. Only the dead weight of the footing is used in determining the values.

Table A-20 Reinforcing Bar Development Lengths, f'_c = 3000 psi

The required development length for hooked and straight reinforcing bars are shown in Table A-20. Eqs. 18,19 and 20 with a concrete strength of 3000 psi are used to determine the development lengths.

Table A-21 Reinforcing Bar Development Lengths, f'_c = 4000 psi

Table A-21 is identical to Table A-20 with the exception that a f_c of 4000 psi is used in the calculations.

Table A-22 Dimensions of Type A Anchor Plates and Welds

This table provides plate height, thickness and fillet weld size for an A36 plate Type A, for the cable type and slopes presented. A plate of this geometry and attachment will develop the cable design force using a minimum factor of safety of 3 in selecting the cable. The Type A plate is shown in Figure 5.2.1. The table data was determined using the calculation method in Example 5-2.

Table A-23 Allowable Cable Force, Type A Plate Anchor as Limited by Anchor Rod Capacity

This table provides the maximum Unfactored cable force for the parameters presented based on the calculation method and material strengths in Example 5-4.

Table A-24 Dimensions of Type B Anchor Plates and Welds

This table provides the plate width and thickness for an A36 plate Type B, for the cable types and slopes presented. A plate of this geometry will develop the cable design force using a minimum factor of safety of 3 in selecting the cable. The Type B plate is shown in Figure 5.2.2. The table data was determined using the calculation method in Example 5-3.

Table A-25 Allowable Cable Force, Type B Plate Anchor as Limited by Anchor Rod Capacity

This table provides the maximum Unfactored cable force for the parameters presented based on the calculation method and material strengths in Example 5-6.

MOMENT RESISTANCE, ϕM_n (ft.-kips) BASE PLATES WITH INSET ANCHOR RODS 5/16 inch fillet welds E70XX Electrode				
Shape	**d**	**b_f**	**X-X Axis**	**Y-Y Axis**
W8X24	8	6 1/2	50.8	12.2
W8X28	8	6 1/2	50.7	12.4
W8X31	8	8	60.1	18.5
W8X35	8	8	60.8	18.7
W8X40	8	8	61.7	18.9
W8X48	8	8	63.2	19.1
W10X33	10	8	75.6	18.4
W10X39	10	8	77.0	18.5
W10X45	10	8	78.2	18.7
W10X49	10	10	93.8	29.0
W10X54	10	10	94.7	29.2
W10X60	10	10	95.8	29.5
W10X68	10	10	97.3	29.8
W12X40	12	8	97.3	18.6
W12X45	12	8	98.3	18.8
W12X50	12	8	99.3	18.9
W12X53	12	10	117.8	29.0
W12X58	12	10	118.7	29.1
W12X65	12	12	138.2	41.8
W12X72	12	12	139.5	42.0
W12X79	12	12	140.8	42.3
W12X87	12	12	142.3	42.6
W14X61	14	10	139.2	29.0
W14X68	14	10	140.6	29.2
W14X74	14	10	141.8	29.4
W14X82	14	10	143.4	29.8
W14X90	14	14 1/2	192.4	61.1
W14X99	14	14 5/8	194.1	61.5
W14X109	14	14 5/8	195.8	61.9
W14X120	14	14 5/8	197.9	62.4
W14X132	14	14 3/4	200.1	62.9

Table A-l Moment Resistance of Base Plates with Inset Anchor Rods Based on Weld Strength

