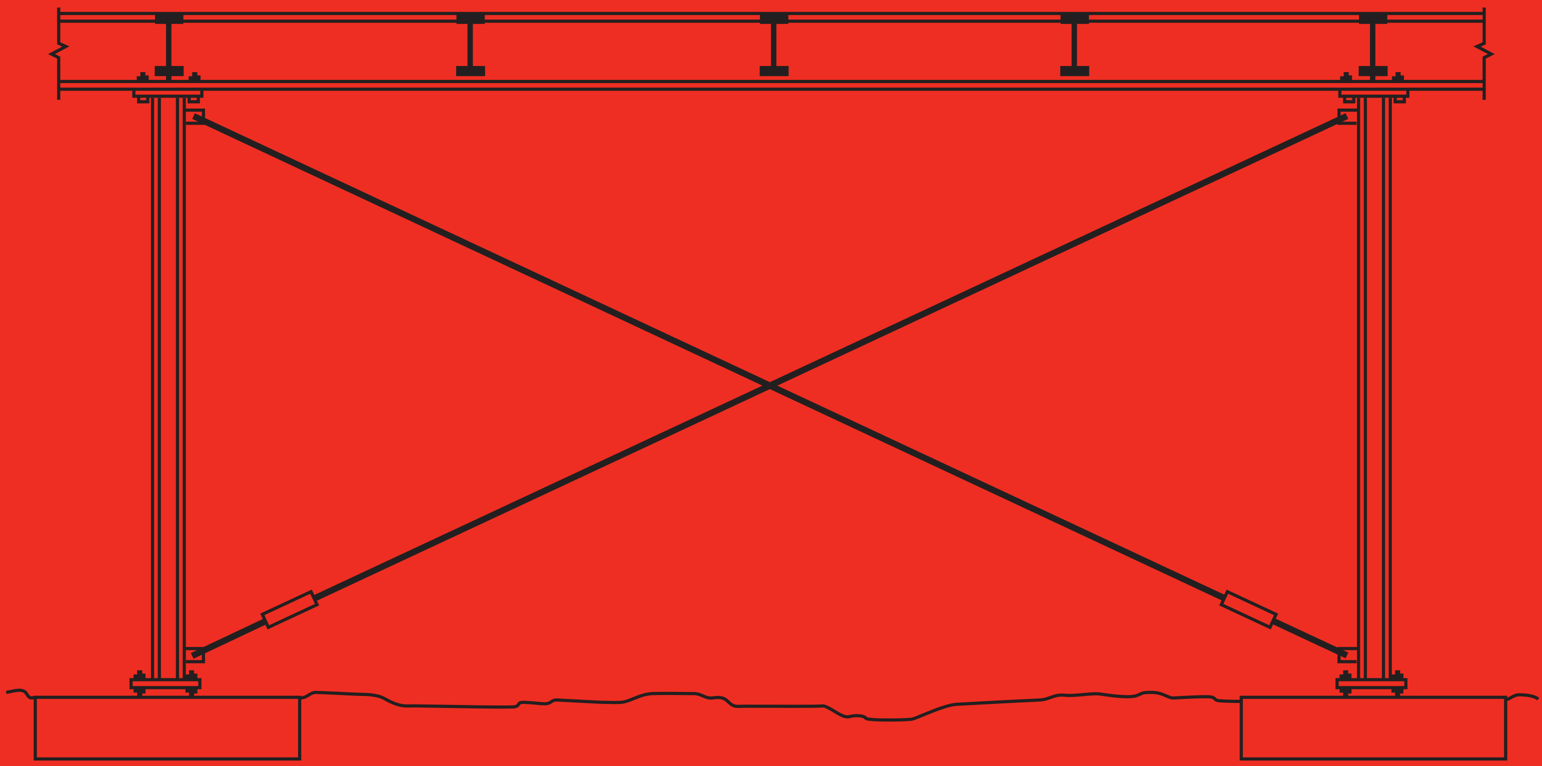

Design Guide 10

Erection Bracing of Low-Rise Structural Steel Buildings

Second Edition

Smarter. Stronger. Steel.

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Preface

This second edition of Design Guide 10 provides guidance for the design of temporary bracing for single-story structural steel buildings. The second edition incorporates the 2016 AISC *Specifications* and the 15th Edition of the AISC *Steel Construction Manual*, as well as the 2019 ACI 318 *Building Code Requirements for Reinforced Concrete and Commentary.* All design examples have been updated to conform to the 2016 AISC *Specification* and the 2019 ACI 318 *Building Code*.

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Chapter 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. The Guide is primarily concerned with establishing adequate strength to sustain the loads required by the AISC *Code of Standard Practice for Buildings and Bridges* (AISC, 2016a), hereafter referred to as the AISC *Code of Standard Practice*, and ASCE/SEI 37, *Design Loads on Structures during Construction* (ASCE, 2015), hereafter referred to as ASCE/SEI 37-14. Unless specific instructions to the contrary are provided in the contract documents, the trade practices defined in the AISC *Code of Standard Practice* govern the fabrication and erection of structural steel. While ASCE/SEI 37-14 is a standard of the American Society of Civil Engineers, the authors are unaware of its incorporation in any code. Thus, in the opinion of the authors it is subsidiary to the AISC *Code of Standard Practice*.

For the purpose of this Design Guide, low-rise buildings are taken to have the following characteristics:

1. Function: general purpose structures for such uses as light manufacturing, crane buildings, warehousing, office, 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 AISC *Specification for Structural Steel Buildings* Section M4.2 (AISC, 2016b), hereafter referred to as the AISC *Specification*, and in AISC *Code of Standard Practice* Section 7.

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 that are not in the scope of the steel erector's work. However, the AISC *Code of Standard Practice* does require that the erector provide temporary supports to secure the bare frame "…against loads that are likely to be encountered during erection…". Other temporary supports such as shoring and cribbing for vertical loads are not included in the scope of this Guide.

The following are excerpts from the AISC *Code of Standard Practice* that apply to temporary supports. The reader is encouraged to become familiar with the AISC *Code of Standard Practice* in its entirety.

1.9. Means, Methods and Safety of Erection

1.9.1. The *erector* shall be responsible for the means, methods and safety of erection of the *structural steel* frame.

1.9.2. The *structural engineer of record* shall be responsible for the structural adequacy of the design of the structure in the completed project. The *structural engineer of record* shall not be responsible for the means, methods and safety of erection of the *structural steel* frame. See also Sections 3.1.4 and 7.10.

7.10. Temporary Support of Structural Steel Frames

7.10.3. Based upon the information provided in accordance with Sections 7.10.1 and 7.10.2, the *erector* shall determine, furnish and install all temporary supports, such as temporary guys, beams, falsework, cribbing or other elements required for the erection operation. These temporary supports shall be sufficient to secure the bare *structural steel* framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

The *erector* need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the *owner's designated representatives for design* and *construction*, nor those that are unpredictable, such as loads due to hurricane, tornado, earthquake, explosion or collision.

Temporary supports that are required during or after the erection of the *structural steel* frame for the support of loads caused by non-*structural steel* elements, including cladding, interior partitions and other such elements that will induce or transmit loads to the *structural steel* frame during or after erection, shall be the responsibility of others.

It should be noted that although the AISC *Code of Standard Practice* does not require consideration of loads due to hurricane, the typical approach for the determination of temporary bracing is to use the mapped wind speed for the site. In coastal areas along the Atlantic, Gulf, and Alaskan coasts, the mapped speeds include historical hurricane wind data. ASCE/SEI 37-14 includes recommendations for the consideration of design wind forces both in and out of hurricane season, and the possibility that a lower design wind speed be used with a supplementary hurricane contingency plan developed using the higher wind speeds associated with hurricane season.

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 relying solely on structural steel elements for the lateral load-resisting system; however, the frames may contain elements such as a roof deck diaphragm, which would change the frame to one that relies on elements other than structural steel for lateral load resistance.

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 and associated diaphragms and horizontal bracing would be designed to transfer the lateral load to the rigid frames. Rigid-frame construction would be characterized as relying solely on structural steel elements for the lateral load-resisting system; however, as in the case of braced construction, the framework may contain nonstructural elements, such as steel deck diaphragms, that are required for lateral load resistance.

Diaphragm construction: This system uses horizontal and/or vertical diaphragms to resist lateral loads. As stated previously, horizontal diaphragms may be used with other bracing systems. Horizontal diaphragms are typically composed of 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 elements contributing to the resistance of the frame to lateral loads.

Cantilever construction: Also called flag pole construction, this system achieves lateral load resistance by means of the flexural strength of the columns and moment-resisting base connections to the foundations. This system would be characterized as relying solely on structural steel elements for the lateral load-resisting system unless the base design required post-erection grouting to achieve its available strength. Because grouting is typically outside the scope of the erector, a design requiring grout would be a nonstructural element that is required for lateral load resistance.

Each of the four bracing systems poses different issues for their erection and temporary support, but they share one thing in common. As presented in the project construction documents, all of the systems 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 (for more information on this topic, see AISC *Code of Standard Practice* Section 7.10.) Structures that incorporate nonstructural steel elements as part of the lateral load-resisting system will require temporary support of the erection by definition unless the nonstructural elements (like concrete or masonry core walls) are in place in advance of steel erection.

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. Since the publication of the first edition of this Guide, the authors have observed industry practice and regret that, based on their observations, limited progress has been made with respect to the proper determination of temporary supports in the low-rise segment of the industry. Low-rise construction still does not command a comparable respect or attention to that of high-rise and bridge construction 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 led to the continuance of 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, all steel frames need bracing. This fallacy is probably a carryover from the era when steel frames were primarily used in heavy framing that 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 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. Figure 1-1 shows a column collapse where the footings were assumed to be adequate for erection loads without evaluation.

4. *Bracing can be removed at any time.* In fact, the temporary supports are an integral part of the framework until it is completed. This condition may not even occur until sometime after the erection work is complete, as in the case of structures incorporating nonstructural-steel elements in the lateral load-resisting system.

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

6. *Plumbing cables are adequate as bracing cables.* 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 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 is intended to be used to determine the requirements for temporary supports to resist lateral forces—that is, stability and wind. The next three chapters present methods by which the temporary supports may be determined by calculation of loads and calculation of resistance. Chapter 2 deals with design loads that would be applicable to the

Fig. 1-1. Erection collapse due to lack of bracing.

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. Chapters 3 and 4 deal with the determination of resistances, both of the permanent structure as it is being erected and any additional temporary supports, which may be needed to complete the temporary support system. The final chapter presents a series of prescriptive requirements for the structure and the temporary supports that, if met, eliminate the need to prepare calculations. The prescriptive requirements of Chapter 5 are based on calculations prepared using the principles presented in Chapters 2, 3, and 4. An appendix is also included that provides tabular values for resistances for various components of the permanent structure.

Chapter 2
Loads for Temporary Supports During Construction

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

- Gravity loads
  - Dead load of the structure itself
  - Superimposed dead loads
  - Live loads and other loads from construction operations
- Environmental loads, i.e., wind
- Stability loads
- Erection operation loads
- Loads from erection apparatus
- Impact loads caused by erection equipment and pieces being raised within the structure

2.1 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; nonuniform, 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 Part 17 of the AISC *Steel Construction Manual* (AISC, 2017a), hereafter referred to as the AISC *Manual*, and ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), hereafter referred to as ASCE/SEI 7-10, Tables C3-l and C3-2. (Note that the 2010 version of ASCE/SEI 7 is referenced in this Guide because ASCE/SEI 37-14 is based on the 2010 version.) 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 incomplete 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.

2.2 ENVIRONMENTAL LOADS

The principal environmental load affecting the design of temporary supports is wind load. 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.

2.2.1 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 structural element. Various codes and standards treat the determination of design wind pressures in slightly different ways; however, the basic concept is common to all methods. What follows is a discussion of the procedure provided in ASCE/SEI 7-10, which will illustrate the basic concept.

In ASCE/SEI 7-10, the equation for basic design wind pressure, p, for the main force-resisting system using the Directional Procedure for a rigid building is:

$$p = qGC_p - q_i(GC_{pi})$$

(ASCE/SEI 7-10 Eq. 27.4-1)

where

C_p = external pressure coefficient from ASCE/SEI 7-10, Figures 27.4-1, 27.4-2, and 27.4-3

G = gust-effect factor from ASCE/SEI 7-10, Section 26.9

GC_{pi} = internal pressure coefficient from ASCE/SEI 7-10, Table 26.11-1

q = q_h for leeward walls, side walls, and roofs, evaluated at height h, lb/ft^2

= q_z for windward walls evaluated at height z above the ground, lb/ft^2

q_i = q_h for windward walls, side walls, leeward walls, and roof of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings, lb/ft^2

= q_z for positive internal pressure evaluation of partially enclosed buildings, lb/ft^2

q_z = $0.00256K_zK_{zt}K_dV^2$, lb/ft^2 (ASCE/SEI 7-10 Eq. 27.3-1)

K_d = wind directionality factor defined in ASCE/SEI 7-10, Section 26.6

K_z = velocity pressure coefficient defined in ASCE/SEI 7-10, Section 27.3.1

K_{zt} = topographic factor defined in ASCE/SEI 7-10, Section 26.8.2

V = basic wind speed, mph, from ASCE/SEI 7-10, Section 26.5

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.

The authors recommend that the basic wind speed (using the ASCE/SEI 7-10 map for Risk Category II) and exposure classification for the project location should be used in the design of the temporary supports. As noted previously, this will result in hurricane-like wind speeds in coastal areas. The provisions of ASCE/SEI 37-14 address wind loads both in and out of hurricane season and alternative means to address the associated wind forces. The use of Risk Category II is permitted in ASCE/SEI 37-14, Section 6.1.

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

ASCE/SEI 7-10 does not specifically address main wind force-resisting system (MWFRS) wind loading on bare frame open structures. It does however address certain other structures in Chapter 29, such as solid freestanding walls and solid signs, other structures, rooftop structures and equipment, parapets, and roof overhangs. It is the judgment of the authors that the provisions for signs are the best analogous geometries to the surfaces of bare frame open structures. Thus, the appropriate equation for design wind force, F, on bare frame structures is ASCE/SEI 7-10, Equation 29.4-1:

$$F = q_h G C_f A_s \qquad \text{(ASCE/SEI 7-10 Eq. 29.4-1)}$$

where

A_s = gross area of the solid freestanding wall or freestanding solid sign, ft^2

C_f = net force coefficient from ASCE/SEI 7-10, Figure 29.4-1

F = design wind force, lb

G = gust-effect factor from ASCE/SEI 7-10, Section 26.9

q_h = velocity pressure determined using ASCE/SEI 7-10, Equation 29.3-1 at height h (defined in ASCE/SEI 7-10, Figure 29.4-1), lb/ft^2

$= 0.00256 K_z K_{zt} K_d V^2$ (ASCE/SEI 7-10 Eq. 29.3-1)

Using ASCE/SEI 37-14, Section 6.2.1, V can be reduced using the 0.75 factor for a construction period of less than 6 weeks, 0.8 for 6 weeks to a year, 0.85 for one to two years, and 0.9 for two to five years. In the case of bare frame 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 force 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. *Metal Building Systems Manual* Part A7.3.3 (MBMA, 2012), published by the Metal Building Manufacturers Association.
2. "Wind Forces on Structures," published by the American Society of Civil Engineers (ASCE, 1961).
3. *Standards for Load Assumptions, Acceptance, and Inspection of Structures* (SIA, 1956), published by the Swiss Association of Engineers and Architects.
4. *Actions on Structures—Part 4: Wind Loads* (DIN, 2005), German Industrial Standard (DIN) 1055, published by the German Institute for Standards. Although this standard is no longer the standard for wind loads in Germany and has been technically withdrawn as a design load standard, its provisions for bare structures have not been replaced by other comparable requirements. The German national supplement to the Eurocode is silent on this topic.

ASCE/SEI 37-14, Section 6.2.2, addresses certain aspects of wind loads on frameworks without cladding. This section's provisions for wind loads on multiple parallel frame lines are illustrated in Chapters 3 and 4 of this Guide.

ASCE/SEI 37-14, Section 6.2.2, *Frameworks without Cladding* states:

> Structures shall resist the effect of wind acting upon successive unenclosed components.
>
> Treatment of staging, shoring, and falsework with a regular rectangular plan as trussed towers in accordance with ASCE/SEI 7-10 shall be permissible. 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%.

3. 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 the foregoing 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 force coefficient for the open-web member that is a function of the solidity ratio. For the types of open-web members used in low-rise construction, an effective area (solidity ratio, φ) equal to 20 to 30% of the projected solid area can be used. The ratio can be calculated if sufficient data is available.

Shielding of multiple parallel elements can also be evaluated using the following equation taken from DIN 1055 *Actions on Structures—Part 4: Wind Loads* (DIN, 2005). See Figures 2-1 and 2-2. Note that the variables a, d, and l used in these figures are in consistent units. The total factored area, A, is determined using Equation 2-1:

$$A = \left[1 + \eta + (n-2)\eta^2\right] A_1 \tag{2-1}$$

where

A_1 = projected area of one element, in.^{2}

n = total number of parallel elements

η = stacking factor taken from Figure 2-2

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

It should be noted that the AISC *Code of Standard Practice* does not require consideration of loads due to hurricanes as a design condition for temporary supports. ASCE/SEI 37-14 does provide relevant provisions for winds in coastal areas. ASCE/SEI 37-14, Section 6.2, has provisions and guidance in the Commentary for situations in which the construction period will fall within the hurricane season (July 1 through October 31). Essentially the standard provides for design for the coastal wind forces per ASCE/SEI 7-10 or the design of a hurricane contingency in which instructions are provided that can be readily implemented when weather predictions indicate that the structure is in the likely path of an approaching hurricane. It is incumbent on the erector and the temporary supports designer to develop a hurricane contingency that can be implemented fully in the limited time between prediction and the time at which the site must be evacuated for safety. Should the erector elect to employ a hurricane contingency plan, this should be coordinated with the owner's designated representative for construction as provided for in the contract documents.

2.2.2 Seismic Loads

The AISC *Code of Standard Practice* Section 7.10.3 excludes seismic forces as a load consideration in the design of temporary supports. However, earthquake forces are addressed in ASCE/SEI 37-14, Section 6.5. ASCE/SEI 37-14, Section 6.5.1, states:

> Earthquake loads need not be considered unless required by the authority having jurisdiction and the mapped Risk-Targeted MCE_R, 5% damped, spectral response acceleration parameter at a period of 1 s, S_1, defined in Section 11.4.1 of ASCE 7-10 equals or exceeds 0.40.

As the commentary to this section states, "It is not reasonable to require seismic resistance for temporary works where large earthquakes are infrequent or not considered probable." When ASCE/SEI 7-10 earthquake provisions do apply, ASCE/SEI 37-14 provides certain modifications to the provisions in Section 6.5.2 and its commentary. The limited, albeit significant, situations where ASCE/SEI 7-10 earthquake load provisions apply and the related design consequences are beyond the scope of this Design Guide, especially in light of the exclusion of earthquake forces in AISC *Code of Standard Practice* Section 7.10.3.

2.3 DESIGN FOR STABILITY AND HORIZONTAL CONSTRUCTION LOADS

AISC *Specification* Chapter C, Appendix 7 and Appendix 8, provide requirements for the analysis and design for stability. The reader is referred to the AISC *Specification* for these requirements. Also, the reader is referred to Example 4.1.1 of this Guide for illustration of the application of the direct analysis method, including second order effects, to the design of a temporary wire rope diagonal brace.

ASCE/SEI 37-14, Section 4.4, provides for minimum horizontal construction loads, C_H. Item 4 in Section 4.4 provides for a horizontal construction load of 2% of the total vertical load. For the bare frames that are the subject of this Guide, the total load is essentially the dead load of the steel frame. The provision states that this load, C_H, need not be applied concurrently with wind or seismic load.

2.4 ERECTION OPERATION LOADS

Certain 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 may result in incidental loads on the partially completed frame. These small

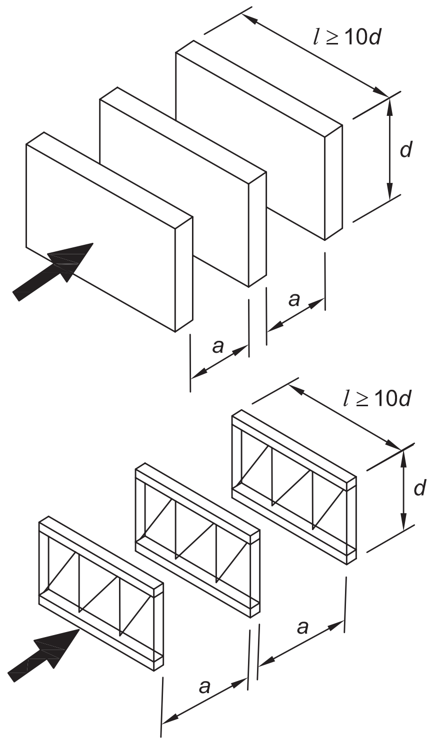


Fig. 2-1. Parameters for use with Fig. 2-2 (DIN, 2005).

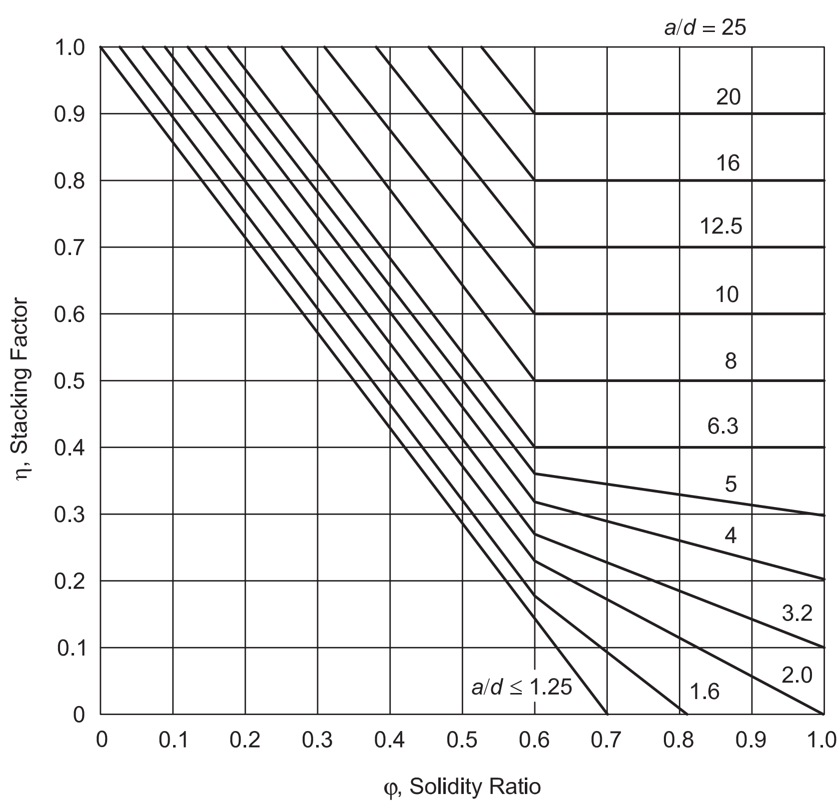


Fig. 2-2. Stacking factor versus solidity ratio (DIN, 2005).

loads are generally resisted by the frame with minimum connections provided. If significant prying, pulling, or jacking is required, the resulting forces 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 lb/ft be applied to the perimeter of the framework.