BASE PLATE BENDING RESISTANCE, ϕM_n (ft.-kips) WITH INSET ANCHOR RODS								
Shape	Anchor Rod Spacing	Base Plate Plan Size	X-X Axis			Y-Y Axis		
			Plate Thickness			Plate Thickness		
		L X W	3/4	1	1 1/4	3/4	1	1 1/4
W8X24	4	9 X 7	11.3	20.1	31.4	11.3	20.1	31.4
W8X28	4	9 X 7	11.3	20.1	31.4	11.3	20.1	31.4
W8X31	4	9 X 9	12.6	22.3	34.9	12.6	22.3	34.9
W8X35	4	9 X 9	12.5	22.2	34.8	12.5	22.2	34.8
W8X40	4	9 X 9	12.5	22.2	34.7	12.5	22.2	34.7
W8X48	4	9 X 9	12.4	22.0	34.4	12.4	22.0	34.4
W10X33	4	11 X 9	10.7	19.0	29.6	10.7	19.0	29.6
W10X33	5	11 X 9	14.1	25.1	39.2	14.1	25.1	39.2
W10X39	4	11 X 9	10.6	18.8	29.4	10.6	18.8	29.4
W10X39	5	11 X 9	14.0	25.0	39.0	14.0	25.0	39.0
W10X45	4	11 X 9	10.5	18.7	29.2	10.5	18.7	29.2
W10X45	5	11 X 9	14.0	24.9	38.9	14.0	24.9	38.9
W10X49	4	11 X 11	11.7	20.8	32.4	11.7	20.8	32.4
W10X49	5	11 X 11	15.7	28.0	43.7	15.7	28.0	43.7
W10X49	6	11 X 11	20.5	36.4	56.9	20.5	36.4	56.9
W10X54	4	11 X 11	11.6	20.7	32.3	11.6	20.7	32.3
W10X54	5	11 X 11	15.7	27.9	43.6	15.7	27.9	43.6
W10X54	6	11 X 11	20.3	36.2	56.5	20.3	36.2	56.5
W10X60	4	11 X 11	11.6	20.6	32.2	11.6	20.6	32.2
W10X60	5	11 X 11	15.7	27.9	43.5	15.7	27.9	43.5
W10X60	6	11 X 11	20.2	35.9	56.2	20.2	35.9	56.2
W10X68	4	11 X 11	11.5	20.5	32.1	11.5	20.5	32.1
W10X68	5	11 X 11	15.6	27.8	43.4	15.6	27.8	43.4
W10X68	6	11 X 11	20.0	35.6	55.6	20.0	35.6	55.6
W12X40	4	13 X 9	9.3	16.6	26.0	9.3	16.6	26.0
W12X40	5	13 X 9	12.3	21.9	34.2	12.3	21.9	34.2
W12X45	4	13 X 9	9.3	16.6	25.9	9.3	16.6	25.9
W12X45	5	13 X 9	12.3	21.9	34.2	12.3	21.9	34.2
W12X50	4	13 X 9	9.3	16.6	25.9	9.3	16.6	25.9
W12X50	5	13 X 9	12.3	21.8	34.1	12.3	21.8	34.1
W12X53	4	13 X 11	10.1	18.0	28.1	10.1	18.0	28.1
W12X53	5	13 X 11	13.4	23.9	37.3	13.4	23.9	37.3
W12X53	6	13 X 11	17.0	30.3	47.3	17.0	30.3	47.3
W12X58	4	13 X 11	10.1	18.0	28.1	10.1	18.0	28.1
W12X58	5	13 X 11	13.4	23.8	37.2	13.4	23.8	37.2
W12X58	6	13 X 11	17.0	30.2	47.2	17.0	30.2	47.2
W12X65	4	13 X 13	10.9	19.4	30.3	10.9	19.4	30.3
W12X65	5	13 X 13	14.6	25.9	40.5	14.6	25.9	40.5
W12X65	6	13 X 13	18.7	33.2	51.8	18.7	33.2	51.8

Table A-2 Moment Resistance of Base Plates with Inset Anchor Rods Based on Plate Strength

BASE PLATE BENDING RESISTANCE, ϕM_n (ft.-kips) WITH INSET ANCHOR RODS								
Shape	Anchor Rod Spacing	Base Plate Plan Size	X-X Axis			Y-Y Axis		
			Plate Thickness			Plate Thickness		
		L X W	3/4	1	1 1/4	3/4	1	1 1/4
W12X72	4	13 X 13	10.9	19.4	30.3	10.9	19.4	30.3
W12X72	5	13 X 13	14.5	25.9	40.4	14.5	25.9	40.4
W12X72	6	13 X 13	18.6	33.1	51.7	18.6	33.1	51.7
W12X79	4	13 X 13	10.9	19.3	30.2	10.9	19.3	30.2
W12X79	5	13 X 13	14.5	25.8	40.3	14.5	25.8	40.3
W12X79	6	13 X 13	18.6	33.1	51.6	18.6	33.1	51.6
W12X87	4	13 X 13	10.8	19.3	30.1	10.8	19.3	30.1
W12X87	5	13 X 13	14.4	25.7	40.1	14.4	25.7	40.1
W12X87	6	13 X 13	18.6	33.0	51.6	18.6	33.0	51.6
W14X61	4	15 X 11	9.4	16.6	26.0	9.4	16.6	26.0
W14X61	5	15 X 11	12.2	21.7	33.9	12.2	21.7	33.9
W14X61	6	15 X 11	15.4	27.4	42.8	15.4	27.4	42.8
W14X68	4	15 X 11	9.3	16.6	26.0	9.3	16.6	26.0
W14X68	5	15 X 11	12.2	21.6	33.8	12.2	21.6	33.8
W14X68	6	15 X 11	15.4	27.3	42.7	15.4	27.3	42.7
W14X74	4	15 X 11	9.3	16.6	25.9	9.3	16.6	25.9
W14X74	5	15 X 11	12.2	21.6	33.8	12.2	21.6	33.8
W14X74	6	15 X 11	15.3	27.2	42.6	15.3	27.2	42.6
W14X82	4	15 X 11	9.3	16.6	25.9	9.3	16.6	25.9
W14X82	5	15 X 11	12.1	21.6	33.7	12.1	21.6	33.7
W14X82	6	15 X 11	15.3	27.2	42.5	15.3	27.2	42.5
W14X90	4	15 X 15	10.8	19.2	30.0	10.8	19.2	30.0
W14X90	5	15 X 15	14.2	25.3	39.5	14.2	25.3	39.5
W14X90	6	15 X 15	18.2	32.3	50.5	18.2	32.3	50.5
W14X90	8	15 X 15	26.7	47.5	74.2	26.7	47.5	74.2
W14X99	4	15 X 15	10.8	19.2	30.0	10.8	19.2	30.0
W14X99	5	15 X 15	14.2	25.2	39.4	14.2	25.2	39.4
W14X99	6	15 X 15	18.1	32.2	50.3	18.1	32.2	50.3
W14X99	8	15 X 15	26.6	47.3	73.9	26.6	47.3	73.9
W14X109	4	15 X 15	10.8	19.1	29.9	10.8	19.1	29.9
W14X109	5	15 X 15	14.1	25.2	39.3	14.1	25.2	39.3
W14X109	6	15 X 15	18.0	32.1	50.1	18.0	32.1	50.1
W14X109	8	15 X 15	26.5	47.1	73.5	26.5	47.1	73.5
W14X120	4	15 X 15	10.7	19.1	29.8	10.7	19.1	29.8
W14X120	5	15 X 15	14.1	25.1	39.2	14.1	25.1	39.2
W14X120	6	15 X 15	18.0	32.0	49.9	18.0	32.0	49.9
W14X120	8	15 X 15	26.4	46.9	73.2	26.4	46.9	73.2
W14X132	4	15 X 15	10.7	19.1	29.8	10.7	19.1	29.8
W14X132	5	15 X 15	14.1	25.0	39.1	14.1	25.0	39.1
W14X132	6	15 X 15	17.9	31.8	49.7	17.9	31.8	49.7
W14X132	8	15 X 15	26.2	46.6	72.9	26.2	46.6	72.9

Table A-2 Moment Resistance of Base Plates with Inset Anchor Rods Based on Plate Strength

MOMENT RESISTANCE - OUTSET RODS, ϕMn (ft.-kips)

5/16 inch fillet welds
E70XX Electrode

Shape	d	bf	Rod Pattern		X-X Axis			Y-Y Axis		
			L	W	Plate Thickness			Plate Thickness		
					3/4	1	1 1/4	3/4	1	1 1/4
W8X24	8	6 1/2	12	10 1/2	12.9	19.9	27.8	11.3	17.4	24.3
W8X28	8	6 1/2	12	10 1/2	12.9	19.9	27.8	11.3	17.4	24.3
W8X31	8	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8X35	8	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8X40	8	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8X48	8	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W10X33	10	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10X39	10	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10X45	10	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10X49	10	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10X54	10	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10X60	10	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10X68	10	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W12X40	12	8	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12X45	12	8	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12X50	12	8	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12X53	12	10	16	14	17.2	26.5	37.1	15.1	23.2	32.4
W12X58	12	10	16	14	17.2	26.5	37.1	15.1	23.2	32.4
W12X65	12	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12X72	12	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12X79	12	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12X87	12	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W14X61	14	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14X68	14	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14X74	14	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14X82	14	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14X90	14	14 1/2	18	18 1/2	19.4	29.8	41.7	19.9	30.7	42.8
W14X99	14	14 5/8	18	18 5/8	19.4	29.8	41.7	20.0	30.9	43.1
W14X109	14	14 5/8	18	18 5/8	19.4	29.8	41.7	20.0	30.9	43.1
W14X120	14	14 5/8	18	18 5/8	19.4	29.8	41.7	20.0	30.9	43.1
W14X132	14	14 3/4	18	18 3/4	19.4	29.8	41.7	20.2	31.1	43.4