ASCE/SEI 37-14 refers to forces due to erection as erection and fitting forces, C_F, but the specific provisions only refer to the requirement in OSHA 1927.755(a)(2) requiring that column and anchor rod assemblies, including the welding of the column to the base plate, shall be designed to resist a 300-lb eccentric load located 18 in. 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*.

2.5 LOAD COMBINATIONS

ASCE/SEI 7-10, Section 2.3.2, provides seven basic load combinations to be investigated in load and resistance factor design (LRFD):

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

ASCE/SEI 7-10, Section 2.4.1, provides nine basic load combinations to be investigated in allowable strength design (ASD):

1. D
2. $D + L$
3. $D + (L_r \text{ or } S \text{ or } R)$
4. $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
5. $D + (0.6W + 0.7E)$

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

6b. $D + 0.75L + 0.75(0.7E) + 0.75S$

7. $0.6D + 0.6W$
8. $0.6D + 0.7E$

The nominal loads to be considered are:

D = dead load
E = earthquake load
L = live load
L_r = roof live load
R = rain load
S = snow load
W = wind load

Loads and load combinations are also presented in ASCE/SEI 37-14, Chapter 2. This presentation is compatible with ASCE/SEI 7-10, but includes additional detail to reflect the scope of ASCE/SEI 37-14, such as accounting for equipment weights, the design of work platforms, and consideration of conditions where one load may be required to counterbalance another in a design or evaluation. The reader is referred to ASCE/SEI 37-14, Chapter 2, and its Commentary.

Chapter 3
Resistance to Loads by the Permanent Structure During Construction

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 construction progresses. Generally, the resistance is complete when the steel erection is complete. In a structure that incorporates nonstructural elements in the lateral load-resisting system, resistance (temporary supports) will be required after the completion of the erector's work and will be needed until the other nonstructural-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 determined to be less than adequate, it must be supplemented by temporary supports.

Temporary support of a column by means of leveling nuts and shims is considered in this section of the Guide, because their use induces forces on permanent elements of the structure such as anchor rods and foundations as illustrated in the text and examples. Determination of the loadings and associated strengths required when leveling nuts and/or shims are used is the responsibility of the erector.

Elements of the permanent structure that may be used to resist loads during construction include:

1. Columns (Section 3.1, Columns)
2. Column bases (Section 3.2, Column Bases, and Section 3.3, Design Examples)
3. Tie members—Beams and Joists (Section 3.4, Tie Members)
4. Diagonal bracing (Section 3.5, Use of Permanent Bracing)
5. Connections (Section 3.6, Beam-to-Column Connections)
6. Diaphragms (Section 3.7, Diaphragms)

These elements are briefly discussed in the following sections and additionally in the sections cited in the preceding list.

1. Columns

Typically, columns will have the same strength in the partially completed work as in the completed work, so their axial available strength would be the same during erection as the completed work. The exceptions would be:

a. Columns that are free standing on their bases before other framing and bracing are installed
b. Columns supported on leveling nuts or shims prior to grouting
c. Columns that are to be laterally braced by girts or struts
d. Columns that have additional axial load due to the temporary support system

2. 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 then to the ground.

The components of a column base are:

a. Base plate and its attachment to the column shaft

Column base plates are square or rectangular plates that 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 to the column shafts. The column base reaction is transferred to the column by bearing for compression forces and by the column-to-base plate welds for tension and shear.

b. 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 to distinguish between bolts (generally available in lengths up to 8 in.) 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 and horizontal shear forces when induced by net tension in the column, base bending moments, and shear. During erection operations and prior to base plate grouting, anchor rods may also resist compression loads and horizontal 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 weld to prevent turning. If welding is used it should be placed below the nut. Alternatively, the nut can be secured by means of a jammed double nut or deformed threads above and below the nut.

Calculations and examples are presented in this guide using hooked anchor rods. This was done because hooked anchor rods are frequently used in light construction. However, the authors strongly recommend that headed anchor rods be used due to their superior strength.

c. Base plate grout

High-strength, nonshrink grout is placed between the column base plate and the supporting foundation. Where base plates are shipped loose, the base plates are typically 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 with washer plates on the anchor rods beneath the base plates
2. The use of shim stacks between the base plate bottom and top of concrete supports
3. The use of ¼-in.-thick steel leveling plates that 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 or washer stacks are used, the compressive components of the column base reaction are carried by the shims and the tensile and shear components are carried by the anchor rods. In this Guide, "shims" and "washer stacks" should be taken by the reader as interchangeable terms. When used, washer stacks have the advantage of not being displaced; however, their location is limited to the location of the anchor rods.

Leveling nuts bear the weight of the frame until grouting of the bases. Because the anchor rod, nut, and washers have limited available strength, grouting must be completed before this available strength would be exceeded by the accumulated weight of the frame. For example, the available strength of the anchor rods 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 steel floor deck.

Properly installed shim stacks can support significant vertical load. There are two types of shims—those that are placed on (washer) or around (horseshoe) the anchor rods and shim stacks that 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 welded to maintain the integrity of the stacks. When shim stacks are used, care must be taken to ensure that the stacks cannot topple, shift, or become dislodged until grouting.

Pre-grouted 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 AISC *Manual* and AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006).

In some instances, columns are deigned to be installed without grout with the base plate bearing directly on the foundation. The analysis procedures for the assessment of the base plate, anchors, and foundation presented herein can be adapted for these cases.

d. Supporting foundation

Building foundations are cast-in-place concrete structures. The element which typically 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 that generally adopt the provisions of the American Concrete Institute standards such as ACI 318 *Building Code Requirements for Reinforced Concrete and Commentary* (ACI, 2019), hereafter referred to as ACI 318-19. 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 are resisted by deformed reinforcing bars. If erection occurs prior to the concrete 28-day strength being obtained, a reduced value must be used in calculations. OSHA 1926.752(a) (OSHA, 2011) requires a minimum of 75% of the specified strength to be obtained.

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* (ACI, 2013) 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 available 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 erection operation, sequence of the work, reactions from temporary supports, and the timing of grouting may cause forces in the anchor rods and foundation that 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 wire ropes for temporary lateral support also induce tensions and shears in the column base that 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 that is not counterbalanced by the dead load. A net uplift in the column base may induce forces in the base plates and welds, anchor rods, and foundation that exceed those for which the structure in its completed state was designed.

3. Tie Members—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 compressive 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 that is in place at the time of load application.

4. 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 available strength may be limited by the hole type and tightening achieved. OSHA does not permit the use of "pins" for bracing during erection; however, one bolt minimum connections are required. The strength of such connections are dependent on the bolt type and diameter. The bracing available 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.

5. Connections

Structural steel frames are held together by a multitude of connections that transfer axial force, shear, and moment from component to component. During erection, connections may 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 textbooks and the AISC *Manual,* partially completed connections can be evaluated for adequacy during erection.

6. 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—that is, 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.

3.1 COLUMNS

Exceptions were listed earlier wherein the columns may not have the same effective 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.

3.2 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 are 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.

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 that could occur. It is not the intent of this Design Guide to provide an engineering treatise on each of these failure modes, but rather to provide a general overview of each mode and to illustrate the use of fundamental engineering principles to determine the load and resistance associated with each mode. Equations are provided to obtain the available strength for each mode based on structural engineering principles and the AISC *Specification*. The most likely modes of failure are listed as follows:

1. Fracture of the fillet weld connecting the column to the base plate.
2. Bending failure of the base plate.
3. Tensile 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 breakout from the concrete pier or footing. Breakout as used herein refers to what ACI refers to as concrete breakout.
7. Anchor rod pullout.
8. Anchor rod pushout at the bottom of the footing.
9. Pier bending failure.
10. Footing overturning.

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

3.2.1 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 are secondary—that is, some other mode of failure initiated the collapse, and weld failure occurred after the initial failure.

Fracture occurs when the weld strength is exceeded. A fillet weld fracture at the column base plate is depicted in Figure 3-1. This normally occurs for forces acting about the weak axis of the column, because the weld group has a lower available strength about the weak axis, and because the wind forces are greater when acting against the weak axis.

The available strength of the weld between the column and the base plate can be determined by calculating the available flexural strength of the weld group. Applied shear forces on the weld are small and can be neglected in these calculations.

The available flexural strength is ϕM_n for LRFD and M_n/Ω for ASD.

For bending about the column major axis:

$$M_n = F_{nw} S_x \tag{3-1}$$

For bending about the column minor axis:

$$M_n = F_{nw} S_y \tag{3-2}$$

where

F_{nw} = nominal stress of the weld metal, ksi
= $0.60F_{EXX}(1.0 + 0.50\sin^{1.5}\theta)$ (*Spec.* Eq. J2-5)

F_{EXX} = filler metal classification strength, ksi

M_n = nominal flexural strength of the weld, kip-in.

S_x = elastic section modulus of the weld group taken about its x-axis, in.^{3}

S_y = elastic section modulus of the weld group taken about its y-axis, in.^{3}

ϕ = 0.75

Ω = 2.00

θ = angle between the line of action of the required force and the weld longitudinal axis, degrees

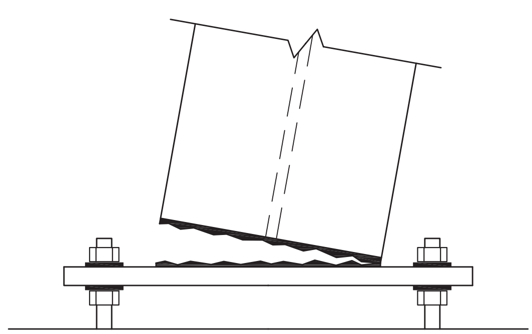


Fig. 3-1. Failure mode 1—fracture of weld.

For fillet welds loaded at 90° to the weld longitudinal axis:

$$F_{nw} = (0.60F_{EXXX})(1.5)$$
$$= 0.90F_{EXX} \tag{3-3}$$

3.2.2 Bending Failure of the Base Plate

Ordinarily a bending failure, such as illustrated in Figure 3-2, 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 available 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 available 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 available flexural 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. Alternatively, the procedures contained in AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006), can be used.

The available strength of the base plate is determined using two yield lines. Two yield line lengths b_1 and b_2 are shown in Figure 3-3. The length b_1 is taken as two times d_1, the distance of the anchor rod to the center 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 anchor rod centerline. The yield lines shown are for the calculation of M_y.

Using the dimensions shown in Figure 3-3, the available strength, ϕP_n or P_n/Ω, for a single anchor rod is based on:

$$P_n = \frac{M_{1n}}{d_1} + \frac{M_{2n}}{d_2} \tag{3-4}$$

where

M_{1n} = plastic moment resistance based on b_1, kip-in.

M_{2n} = plastic moment resistance based on b_2, kip-in.

P_n = anchor rod nominal force that causes the base plate to reach its nominal strength, kips

ϕ = 0.90

Ω = 1.67

Equation 3-4 is based on d_1 and d_2 being approximately equal. P_n is multiplied by 2 if the base condition consists of two anchor rods in tension. The available flexural strength is ϕM_n or M_n/Ω, where M_n is determined by multiplying P_n by the appropriate lever arm, g_1 or g_2. In some cases, the strength of the base plate may actually be governed by compression in the column. However, only the tension case is presented here because it illustrates all the essential considerations. For the case where there are two anchor rods in tension:

$$M_n = 2P_n g_1 \text{ or } 2P_n g_2 \tag{3-5}$$

If leveling nuts are used under the base plate, the lever arm, d, is the distance between the anchor rods, as shown in Figure 3-4.

If shim stacks are used, then the lever arm, d, is the distance from the anchor rods to the center of the shim stack, as shown in Figure 3-5. See discussion of the use of shims at the beginning of this section.

For most wide-flange columns subject to axial compression only, welding on one side of each flange, as

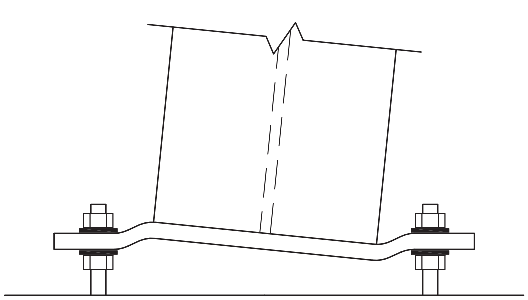


Fig. 3-2. Failure mode 2—bending failure of base plate.

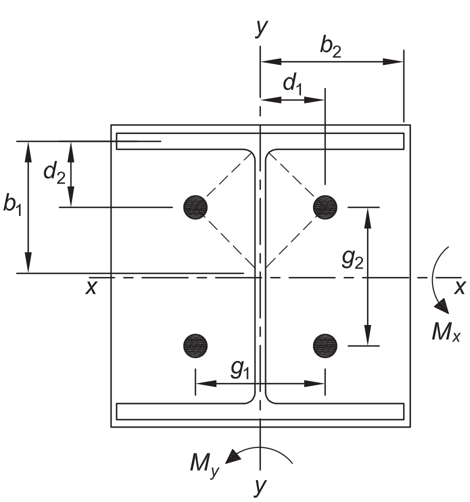


Fig. 3-3. Base plate dimensions.

recommended in AISC *Manual* Part 14, will provide adequate strength and the most economical detail. Without the web weld, only the length b_2 would be used in the strength calculations. See Figure 3-3.

Case B: Outset Anchor Rods—Wide-Flange Columns

The authors are unaware of any published solutions to determine base plate thickness or weld available strength for the base plate anchor rod condition shown in Figure 3-6, where the anchor rods are outset. By examining Figure 3-6 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 analyses in order to establish a conservative design procedure to determine the required base plate thickness and weld available strength for this condition. The following conclusions are based on the finite element studies:

1. The effective width of the base plate, b_e, for flexural strength checks should be taken as $2L$, but not to exceed 5 in.
2. A maximum weld length of 2 in. 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 4 in. of weld can be used.
3. The base plate thickness should be of similar thickness to the flange thickness so as not to over strain the welds.

In equation format, the available strength for a single outset anchor rod is determined as follows:

Based on the plate effective width, the available strength is ϕP_n or P_n/Ω,

where

$$P_n = \frac{F_y b_e t_p^2}{4L} \tag{3-6}$$

t_p = plate thickness, in.

Based on weld strength, the available strength is $\phi_w P_n$ or P_n/Ω_w,

where

$$P_n = F_w t_{weld} \text{ (2 in.)} \tag{3-7}$$

F_w = nominal weld stress, ksi
= $0.90F_{EXX}$ (90° loading)

t_{weld} = throat width of weld, in.

$\phi_w = 0.75$

$\Omega_w = 2.00$

Based on weld strain, the available strength is $\phi_w P_n$ or P_n/Ω_w,

where

$$P_n = 50t_{weld}(t_p)^{1.5} \tag{3-8}$$

$\phi_w = 0.90$

$\Omega_w = 1.67$

Using the controlling value for P_n and d, for two anchor rods in tension, the available flexural strength, ϕM_n or M_n/Ω, is based on the following:

$$M_n = 2P_n d \tag{3-9}$$

Case C: Outset Anchor Rods with Hollow Structural Section (HSS) Columns

When HSS columns are used, Equations 3-6 and 3-8 can be used to calculate P_n; however, if fillet welds exist on all four sides of the column, then 4 in. of weld length at the corner of the HSS can be used for the calculation of P_n in Equation 3-7.

Then the available strength of a single anchor rod is $\phi_w P_n$ or P_n/Ω_w,

where

$$P_n = F_w t_{weld}(4) \tag{3-10}$$

$\phi_w = 0.75$

$\Omega_w = 2.00$

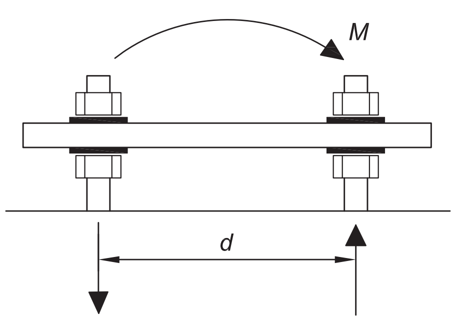


Fig. 3-4. Base plate with leveling nuts.

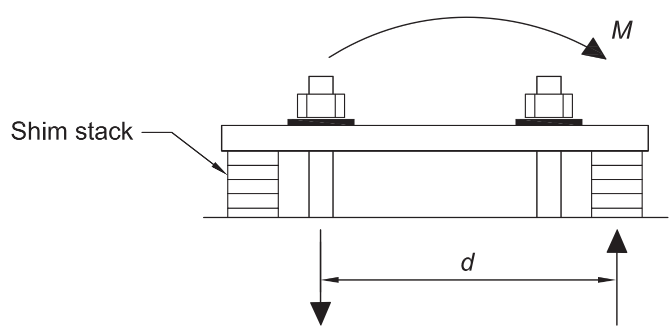


Fig. 3-5. Base plate with shim stacks.

3.2.3 Tensile Rupture of the Anchor Rods

A tensile rupture of the anchor rods is often observed after an erection collapse. This failure occurs when the overturning forces exceed the available strength of the anchor rods. Rupture typically occurs in the root of the anchor rod threads 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. An anchor rod tension failure is shown in Figure 3-7. The available tensile rupture strength, $\phi_t P_n$ or P_n/Ω_t, for rods is determined in accordance with the AISC *Specification* Section D2,

where

$$P_n = F_u A_b \tag{3-11}$$

A_b = area of anchor rod, in.^{2}

F_u = specified minimum tensile strength of the rod, ksi

ϕ_t = 0.75

Ω_t = 2.00

For two anchor rods in tension, the available flexural strength of the base plate can again be determined based on:

$$M_n = 2P_n d \tag{3-12}$$

3.2.4 Buckling of the Anchor Rods

The failure mode of anchor rod buckling is illustrated in Figure 3-8.

The buckling strength of anchor rods can be calculated using AISC *Specification* Chapter E or Section J4.4. For base plates set using leveling nuts, a reasonable value for the unbraced length of the anchor rods is the distance from the top of the concrete pier or footing to the bottom of the leveling nut. 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 or equal to ¾ in. used in conjunction with grout thickness not exceeding 5 in., the authors have determined that the buckling strength of the anchor rods is approximately equal to the tensile strength of the rods. Thus, this failure mode need not be checked for most situations.

3.2.5 Anchor Rod Nut 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, as shown in Figure 3-9. To address these conditions, washers of sufficient size (diameter and thickness) should be used on both sides of the base plate. The reader is referred to AISC *Manual* Table 14-2 for recommended washer sizes. Special consideration must be given to base plate holes that have been enlarged to accommodate misplaced anchor rods.

3.2.6 Anchor Rod Breakout and Pullout

The available strength for anchor rod breakout is based on ACI equations, thus the available strength can only be used with LRFD calculations.

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 tensile strength of the rod as illustrated in Figures 3-10 and 3-11. As indicated earlier, ACI refers to these modes of failure as "concrete breakout strength." The failure occurs in the concrete along the surface of a truncated pyramid (cones), surrounding area of the anchor head, or for the case of hooked rods, the projected area of the hook.

The dotted lines in Section A-A of Figure 3-12 represent the failure lines of the truncated pyramid profile. Note that for rods in tension, the truncated pyramids will be pulled out of the footing or pier top, whereas the truncated pyramids beneath the rods in compression will be pushed out of the footing bottom. This latter failure mode will be discussed in the next section. Depending on the spacing of the anchor

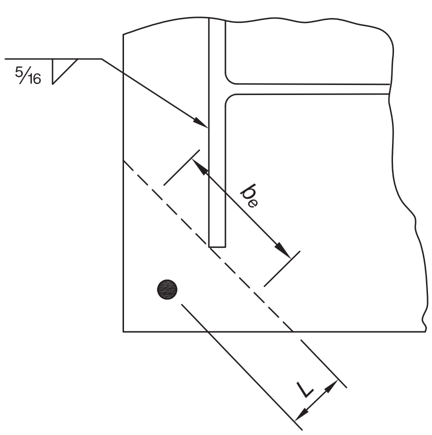


Fig. 3-6. Effective width.

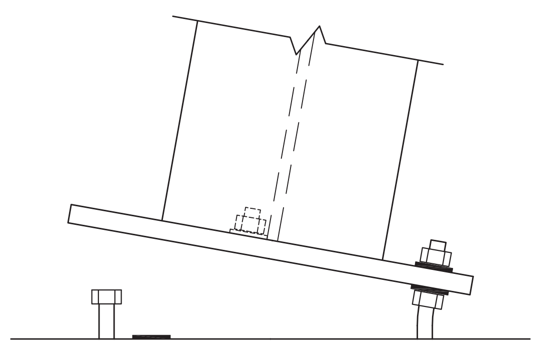


Fig. 3-7. Failure mode 3—tensile rupture of anchor rods.

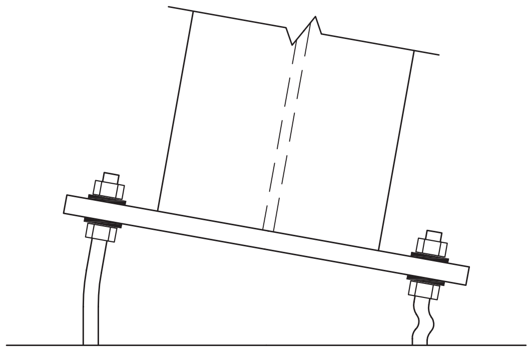


Fig. 3-8. Failure mode 4—anchor rod buckling.

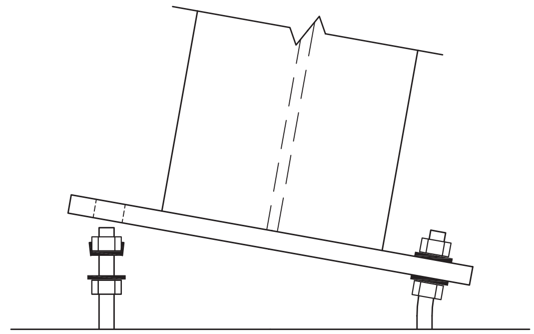


Fig. 3-9. Failure mode 5—anchor rod nut pull through.

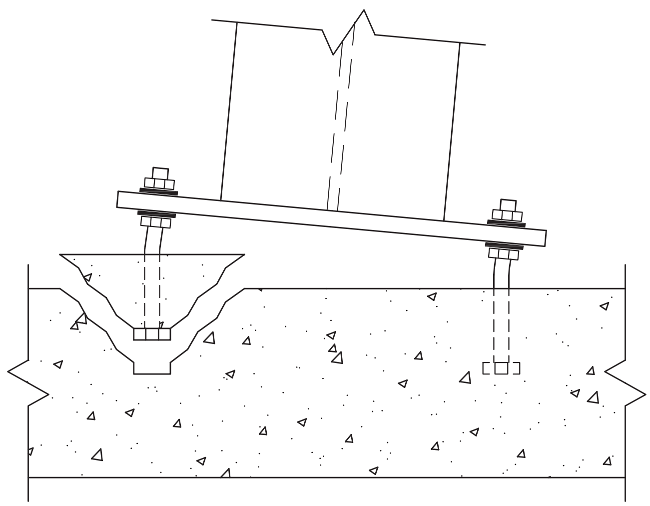


Fig. 3-10. Failure mode 6—anchor rod breakout.