Table A-3 Moment Resistance of Base Plates with Outset Anchor Rods

MOMENT RESISTANCE, ϕMn (ft.-kips) TUBE COLUMN BASE PLATE 5/16 inch fillet welds E70XX Electrode				
Nominal TS Size	**Anchor Rod Spacing**	**X-X or Y-Y Axis**		
		Plate Thickness		
		3/4	**1**	**1 1/4**
4X4	8	8.6	13.3	18.5
5X5	9	9.7	14.9	20.8
6X6	10	10.8	16.6	23.2
8X8	12	12.9	19.9	27.8
10X10	14	15.1	23.2	32.4
12X12	16	17.2	26.5	37.1

Table A-4 Moment Resistance of Tube Column Base Plates

TENSION RESISTANCE, ϕP_n (kips/rod) Single A36 Anchor Rods				
Rod Diameter, in.				
3/4	**7/8**	**1**	**1-1/8**	**1-1/4**
Tension Area, in.^{2}				
0.4418	**0.6013**	**0.7854**	**0.9940**	**1.2270**
14.4	19.6	25.6	32.4	40.0

Table A-5 Tension Resistance of Anchor Rods Based on Anchor Rod Strength

TENSION RESISTANCE, ϕP_n (kips/rod) Single A36 Hooked Anchor Rods f'_c = 3000 psi					
Hook Length in.	Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Tension Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2270
3	8.0	9.4	10.7	12.0	13.4
4	10.7	12.5	14.3	16.1	17.9
5	----	----	17.9	20.1	22.3
6	----	----	----	24.1	26.8

Table A-6 Tension Resistance of Hooked Anchor Rods Based on Hook Length, f'_c = 3000 psi

TENSION RESISTANCE, ϕP_n (kips/rod) Single A36 Hooked Anchor Rods f'_c = 4000 psi					
Hook Length in.	Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Tension Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2270
3	10.7	12.5	14.3	16.1	17.9
4	14.3	16.7	19.0	21.4	23.8
5	----	----	23.8	26.8	29.8
6	----	----	----	32.1	35.7

Table A-7 Tension Resistance of Hooked Anchor Rods Based on Hook Length, f'_c = 4000 psi

CONCRETE PULL OUT RESISTANCE, ϕP_n (kips/rod) Single Nutted Anchor Rods					
Embed. Depth	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
9	54.0	55.0	55.9	56.9	57.9
12	93.0	94.3	95.7	97.0	98.3
15	142.6	144.3	145.9	147.5	149.2
18	202.7	204.7	206.7	208.6	210.6

Table A-8 Tension Resistance of Single Anchor Rods Based on Concrete Pull Out Capacity

CONCRETE PULL OUT RESISTANCE, ϕP_n (kips/rod) Anchor Rods in Groups Embedment Depth = 9 inches					
Anchor Rod Spacing in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2272
4	33.7	34.2	34.7	35.2	35.7
5	35.4	35.9	36.4	36.8	37.3
6	37.0	37.5	38.0	38.5	39.0
8	40.4	40.9	41.4	41.9	42.4
12	47.1	47.6	48.1	48.6	49.1
14	50.4	50.9	51.4	51.9	52.4
16	53.8	54.3	54.8	55.3	55.8
18	54.0	55.0	55.9	56.9	57.9
20	54.0	55.0	55.9	56.9	57.9

Table A-9 Tension Resistance of Two Anchor Rods Based on Spacing and Embedment

CONCRETE PULL OUT RESISTANCE, ϕPn (kips/rod) Anchor Rods in Groups Embedment Depth = 12 inches					
Anchor Rod Spacing in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2272
4	55.4	55.3	55.2	55.1	55.0
5	57.7	57.5	57.4	57.2	57.1
6	59.9	59.7	59.6	59.4	59.2
8	64.4	64.1	63.9	63.7	63.4
12	73.3	73.0	72.6	72.2	71.9
14	77.8	77.4	76.9	76.5	76.1
16	82.3	81.8	81.3	80.8	80.4
18	86.7	86.2	85.6	85.1	84.6
20	91.2	90.6	90.0	89.4	88.8

Table A-10 Tension Resistance of Two Anchor Rods Based on Spacing and Embedment

CONCRETE PULL OUT RESISTANCE, ϕPn (kips/rod) Anchor Rods in Groups Embedment Depth = 15 inches					
Anchor Rod Spacing in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2272
4	82.5	82.3	82.2	82.1	82.0
5	85.3	85.1	85.0	84.8	84.7
6	88.1	87.9	87.7	87.5	87.3
8	93.6	93.4	93.1	92.9	92.7
12	104.8	104.4	104.1	103.7	103.3
14	110.4	110.0	109.5	109.1	108.7
16	116.0	115.5	115.0	114.5	114.0
18	121.6	121.0	120.4	119.9	119.4
20	127.2	126.5	125.9	125.3	124.7

Table A-11 Tension Resistance of Two Anchor Rods Based on Spacing and Embedment

CONCRETE PULL OUT RESISTANCE, ϕP_n (kips/rod) Anchor Rods in Groups Embedment Depth = 18 inches					
Anchor Rod Spacing in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
	Area, in.^{2}				
	0.4418	0.6013	0.7854	0.9940	1.2272
4	114.8	114.6	114.5	114.4	114.3
5	118.1	118.0	117.8	117.6	117.5
6	121.5	121.3	121.1	120.9	120.7
8	128.2	127.9	127.7	127.4	127.2
12	141.6	141.2	140.8	140.4	140.1
14	148.3	147.8	147.4	147.0	146.5
16	155.0	154.5	154.0	153.5	153.0
18	161.7	161.1	160.5	160.0	159.4
20	168.4	167.8	167.1	166.5	165.9

Table A-12 Tension Resistance of Two Anchor Rods Based on Spacing and Embedment

PUSH OUT RESISTANCE, ϕP_n (kips/rod) Single Anchor Rods					
Clear Cover, in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
3	7.5	7.8	8.1	8.4	8.8
6	25.4	26.1	26.8	27.4	28.1
9	54.0	55.0	55.9	56.9	57.9
12	93.0	94.3	95.7	97.0	98.3

Table A-13 Compression Resistance of Single Anchor Rods Based on Concrete Push Out

PUSH OUT SHEAR RESISTANCE, ϕP_n (kips/rod) Group of Two Anchor Rods Clear Cover = 3 inches					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
4	6.0	6.1	6.3	6.5	6.6
5	6.5	6.7	6.9	7.0	7.2
6	7.1	7.2	7.4	7.6	7.7
8	7.5	7.8	8.1	8.4	8.8
12	7.5	7.8	8.1	8.4	8.8
14	7.5	7.8	8.1	8.4	8.8
16	7.5	7.8	8.1	8.4	8.8
18	7.5	7.8	8.1	8.4	8.8
20	7.5	7.8	8.1	8.4	8.8

Table A-14 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

PUSH OUT RESISTANCE, ϕP_n (kips/rod) Group of Two Anchor Rods Clear Cover = 6 inches					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
4	17.2	17.5	17.9	18.2	18.5
5	18.3	18.6	19.0	19.3	19.6
6	19.4	19.8	20.1	20.4	20.7
8	21.7	22.0	22.3	22.7	23.0
12	25.4	26.1	26.8	27.1	27.4
14	25.4	26.1	26.8	27.4	28.1
16	25.4	26.1	26.8	27.4	28.1
18	25.4	26.1	26.8	27.4	28.1
20	25.4	26.1	26.8	27.4	28.1

Table A-15 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

PUSH OUT RESISTANCE, ϕP_n (kips/rod) Group of Two Anchor Rods Clear Cover = 9 inches					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
4	33.7	34.2	34.7	35.2	35.7
5	35.4	35.9	36.4	36.8	37.3
6	37.0	37.5	38.0	38.5	39.0
8	40.4	40.9	41.4	41.9	42.4
12	47.1	47.6	48.1	48.6	49.1
14	50.4	50.9	51.4	51.9	52.4
16	53.8	54.3	54.8	55.3	55.8
18	54.0	55.0	55.9	56.9	57.9
20	54.0	55.0	55.9	56.9	57.9