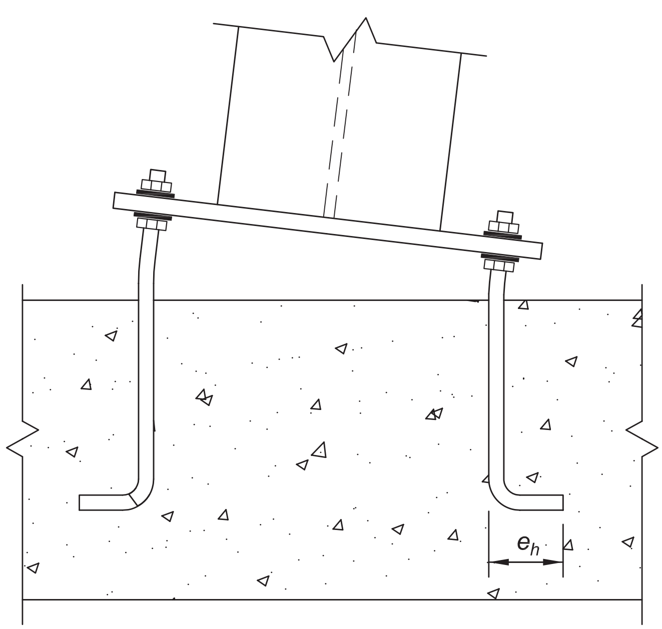


Fig. 3-11. Failure mode 7—anchor rod pullout.

rods and the depth of embedment of the rods in the concrete, the failure truncated pyramids may overlap. The overlapping of the failure truncated pyramids makes the calculation more complex.

Based on ACI 318-19, Chapter 17, the nominal breakout strength in tension of a group of anchors, N_{cbg}, should not exceed (ACI Equation 17.6.2.1b):

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \tag{3-13}$$

where

A_{Nc} = projected concrete failure area of a group of anchors, for calculation of strength in tension, in.^{2}, from ACI 318-19, Figure R17.6.2.1

A_{Nco} = projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, in.^{2}

N_b = basic concrete breakout strength in tension of a single anchor in cracked concrete, lb

c_{s1} = distance from the center of anchor shaft to edge of concrete, in.

c_{s2} = distance from center of anchor shaft to the edge of concrete in the direction perpendicular to c_{s1}, in.

s_1, s_2 = anchor spacing in each direction, in.

$\psi_{c,N}$ = factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete

$\psi_{cp,N}$ = factor used to modify tensile strength of post installed anchors intended for use in uncracked concrete without supplementary reinforcement to account for the splitting tensile stresses due to installation.

$\psi_{ec,N}$ = factor used to modify tensile strength of anchors based on eccentricity of applied loads

$\psi_{ed,N}$ = factor used to modify tensile strength of anchors based on proximity to edges of concrete member

Also from ACI 318-19 (Equation 17.6.2.1.4):

$A_{Nco} = 9h_{ef}^2$, in.^{2}

h_{ef} = effective embedment depth of anchor, in.

Where anchors are located less than $1.5h_{ef}$ from three or more edges, the value of h_{ef} used in the calculations for A_{Nc} shall be the larger of $c_{a,max}/1.5$ or $s/3$,

where

$c_{a,max}$ = largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, in.

s = maximum spacing between anchors within a group, in.

When $c_{a,min} \geq 1.5h_{ef}$ (ACI Equation 17.6.2.4.1a)

$$\psi_{ed,N} = 1.0 \tag{3-14a}$$

When $c_{a,min} < 1.5h_{ef}$ (ACI Equation 17.6.2.4.1b)

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \tag{3-14b}$$

where

$c_{a,min}$ = minimum distance from center of an anchor shaft to the edge of concrete, in.

The basic concrete breakout strength for a single anchor in tension in cracked concrete, N_b, is (ACI Equation 17.6.2.2.1):

$$N_b = k_c \lambda_a \sqrt{f_c'} h_{ef}^{1.5} \tag{3-15}$$

where

f_c' = specified compressive strength of concrete, psi

k_c = coefficient for basic concrete breakout strength in tension

λ_a = modification factor to reflect the reduced mechanical properties of lightweight concrete

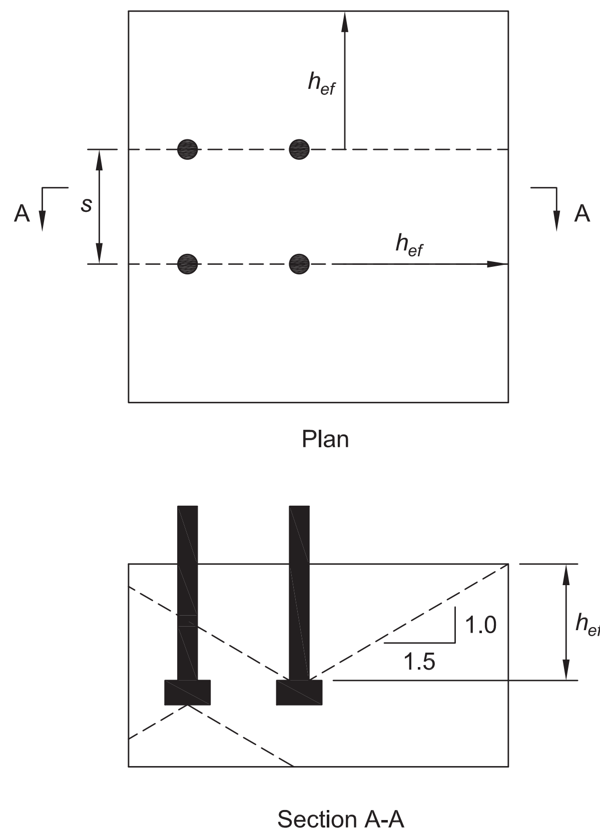


Fig. 3-12. Failure truncated pyramids.

Table 3-1a. Headed Anchor Concrete Pullout Strength, ϕN_p, kips					
Anchor Rod Diameter, in.	Anchor Rod Area, A_r, in.^{2}	Bearing Area, A_{brg}, in.^{2}	f'_c = 3,000 psi	f'_c = 4,000 psi	f'_c = 5,000 psi
¾	0.442	0.911	15.3	20.4	25.5
⅞	0.601	1.19	20.0	26.6	33.3
1	0.785	1.50	25.2	33.6	42.0
1⅛	0.994	1.85	31.1	41.5	51.8
1¼	1.23	2.24	37.6	50.1	62.6

For use in this Design Guide the values are:

k_c = 24 (for cast-in anchors)

λ_a = 1.0

$\psi_{ec,N}$ = 1.0 (no eccentricity of loading)

$\psi_{ed,N}$ = 1.0 (for anchor rods in footings)

= 0.7 (conservatively used for anchor rods in piers)

$\psi_{c,N}$ = 1.25 (concrete is considered uncracked at service loads)

When $h_{ef} < 11$ in.

$$N_b = 24\sqrt{f'_c}\left(h_{ef}\right)^{1.5} \tag{3-16a}$$

When 11 in. $\le h_{ef} \le$ 25 in.

$$N_b = 16\sqrt{f'_c}\left(h_{ef}\right)^{5/3} \tag{3-16b}$$

For footings where $c_{a,min} \ge 1.5h_{ef}$ (most cases)

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}(1.0)(1.0)(1.25)(1.0)N_b$$

$$= 1.25\left(\frac{A_{Nc}}{A_{Nco}}\right)N_b \tag{3-17}$$

For piers

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}(1.0)(\psi_{ed,N})(1.25)(1.0)N_b$$

$$= 1.25\psi_{ed,N}\left(\frac{A_{Nc}}{A_{Nco}}\right)N_b \tag{3-18}$$

The required breakout strength, P_r, is

$$P_r \le \phi N_{cbg} \tag{3-19}$$

where

ϕ = 0.70

In addition to concrete breakout, concrete pullout can also occur when hooked rods straighten or headed anchor rods locally crush the concrete in bearing. ACI 318-19 requires that the nominal pullout strength, N_{pn}, of a single cast-in anchor rod shall not exceed (ACI Equation 17.6.3.1):

$$N_{pn} = \psi_{c,P} N_p \tag{3-20}$$

where

N_p = pullout strength in tension of a single headed anchor rod or hooked bolt, kips

For a single headed anchor rod (ACI Equation 17.6.3.2.2a)

$$N_p = 8A_{brg} f'_c \tag{3-21}$$

For a single hooked bolt (ACI Equation 17.6.3.2.2b)

$$N_p = 0.9 f'_c e_h d_a \tag{3-22}$$

where

A_{brg} = net bearing area of the head of anchor rod, in.^{2}

$= A_n - A_r$

A_n = nut area with no hole, in.^{2}

$= 6\left(\frac{w_n \tan 30°}{2}\right)\left(\frac{w_n}{2}\right)$

A_r = anchor rod area, in.^{2}

w_n = nut width, in. based on AISC *Manual* Table 7-19

d_a = anchor rod diameter, in.

e_h = distance from inner surface of the shaft of the J- or L-bolt to outer tip of the J- or L-bolt (length of hook, see Figure 3-11), in., $3d_a \le e_h \le 4.5d_a$

$\psi_{c,P}$ = 1.4 for uncracked concrete and 1.0 for cracked concrete

Concrete pullout strengths for headed anchor rods based on the nut net bearing area of the head of the anchor rod are shown in Table 3-1a. These strengths are available only with sufficient embedment of the anchor rod. Some designers use a heavy plate washer to increase the bearing area. Shown in Table 3-1b are concrete pullout strengths for hooked bolts (J bolts). Note the significant increase in strength by using headed rods.

The available strength obtained from Equation 3-20 must be compared to the available strength obtained from the

Table 3-1b. Hooked Bolt Concrete Pullout Strength, ϕN_p, kips				
Bolt Diameter, in.	Hook Length, e_h, in.	$f_c' = 3{,}000$ psi	$f_c' = 4{,}000$ psi	$f_c' = 5{,}000$ psi
¾	3⅜	4.78	6.38	7.97
⅞	4	6.62	8.82	11.0
1	4½	8.51	11.3	14.2
1⅛	5	10.6	14.2	17.7
1¼	5½	13.0	17.3	21.7

failure truncated pyramids. The lower of the nominal strengths from these two limit states, T_n, provides the nominal strength to be used in the calculation for the available flexural strength associated with rod breakout and pullout:

$$\phi M_n = \phi 2 T_n d \tag{3-23}$$

where
$\phi = 0.70$

On occasion, post-installed anchors are used during erection. The breakout and pullout strengths of post-installed anchors is treated in ACI 318-19.

3.2.7 Anchor Rod Pushout at the Bottom of the Footing

Anchor rod pushout can occur when the rod is loaded to the point where a truncated pyramid of concrete below the anchor rod breaks away from the footing, as shown in Figure 3-13. This failure mode is identical to anchor rod pull out, but is due to a compressive force in the rod rather than a tensile force. The pushout design strength for hooked anchor rods is essentially equal to that of the nutted rod.

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 3-14.

3.2.8 Pier Bending Failure

The available flexural strength of a reinforced concrete pier in bending, as shown in Figure 3-15, is calculated using reinforced concrete principles as shown in the following.

Determine the depth of the compression area:

$$C = T \tag{3-24}$$

where

$$C = 0.85 f_c' ab \tag{3-25}$$

$$T = F_y A_s \tag{3-26}$$

A_s = area of the reinforcement, in.^{2}

F_y = specified minimum yield strength of the reinforcement, psi

a = depth of the compression block, in.

b = width of the pier, in.

Setting C equal to T and solving for a:

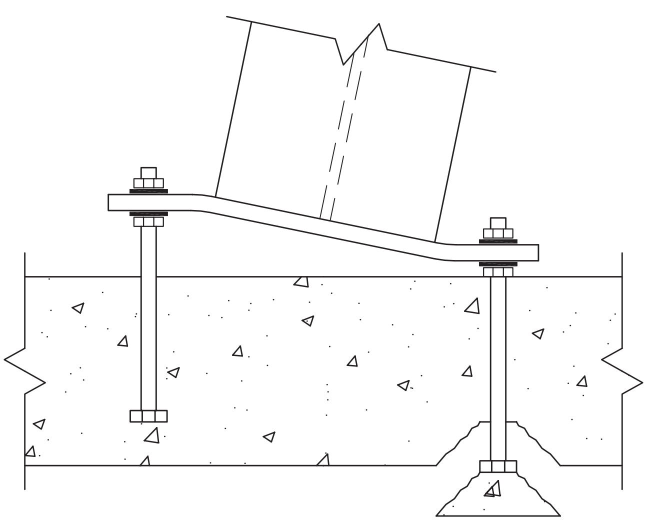


Fig. 3-13. Failure mode 8—anchor rod pushout.

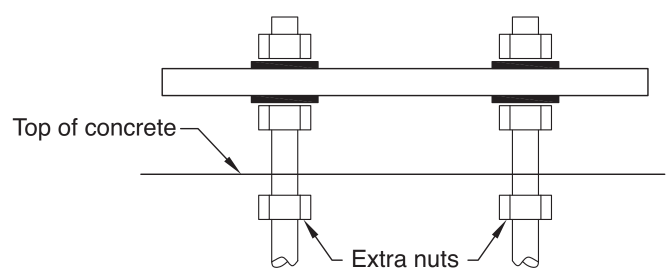


Fig. 3-14. Prevention of pushout.

$$a = \frac{F_y A_s}{0.85 f_c' b} \tag{3-27}$$

The available flexural strength of the pier is:

$$\phi M_n = \phi C \left(d - \frac{a}{2} \right) \tag{3-28}$$

where
d = effective depth of reinforcement, in.
ϕ = 0.90

In addition, to ensure that the reinforcement can develop the moment, the vertical reinforcement must be fully developed. Based on ACI 318-19, the required development length can be determined from the following equations. These equations presume that ACI column ties, concrete cover, and minimum spacing criteria are satisfied.

The development length of a standard hooked bar in tension, l_{dh}, measured from the critical section outside end of the hook is calculated using ACI 318-19, Section 25.4.3.1:

$$\left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda \sqrt{f_c'}} \right) d_b^{1.5} \geq 8d_b \text{ and 6 in.} \tag{3-29}$$

For straight bars in the pier, the development length, l_d, is calculated using ACI 318-19, Table 25.4.2.3.

For #6 bars and smaller:

$$\left(\frac{f_y \psi_t \psi_e \psi_g}{25\lambda \sqrt{f_c'}} \right) d_b \geq 12 \text{ in.} \tag{3-30a}$$

For #7 bars and larger:

$$\left(\frac{f_y \psi_t \psi_e \psi_g}{20\lambda \sqrt{f_c'}} \right) d_b \geq 12 \text{ in.} \tag{3-30b}$$

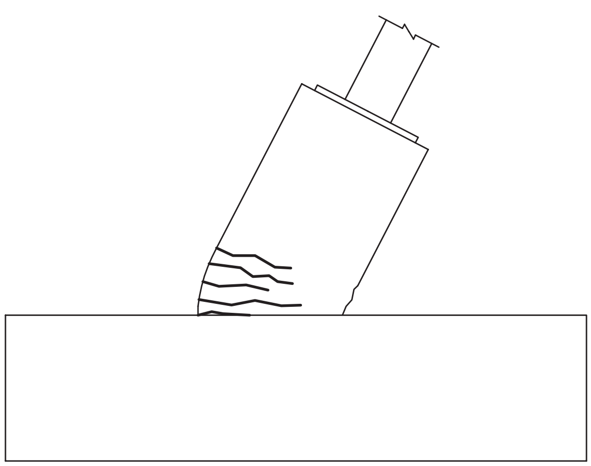


Fig. 3-15. *Failure mode 9—pier bending failure.*

where
d_b = bar diameter, in.
f_y = reinforcement bar yield strength, ksi
λ = 1.0
ψ_c = concrete strength modification factor
$= \frac{f_c'}{15{,}000} + 0.6$
ψ_e = epoxy modification factor
= 1.0 for uncoated or zinc-coated (galvanized) reinforcement
ψ_g = reinforcement grade modification factor
= 1.0 for Grade 40 or Grade 60
ψ_o = location modification factor
= 1.0
ψ_r = confining reinforcement modification factor
= 1.0
ψ_t = casting position modification factor
= 1.0 for vertical reinforcement

Figure 3-16 illustrates the development length, l_{dh}. If the actual bar embedment length is less than the value obtained from these equations, the strength requires further investigation.

3.2.9 Footing Overturning

The resistance of a column footing to overturning is dependent on the weight of the footing, pier, soil overburden, if any, and the length of the footing in the direction of overturning. (Note that soil overburden is the weight of the soil sitting on the top of the footing.) During construction, the overburden is often not present and thus is not included in the overturning calculation.

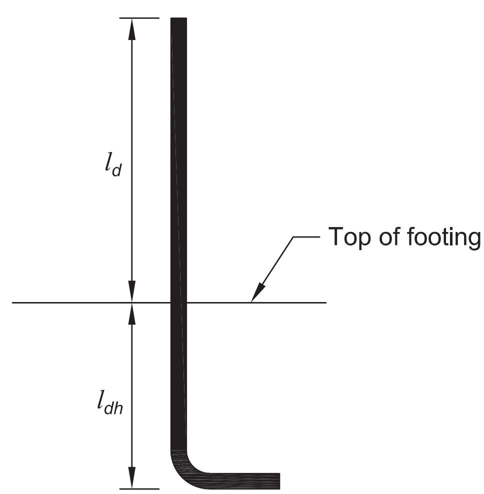


Fig. 3-16. Development lengths.

Shown in Figure 3-17 is a footing subjected to an overturning moment. The unfactored overturning resistance, M_o, equals the weight of the footing and all loads supported on it, W, multiplied by the length, L, of the footing in the direction of overturning divided by 2:

$$M_o = W\left(\frac{L}{2}\right) \tag{3-31}$$

where

$W = P_1 + P_2 + P_3$, kips (3-32)

and

P_1 = weight of superimposed loads, kips

P_2 = weight of the pier, kips

P_3 = weight of the footing, kips

The unfactored overturning resistance, M_o, is then compared to the required overturning moment obtained using the appropriate ASCE/SEI 7-10 load factors for wind or live load acting alone, M_{rOT}, to determine the suitability of the cantilevered column.

$$M_{rOT} \leq M_{nOT} \tag{3-33}$$

where

$M_{nOT} = \gamma_d M_o$ (3-34)

γ_d = load factor for LRFD or ASD load combinations

For dead loads, $\gamma_d = 0.9$ for LRFD from ASCE/SEI 7-10, Section 2.3.2, and $\gamma_d = 0.6$ for ASD from ASCE/SEI 7-10, Section 2.4.1.

Therefore,

$M_{nOT} = 0.9M_o$ (LRFD) (3-35a)

$= 0.6M_o$ (ASD) (3-35b)

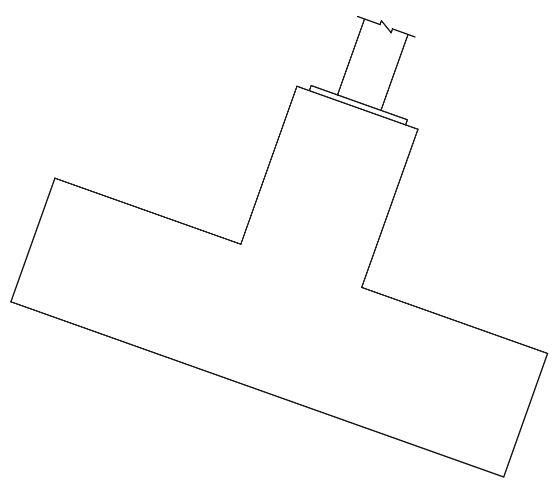


Fig. 3-17. Failure mode 10—footing overturning.

3.3 DESIGN EXAMPLES

The design examples that follow are shown in LRFD only because concrete calculations are involved. These examples only use uniaxial moments for simplicity; the reader is cautioned that biaxial moments should also be checked.

Example 3.3.1—Overturning of a Free-Standing Column with Inset Anchor Rods

Given:

Determine the overturning resistance of a W12×65, free-standing cantilever column 40 ft in length.

Use four ¾-in. ASTM F1554 Grade 36 hooked anchor rods with 1-ft embedment and a 4-in. hook. Leveling nuts and washers are used. The base plate material is ASTM A36. Reinforcing bars are 60-ksi material. Use 70-ksi weld electrodes.

The pier dimensions are 1'-4"×1'-4" with four #6 vertical bars and #3 ties spaced at 12 in. on center. The footing dimensions are 6'-0"×6'-0"×1'-3". Normal weight concrete (150 lb/ft^3) is used with $f_c' = 3{,}000$ psi. The weight of superimposed loads is 65 lb/ft and there is no soil overburden.

Foundation details are shown in Figure 3-18, and base plate details are shown in Figure 3-19.

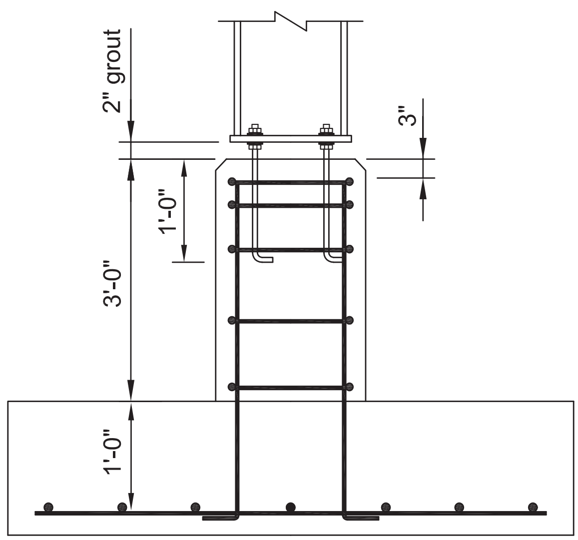


Fig. 3-18. Foundation detail for Example 3.3.1.

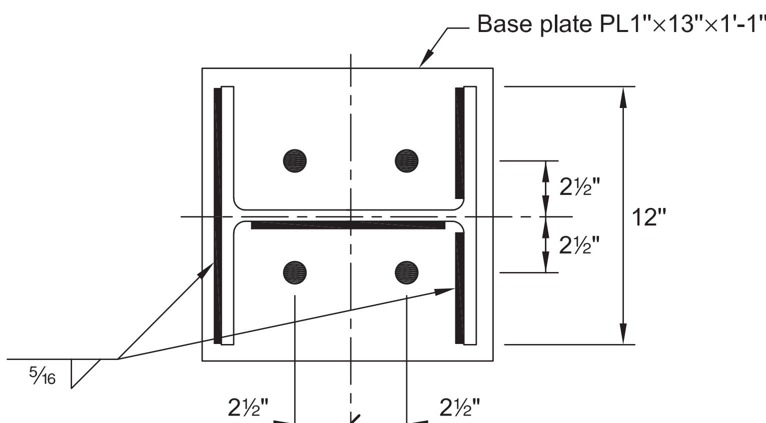


Fig. 3-19. Base plate detail for Example 3.3.1.

Note: The weld pattern shown in Figure 3-19 is preferred by many fabricators because the column base plate assembly need not be turned over in the shop to make all of the welds.

Solution:

From AISC *Manual* Table 2-5, the plate material properties are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 2-6, the anchor rod properties are as follows:

ASTM F1554 Grade 36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties for the W12×65 column are as follows:

$b_f = 12.0$ in.
$d = 12.1$ in.
$t_f = 0.605$ in.
$t_w = 0.390$ in.

Failure Mode 1—Fracture of the Weld

The available flexural strength of the weld group is determined using Equation 3-2:

$$M_n = F_{nw} S_y \tag{3-2}$$

From Equation 3-3:

$$\begin{aligned} F_w &= 0.90 F_{EXX} \\ &= 0.90(70 \text{ ksi}) \\ &= 63.0 \text{ ksi} \end{aligned} \tag{3-3}$$

Calculate the section modulus of the weld group, S_y (neglecting web weld):

$$\begin{aligned} I_y &= \frac{2 t_{weld} b_f^{\,3}}{12} \\ &= \frac{2(0.707)(5/16 \text{ in.})(12.0 \text{ in.})^3}{12} \\ &= 63.6 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} S_y &= \frac{I_y}{b_f/2} \\ &= \frac{63.6 \text{ in.}^4}{12.0 \text{ in.}/2} \\ &= 10.6 \text{ in.}^3 \end{aligned}$$

The available flexural strength of the weld group is

$$\begin{aligned} \phi M_n &= 0.75(63.0 \text{ ksi})(10.6 \text{ in.}^3)(1 \text{ ft}/12 \text{ in.}) \\ &= 41.7 \text{ kip-ft} \end{aligned}$$

Failure Mode 2—Bending Failure of the Base Plate

Case A: Inset Anchor Rods—Weak-Axis Available Strength

The available strength of the base plate is determined following the procedure given in Section 3.2.2, Case A.

Based on the weld pattern and the geometry provided in Figure 3-19:

$$g_1 = 5.00 \text{ in.}$$
$$g_2 = 5.00 \text{ in.}$$

$$\begin{aligned} d_1 &= \frac{g_1}{2} \\ &= \frac{5.00 \text{ in.}}{2} \\ &= 2.50 \text{ in.} \end{aligned}$$

$$\begin{aligned} d_2 &= \frac{d - g_2 - t_f}{2} \\ &= \frac{12.1 \text{ in.} - 5.00 \text{ in.} - 0.605 \text{ in.}}{2} \\ &= 3.25 \text{ in.} \end{aligned}$$

$$\begin{aligned} b_1 &= 2d_1 \le \frac{d - t_f}{2} \\ &= 2(2.50 \text{ in.}) \le \frac{12.1 \text{ in.} - 0.605 \text{ in.}}{2} \\ &= 5.00 \text{ in.} < 5.75 \text{ in.} \\ &= 5.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} b_2 &= \frac{b_f}{2} \\ &= \frac{12.0 \text{ in.}}{2} \\ &= 6.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} Z_1 &= \frac{b_1 t_p^2}{4} \\ &= \frac{(5.00 \text{ in.})(1 \text{ in.})^2}{4} \\ &= 1.25 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} \phi M_{1n} &= \phi F_y Z_1 \\ &= 0.90(36 \text{ ksi})(1.25 \text{ in.}^3) \\ &= 40.5 \text{ kip-in.} \end{aligned}$$

$$\begin{aligned} Z_2 &= \frac{b_2 t_p^2}{4} \\ &= \frac{(6.00 \text{ in.})(1 \text{ in.})^2}{4} \\ &= 1.50 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned}\phi M_{2n} &= \phi F_y Z_2 \\ &= 0.90(36\text{ ksi})(1.50\text{ in.}^3) \\ &= 48.6\text{ kip-in.}\end{aligned}$$

From Equation 3-4, the available strength for a single anchor rod is:

$$\begin{aligned}\phi P_n &= \frac{\phi M_{1n}}{d_1} + \frac{\phi M_{2n}}{d_2} \qquad (3\text{-}4) \\ &= \frac{40.5\text{ kip-in.}}{2.50\text{ in.}} + \frac{48.6\text{ kip-in.}}{3.25\text{ in.}} \\ &= 31.2\text{ kips}\end{aligned}$$

From Equation 3-5, the available flexural strength of the base plate is:

$$\begin{aligned}\phi M_n &= 2\phi P_n g_1 \qquad (3\text{-}5) \\ &= 2(31.2\text{ kips})(5.00\text{ in.})(1\text{ ft}/12\text{ in.}) \\ &= 26.0\text{ kip-ft}\end{aligned}$$

Failure Mode 3—Tensile Rupture of Anchor Rods

From AISC *Specification* Table J3.2, the nominal tensile strength of each anchor is:

$$\begin{aligned}F_n &= 0.75F_u \\ &= 0.75(58\text{ ksi}) \\ &= 43.5\text{ ksi}\end{aligned}$$

$$\begin{aligned}A_b &= \frac{\pi d_b^2}{4} \\ &= \frac{\pi(\tfrac{3}{4}\text{ in.})^2}{4} \\ &= 0.442\text{ in.}^2\end{aligned}$$

The available tensile rupture strength for each anchor rod is:

$$\begin{aligned}\phi P_n &= \phi F_n A_b \qquad \text{(from } \textit{Spec}\text{. Eq. J3-1)} \\ &= 0.75(43.5\text{ ksi})(0.442\text{ in.}^2) \\ &= 14.4\text{ kips}\end{aligned}$$

From Equation 3-5, the available strength of the base plate based on tensile rupture of the anchor rods is:

$$\begin{aligned}\phi M_n &= 2\phi P_n g_1 \qquad \text{(from Eq. 3-5)} \\ &= 2(14.4\text{ kips})(5.00\text{ in.})(1\text{ ft}/12\text{ in.}) \\ &= 12.0\text{ kip-ft}\end{aligned}$$

Failure Mode 4— Buckling of Anchor Rods

Because the grout thickness does not exceed 5 in. and the anchor rod diameter is greater than or equal to ¾ in., this failure mode does not govern (see Section 3.2.4).

Failure Mode 5—Anchor Rod Nut Pull Through

Use the minimum washer size per AISC *Manual* Table 14-2 to address this failure mode. For ¾-in.-diameter rods, use a ¼"×2"×2"washer plate.

Failure Modes 6 and 7—Anchor Rod Breakout and Pullout

Concrete breakout strength of an anchor rod in tension is determined using Equation 3-18.

$$N_{cbg} = 1.25\psi_{ed,N}\left(\frac{A_{Nc}}{A_{Nco}}\right)N_b \tag{3-18}$$

From Figure 3-20:

$$c_{a,max} = 10\tfrac{1}{2}\text{ in.}$$

$$\begin{aligned} h_{ef} &= \frac{c_{a,max}}{1.5} \geq \frac{s}{3} \\ &= \frac{10\tfrac{1}{2}\text{ in.}}{1.5} \geq \frac{5.00\text{ in.}}{3} \\ &= 7.00\text{ in.} > 1.67\text{ in.} \end{aligned}$$

Therefore, use $h_{ef} = 7.00$ in.

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 \\ &= 9(7.00\text{ in.})^2 \\ &= 441\text{ in.}^2 \end{aligned}$$

Because $h_{ef} < 11$ in., the breakout strength for a single anchor in tension in determined from Equation 3-16a:

$$\begin{aligned} N_b &= 24\sqrt{f_c'}\,h_{ef}^{1.5} \\ &= 24\sqrt{3{,}000\text{ psi}}\,(7.00\text{ in.})^{1.5} \\ &= 24{,}300\text{ lb} \end{aligned} \tag{3-16a}$$

From Figure 3-20:

$$c_{a,min} = 5\tfrac{1}{2}\text{ in.}$$

$$\begin{aligned} 1.5h_{ef} &= 1.5(7.00\text{ in.}) \\ &= 10.5\text{ in.} \end{aligned}$$

Because $c_{a,\,min} < 1.5h_{ef}$, use Equation 3-14b:

$$\begin{aligned} \psi_{ed,N} &= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \\ &= 0.7 + 0.3\left(\frac{5\tfrac{1}{2}\text{ in.}}{10.5\text{ in.}}\right) \\ &= 0.857 \end{aligned} \tag{3-14b}$$

The truncated pyramid area is determined from the geometry shown in Figure 3-20.

$$\begin{aligned} A_{Nc} &= (1.5h_{ef} + c_{s1})(c_{s2} + s_2 + c_{s2}) \\ &= (10.5\text{ in.} + 5\tfrac{1}{2}\text{ in.})(5\tfrac{1}{2}\text{ in.} + 5\text{ in.} + 5\tfrac{1}{2}\text{ in.}) \\ &= 256\text{ in.}^2 \end{aligned}$$

$$N_{cbg} = 1.25\psi_{ed,N}\left(\frac{A_{Nc}}{A_{Nco}}\right)N_b \tag{3-18}$$

$$= 1.25(0.857)\left(\frac{256\text{ in.}^2}{441\text{ in.}^2}\right)(24{,}300\text{ lb})$$

$$= 15{,}100\text{ lb}$$

The available breakout strength per anchor rod is:

$$\phi N_{cbg} = 0.70(15{,}100\text{ lb})(1\text{ kip}/1{,}000\text{ lb})$$
$$= 10.6\text{ kips}$$

The available breakout strength for two anchor rods is:

$$\phi N_{cbg} = 2(10.6\text{ kips})$$
$$= 21.2\text{ kips}$$

Check Hook Bearing Strength

Check if $3d_a \leq e_h \leq 4.5d_a$

d_a = ¾ in.

$3d_a$ = 3(¾ in.)
= 2.25 in.

$4.5d_a$ = 4.5(¾ in.)
= 3.38 in.

e_h = 4 in.

Therefore, use e_h = 3.38 in.

The pullout strength of a single hooked bolt is:

$$N_p = 0.9f'_c e_h d_a \tag{3-22}$$
$$= 0.9(3{,}000\text{ psi})(3.38\text{ in.})(3/4\text{ in.})$$
$$= 6{,}840\text{ lb}$$

The nominal pullout strength is:

$\psi_{c,P}$ = 1.4 (for uncracked concrete)

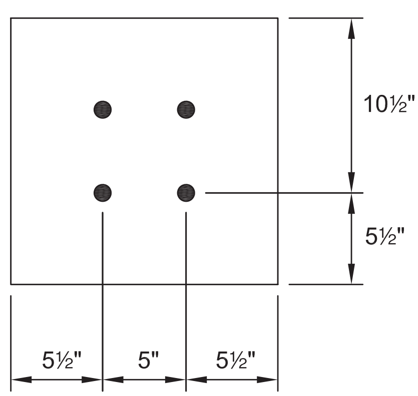


Fig. 3-20. Dimensions of truncated pyramid for Example 3.3.1.

$$
\begin{aligned}
N_{pn} &= \psi_{c,P} N_p \\
&= 1.4(6{,}840 \text{ lb}) \\
&= 9{,}580 \text{ lb}
\end{aligned}
\tag{3-20}
$$

The available pullout strength for two anchor rods is:

$$
\begin{aligned}
\phi N_{pn} &= 2(0.70)(9{,}580 \text{ lb})(1 \text{ kip}/1{,}000 \text{ lb}) \\
&= 13.4 \text{ kips}
\end{aligned}
$$

From Equation 3-23, the available flexural strength based on anchor rod pullout is

$$
\begin{aligned}
\phi M_n &= \phi N_{pn} d \\
&= (13.4 \text{ kips})(5.00 \text{ in.})(1 \text{ ft}/12 \text{ in.}) \\
&= 5.58 \text{ kip-ft}
\end{aligned}
\tag{from Eq. 3-23}
$$

Failure Mode 8—Anchor Rod Pushout

Anchor rod push through will not occur due to the presence of the pier.

Failure Mode 9—Pier Bending

Determine the depth of the compression area:

$$
\begin{aligned}
a &= \frac{F_y A_s}{0.85 f'_c b} \\
&= \frac{(60{,}000 \text{ psi})(2)(0.442 \text{ in.}^2)}{0.85(3{,}000 \text{ psi})(16 \text{ in.})} \\
&= 1.30 \text{ in.}
\end{aligned}
\tag{3-27}
$$

$$
\begin{aligned}
C &= 0.85 f'_c ab \\
&= 0.85(3{,}000 \text{ psi})(1.30 \text{ in.})(16 \text{ in.})(1 \text{ kip}/1{,}000 \text{ lb}) \\
&= 53.0 \text{ kips}
\end{aligned}
\tag{3-25}
$$

The available flexural strength of the pier is determined as follows, where d is the effective depth for the reinforcing bars based on clear cover and cage dimensions (see Figure 3-18).

$$
\phi M_n = C\left(d - \frac{a}{2}\right) \tag{3-28}
$$

The available flexural strength of the pier is:

$$
\begin{aligned}
\phi M_n &= 0.90(53.0 \text{ kips})\left(13\tfrac{3}{4} \text{ in.} - \frac{1.30 \text{ in.}}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\
&= 52.1 \text{ kip-ft}
\end{aligned}
$$

Check Reinforcing Development Length

The required reinforcing bar length in the pier is checked as follows for hooked bars:

$$
l_{dh} = \left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}}\right) d_b^{1.5} \geq 8d_b \text{ and } 6 \text{ in.} \tag{3-29}
$$

$$\psi_e = \frac{f'_c}{15{,}000} + 0.6$$

$$= \frac{3{,}000 \text{ psi}}{15{,}000} + 0.6$$

$$= 0.8$$

$$l_{dh} = \left[\frac{(60{,}000 \text{ psi})(0.8)(1.0)(1.0)(1.0)}{55(1.0)\sqrt{3{,}000 \text{ psi}}} \right] (\tfrac{3}{4} \text{ in.})^{1.5}$$

$$= 10.3 \text{ in.} < 12 \text{ in.} \quad \textbf{o.k.}$$

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

$$l_d = \left(\frac{f_y \psi_t \psi_e \psi_g}{25\lambda\sqrt{f'_c}} \right) d_b \geq 12 \text{ in.} \tag{3-30a}$$

$$= \left[\frac{(60{,}000 \text{ psi})(1.0)(1.0)(1.0)}{25(1.0)\sqrt{3{,}000 \text{ psi}}} \right] (\tfrac{3}{4} \text{ in.})$$

$$= 32.9 \text{ in.} < 33 \text{ in.} \quad \textbf{o.k.}$$

Failure Mode 10—Footing Overturning

The overturning resistance is determined using Equation 3-31:

$$M_o = W\left(\frac{L}{2}\right) \tag{3-31}$$

For the weight of superimposed loads:

$$P_1 = (65 \text{ lb/ft})(40 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 2.60 \text{ kips (column)}$$

For the weight of the pier:

$$P_2 = (150 \text{ lb/ft}^3)(1.33 \text{ ft})(1.33 \text{ ft})(3.00 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 0.796 \text{ kip (pier)}$$

For the weight of the footing:

$$P_3 = (150 \text{ lb/ft}^3)(1.25 \text{ ft})(6 \text{ ft})(6 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 6.75 \text{ kips (footing)}$$

$$W = P_1 + P_2 + P_3 \tag{3-32}$$

$$= 2.60 \text{ kips} + 0.796 \text{ kip} + 6.75 \text{ kips}$$

$$= 10.1 \text{ kips}$$

$$M_o = (10.1 \text{ kips})\left(\frac{6 \text{ ft}}{2}\right)$$

$$= 30.3 \text{ kip-ft}$$

$$M_{nOT} = 0.9M_o \tag{3-35a}$$

$$= 0.9(30.3 \text{ kip-ft})$$

$$= 27.3 \text{ kip-ft}$$

Comparing these failure modes, the controlling case is anchor rod pullout. The overturning resistance of the column is, therefore, equal to $\phi M_n = 5.58$ kip-ft.

Example 3.3.2—Overturning of a Free-Standing Column with Outset Anchor Rods

Given:

Repeat Example 3.3.1 using outset ¾-in. anchor rods with embedded nuts. Increase the pier size to 1'-8"×1'-8" to accommodate the base plate. Increase the vertical reinforcement to eight #6 bars. The distance from the anchor rod to the flange tip, L, is 2.83 in. as shown in Figure 3-21. The anchor rod layout is given in Figure 3-22(a) and the pier reinforcing layout is shown in Figure 3-22(b).

Solution:

Failure Mode 1—Fracture of the Weld

From Example 3.3.1:

$$\phi M_n = 41.7 \text{ kip-ft}$$

Failure Mode 2—Bending Failure of the Base Plate

Case B: Outset Anchor Rods

Failure based on plate effective width (see Figure 3-21):

$$b_e = 2L \leq 5 \text{ in.}$$

$$= 2(2.83 \text{ in.})$$

$$= 5.66 \text{ in.} > 5 \text{ in.}$$

Therefore, use $b_e = 5$ in.

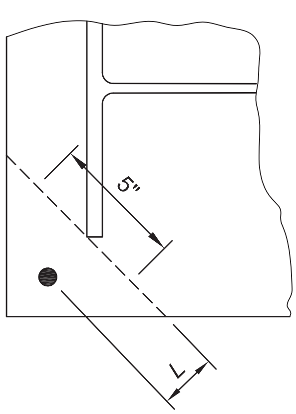
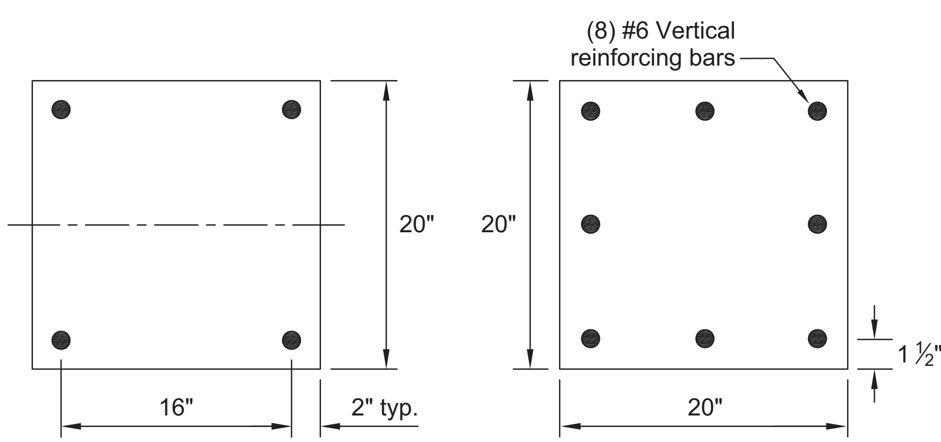


Fig. 3-21. Base plate yield line for Example 3.3.2.



(a) Anchor rod layout

(b) Pier reinforcing layout

Fig. 3-22. Anchor rod and pier reinforcing layout for Example 3.3.2.

The available strength for a single anchor rod based on the plate effective width is:

$$\phi P_n = \frac{\phi F_y b_e t_p^2}{4L} \quad \text{(from Eq. 3-6)}$$

$$= \frac{0.90(36 \text{ ksi})(5 \text{ in.})(1 \text{ in.})^2}{4(2.83 \text{ in.})}$$

$$= 14.3 \text{ kips}$$

The available strength for a single anchor rod based on weld strength is:

$$\phi P_n = \phi_w F_w t_{weld}(2 \text{ in.}) \quad \text{(from Eq. 3-7)}$$

$$= 0.75(70 \text{ ksi})(0.707)(\tfrac{5}{16} \text{ in.})(2 \text{ in.})$$

$$= 23.2 \text{ kips}$$

The available strength for a single anchor rod based on weld strain is:

$$\phi P_n = \phi_w 50 t_{weld}(t_p)^{1.5} \quad \text{(from Eq. 3-8)}$$

$$= 0.90(50)(0.707)(\tfrac{5}{16} \text{ in.})(1 \text{ in.})^{1.5}$$

$$= 9.94 \text{ kips (controls)}$$

Therefore, the available flexural strength of the base plate is:

$$\phi M_n = 2\phi P_n d \quad \text{(from Eq. 3-9)}$$

$$= 2(9.94 \text{ kips})(16.0 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$

$$= 26.5 \text{ kip-ft}$$

Failure Mode 3—Tensile Rupture of Anchor Rods

From Example 3.3.1, the available tensile rupture strength for each anchor rod is:

$$\phi P_n = 14.4 \text{ kips/rod}$$

The available flexural strength of the base plate is:

$$\phi M_n = 2\phi P_n d \quad \text{(from Eq. 3-12)}$$

$$\phi M_n = 2(14.4 \text{ kips/rod})(16 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$

$$= 38.4 \text{ kip-ft}$$

Failure Mode 4—Buckling of Anchor Rods

Because the grout thickness does not exceed 5 in. and the anchor rod diameter is greater than or equal to ¾ in., this failure mode does not govern (see Section 3.2.4).

Failure Mode 5—Anchor Rod Nut Pull Through

Use the minimum washer size according to AISC *Manual* Table 14-2 to address this failure mode. For ¾-in.-diameter rods, use a ¼"×2"×2" washer plate.

Failure Modes 6 and 7—Anchor Rod Breakout and Pullout

As shown in Figure 3-22, the maximum spacing between anchor rods is s =16.0 in.

Concrete Breakout Strength of Anchor Rods in Tension

$$\begin{aligned} A_{Nc} &= (18 \text{ in.} + 2 \text{ in.})(2 \text{ in.} + 16 \text{ in.} + 2 \text{ in.}) \\ &= 400 \text{ in.}^2 \end{aligned}$$

From Figure 3-22:

$$c_{a,max} = 18 \text{ in.}$$

The value of $c_{a,max}$ must be less than or equal to $1.5h_{ef}$; therefore, determine h_{ef}, the effective embedment depth of the anchor, as follows.

$$\begin{aligned} h_{ef} &= \frac{c_{a,max}}{1.5} \geq \frac{s}{3} \\ &= \frac{18 \text{ in.}}{1.5} \geq \frac{16 \text{ in.}}{3} \\ &= 12.0 \text{ in.} > 5.33 \text{ in.} \end{aligned}$$

Therefore, use h_{ef} = 12.0 in.