Table A-16 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

PUSH OUT RESISTANCE, ϕP_n (kips/rod) Group of Two Anchor Rods Clear Cover = 12 inches					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	3/4	7/8	1	1-1/8	1-1/4
4	55.4	56.1	56.8	57.4	58.1
5	57.7	58.3	59.0	59.7	60.3
6	59.9	60.6	61.2	61.9	62.6
8	64.4	65.0	65.7	66.4	67.0
12	73.3	74.0	74.6	75.3	76.0
14	77.8	78.5	79.1	79.8	80.4
16	82.3	82.9	83.6	84.2	84.9
18	86.7	87.4	88.1	88.7	89.4
20	91.2	91.9	92.5	93.2	93.8

Table A-17 Compression Resistance of Two Anchor Rods Based on Concrete Push Out

BENDING RESISTANCE, ϕM_n (ft.-kips) Single Square Piers				
Size, in.	Reinforcing			
	0.5%		1.0%	
	Bars	ϕMn	Bars	ϕMn
12 x 12	4 - #5	25.3	4 - #6	34.7
14 x 14	4 - #5	31.2	4 - #7	57.0
16 x 16	4 - #6	51.4	4 - #8	87.7
18 x 18	4 - #6	59.6	4 - #9	127.2
20 x 20	4 - #7	91.0	4 - #9	146.3
22 x 22	4 - #8	132.6	8 - #8	194.5
24 x 24	4 - #8	147.4	8 - #8	216.9

Table A-18 Concrete Pier Bending Resistance

OVERTURNING RESISTANCE, M_o (ft.-kips) Single Square Footings				
Size	Thickness Group A		Thickness Group B	
	Footing Thickness in.	M_o	Footing Thickness in.	M_o
4'-0"	12	4.8	13	5.2
4'-6"	12	6.8	13	7.4
5'-0"	12	9.4	15	11.7
5'-6"	13	13.5	17	17.7
6'-0"	14	18.9	18	24.3
6'-6"	16	27.5	20	34.3
7'-0"	17	36.4	21	45.0
7'-6"	18	47.5	23	60.6
8'-0"	19	60.8	24	76.8
8'-6"	20	76.8	26	99.8
9'-0"	21	95.7	27	123.0
9'-6"	22	117.9	28	150.0
10'-0"	24	150.0	30	187.5

Table A-19 Concrete Footing Overturning Resistance

Required Development Length (inches) f'_c = 3000 psi			
Bar Size	**Standard Hooked Bar**	**Straight bars**	
		#6 & smaller	**#7 & larger**
3	6	16	----
4	8	22	----
5	10	27	----
6	12	33	----
7	13	----	48
8	15	----	55
9	17	----	62
10	19	----	68
11	21	----	75

Table A-20 Reinforcing Bar Development Lengths, f'_c = 3,000

Required Development Length (inches) f'_c = 4000 psi			
Bar Size	**Standard Hooked Bar**	**Straight bars**	
		#6 & smaller	**#7 & larger**
3	6	14	----
4	7	19	----
5	8	24	----
6	10	28	----
7	12	----	42
8	13	----	47
9	15	----	53
10	17	----	59
11	18	----	65

Table A-21 Reinforcing Bar Development Lengths, f'_c = 4,000

PLATE ANCHOR - TYPE A Plate in Tension & Bending									
Cable, 6x7 FC (IPS)	Slope, deg. from horizontal								
	30			45			60		
Diam. in.	L in.	t in.	Fillet Weld	L in.	t in.	Fillet Weld	L in.	t in.	Fillet Weld
1/2	3	5/16	3/16	3	3/8	3/16	3	3/8	3/16
5/8	4	5/16	3/16	4	5/16	3/16	4	3/8	3/16
3/4	5	5/16	3/16	5	5/16	3/16	5	3/8	3/16
Cable, 6x7 IWRC (IPS)	Slope, deg. from horizontal								
	30			45			60		
Diam. in.	L in.	t in.	Fillet Weld	L in.	t in.	Fillet Weld	L in.	t in.	Fillet Weld
1/2	3	5/16	3/16	3	3/8	3/16	3	1/2	3/16
5/8	4	5/16	3/16	4	3/8	3/16	4	3/8	3/16
3/4	5	5/16	3/16	5	3/8	3/16	5	3/8	3/16

Table A-22 Dimensions of Type A Anchor Plates and Welds

PLATE ANCHOR ALLOWABLE CABLE FORCE, P (lbs.) Anchor Rods in Tension & Bending									
Grout Depth in.	Anchor Rod Diameter, in.								
	3/4			1			1-1/4		
	slope, deg.			slope, deg.			slope, deg.		
	30	45	60	30	45	60	30	45	60
2	13690	15064	18118	34311	36248	41408	71093	71886	78078
3	7417	8560	11006	18114	20464	25513	36484	40313	48753
4	5087	5979	7904	12305	14256	18436	24538	28011	35442
5	3870	4594	6166	9317	10938	14432	18486	21462	27841
6	3124	3729	5054	7497	8873	11857	14828	17394	22924
7	2618	3139	4282	6272	7464	10062	12379	14623	19483
8	2254	2710	3715	5391	6441	8739	10624	12614	16941

Table A-23 Allowable Cable Force, Type A Plate Anchor
as Limited by Anchor Rod Capacity

BENT PLATE ANCHOR - TYPE B Plate in Tension & Bending						
Cable, 6x7 FC, (IPS)	Slope, deg. from horizontal					
	30		45		60	
Diam. in.	B in.	t in.	B in.	t in.	B in.	t in.
1/2	4	1/2	4	1/2	4	5/8
5/8	5	1/2	5	5/8	5	5/8
3/4	6	1/2	6	5/8	6	5/8
Cable, 6x7 IWRC, (IPS)	Slope, deg. from horizontal					
	30		45		60	
Diam. in.	B in.	t in.	B in.	t in.	B in.	t in.
1/2	4	1/2	4	1/2	4	5/8
5/8	5	1/2	5	5/8	5	5/8
3/4	6	5/8	6	5/8	6	3/4

Table A-24 Dimensions of Type B Anchor Plate and Welds

	BENT PLATE ANCHOR ALLOWABLE CABLE FORCE, P (lbs.) Anchor Rods in Tension & Bending								
Grout Depth in.	Anchor Rod Diameter, in.								
	3/4			1			1-1/4		
	Slope, deg.			Slope, deg.			Slope, deg.		
	30	45	60	30	45	60	30	45	60
2	9357	8755	8789	20760	18351	17512	38106	32123	29501
3	5930	6073	6692	13471	13197	13860	25262	23795	24038
4	4340	4649	5402	9971	10304	11468	18893	18896	20282
5	3422	3766	4530	7914	8451	9781	15089	15670	17541
6	2825	3165	3900	6561	7163	8526	12560	13385	15453
7	2405	2729	3423	5603	6216	7557	10757	11682	13809
8	2094	2399	3051	4889	5490	6785	9407	10363	12481

Table A-25 Allowable Cable Force, Type B Plate Anchor as Limited by Anchor Rod Capacity

NOTES:

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Revisions and Errata List
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October 15, 2012

The following list represents corrections to the revision dated October 2003 of the first printing (1997) of AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Buildings.*

Page(s)	Item
29	Equation 5-1 should read:

$$\Delta_1 = \frac{(0.2\text{NBS} - \text{P})\text{L}}{\text{A}(0.9)\text{E}}$$

33 — Near the middle of the right column, the sentence beginning “Per Caltrans...” should read:

Per Caltrans (9) the maximum cable drape (A) should be 2.75 in.

In the 9th line from the bottom of the right column, the corresponding calculation of P should read:

$$\text{P} = (0.84)(40)^2 \Big/ \left[8\left(\frac{2.75}{12}\right)(0.847)\right]$$
$$= 866 \text{ lbs.}$$

The horizontal and vertical components of the preload force are 734 pounds and 460 pounds, respectively.

34 — In the left column, 2nd line, the calculation for Δ_1 should read:

$$\Delta_1 = \frac{\left[0.2(45{,}400) - 866\right](47.2)}{0.216(0.9)(13{,}000{,}000)} \qquad \text{(Eq. 5-1)}$$
$$= 0.15 \text{ ft}$$

The calculation in the 3rd line from the bottom of the left column should read:

$$(\sin\theta)a = (\sin 0.9°)(25) = 0.393 \text{ ft}$$

At the top of the right column, replace the first eight lines with the following:

$$\text{R} = \frac{81{,}120(0.393)}{25}$$
$$= 1{,}275 \text{ lbs.}$$

$$1{,}275(47.51/40) = 1{,}514 \text{ lbs.}$$

Cable force including PΔ effects:

$11,013 + 1,514 + 866 = 13,393$ lbs.

Cable force: 13,393 lbs.

Allowable cable force = 45,400/3 = 15,133 > 13,393 lbs.

Therefore, use a ¾ in. diameter cable.