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 \\ &= 9(12.0 \text{ in.})^2 \\ &= 1{,}300 \text{ in.}^2 \end{aligned}$$

Because 11 in. < h_{ef} < 23 in., the breakout strength for a single anchor in tension in determined from Equation 3-16b:

$$N_b = 16\sqrt{f_c'}\,(h_{ef})^{5/3} \tag{3-16b}$$

$$\begin{aligned} &= 16\sqrt{3{,}000 \text{ psi}}\,(12.0 \text{ in.})^{5/3} \\ &= 55{,}100 \text{ lb} \end{aligned}$$

From Figure 3-22:

$$c_{a,min} = 2 \text{ in.}$$

$$\begin{aligned} 1.5h_{ef} &= 1.5(12.0 \text{ in}) \\ &= 18.0 \text{ in.} \end{aligned}$$

Because $c_{a,min} < 1.5h_{ef}$, use Equation 3-14b:

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \tag{3-14b}$$

$$\begin{aligned} &= 0.7 + 0.3\left(\frac{2 \text{ in.}}{18.0 \text{ in.}}\right) \\ &= 0.733 \end{aligned}$$

For piers:

$$N_{cbg} = 1.25\psi_{ed,N}\left(\frac{A_{Nc}}{A_{Nco}}\right)N_b \tag{3-18}$$

$$\begin{aligned} &= 1.25(0.733)\left(\frac{400 \text{ in.}^2}{1{,}300 \text{ in.}^2}\right)(55{,}100 \text{ lb}) \\ &= 15{,}500 \text{ lb} \end{aligned}$$

The available breakout strength per anchor rod is:

$$\phi N_{cbg} = 0.70(15{,}500 \text{ lb})(1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 10.9 \text{ kips}$$

The available breakout strength for two anchor rods is:

$$\phi N_{cbg} = 2(10.9 \text{ kips})$$
$$= 21.8 \text{ kips}$$

ACI 318-19 also requires that the available pullout strength based on Equation 3-20 be checked.

From Table 3-1a, the headed anchor concrete pullout strength is:

$$\phi N_p = 15.3 \text{ kips/rod}$$

The nominal pullout strength is:

$$\psi_{c,p} = 1.4 \text{ (for uncracked concrete)}$$

$$\phi N_{pn} = \psi_{c,p}\phi N_p \qquad \text{(from Eq. 3-20)}$$
$$= 1.4(15.3 \text{ kips/rod})$$
$$= 21.4 \text{ kips/rod}$$

The available pullout strength for two anchor rods is:

$$\phi N_{pn} = 2(21.4 \text{ kips})$$
$$= 42.8 \text{ kips}$$

Comparing all failure modes, the controlling case is anchor rod breakout. The available flexural strength is determined from Equation 3-23, where $N_{cbg} = 2T_n$:

$$\phi M_n = \phi N_{cbg} d \qquad \text{(from Eq. 3-23)}$$
$$= (21.8 \text{ kips})(16.0 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$
$$= 29.1 \text{ kip-ft}$$

Failure Mode 8—Anchor Rod Pushout

Anchor rod pushout will not occur due to the presence of the pier.

Failure Mode 9—Pier Bending

Determine the depth of the compression area:

$$a = \frac{3F_y A_b}{0.85 f'_c b} \qquad \text{(from Eq. 3-27)}$$
$$= \frac{3(60{,}000 \text{ psi})(0.442 \text{ in.}^2)}{(0.85)(3{,}000 \text{ psi})(20 \text{ in.})}$$
$$= 1.56 \text{ in.}$$

$$C = 0.85 f'_c ab \qquad \text{(3-25)}$$
$$= 0.85(3{,}000 \text{ psi})(1.56 \text{ in.})(20 \text{ in.})(1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 79.6 \text{ kips}$$

From the given geometry, $d = 17\frac{3}{4}$ in.

$$M_n = C\left(d - \frac{a}{2}\right) \qquad \text{(from Eq. 3-28)}$$

$$= (79.6 \text{ kips})\left(17\tfrac{3}{4} \text{ in.} - \frac{1.56 \text{ in.}}{2}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 113 \text{ kip-ft}$$

The available flexural strength is:

$$\phi M_n = 0.90(113 \text{ kip-ft})$$

$$= 102 \text{ kip-ft}$$

Check Reinforcing Development Length

The development length is verified in Example 3.3.1.

Failure Mode 10—Footing Overturning

$$M_o = W\left(\frac{L}{2}\right) \qquad \text{(3-31)}$$

For the weight of superimposed loads:

$$P_1 = (65 \text{ lb/ft})(40 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 2.60 \text{ kips (column)}$$

For the weight of the pier:

$$P_2 = (150 \text{ lb/ft}^3)(1.67 \text{ ft})(1.67 \text{ ft})(3 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 1.26 \text{ kips (pier)}$$

For the weight of the footing:

$$P_3 = (150 \text{ lb/ft}^3)(6 \text{ ft})(6 \text{ ft})(1.25 \text{ ft})(1 \text{ kip/1,000 lb})$$

$$= 6.75 \text{ kips (footing)}$$

$$W = P_1 + P_2 + P_3 \qquad \text{(3-32)}$$

$$= 2.60 \text{ kips} + 1.26 \text{ kips} + 6.75 \text{ kips}$$

$$= 10.6 \text{ kips}$$

$$M_o = (10.6 \text{ kips})\left(\frac{6 \text{ ft}}{2}\right)$$

$$= 31.8 \text{ kip-ft}$$

$$M_{nOT} = 0.9M_o \qquad \text{(3-35a)}$$

$$= 0.9(31.8 \text{ kip-ft})$$

$$= 28.6 \text{ kip-ft}$$

Comparing these failure modes, the controlling case is weld strain. The available flexural strength is $\phi M_n = 26.5$ kip-ft.

Example 3.3.3—Overturning of a Free-Standing Column with Inset Anchor Rods Utilizing the Tables Provided in the Appendix

Given:

Repeat Example 3.3.1 using the tables in the Appendix.

Solution:

Failure Mode 1—Fracture of the Weld

From Table A-1, for a W12×65, the available flexural strength of the base plate with inset anchor rods based on weld strength is:

$$\phi M_{ny} = 41.8 \text{ kip-ft}$$

Failure Mode 2—Bending Failure of the Base Plate

From Table A-2, for a W12×65 with an anchor rod spacing of 5"×5", and a 1"×13"×13" base plate, the available flexural strength of the base plate is:

$$\phi M_{nx} = 26.0 \text{ kip-ft}$$

Failure Mode 3—Tensile Rupture of Anchor Rods

From Table A-5, for a ¾-in. ASTM F1554 Grade 36 anchor rod, the available tensile strength is:

$$\phi P_n = 14.4 \text{ kips}$$

The available flexural strength of the base plate is:

$$\phi M_n = 2\phi P_n d \qquad \text{(from Eq. 3-12)}$$

$$= 2(14.4 \text{ kips})(5.00 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$

$$= 12.0 \text{ kip-ft}$$

Failure Mode 4—Buckling of Anchor Rods

This failure mode does not govern, see Section 3.2.4.

Failure Mode 5—Anchor Rod Nut Pull Through

Use the minimum washer size according to AISC *Manual* Table 14-2 to address this failure mode. For ¾-in.-diameter rods, use a ¼"×2"×2" washer plate.

Failure Modes 6 and 7—Anchor Rod Breakout and Pullout

The procedures presented in Section 3.2.6 must be used because tables are not available for the anchor rod breakout in piers. From Example 3.3.1:

$$\phi N_{cbg} = 21.2 \text{ kips}$$

From Table A-6a, the available concrete pullout strength due to a ¾-in. rod with a 4-in. hook, in tension, is:

$$\phi P_n = 6.70 \text{ kips}$$

Therefore, the flexural resistance is controlled by anchor rod pullout. The available flexural strength is:

$$\phi M_n = 2\phi P_n d \qquad \text{(from 3-23)}$$

$$= 2(6.70 \text{ kips})(5 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$

$$= 5.58 \text{ kip-ft}$$

Failure Mode 8—Anchor Rod Pushout

Anchor rod pushout will not occur due to the presence of the pier.

Failure Mode 9—Pier Bending

The reinforcement ratio for the 1'-4"×1'-4" pier with four #6 bars is:

$$\left[\frac{4\left(0.442 \text{ in.}^2\right)}{(16 \text{ in.})^2}\right](100) = 0.691\%$$

From Table A-14, the available flexural strength for a pier with 0.5% reinforcing is:

$$\phi M_n = 51.4 \text{ kip-ft}$$

The development length of the reinforcing must also be checked. From Table A-16, for #6 hooked bars the required development length is 11 in. The provided length of 12 in. is sufficient.

From Table A-16, the required straight bar development length is 33 in., which is less than the 36-in. length provided.

Failure Mode 10—Footing Overturning

From Table A-15, the available overturning resistance for the 6'-0"×6'-0"×1'-3" footing can be conservatively based on the table value for a 6'-0"×6'-0"×1'-2" footing (not including the weight of the column and pier).

$$M_o = 18.9 \text{ kip-ft}$$

$$M_{nOT} = \gamma_d M_o \qquad \text{(3-34)}$$

$$= 0.9(18.9 \text{ kip-ft})$$

$$= 17.0 \text{ kip-ft}$$

Because the failure mode was based on conservative values taken from Table A-15, which do not include the pier or column weight, the results differ from the more exact calculation in Example 3.3.1:

$$M_{nOT} = 27.3 \text{ kip-ft}$$

Comparing all failure modes, the controlling case is anchor rod pullout, and $\phi M_n = 5.58$ kip-ft.

Example 3.3.4—Overturning of a Free-Standing Column with Outset Anchor Rods Utilizing the Tables Provided in the Appendix

Given:

Repeat Example 3.3.2 using the tables in the Appendix.

Solution:

Failure Mode 1—Fracture of the Weld

From Table A-1, for a W12×65, the available flexural strength of the base plate with outset anchor rods based on weld strength is:

$\phi M_{ny} = 41.8$ kip-ft

Failure Mode 2—Bending Failure of the Base Plate

From Table A-3a, for a W12×65 with an anchor rod spacing of 16"×16" and a 1-in.-thick base plate, the available flexural strength of the base plate with outset anchor rods is:

$\phi M_n = 26.5$ kip-ft

Failure Mode 3—Tensile Rupture of Anchor Rods

From Table A-5, for a ¾-in. ASTM F1554 Grade 36 anchor rod, the available tensile strength is:

$\phi P_n = 14.4$ kips/rod

The available flexural strength of the base plate is:

$$\phi M_n = 2\phi P_n d \qquad \text{(from Eq. 3-12)}$$

$$= 2(14.4 \text{ kips})(16 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$

$$= 38.4 \text{ kip-ft}$$

Failure Mode 4— Buckling of Anchor Rods

Because the grout thickness does not exceed 5 in. and the anchor rod diameter is greater than or equal to ¾ in., this failure mode does not govern (see Section 3.2.4).

Failure Mode 5—Anchor Rod Nut Pull Through

Use the minimum washer size according to AISC *Manual* Table 14-2 to address this failure mode. For ¾-in.-diameter rods, use a ¼"×2"×2" washer plate.

Failure Modes 6 and 7—Anchor Rod Breakout and Pullout

From Example 3.3.2, the available flexural strength due to anchor rod breakout and pullout is:

$\phi M_n = 29.1$ kip-ft

Failure Mode 8—Anchor Rod Pushout

Anchor rod pushout does not occur due to the presence of the pier.

Failure Mode 9—Pier Bending

From Example 3.3.2, the available flexural strength is:

$\phi M_n = 102$ kip-ft

Check Reinforcing Development Length

The development length is verified in Example 3.3.2.

Failure Mode 10—Footing Overturning

From Table A-15, the available overturning resistance for the 6'-0"×6'-0"×1'-3" footing can be conservatively based on the table value for a 6'-0"× 6'-0"×1'-2" footing (not including the weight of the column and pier).

$$M_o = 18.9 \text{ kip-ft}$$

$$\begin{aligned} M_{nOT} &= \gamma_d M_o \qquad (3\text{-}34) \\ &= 0.9(18.9 \text{ kip-ft}) \\ &= 17.0 \text{ kip-ft} \end{aligned}$$

Because the failure mode was based on conservative values taken from Table A-15, which do not include the pier or column weight, the results differ from the more exact calculation performed in Example 3.3.2:

$$M_{nOT} = 28.6 \text{ kip-ft}$$

Comparing these failure modes, the controlling failure mode is flexure of the base plate, and $\phi M_n = 26.5$ kip-ft.

Example 3.3.5—Wind Loads on Columns

Given:

For the column/pier/footing detail provided in Example 3.3.1, determine if a W12×65 column could safely resist the overturning moment from an anticipated 35-mph wind using column heights of 25 ft and 40 ft. Use Exposure B conditions. The wind load is not applied to the pier or footing.

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

Solution:

Wind Calculations

The design wind force is determined as:

$$F = q_h G C_f A_s \qquad \text{(ASCE/SEI 7-10 Eq. 29.4-1)}$$

From ASCE/SEI 7-10, Table 29.3-1, Exposure B:

$K_z = 0.66$ at the 25-ft elevation

From ASCE/SEI 7-10, Section 26.8.2:

$$K_{zt} = 1.0$$

From ASCE/SEI 7-10, Table 26.6-1:

$$K_d = 0.85$$

For design, convert the expected wind speed to a 3-second gust according to ASCE/SEI 37-14, Section 6.2.1.2.

$$\begin{aligned} V &= 1.2(35 \text{ mph})(1.26) \\ &= 52.9 \text{ mph} \end{aligned}$$

$$\begin{aligned} q_h &= 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE/SEI 7-10 Eq. 29.3-1)} \\ &= 0.00256(0.66)(1.0)(0.85)(52.9 \text{ mph})^2 \\ &= 4.02 \text{ lb/ft}^2 \end{aligned}$$

The gust-effect factor from ASCE/SEI 7-10, Section 26.9, is:

$$G = 0.85$$

The force coefficient from ASCE/SEI 7-10, Figure 29.4-1, with $s/h = 1$ and $B/s \le 0.05$ is:

$$C_f = 1.80$$

For the 25-ft column

The gross area of the 25-ft column is:

$$\begin{aligned} A_s &= (25 \text{ ft})(12.1 \text{ in.})(1 \text{ ft}/12 \text{ in.}) \\ &= 25.2 \text{ ft}^2 \end{aligned}$$

The design wind force is:

$$\begin{aligned} F &= q_h G C_f A_s && \text{(ASCE/SEI 7-10 Eq. 29.4-1)} \\ &= (4.02 \text{ lb/ft}^2)(0.85)(1.80)(25.2 \text{ ft}^2) \\ &= 155 \text{ lb} \end{aligned}$$

The lever arm, L, for overturning at the column base is the location of F shown in the cross-section view in ASCE/SEI 7-10, Figure 29.4-1:

$$\begin{aligned} L &= \frac{25 \text{ ft}}{2} + (0.05)(25 \text{ ft}) \\ &= 13.8 \text{ ft} \end{aligned}$$

The overturning moment is:

$$\begin{aligned} M_{rOT} &= FL \\ &= (155 \text{ lb})(13.8 \text{ ft})(1 \text{ kip}/1{,}000 \text{ lb}) \\ &= 2.14 \text{ kip-ft} \end{aligned}$$

The overturning moment is compared to the governing available flexural strength (from Example 3.3.1):

$\phi M_n = 5.58 \text{ kip-ft} > 2.14 \text{ kip-ft}$ **o.k.**

From ASCE/SEI 7-10, Figure 29.4-1, the lever arm, L, for overturning of the 1.25-foot-thick footing:

$$\begin{aligned} L &= \frac{25 \text{ ft}}{2} + (0.05)(25 \text{ ft}) + 3.00 \text{ ft} + 1.25 \text{ ft} \\ &= 18.0 \text{ ft} \end{aligned}$$

$$\begin{aligned} M_{rOT} &= FL \\ &= (155 \text{ lb})(18.0 \text{ ft})(1 \text{ kip}/1{,}000 \text{ lb}) \\ &= 2.79 \text{ kip-ft} \end{aligned}$$

The overturning moment is compared to the overturning resistance calculated in Example 3.3.1.

$M_{nOT} = 27.3 \text{ kip-ft} > 2.79 \text{ kip-ft}$ **o.k.**

For the 40-ft column

The gross area of the 40-ft column is:

$$A_s = (40 \text{ ft})(12.1 \text{ in.})(1 \text{ ft}/12 \text{ in.})$$
$$= 40.3 \text{ ft}^2$$

From ASCE/SEI 7-10, Table 29.3-1, Exposure B:

$K_z = 0.76$ at the 40-ft elevation

$$q_h = 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(ASCE/SEI 7-10 Eq. 29.3-1)}$$
$$= 0.00256(0.76)(1.0)(0.85)(52.9 \text{ mph})^2$$
$$= 4.63 \text{ lb/ft}^2$$

The design wind force is:

$$F = q_h G C_f A_s \qquad \text{(ASCE/SEI 7-10 Eq. 29.4-1)}$$
$$= (4.63 \text{ lb/ft}^2)(0.85)(1.80)(40.3 \text{ ft}^2)$$
$$= 285 \text{ lb}$$

The lever arm, L, for overturning at the column base is the location of F shown in the cross-section view in ASCE/SEI 7-10, Figure 29.4-1:

$$L = \frac{40 \text{ ft}}{2} + (0.05)(40 \text{ ft})$$
$$= 22.0 \text{ ft}$$

$$M_{rOT} = FL$$
$$= (285 \text{ lb})(22.0 \text{ ft})(1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 6.27 \text{ kip-ft}$$

The overturning moment is compared to the overturning resistance of the 40-ft-long W12×65 column, which is equal to the governing available flexural strength (from Example 3.3.1):

$\phi M_n = 5.58$ kip-ft < 6.27 kip-ft **n.g.**

From ASCE/SEI 7-10, Figure 29.4-1, the lever arm, L, for overturning of the 1.25-ft-thick footing:

$$L = \frac{40 \text{ ft}}{2} + (0.05)(40 \text{ ft}) + 3.00 \text{ ft} + 1.25 \text{ ft}$$
$$= 26.3 \text{ ft}$$

$$M_{rOT} = FL$$
$$= (285 \text{ lb})(26.3 \text{ ft})(1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 7.50 \text{ kip-ft}$$

The overturning moment is compared to the overturning resistance calculated in Example 3.3.1.

$M_{nOT} = 27.3$ kip-ft > 7.50 kip-ft **o.k.**

Therefore, the W12×65 column can safely resist the 35-mph wind at a height of 25 ft but not at a height of 40 ft.

Example 3.3.6—Erection Loads on Column Pier and Footing

Given:

Verify that the column from Example 3.3.5 can safely support a 300-lb live load located 18 in. off of the column face.

Solution:

From AISC *Manual* Table 1-1, the flange width for the W12×65 column is:

$b_f = 12.0$ in.

For LRFD, the factored live load is:

$$P_u = 1.6(300 \text{ lb})$$
$$= 480 \text{ lb}$$

$$M_u = P_u\left(\frac{b_f}{2} + 18 \text{ in.}\right)$$
$$= (480 \text{ lb})\left(\frac{12.0 \text{ in.}}{2} + 18 \text{ in.}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)\left(\frac{1 \text{ kip}}{1{,}000 \text{ lb}}\right)$$
$$= 0.960 \text{ kip-ft}$$

The moment is compared to the overturning resistance of the column in Example 3.3.1, which is controlled by the available anchor rod pullout strength from Example 3.3.1:

$\phi M_n = 5.58 \text{ kip-ft} > M_u = 0.960 \text{ kip-ft}$ **o.k.**

The footing is adequate.

3.4 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 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.

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

1. Tensile strength
2. Compressive strength
3. Connection strength

3.4.1 Wide-Flange Beam Tensile Strength

The tensile strength of any wide-flange beam acting as a tie member will typically not require detailed evaluation. The 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 strength of a tie member must be determined, it can be determined as the lesser value of the following:

From AISC *Specification* Section D2, the nominal tensile yielding strength of the gross section is:

$$P_n = F_y A_g \qquad (\textit{Spec.} \text{ Eq. D2-1})$$
$$\phi_t = 0.90$$
$$\Omega_t = 1.67$$

From AISC *Specification* Section D2, the nominal tensile rupture strength of the net section is:

Table 3-2. Wide-Flange Available Compressive Strength About Weak Axis, F_y = 36 ksi			
Span, ft	Nominal Depth*, in.	LRFD, kips	ASD, kips
20	14	20.0	13.3
25	16	20.0	13.3
30	18	25.0	16.7
35	21	25.0	16.7
40	24	25.0	16.7
45	27	60.0	40.0
50	30	65.0	43.3
* based on lightest wide-flange shape			

Table 3-3. Available Tensile Strength of Connection to a Wide-Flange Beam			
Connection Type	LRFD, kips	ASD, kips	Controlling Element
Beams on columns	35.8	23.8	Bolt shear
¼-in. framing angles	6.53	4.35	Framing angles
⁵⁄₁₆-in. framing angles	10.2	6.78	Framing angles
⅜-in. framing angles	14.7	9.78	Framing angles
¼-in. single-plate shear connections	38.5	23.8	Bolt shear

$$P_n = F_u A_e \qquad (Spec.\ \text{Eq. D2-2})$$
$$\phi_t = 0.75$$
$$\Omega_t = 2.00$$

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

3.4.2 Wide-Flange Beam Compressive Strength

For compression loading, wide-flange tie beams can buckle because they are not laterally supported. Weak-axis buckling strength for the lightest ASTM A36 wide-flange shapes for the depths and spans given are shown in Table 3-2. The values are based on AISC *Specification* Chapter E. These values cannot exceed the connection value for the type of connection used.

The compressive strength for specific wide-flange beams can be determined using AISC *Specification* Chapter E and the AISC *Manual*.

3.4.3 Wide-Flange Beam Connection Strength

Common connection types are shown in Figure 3-23 and consist of:

1. Beams resting on column tops
2. Framing angle connections (double angles)
3. Single-plate shear connections

Presented in Table 3-3 are strengths for these connections. These strengths are based on the installation of two ¾-in.-diameter ASTM F3125 Grade A325 bolts snug tight in each connection. An edge distance of 1½ in. is used for bolt shear conditions. The framing angle strengths are based on the prying action strength using a 1½-in. length from the angle face to the bolt line and a 3-in. angle length (see AISC *Manual* Part 9). ASTM A36 material was used for the framing angles and the single-plate shear connection. The controlling element is also indicated.

The axial strength of tie beams or tie joists can be significantly increased once the decking is installed and connected. Additional strength is also obtained once a concrete slab is poured. These strength increases can be used to stabilize the structure during construction based on calculations and the timing of decking installation.

3.4.4 Steel Joist Tensile Strength

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

Table 3-4. Joist Available Compressive Strength				
Span, ft	Joist Designation	Rows of Bridging	LRFD, kips	ASD, kips
20	10K1	2	10.8	7.20
25	14K1	2	7.00	4.60
30	18K3	3	11.1	7.40
35	20K4	3	11.0	7.33
40	20K5	4	11.1	7.40
45	26K5	4	11.1	7.40
50	28K7	4	11.1	7.40

$$T_n = F_y A_g$$
$$\phi_t = 0.90$$
$$\Omega_t = 1.67$$

It is suggested that only the top chord area be used for A_g 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 $F_y = 50$ ksi.

3.4.5 Steel Joist Compressive Strength

Because the available compressive strength of unbridged joists is low, unbridged 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 lb range. Once the joists are bridged they have considerably greater compressive strength. Compressive strengths are shown in Table 3-4 for several spans and joist sizes. The bridging is in place and anchored. Compressive available strengths for other spans and joist sizes can be obtained from the joist supplier.

3.4.6 Steel Joist Connection Strength

Tie joists are typically connected to column tops using two ½-in.-diameter ASTM A307 bolts. Many erectors also weld the joists to their supports using the Steel Joist Institute's minimum weld requirements (two ⅛-in. fillet welds 2 in. long). Because most joist manufacturers supply long-slotted holes in the joist seats, welding is required to hold the joists in place. The available shear strength for the two ⅛-in. fillet welds is 11.1 kips (LRFD) and 7.42 kips (ASD), based on using 70-ksi weld 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 shear strength for other weld sizes can be determined from the AISC *Specification*. For 70-ksi electrodes the available shear strength per inch of weld length using LRFD can be calculated by multiplying the fillet weld size in sixteenths by 1.392. The available shear strength using ASD is determined by multiplying the fillet weld size in sixteenths by 0.928.

3.4.7 Joist Girder Tensile Strength

Many of the same comments for joists also apply to joist girders acting as tension ties. Connection strength will again typically control the design.

3.4.8 Joist Girder Compressive Strength

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

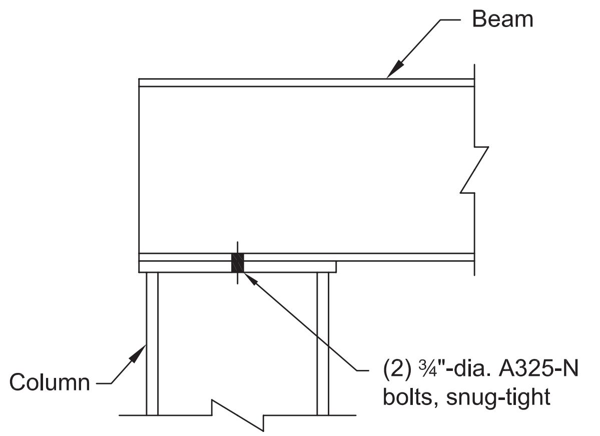
LRFD available compressive strength values for joist girders with top chord angles are shown in Table 3-5a. Table 3-5b provides the ASD values. In all cases the minimum available thickness of the angles has been assumed in calculating the values provided in the table.

3.4.9 Joist Girder Connection Strength

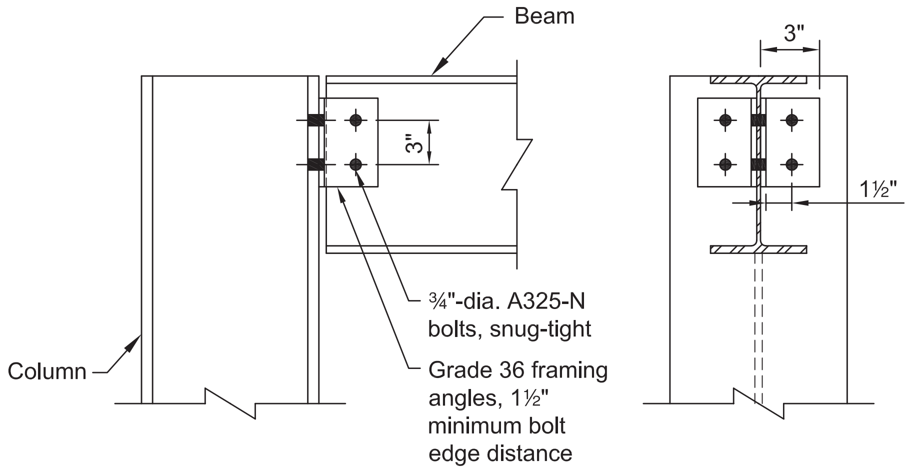
Tie joist girders are typically connected to column tops using two ¾-in.-diameter F3125, Grade A325 bolts. The minimum size SJI welds consist of two ¼-in. fillet welds 2 in. long. Long-slotted holes are generally provided in the joist girder seats. The available shear strength for two ¼-in. fillet welds is 22.3 kips (LRFD) and 14.8 kips (ASD), based on using 70-ksi weld electrodes.

3.5 USE OF PERMANENT BRACING

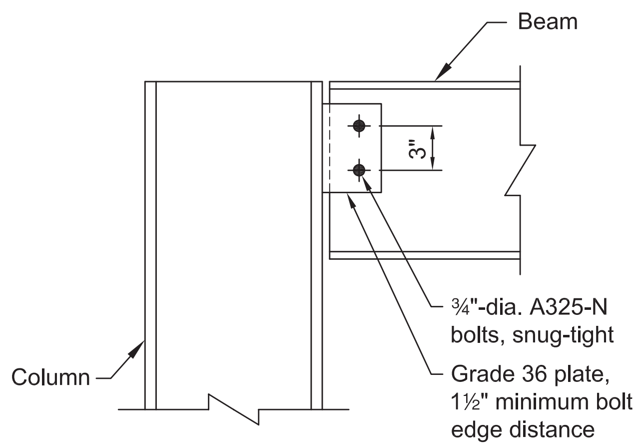
The design procedure for temporary bracing can be applied to permanent bracing used as a part of the temporary bracing scheme. It involves the determination of a design lateral



(a) Beam resting on top of column



(b) Double-angle connection



(c) Single shear-plate connection

Fig. 3-23. Common wide-flange beam-to-column connections.

Table 3-5a. Joist Girder Available Compressive Strength (LRFD), kips						
	Top Chord Angle Leg Length, in.					
Span, ft	**2½**	**3**	**3½**	**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 3-5b. Joist Girder Available Compressive Strength (ASD), kips						
	Top Chord Angle Leg Length, in.					
Span, ft	**2½**	**3**	**3½**	**4**	**5**	**6**
30	2.0	4.0	8.0	12.0	28.7	49.3
35	1.3	2.7	6.0	8.7	21.3	36.7
40	1.3	2.0	4.7	6.7	16.0	28.0
45	0.7	1.3	3.3	5.3	12.7	22.0
50	0.7	1.3	2.7	4.0	10.7	18.0
55	—	1.3	2.7	3.3	8.7	14.7
60	—	—	2.0	2.7	7.3	12.7

force (wind or stability) and confirmation of adequate resistance. The design procedure is illustrated in the following examples. The difference in stiffness of the bracing should be considered when designing the use of temporary and permanent bracing on the same frame line.

The designer of the temporary support system may also want to consider the stability provided by connections that are generally assumed to be simple connections installed during erection of the structure. The strength of these connections would need to be calculated as well as the strength and drift of the overall framing system when using this approach. Such calculations are beyond the scope of this Design Guide.

Example 3.5.1—Check Permanent Vertical Bracing for Wind and Erection Loading

Given:

Verify that the permanent vertical bracing of a single-story building is adequate to resist wind and construction loading during erection.

The building plan is six 40-ft bays with seven transverse 40-ft bays as shown in Figure 3-24. One frame line is braced with permanent bracing. The eave height of the building is 25 ft as shown in Figure 3-25. Members used in the building are as follows:

Columns: W8×40
Tie beams: W18×35
Girders: W24×68
Joists: 22K9 @ 5'-0" o.c.

Columns, tie beams, and girders are ASTM A992 material. The permanent vertical bracing members are ASTM A36 2L3×3×¼. End connections for the braces use four ¾-in.-diameter ASTM F3125, Grade A325 bolts (thread condition N). The required force in the permanent brace is 63 kips (LRFD) and 38 kips (ASD).

The wind direction is normal to the tie beams and joists. The wind pressure is determined using ASCE/SEI 7-10 and ASCE/SEI 37-14.

V = 115 mph (Exposure B, Risk Category II)
$s/h = 0.08$
$B/s = 20$
Case A (s = 2 ft, h = 25 ft, B = 40 ft)

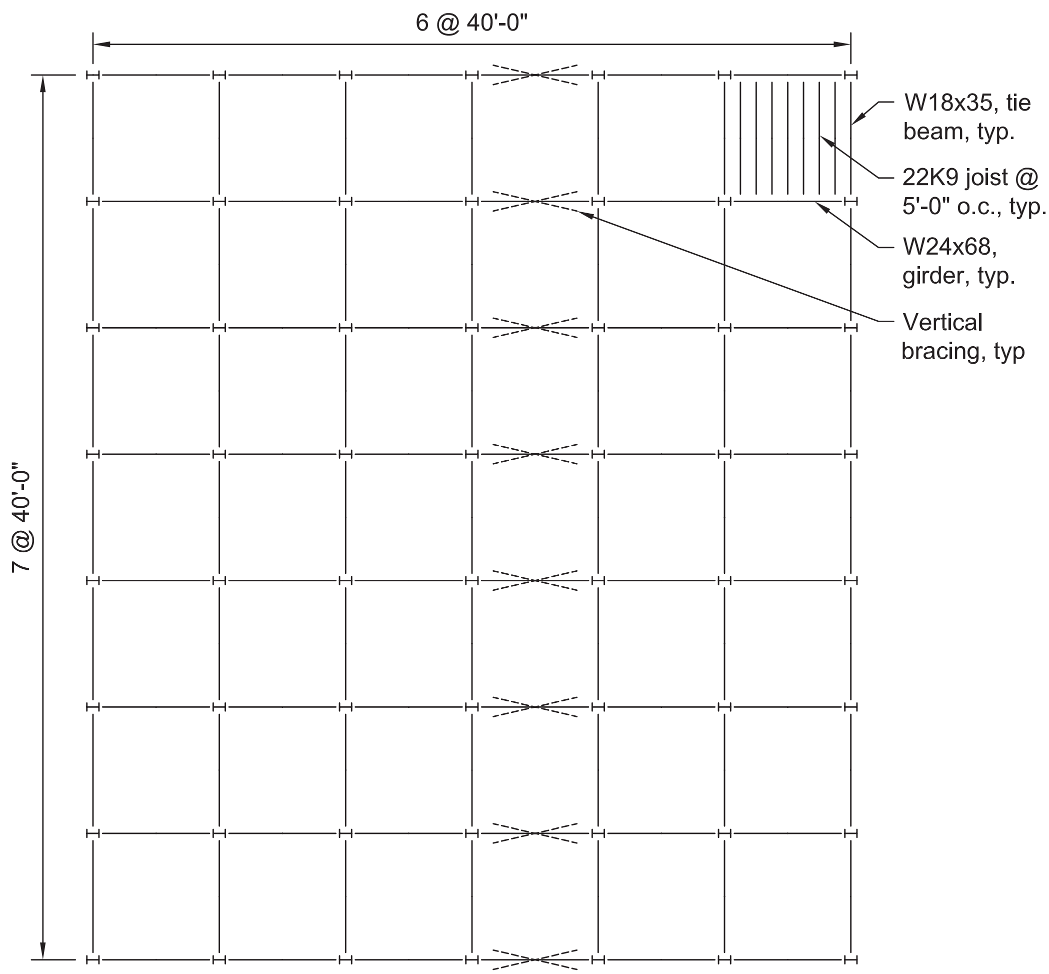


Fig. 3-24. Building plan for Example 3.5.1.

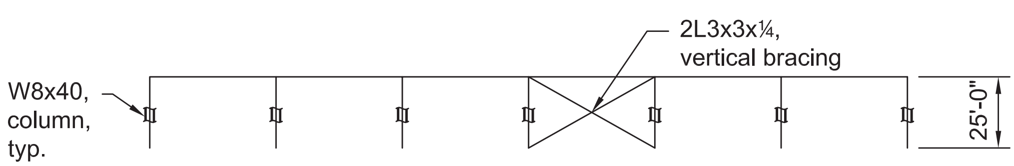


Fig. 3-25. Building elevation for Example 3.5.1.

Solution:

From AISC *Manual* Table 2-4, the member properties are as follows:

Column, tie beams, girders
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Bracing
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

Determination of Wind Load

The design wind force is:

$$F = q_h G C_f A_s \qquad \text{(ASCE/SEI 7-10 Eq. 29.4-1)}$$

From ASCE/SEI 7-10, Figure 29.4-1, with $s/h = 0.08$, $B/s = 20$, and case A ($s = 2$ ft, $h = 25$ ft, $B = 40$ ft)

$$C_f = 1.90$$

From ASCE/SEI 7-10, Section 26.9.1, the gust-effect factor is:

$$G = 0.85$$

From ASCE/SEI 7-10, Section 26.6 and Table 26.6-1, the wind directional factor is:

$$K_d = 0.85$$

From ASCE/SEI 7-10, Section 29.3.1 and Table 29.3-1, with Exposure B at 25 ft:

$$K_z = 0.66$$

From ASCE/SEI 7-10, Section 26.8.2, the topographic factor is:

$K_{zt} = 1.0$ for no topographic effects

According to ASCE/SEI 37-14, Section 6.2.1, V can be reduced using the 0.75 factor for a construction period of less than 6 weeks. The velocity pressure, q_h, is determined as follows:

$$\begin{aligned} q_h &= 0.00256 K_z K_{zt} K_d V^2 \qquad \text{(from ASCE/SEI 7-10 Eq.29.3-1)} \\ &= 0.00256(0.66)(1.0)(0.85)\left[(0.75)(115 \text{ mph})\right]^2 \\ &= 10.7 \text{ lb/ft}^2 \end{aligned}$$

$$\begin{aligned} F &= (10.7 \text{ lb/ft}^2)(0.85)(1.90)A_s \\ &= 17.3A_s \end{aligned}$$

Determination of A_s

The frame in this example has the following surface area subjected 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. For the first frame:

$$\begin{aligned} A_F &= 2(40 \text{ ft})(0.5)(18 \text{ in.})(1 \text{ ft/12 in.}) + (25 \text{ ft})(0.5)(8 \text{ in.})(1 \text{ ft/12 in.}) \\ &= 60.0 \text{ ft}^2 + 8.33 \text{ ft}^2 \\ &= 68.3 \text{ ft}^2 \end{aligned}$$

The second through seventh frames have the same area. The total frame area, including the 0.15 reduction allowed for the fourth and subsequent rows in ASCE/SEI 37-14, Section 6.2.2, is determined as follows:

$$A_{F\text{-}Tot} = 3(68.3\text{ ft}^2) + 4(68.3\text{ ft}^2)(1.0 - 0.15)$$
$$= 437\text{ ft}^2$$

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

$$A_{j\text{-}gr} = (22\text{ in.})(1\text{ ft/12 in.})(40\text{ ft})(7\text{ joists})(6\text{ bays})$$
$$= 3{,}080\text{ ft}^2$$

The shielding reduction, based on Equation 2-1, is:

$$\left[1 + \eta + (n - 2)\eta^2\right]/n \qquad \text{(from Eq. 2-1)}$$

As discussed in Section 2.2.1, the effective solid area would be the gross projected area multiplied by 0.2 for net area, therefore:

$$\varphi = 0.20$$

From the geometry shown in Figure 2-1:

$$a = 60\text{ in.}$$
$$d = 22\text{ in.}$$

The stacking factor, η, is determined using Figure 2-2:

$$a/d = 60\text{ in.}/22\text{ in.}$$
$$= 2.73$$

$$\eta = 0.83$$

With seven joists per bay in six bays, the total number of parallel elements is:

$$n = (7)(6)$$
$$= 42$$

$$[1+\eta+(n-2)\eta^2]/n = [1+0.83+(42-2)(0.83)^2]/42$$
$$= 0.700$$

Therefore, the total effective area of the joists is the gross joist area, $A_{j\text{-}gr}$, reduced by the factors for net area and for shielding due to parallel elements:

$$A_{j\text{-}eff} = A_{j\text{-}gr}\varphi\left\{\left[1+\eta+(n-2)\eta^2\right]/n\right\}$$
$$= (3{,}080\text{ ft}^2)(0.20)(0.700)$$
$$= 431\text{ ft}^2$$

The total steel area is the total frame area added to the effective joist area:

$$A_s = A_{F\text{-}Tot} + A_{j\text{-}eff}$$
$$= 437\text{ ft}^2 + 431\text{ ft}^2$$
$$= 868\text{ ft}^2$$

The design wind force, F, at the level of the roof is determined as follows. Note that for ASD the wind loads are multiplied by 0.6 per ASCE/SEI 7-10, Section 2.4.1, load combination 5.

LRFD	ASD
$F_u = 17.3A_s$ $= 17.3(868\text{ ft}^2)(1\text{ kip}/1{,}000\text{ lb})$ $= 15.0\text{ kips}$	$F_a = (0.6)(17.3)A_s$ $= (0.6)(17.3)(868\text{ ft}^2)(1\text{ kip}/1{,}000\text{ lb})$ $= 9.01\text{ kips}$

Determination of Horizontal Construction Loading, C_H

ASCE/SEI 37-14, Section 4.4, requires 2% of the total dead load to be applied horizontally along the structure edge to account for horizontal construction loading.

Total vertical supported dead load:

7 columns:	7(40 lb/ft)(25 ft)	=	7,000 lb
7 beams:	7(35 lb/ft)(40 ft)	=	9,800 lb
6 girders:	6(68 lb/ft)(40 ft)	=	16,300 lb
Roof framing*:	6(40 ft)(40 ft)(5 psf)	=	48,000 lb
Total		=	81,100 lb

*Joists and bundled deck

Thus, the horizontal design value is:

LRFD	ASD
$C_{Hu} = 1.4(0.02)(81{,}100\text{ lb})(1\text{ kip}/1{,}000\text{ lb})$ $= 2.27\text{ kips}$	$C_{Ha} = (0.02)(81{,}100\text{ lb})(1\text{ kip}/1{,}000\text{ lb})$ $= 1.62\text{ kips}$

Because these values are less than the design wind forces, the force in the diagonal is calculated using 15.0 kips (LRFD) or 9.01 kips (ASD):

LRFD	ASD
$P_{Du} = (15.0\text{ kips})\left(\dfrac{47.2\text{ ft}}{40\text{ ft}}\right)$ $= 17.7\text{ kips} < 63\text{ kips}$ **o.k.**	$P_{Da} = (9.01\text{ kips})\left(\dfrac{47.2\text{ ft}}{40\text{ ft}}\right)$ $= 10.6\text{ kips} < 38\text{ kips}$ **o.k.**

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 is a W24×68 spanning 40 ft. From Table 3-2, the available compressive strength is 25.0 kips (LRFD) and 16.7 kips (ASD), and thus this member is adequate to carry the strut force.

A check of P-Δ 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 vertical component of the temporary brace force is:

LRFD	ASD
$P_u = 17.7\text{ kips}\left(\dfrac{25\text{ ft}}{47.2\text{ ft}}\right)$ $= 9.38\text{ kips}$	$P_a = 10.6\text{ kips}\left(\dfrac{25\text{ ft}}{47.2\text{ ft}}\right)$ $= 5.61\text{ kips}$

Using AISC *Manual* Table 4-1a, the available compressive strength of the W8×40 column is:

LRFD	ASD
$\phi_c P_n = 123 \text{ kips} > P_u = 9.38 \text{ kips}$	$P_n/\Omega_c = 81.7 \text{ kips} > P_a = 5.61 \text{ kips}$

The column is adequate for the vertical component of the temporary bracing force.

3.6 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 available connection strength obtained using the minimum number of fasteners, additional strength can be obtained by installing more fasteners up to the full available strength. This additional available 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 developed in the AISC *Manual*. The reader is referred to AISC V15.0 *Design Examples* (AISC, 2017b) and AISC Design Guide 29 *Vertical Bracing Connections—Analysis and Design* (Muir and Thornton, 2014) for useful guidance on these topics.

3.7 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 *Diaphragm Design Manual* (SDI, 2015).

Chapter 4
Resistance to Loads Using Temporary Supports During Construction

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 wire ropes or struts, the use of the permanent bracing, or a combination thereof. Temporary diagonal struts that carry both tension and compression or just compression are rarely used. Wire rope 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 wire ropes to provide temporary lateral bracing in a frame line requires that the following conditions be met:

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

The development of the beams or joists as functional tie elements requires a check of their available strength as unbraced compression elements, because 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 because the connections will likely only be minimally bolted at the initial stage of loading. The evaluation of strut members is discussed in detail in Chapter 3 of this Design Guide.

The development of the wire rope 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 multistory frame or a one-story frame, the lower end of the wire rope would be attached to the base of the adjacent column or to the foundation itself.

4.1 WIRE ROPE DIAGONAL BRACING

Bracing wire ropes are composed of wire rope and anchorage accessories. Wire rope consists of three components: (1) individual wires forming strands, (2) a core, and (3) multi-wire strands laid helically around the core. The wires that 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 counterclockwise 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 regular lay, right lay. Wire rope is designated by the number of strands, the number of wires per strands, the strand pattern (construction), the type of core, the 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 authors are advised that fiber or synthetic core wire rope is typically selected in applications where maximum flexibility and elasticity is desired, whereas the use of fiber core or synthetic core rope for cable bracing is not a good application and is discouraged. A wire or strand core typically contributes to 7 to 7.5% of overall wire rope strength and is recommended for use in temporary bracing and plumbing applications.

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 nominal breaking strength by a safety factor. This safety factor ranges between 2 and 6 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 User's Manual* published by the Wire Rope Technical Board (WRTB, 2005) and the *Falsework Manual* published by the State of California Department of Transportation (Caltrans, 2016). The Wire Rope Technical Board does not set a factor of safety for wire rope used as temporary lateral supports. However, the *Wire Rope User's 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 Design Guide. The authors recommend the use of a resistance factor, ϕ, of 0.50 for LRFD and a safety factor, Ω, of 3.00 for ASD. The Caltrans *Falsework Manual* uses a safety factor of 2.00, but

Table 4-1. U-Bolt Wire Rope Clips OSHA CFA 1926.251 (OSHA, 2011) TABLE H-2 NUMBER AND SPACING OF U-BOLT WIRE ROPE CLIPS			
Improved Plow Steel Rope Diameter, in.	Number of Clips		Maximum Spacing, in.
	Drop Forged	Other Material	
½	3	4	3
⅝	3	4	3¾
¾	4	5	4½
⅞	4	5	5¼
1	5	6	6
1⅛	6	6	6¾
1¼	6	7	7½
1⅜	7	7	8¼
1½	7	8	9

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 (with thimble)	80%
Knot and clip (contractor's knot)	50%
Plate clamp—three bolt type (with thimble)	80%
Spliced eye and thimble	
⅜ in. to ¾ in.	96%
⅞ in. to 1 in.	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 of a U-bolt and a forged clip. Clips should be drop forged. Malleable iron clips should not be used in temporary bracing or plumbing applications. Table 4-1 is taken from OSHA 1926.251 (OSHA, 2011). 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. Double saddle clips (fist grips) can be used; however, the reader is cautioned that they are rarely installed correctly.

The use of wire rope(s) 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 wire rope under load. This elongation has two sources— elastic stretch, approximately PL/AE, and constructional stretch, Δ_{CS}, 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 wire rope 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% of the breaking strength, a reduced E equal to $0.9E$ is used in industry practice. This stretch is Δ_1. When the wire rope load exceeds 20% of the breaking strength, the elastic stretch is the sum of Δ_1 and Δ_2 as defined in the following equations, which are taken from the *Wire Rope User's Manual* (WRTB, 2005).

$$\Delta_1 = \frac{0.2(NBS - P)L}{A(0.9)E} \tag{4-1}$$

$$\Delta_2 = \frac{(CDF - 0.2NBS)(L + \Delta_1)}{AE} \tag{4-2}$$

where

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

CDF = wire rope design force, lb

E = nominal modulus of elasticity, psi

L = wire rope length, ft

NBS = nominal breaking strength, lb

P = wire rope preload, lb

Δ_1, Δ_2 = wire rope stretch, ft

Constructional stretch, Δ_{CS}, is given by the following formula from the Caltrans *Falsework Manual* (Caltrans, 2016):

$$\Delta_{CS} = \left(\frac{\text{Applied load}}{0.65NBS}\right)(CS)(L) \tag{4-3}$$

where

CS = constructional stretch percentage supplied by the manufacturer, % (typically between 0.75% and 1.0%)

L = wire rope length, ft

The load and wire rope strength are in pounds.

In order for wire rope to perform properly, it is necessary to provide an initial preload by drawing it up to a maximum initial drape. The Caltrans *Falsework Manual* (Caltrans, 2016) provides the following maximum drapes for the wire rope sizes shown:

Wire Rope Size, in.	Maximum Drape, *A*, in.
3⁄8	1
½	2
¾	2¾

The wire rope drape, A, is a vertical distance measured at mid-bay between the two wire rope end points.

Drawing up the wire rope to the maximum allowed drape induces a force in the wire rope which can be calculated from the following equation presented in the *Falsework Manual*. The wire rope preload, P, is:

$$P = \frac{qx^2}{8A\cos\psi} \tag{4-4}$$

where

A = wire rope drape, ft

q = wire rope weight, lb/ft

x = horizontal distance between connection points, ft

ψ = angle between horizontal and wire rope (if straight), deg

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

Note that installers should be cautioned not to overdraw the wire rope as this may pull the frame out of plumb or may overload components of the frame.

The following eight tables (Tables 4-2 through 4-9) present wire rope data taken from the *Wire Rope User's Manual* (WRTB, 2005) for various classifications, core types, and steel grades. The values for weight and metallic area are labeled approximate because 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; WS = Warrington Seale). The nominal breaking strength given is the industry consensus value. Galvanized wire is rated at 10% 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. The reader is reminded of the earlier advice regarding the preferable use of wire or strand core 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 P-Δ effect. In the case of a wire rope 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 typically addressed with a strength design of the bracing for an appropriate equivalent lateral static force, commonly 2% of the supported gravity load. As recommended in Section 2.4, a 100 lb/ft lateral load in each principal axis is applied to the structure to be braced to account for incidental load during erection. This stability check would not normally govern the design of temporary bracing.

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

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

Table 4-2. Nominal Breaking Strength of Wire Rope*			
6×7 Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 13,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.^{2}**	**Nominal Breaking Strength,[1] lb**
3/8	0.21	0.054	11,720
7/16	0.29	0.074	15,800
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

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)

[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-3. Nominal Breaking Strength of Wire Rope*			
6×19 (S) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 12,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.^{2}**	**Nominal Breaking Strength,[1] lb**
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

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)

[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-4. Nominal Breaking Strength of Wire Rope*			
6×36 (WS) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 11,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.^{2}**	**Nominal Breaking Strength,[1] lb**
3/8	0.24	0.059	12,200
7/16	0.32	0.080	16,500
1/2	0.42	0.105	21,400
9/16	0.53	0.133	27,000
5/8	0.66	0.164	33,400
3/4	0.95	0.236	47,600
7/8	1.29	0.321	64,400
1	1.68	0.419	83,600

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-5. Nominal Breaking Strength of Wire Rope*			
8×19 (W) Classification/Bright (Uncoated), Fiber Core, Improved Plow Steel, E = 9,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.^{2}**	**Nominal Breaking Strength,[1] lb**
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

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-6. Nominal Breaking Strength of Wire Rope*			
6×19 (S) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter, in.	Approximate Weight, lb/ft	Approximate Metallic Area, in.^{2}	Nominal Breaking Strength,[1] lb
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

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-7. Nominal Breaking Strength of Wire Rope*			
6×19 (S) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 15,000,000 psi			
Nominal Diameter, in.	Approximate Weight, lb/ft	Approximate Metallic Area, in.^{2}	Nominal Breaking Strength,[1] lb
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	103,800

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] $\phi = 0.50$ for LRFD, $\Omega = 3.00$ for ASD

Table 4-8. Nominal Breaking Strength of Wire Rope*			
6×36 (WS) Classification/Bright (Uncoated), IWRC, Improved Plow Steel, E = 13,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.²**	**Nominal Breaking Strength,[1] lb**
3⁄8	0.26	0.068	13,120
7⁄16	0.35	0.093	17,780
1⁄2	0.46	0.121	23,000
9⁄16	0.59	0.153	29,000
5⁄8	0.72	0.189	35,800
3⁄4	1.04	0.273	51,200
7⁄8	1.42	0.360	69,200
1	1.85	0.485	89,800

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] ϕ = 0.50 for LRFD, Ω = 3.00 for ASD

Table 4-9. Nominal Breaking Strength of Wire Rope*			
6×36 (WS) Classification/Bright (Uncoated), IWRC, Extra Improved Plow Steel, E = 13,000,000 psi			
Nominal Diameter, in.	**Approximate Weight, lb/ft**	**Approximate Metallic Area, in.²**	**Nominal Breaking Strength,[1] lb**
3⁄8	0.26	0.068	15,310
7⁄16	0.35	0.093	20,400
1⁄2	0.46	0.121	26,600
9⁄16	0.59	0.153	33,600
5⁄8	0.72	0.189	41,200
3⁄4	1.04	0.273	58,800
7⁄8	1.42	0.360	79,600
1	1.85	0.485	103,400

* Data taken from the *Wire Rope User's Manual* (WRTB, 2005)
[1] ϕ = 0.50 for LRFD, Ω = 3.00 for ASD

Example 4.1.1—Temporary Wire Rope Diagonal Bracing (LRFD)

Given:

Using the tables in this chapter, select a 6×7 fiber core-improved plow steel (FC-IPS) wire rope for temporary bracing for wind and construction loading. The building and design forces are the same as Example 3.5.1, except one frame line is braced using wire rope. The wind direction is normal to the tie beams and joists. The wind pressure is determined using ASCE/SEI 7-10 and ASCE/SEI 37-14.

From Example 3.5.1:

Bays: 6 bays of 40'-0"
Transverse bays: 40'-0" each side of frame, 25-ft eave height
Columns: W8×40
Tie beams: W18×35
Girders: W24×68
Joists: 22K9 @ 5'-0" o.c.
Basic wind speed: $V = 115$ mph (Exposure B, Risk Category II)
Horizontal design wind force: $F = 15{,}000$ lb

Solution:

Design of Diagonal Wire Rope

The geometry of the wire rope is:

Vertical: 25 ft (column height)
Horizontal: 40 ft (bay width)

The diagonal length of the wire rope is:

$$\sqrt{(25 \text{ ft})^2 + (40 \text{ ft})^2} = 47.2 \text{ ft}$$

The strut force in the braced bay is 15,000 lb.

The vertical force in the column is:

$$(15{,}000 \text{ lb})\left(\frac{25 \text{ ft}}{40 \text{ ft}}\right) = 9{,}380 \text{ lb}$$

The force in the diagonal wire rope is:

$$(15{,}000 \text{ lb})\left(\frac{47.2 \text{ ft}}{40 \text{ ft}}\right) = 17{,}700 \text{ lb}$$

Using $\phi = 0.50$ as discussed in Section 4.1, the minimum nominal breaking strength required is:

$$\frac{17{,}700 \text{ lb}}{0.50} = 35{,}400 \text{ lb}$$

Try a ¾-in.-diameter 6×7 FC-IPS wire rope. From Table 4-2:

Nominal breaking strength = 45,400 lb
Area = 0.216 in.^{2}
Weight per foot = 0.84 lb/ft
Modulus of elasticity = 13,000 ksi (nominal)

Second-Order Analysis

In most braced frames where deflections are due to axial elongations, the effects of second-order effects are minimal; however, for the case of wire-rope supported structures, the second-order deflections can be significant because of the relatively low effective elastic modulus, E, of the wire rope. For the applied loads on the wire rope in this example, a second-order analysis was conducted to determine the wire rope force using the direct analysis method from the AISC *Specification*. The columns and tie beams were considered axially rigid because their contribution to frame deflection is trivial. The wire rope's modulus, E = 13,000,000 psi, was reduced in accordance with AISC *Specification* Section C2.3(a) as follows:

$$
\begin{aligned}
E &= 0.80(13{,}000 \text{ ksi}) \\
&= 10{,}400 \text{ ksi}
\end{aligned}
$$

Based on first-order analysis, the wire rope force is 17,700 lb, and the first-order drift is 4.21 in. From a second-order analysis, the wire rope force is 20,200 lb and the total drift is 6.02 in.

Based on these results, the minimum required wire rope strength is:

$$
\begin{aligned}
P_r &= \frac{20{,}200 \text{ lb}}{\phi} \\
&= \frac{20{,}200 \text{ lb}}{0.50} \\
&= 40{,}400 \text{ lb}
\end{aligned}
$$

Thus, a ¾-in.-diameter 6×7 FC-IPS wire rope with a nominal breaking strength of 45,400 lb is adequate.

4.2 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 to terminate the wire rope diagonal, then a spelter socket, wedge socket, or clevis end fitting can be used, or a wire rope with a thimble eye can be attached with a shackle. Wire ropes can also be secured to columns by wrapping the column, either with a section of wire rope (commonly called a choker) to which a hook end turnbuckle is attached or with the end of the diagonal wire rope itself that is secured by wire rope clamps. If wire ropes are wrapped around an element, such as a column, a positive mechanism should be provided to prevent the wire rope from slipping along the column or beam. Also, when wire ropes are terminated by wrapping, care should be taken to avoid damage to the wire rope by kinking or crushing. Wire ropes 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 wire rope force. Wire ropes are tensioned and adjusted using 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 AISC *Manual* Part 15. When structure vibration or wind-induced wire rope bracing vibration occur, it is important to lock the turnbuckle frame to the end fittings to prevent it from turning and loosening. Jamb nuts or lock nuts should not be used, but instead, wire-seize one end-fitting to the frame of the turnbuckle to prevent rotation. Wire rope and rope pullers (come-alongs) are also used to draw up wire rope diagonals. In the following text, two types of wire rope terminations are discussed—projecting plate (Type A) and bent attachment plate (Type B).

4.2.1 Projecting Plate (Type A)

The design of a projecting plate from the face of a column is illustrated in the following example. Available strengths for various conditions of wire rope size, type, and angle 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. It should be noted that if, for example, anchor rod holes such as those tabulated in AISC *Manual* Table 14-2 are used, the force in the wire rope diagonal may cause the column and base plate to shift until the anchor rod bears against the side of the hole. If this potential is of concern, the temporary bracing designer should consider welding the anchor rod washers to the base plate. Also, note that with the exception of cases in which the designer elects to resist the lateral force component with two anchor rods, this is of no concern for Type B anchors because the anchor is placed directly on the anchor rod and a standard hole can be used.

The allowable tensile strength through the pinhole, P_t, is determined using ASME BTH-1-2017, *Design of*

Below-the-Hook Lifting Devices (ASME, 2017):

$$P_t = C_r \frac{F_u}{1.20N_d} 2tb_{eff} \tag{4-5}$$

where

$$C_r = 1 - 0.275\sqrt{1 - \frac{D_p^2}{D_h^2}} \tag{4-6}$$

D_h = hole diameter, in.

D_p = pin diameter, in.

F_u = specified minimum tensile stress of the plate, ksi

F_y = specified minimum yield stress of the plate, ksi

N_d = nominal design factor

$$b_{eff} = b_e 0.6 \frac{F_u}{F_y}\sqrt{\frac{D_h}{b_e}} \le b_e \tag{4-7}$$

(b_{eff} is the smaller of the values calculated from Equation 4-7 and 4t)

b_e = actual width of a pin-connected plate between the edge of the hole and edge of the plate on a line perpendicular to the line of action of the applied load, in.

t = plate thickness, in.

The nominal design factor, N_d, varies depending on the lifter design category. For Design Category A, which is designated when the magnitude and variation of loads applied to the lifter are predictable and the loading and environmental conditions are accurately defined and not severe, the minimum N_d value in allowable stress equations is 2.00. There are also Design Categories B and C (see ASME BTH-1 Section 3-1.3) but for the purposes of this Guide, Design Category A will be used.

The allowable single-plane fracture strength beyond the pin hole in kips per in., P_b, is determined using ASME BTH-1-2017:

$$P_b = C_r \frac{F_u}{1.20N_d}\left[1.13\left(R - \frac{D_h}{2}\right) + \frac{0.92b_e}{1 + b_e/D_h}\right]t \tag{4-8}$$

where

R = distance from the center of the hole to the edge of the plate in the direction of the applied load, in.

The allowable plane shear strength beyond the pin hole, P_v, is determined using ASME BTH-1 Equations 3-50, 3-51, and 3-52:

$$P_v = \frac{0.70F_u}{1.20N_d} A_v \tag{4-9}$$

where

A_v = total area of the two shear planes beyond the pinhole, in.^{2}

$$= 2\left[a + \frac{D_p}{2}(1 - \cos\phi)\right]t \tag{4-10}$$

a = distance from edge of the pinhole to the edge of the plate in the direction of the applied load, in.

$$\phi = 55\frac{D_p}{D_h} \tag{4-11}$$

Example 4.2.1.1—Wire Rope Projecting Plate Design

Given:

For the projecting plate shown in Figure 4-1, design the projecting plate attachment (Type A) for the wire rope force indicated using the provisions of ASCE/SEI 7-10. A 1-in.-diameter pin is used. Use ASTM A36 plate material and 70-ksi weld electrodes. Assume Design Category A lifter.

Solution:

From AISC *Manual* Table 2-5, properties of the plate are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

Weld Design

Using 3⁄16-in. fillet welds along each side of the wing plate, calculate l_{min} according to AISC *Manual* Table 8-4 (Angle = 45°).

$$l_{min} = \frac{P_u}{\phi C C_1 D}$$

where

$C_1 = 1.0$ (for E70XX electrodes from AISC *Manual* Table 8-3)

$D = 3$

$P_u = 14.4$ kips

$\phi = 0.75$

Assuming the distance between the welds is 0 in. and given the eccentricity of the loading:

$k = 0$

$$e_x = al$$
$$= 3.00 \text{ in.}$$

Try $l = 4$ in., using AISC *Manual* Table 8-4:

$C = 2.11$

$a = 0.75$

$$l_{min} = \frac{14.4 \text{ kips}}{0.75(2.11)(1.00)(3)}$$
$$= 3.03 \text{ in.}$$

Use 4-in.-long, $\frac{3}{16}$-in. fillet welds on each side of the plate.

Plate Design

Try a ½-in. plate.

Component bending of the plate (vertical):

$$P_{uv} = (14.4 \text{ kips})(\sin 32°)$$
$$= 7.63 \text{ kips}$$

$$M_u = (7.63 \text{ kips})(3.00 \text{ in.})$$
$$= 22.9 \text{ kip-in.}$$

Component tensioning the plate (horizontal):

$$P_{uh} = (14.4 \text{ kips})(\cos 32°)$$
$$= 12.2 \text{ kips}$$

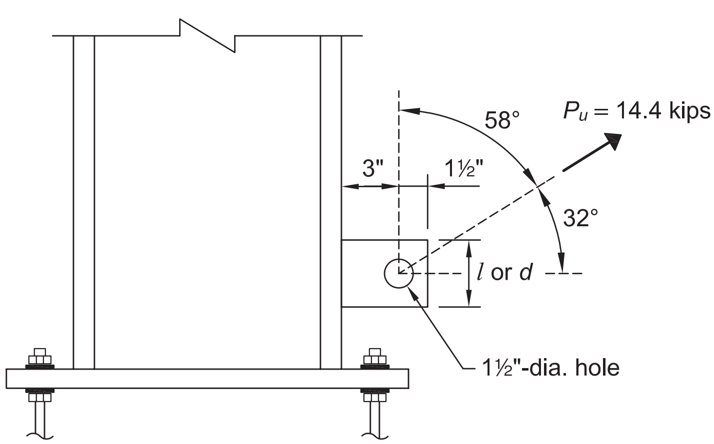


Fig. 4-1. Projecting plate geometry for Example 4.2.1.1.

Check plate local buckling:

$$\frac{b}{t} = \frac{3.00\text{ in.}}{\frac{1}{2}\text{ in.}}$$
$$= 6.00$$

AISC *Specification* Table B4.1a, Case 1, is applicable:

$$0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}$$
$$= 15.8 > 6.00$$

Therefore, the plate will not buckle locally.

Check the plate for flexural yielding and lateral-torsional buckling using AISC *Specification* Section F11.

If $\dfrac{L_b d}{t^2} \le \dfrac{0.08E}{F_y}$, then the limit state of yielding controls,

where

L_b = unbraced length of the plate
= 3.00 in.

d = plate height, in.
= 4.00 in.

t = plate thickness, in.
= ½ in.

$$\frac{(3.00\text{ in.})(4.00\text{ in.})}{(\frac{1}{2}\text{ in.})^2} \le \frac{(0.08)(29{,}000\text{ ksi})}{36\text{ ksi}}$$
$$48 < 64.4$$

Therefore, the limit state of yielding controls.

Using AISC *Specification* Section F11.1, the available flexural strength based on the limit state of yielding is:

$$\phi M_n = \phi F_y Z \le \phi 1.6 F_y S_x \qquad \text{(from \textit{Spec}. Eq. F11-1)}$$
$$= 0.90(36\text{ ksi})\left[\frac{(\frac{1}{2}\text{ in.})(4.00\text{ in.})^2}{4}\right] < 0.90(1.6)(36\text{ ksi})\left[\frac{(\frac{1}{2}\text{ in.})(4.00\text{ in.})^2}{6}\right]$$
$$= 64.8\text{ kip-in.} < 69.1\text{ kip-in.}$$

Therefore, $\phi M_n = 64.8$ kip-in.

Check the plate for tensile yielding and tensile rupture using AISC *Specification* Section J4.1. The available tensile strength based on the limit state of tensile yielding is:

$$\phi P_n = \phi F_y A_g \qquad \text{(from \textit{Spec}. Eq. J4-1)}$$
$$= 0.90(36\text{ ksi})(\tfrac{1}{2}\text{ in.})(4.00\text{ in.})$$
$$= 64.8\text{ kips}$$

Using AISC *Specification* Section J4.1, the available tensile strength based on the limit state of tensile rupture is:

$$A_e = (4.00\text{ in.} - 1\tfrac{1}{2}\text{ in.})(\tfrac{1}{2}\text{ in.})$$
$$= 1.25\text{ in.}^2$$

$$\phi P_n = \phi F_u A_e \qquad \text{(from } \textit{Spec}\text{. Eq. J4-2)}$$
$$= 0.75(58 \text{ ksi})(1.25 \text{ in.}^2)$$
$$= 54.4 \text{ kips}$$

Interaction is checked using AISC *Specification* Section H1.1.

$$\frac{P_r}{P_c} = \frac{12.2 \text{ kips}}{54.4 \text{ kips}}$$
$$= 0.224$$

For $\dfrac{P_r}{P_c} \geq 0.2$:

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = \frac{12.2 \text{ kips}}{54.4 \text{ kips}} + \frac{8}{9}\left(\frac{22.9 \text{ kip-in.}}{64.8 \text{ kip-in.}} + 0\right) \qquad (\textit{Spec}\text{. Eq. H1-1a})$$
$$= 0.538 < 1.0 \quad \textbf{o.k.}$$

Pinhole Strength

The available tensile strength through the pinhole is determined as follows. Based on the geometry of the plate, b_e is approximated to be 1.50 in. The effective width is:

$$b_{eff} = b_e 0.6 \frac{F_u}{F_y}\sqrt{\frac{D_h}{b_e}} \leq b_e \tag{4-7}$$
$$= (1.50 \text{ in.})(0.6)\left(\frac{58 \text{ ksi}}{36 \text{ ksi}}\right)\sqrt{\frac{1\frac{1}{2} \text{ in.}}{1.50 \text{ in.}}} \leq 1.50 \text{ in.}$$
$$= 1.45 \text{ in.}$$

The effective width, b_{eff}, is the smaller of the values calculated from Equation 4-7 and $4t$.

$$4t = 4(\tfrac{1}{2} \text{ in.})$$
$$= 2.00 \text{ in.} > 1.45 \text{ in.}$$

Therefore, use $b_{eff} = 1.45$ in.

$$C_r = 1 - 0.275\sqrt{1 - \frac{D_p^2}{D_h^2}} \tag{4-6}$$
$$= 1 - 0.275\sqrt{1 - \frac{(1 \text{ in.})^2}{(1\frac{1}{2} \text{ in.})^2}}$$
$$= 0.795$$

The allowable tensile strength through the pinhole is:

$$P_t = C_r \frac{F_u}{1.20 N_d} 2tb_{eff} \tag{4-5}$$

where
N_d = 2.00 for Design Category A

$$P_t = 0.795\left[\frac{58\text{ ksi}}{1.20(2.00)}\right](2)(½\text{ in.})(1.45\text{ in.})$$
$$= 27.9\text{ kips}$$

Because P_t is an allowable strength (ASD) value, the LRFD factored load of 14.4 kips must be adjusted for use in the comparison using ASCE/SEI 7-10, ASD Load Combination 7, 0.6D + 0.6W:

$$P_a = (14.4\text{ kips})(0.6)$$
$$= 8.64\text{ kips}$$

$P_t = 27.9\text{ kips} > P_a = 8.64\text{ kips}$ **o.k.**

The allowable single-plane fracture strength beyond the pin hole, P_b, is determined as follows:

$$P_b = C_r \frac{F_u}{1.20N_d}\left[1.13\left(R - \frac{D_h}{2}\right) + \frac{0.92b_e}{1 + b_e/D_h}\right]t \tag{4-8}$$

$$R = \frac{1.50\text{ in.}}{\cos 32°}$$
$$= 1.77\text{ in.}$$

For a ½-in.-thick plate:

$$P_b = (0.795)\left[\frac{58\text{ ksi}}{1.20(2.00)}\right]\left[1.13\left(1.77\text{ in.} - \frac{1½\text{ in.}}{2}\right) + \frac{0.92(1.50\text{ in.})}{1 + (1.50\text{ in.}/1½\text{ in.})}\right](½\text{ in.})$$
$$= 17.7\text{ kips} > 8.64\text{ kips}$$ **o.k.**

The plane shear strength beyond the pin hole, P_v, is determined as follows:

$$P_v = \frac{0.70F_u}{1.20N_d}A_v \tag{4-9}$$

where

$$a = 1.77\text{ in.} - \frac{D_h}{2}$$
$$= 1.77\text{ in.} - \frac{1½\text{ in.}}{2}$$
$$= 1.02\text{ in.}$$

$$\phi = 55\frac{D_p}{D_h} \tag{4-11}$$
$$= 55\left(\frac{1\text{ in.}}{1½\text{ in.}}\right)$$
$$= 36.7°$$

$$A_v = 2\left[a + \frac{D_p}{2}(1 - \cos\phi)\right]t \tag{4-10}$$
$$= 2\left[1.02\text{ in.} + \left(\frac{1\text{ in.}}{2}\right)(1 - \cos 36.7°)\right](½\text{ in.})$$
$$= 1.12\text{ in.}^2$$

$$P_v = \frac{0.70(58\text{ ksi})}{1.20(2.00)}(1.12\text{ in.}^2)$$
$$= 18.9\text{ kips} > 8.64\text{ kips} \quad \textbf{o.k.}$$

Therefore the ½-in. plate is adequate.

The plate and weld can also be found in Table A-18 for the wire rope type and geometry given.

4.2.2 Bent Attachment Plate (Type B)

Another means of attachment of the diagonal wire rope to the column base is a bent plate on one of the column anchor rods as illustrated in Figure 4-2. The use of this plate requires extra anchor rod length to accommodate it. If the plates are to be left in place, they 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 wire rope attachment is illustrated in the following example. Available strength for various conditions of wire rope size, type, and angle can be read from the accompanying tables.

Example 4.2.2.1—Wire Rope Bent Attachment Plate Design (LRFD)

Given:

Design a 5-in.-wide bent plate attachment (Type B) for the geometry shown in Figure 4-2. As repeated from Example 4.2.1.1, the wire rope force is 14.4 kips (LRFD). Use ASTM A36 plate material. The bent plate is attached to a ¾-in.-diameter anchor rod. A 13⁄16-in.-diameter hole is used in the bent plate.

Solution:

From AISC *Manual* Table 2-5, properties of the plate are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

As calculated in Example 4.2.1.1, the vertical and horizontal components of the wire force are:

$P_v = 7.63$ kips (vertical)
$P_h = 12.2$ kips (horizontal)

The moment due to eccentricity from the bend to the face of the nut is:

$$M_u = P_v e$$
$$= (7.63\text{ kips})(1\text{ in.})$$
$$= 7.63\text{ kip-in.}$$

Try a ½-in.-thick plate.

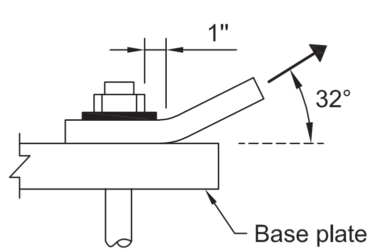


Fig. 4-2. Bent plate on anchor rod.

Check the available flexural strength.

Using AISC *Specification* Section F11.1, the design flexural strength of the plate based on the limit state of yielding is:

$$\phi_b M_n = \phi_b F_y Z \le \phi_b 1.6 F_y S_x \qquad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$

$$= 0.90(36\text{ ksi})\left[\frac{(5.00\text{ in.})(\tfrac{1}{2}\text{ in.})^2}{4}\right] < 0.90(1.6)(36\text{ ksi})\left[\frac{(5.00\text{ in.})(\tfrac{1}{2}\text{ in.})^2}{6}\right]$$

$$= 10.1\text{ kip-in.} < 10.8\text{ kip-in.}$$

Therefore,

$\phi M_n = 10.1$ kip-in. $> M_u = 7.63$ kip-in. **o.k.**

Using AISC *Specification* Section J4.1, the design tensile strength based on the limit state of yielding is:

$$\phi P_n = \phi F_y A_g \qquad \text{(from } \textit{Spec}\text{. Eq. J4-1)}$$

$$= 0.90(36\text{ ksi})(\tfrac{1}{2}\text{ in.})(5.00\text{ in.})$$

$$= 81.0\text{ kips} > 12.2\text{ kips} \quad \textbf{o.k.}$$

Using AISC *Specification* Section J4.1, the design tensile strength based on the limit state of rupture is:

$$A_e = \left[5.00\text{ in.} - (\tfrac{3}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{1}{2}\text{ in.})$$

$$= 2.06\text{ in.}^2$$

$$\phi P_n = \phi F_u A_e \qquad \text{(from } \textit{Spec}\text{. Eq. J4-2)}$$

$$= 0.75(58\text{ ksi})(2.06\text{ in.}^2)$$

$$= 89.6\text{ kips} > 12.2\text{ kips}$$

Interaction is checked using AISC *Specification* Section H1.1.

$$\frac{P_r}{P_c} = \frac{12.2\text{ kips}}{81.0\text{ kips}}$$

$$= 0.151$$

For $\dfrac{P_r}{P_c} < 0.2$:

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = \frac{12.2\text{ kips}}{2(81.0\text{ kips})} + \left(\frac{7.63\text{ kip-in.}}{10.1\text{ kip-in.}} + 0\right) \qquad (\textit{Spec}\text{. Eq. H1-1b})$$

$$= 0.831 < 1.0 \quad \textbf{o.k.}$$

The strength of the plate at the wire rope attachment hole can be determined as in Example 4.2.1.1.

Therefore, use a ½-in.×5-in. plate.

The attachment plate dimensions can also be taken from Appendix Table A-20 for the wire rope type and geometry given.

4.2.3 Anchor Rods

The development of the wire rope force requires that the anchor rods be adequate to transfer the brace force into the footing and also that the footing is adequate to resist the brace force acting as a deadman as discussed in Section 4.3. The adequacy of the anchor rods in tension is discussed in Chapter 3 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 wire rope force is applied, then the procedures presented in

AISC Design Guide 1 can be used. If leveling nuts (or shims) are used and there is no grout at the time of wire rope force application, then another procedure must be used. When leveling nuts (or shims) are used the anchor rods are also subject to bending. In the following design example, a check for anchor rod bending is made. The calculation takes the moment arm as half of the anchor rod height because 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. This condition at the top of the anchors has been idealized as a fixed condition in the calculation of the moment arm.

Example 4.2.3.1—Check Anchor Rods at Column with Wire Rope Projecting Plate (LRFD)

Given:

Check the column anchor rods illustrated in Figure 4-1 for combined tension and shear. The projecting plate is detailed so that the line of action of the wire rope intersects the top of the base plate at mid-depth of the column. The anchor rods are 1⅛-in.-diameter, ASTM F1554 Grade 36. Leveling nuts are used under the base plate.

The base plate design is as follows:

Grout thickness: 1.50 in. (ungrouted for this check)
Base plate thickness: ¾ in.
Plate washer thickness: ⅜ in. (welded to the base plate)
Concrete: $f_c' = 3{,}000$ psi

Note: When the washers are not welded to the base plate, AISC Design Guide 1 recommends that only two of the four anchor rods be used to resist shear due to the anchor rod hole size and potential slip.

Solution:

From AISC *Manual* Table 2-6, the anchor rod material properties are:

ASTM F1554 Grade 36
$F_u = 58$ ksi

The vertical component of force is:

$$P_v = (14.4 \text{ kips})(\sin 32°)$$
$$= 7.63 \text{ kips}$$

The horizontal component of force is:

$$P_h = (14.4 \text{ kips})(\cos 32°)$$
$$= 12.2 \text{ kips}$$

Column axial force

Using the loads from Example 3.5.1, the weight of the frame tributary to one interior column is:

Column:	(40 lb/ft)(25 ft)	= 1,000 lb
Beams:	2(35 lb/ft)(40 ft)(0.5)	= 1,400 lb
Girders:	2(68 lb/ft)(40 ft)(0.5)	= 2,720 lb
Roof framing:	(40 ft)(40 ft)(5 psf)	= 8,000 lb
Total		= 13,100 lb

(13,100 lb)(1 kip/1,000 lb) = 13.1 kips (gravity load)

Anchor Rod Axial Stress

Using ASCE/SEI 7-10, ASD Load Combination 6:

$$P_u = 0.9D + 1.0W$$

Anchor rod axial load due to the column gravity load (resisted by four anchor rods):

$$P_{ud} = 0.9D$$
$$= \frac{(0.9)(13.1 \text{ kips})}{4}$$
$$= 2.95 \text{ kips (compression)}$$

Anchor rod axial uplift load due to wind (resisted by four anchor rods):

$$P_{uw} = \frac{1.0(7.63 \text{ kips})}{4}$$
$$= 1.91 \text{ kips (tension)}$$

Total axial load per anchor rod:

$$P_u = 2.95 \text{ kips} - 1.91 \text{ kips}$$
$$= 1.04 \text{ kips (compression)}$$

The axial stress per anchor rod is:

$$A_b = \frac{\pi d^2}{4}$$
$$= \frac{\pi(1\tfrac{1}{8} \text{ in.})^2}{4}$$
$$= 0.994 \text{ in.}^2$$

$$f_a = \frac{P_u}{A_b}$$
$$= \frac{1.04 \text{ kips}}{0.994 \text{ in.}^2}$$
$$= 1.05 \text{ ksi (compression)}$$

Anchor Rod Bending Stress

The lever arm, L, is equal to the grout height plus the base plate thickness plus one-half of the plate washer thickness, divided by two for reverse curvature bending, as shown in Figure 4-3.

$$L = \left(1.50 \text{ in.} + \tfrac{3}{4} \text{ in.} + \frac{\tfrac{3}{8} \text{ in.}}{2}\right)\left(\frac{1}{2}\right)$$
$$= 1.22 \text{ in.}$$

The shear force per anchor rod is:

$$V_r = \frac{12.2 \text{ kips}}{4}$$
$$= 3.05 \text{ kips}$$

The required flexural strength per anchor rod is:

$$M_r = V_r L$$
$$= (3.05 \text{ kips})(1.22 \text{ in.})$$
$$= 3.72 \text{ kip-in.}$$

From AISC *Manual* Table 17-27, the elastic section modulus, S, of the 1⅛-in.-diameter anchor rod is:

$$S = \frac{\pi d^3}{32}$$
$$= \frac{\pi (1\tfrac{1}{8} \text{ in.})^3}{32}$$
$$= 0.140 \text{ in.}^3$$

The bending stress in the anchor rod is:

$$f_b = \frac{M_r}{S}$$
$$= \frac{3.72 \text{ kip-in.}}{0.140 \text{ in.}^3}$$
$$= 26.6 \text{ ksi (tension)}$$

The combined stress due to axial and bending stresses in the anchor rod is:

$$f_t = f_b + f_a$$
$$= 26.6 \text{ ksi} + (-1.05 \text{ ksi})$$
$$= 25.6 \text{ ksi (tension)}$$

From AISC *Specification* Table J3.2, the nominal tensile strength of the anchor rod with threads not excluded from the shear plane is:

$$F_{nt} = 0.75F_u$$
$$= (0.75)(58 \text{ ksi})$$
$$= 43.5 \text{ ksi}$$

And the available tensile strength is:

$$\phi F_{nt} = (0.75)(43.5 \text{ ksi})$$
$$= 32.6 \text{ ksi} > 25.6 \text{ ksi} \quad \textbf{o.k.}$$

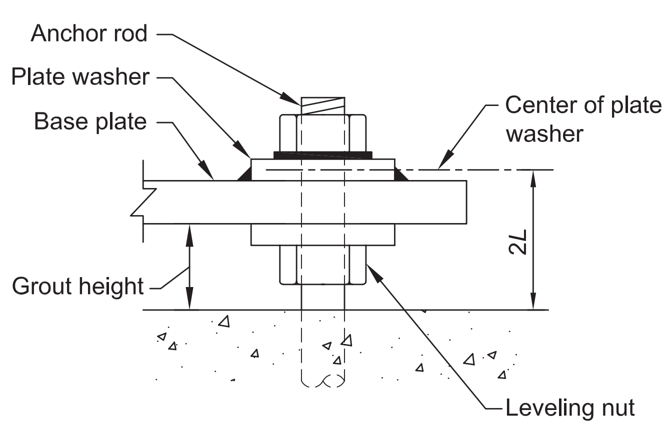


Fig. 4-3. Lever arm.

Anchor Rod Shear Stress

The required shear stress per anchor rod is:

$$f_{rv} = \frac{V_r}{A_b}$$
$$= \frac{3.05 \text{ kips}}{0.994 \text{ in.}^2}$$
$$= 3.07 \text{ ksi}$$

From AISC *Specification* Table J3.2, the nominal shear strength of the anchor rod with threads not excluded from the shear plane is:

$$F_{nv} = 0.450F_u$$
$$= (0.450)(58 \text{ ksi})$$
$$= 26.1 \text{ ksi}$$

And the available shear strength is:

$$\phi F_{nv} = (0.75)(26.1 \text{ ksi})$$
$$= 19.6 \text{ ksi} > 3.07 \text{ ksi} \quad \textbf{o.k.}$$

From AISC *Specification* Section J3.7, the available strength of the anchor rod subjected to shear and tension is determined as follows:

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt} \qquad (\textit{Spec}.\text{ Eq. J3-3a})$$

The User Note in AISC *Specification* Section J3.7 states that when the required stress, f, in either shear or tension, is less than or equal to 30% of the corresponding available stress, the effects of combined stress need not be investigated.

$$0.30(\phi F_{nv}) = 0.30(19.6 \text{ ksi})$$
$$= 5.88 \text{ ksi} > 3.07 \text{ ksi} \quad \textbf{o.k.}$$

Because $f_{rv} < 0.30(\phi F_{nv})$, the interaction check is not required, and the anchor rods are found to be sufficient.

Example 4.2.3.2— Check Anchor Rods at Wire Rope Bent Plate (LRFD)

Given:

Check the column anchor rods as illustrated in Figure 4-2 for combined tension and shear. The force in the wire rope is 14.4 kips (LRFD). The anchor rods are 1⅛-in.-diameter, ASTM F1554 Grade 36. Leveling nuts are used under the base plate.

The base plate design is as follows:

Grout thickness: 1.50 in. (ungrouted for this check)
Base plate thickness: ¾ in.
Plate washer thickness: ⅜ in. (welded to the base plate)
Concrete: $f'_c = 3{,}000$ psi

Note: When the washers are not welded to the base plate, AISC Design Guide 1, Section 3.5.3, recommends that only two of the four anchor rods be used to resist shear due to the anchor rod hole size and potential slip.

Solution:

From AISC *Manual* Table 2-6, the anchor rod material properties are:

ASTM F1554 Grade 36
$F_u = 58$ ksi

The vertical component of force is:

$$P_v = (14.4 \text{ kips})(\sin 32°)$$
$$= 7.63 \text{ kips}$$

The horizontal component of force is:

$$P_h = (14.4 \text{ kips})(\cos 32°)$$
$$= 12.2 \text{ kips}$$

Column Axial Force

From Example 4.2.3.1, the weight of the frame tributary to one interior column is 13.1 kips (gravity).

Anchor Rod Axial Stress

Using ASCE/SEI 7-10, ASD Load Combination 6:

$$P_u = 0.9D + 1.0W$$

Axial load due to dead load (resisted by four anchor rods):

$$P_{ud} = 0.9D$$
$$= \frac{0.9(13.1 \text{ kips})}{4}$$
$$= 2.95 \text{ kips (compression)}$$

Anchor rod axial uplift load due to wind (resisted by one anchor rod):

$$P_{uw} = 7.63 \text{ kips (tension)}$$

Total axial load per anchor rod:

$$P_u = 7.63 \text{ kips} - 2.95 \text{ kips}$$
$$= 4.68 \text{ kips (tension)}$$

The axial stress per anchor rod, f_a, is:

$$A_b = \frac{\pi d^2}{4}$$
$$= \frac{\pi(1\tfrac{1}{8} \text{ in.})^2}{4}$$
$$= 0.994 \text{ in.}^2$$

$$f_a = \frac{P_u}{A_b}$$
$$= \frac{4.68 \text{ kips}}{0.994 \text{ in.}^2}$$
$$= 4.71 \text{ ksi (tension)}$$

Anchor Rod Bending Stress

The lever arm, L, is equal to the grout height plus the base plate thickness plus one-half of the plate washer thickness divided by two for reverse curvature bending, as shown in Figure 4-3.

$$
\begin{aligned}
L &= \left(1.50\text{ in.} + \tfrac{3}{4}\text{ in.} + \frac{\tfrac{3}{8}\text{ in.}}{2}\right)\left(\frac{1}{2}\right) \\
&= 1.22\text{ in.}
\end{aligned}
$$

The shear force per anchor rod is:

$$
\begin{aligned}
V_r &= \frac{12.2\text{ kips}}{2} \\
&= 6.10\text{ kips}
\end{aligned}
$$

The required bending moment per anchor rod is:

$$
\begin{aligned}
M_r &= V_r L \\
&= (6.10\text{ kips})(1.22\text{ in.}) \\
&= 7.44\text{ kip-in.}
\end{aligned}
$$

From AISC *Manual* Table 17-27, the elastic section modulus, S, of the 1⅛-in.-diameter anchor rod is:

$$
\begin{aligned}
S &= \frac{\pi d^3}{32} \\
&= \frac{\pi(1\tfrac{1}{8}\text{ in.})^3}{32} \\
&= 0.140\text{ in.}^3
\end{aligned}
$$

The bending stress, f_b, in the anchor rod is:

$$
\begin{aligned}
f_b &= \frac{M_r}{S} \\
&= \frac{7.44\text{ kip-in.}}{0.140\text{ in.}^3} \\
&= 53.1\text{ ksi (tension)}
\end{aligned}
$$

The combined stress due to axial and bending stresses in the anchor rod is:

$$
\begin{aligned}
f_t &= f_b + f_a \\
&= 53.1\text{ ksi} + 4.71\text{ ksi} \\
&= 57.8\text{ ksi (tension)}
\end{aligned}
$$

As calculated in Example 4.2.3.1, the available tensile stress is:

$$F_t = 32.6\text{ ksi}$$

$$F_t < f_t \quad \textbf{n.g.}$$

The anchor rods need to be larger or of greater tensile strength to resist the forces. Although not shown here, the anchor rods need to be checked for shear and combined shear and tension. Refer to Example 4.2.3.1 for illustration of these calculations.

However, in most cases the anchor rods will already be specified prior to checking their suitability for erection loads, thus the use of the bent plate in this situation may not be feasible.

4.3 DESIGN OF DEADMEN

On occasion the erector must anchor wire rope 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.

4.3.1 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 wire rope makes with the deadman and the location of the bracing wire rope 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 is equal to the total weight of the deadman less the upward force from the bracing wire rope, multiplied by the coefficient of friction between the deadman and the soil. A coefficient of friction, μ, in the range of 0.2 to 0.5 is typically used, which can vary depending on site conditions. The nominal horizontal resistance of the deadman in equation format (Teng, 1962) is:

$$R_n = \mu(W_d - P_r \sin\theta) \tag{4-12}$$

where

P_r = required brace force, lb

W_d = weight of the deadman, lb

θ = angle measured from the horizontal of the bracing wire rope, degrees

μ = coefficient of friction

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

$$R_{all} = \left(\frac{\mu}{1.5}\right)(W_d - P_r \sin\theta) \tag{4-13}$$

In addition to satisfying Equation 4-13, the overturning resistance of the deadman must also be checked. This can be accomplished by taking moments about the top of the deadman. A safety factor of 1.5 is commonly used for overturning.

4.3.2 Short Deadmen Near Ground Surface

On occasion a deadman may also be buried in 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. Using a safety factor of 1.5, the required weight, W_d, in pounds is:

$$W_d = 1.5(P_r \sin\theta) \tag{4-14}$$

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

Granular Soils

Based on soil mechanics principles (Teng, 1962), the total nominal horizontal resistance to sliding, T_n, can be expressed as:

$$T_n \le L(P_p - P_a) + 0.33K_0\gamma\left(\sqrt{K_p} + \sqrt{K_a}\right)H^3 \tan\phi \tag{4-15}$$

where

H = depth of the deadman in soil from adjacent grade, ft

K_a = coefficient of active earth pressure

K_0 = coefficient of earth pressure at rest

K_p = coefficient of passive earth pressure

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

P_a = total active earth pressure, lb per lineal ft

P_p = total passive earth pressure, lb per lineal ft

γ = unit density of the soil, pcf

ϕ = angle of internal friction for the soil, deg

The following values may be used in typical situations:

$K_0 = 0.4$

$\gamma = 120$ pcf

$K_p = 3.0$

$K_a = 0.33$

$\tan\phi = 0.6$

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

$= 320H^2$

Thus, the total sliding resistance is:

$$T_n = 320LH^2 + 22H^3 \tag{4-16}$$

Using a safety factor of 1.5, the allowable resisting force is:

$$T_{all} = 213LH^2 + 15H^3 \tag{4-17}$$

Cohesive Soils

For cohesive soils, the horizontal resistance provided by the deadman can be calculated from the following equation (Teng, 1962):

$$T_n = L(P_p - P_a) + q_u H^2 \tag{4-18}$$

where

q_u = unconfined compressive strength of the soil, psf

From Teng (1962), $(P_p - P_a) = 2q_uH$. Using a conservative value of q_u = 1,500 psf, the equation then becomes:

$$(P_p - P_a) = 2q_uH$$
$$= 3{,}000H$$

Thus, the horizontal resistance of the deadman is:

$$T_n = 3{,}000LH + 1{,}500H^2 \tag{4-19}$$

Using a safety factor of 1.5, the allowable horizontal resistance is:

$$T_{all} = 2{,}000LH + 1{,}000H^2 \tag{4-20}$$

Example 4.3.2.1—Check Footing as Deadman near Ground Surface

Given:

Determine if a 6'-0"× 6'-0"× 1'-6" footing is adequate to use as a deadman near the ground surface for temporary bracing of the building in Example 4.2.3.1. The force in the diagonal is 14.4 kips (LRFD), 32° from horizontal. The soil at the footing is granular type. The footing uses normal weight concrete (γ = 150 lb/ft^3).

Solution:

Check Footing as Surface Deadman

Calculate the weight of the footing, $W_{footing}$:

$$W_{footing} = (150\text{ lb/ft}^3)(6\text{ ft})(6\text{ ft})(1.5\text{ ft})(1\text{ kip/1,000 lb})$$
$$= 8.10\text{ kips}$$

Using the partially erected structure as described in Example 4.2.3.1, the weight of the frame tributary to one interior column is:

$$W_{frame} = 13.1\text{ kips}$$

$$W_d = W_{footing} + W_{frame}$$
$$= 8.10\text{ kips} + 13.1\text{ kips}$$
$$= 21.2\text{ kips}$$

The horizontal component of force is:

$$P_n = P_r\cos\theta$$
$$= (14.4\text{ kips})(\cos 32°)$$
$$= 12.2\text{ kips}$$

The vertical component of force is:

$$P_v = P_r\sin\theta$$
$$= (14.4\text{ kips})(\sin 32°)$$
$$= 7.63\text{ kips}$$

Because this method uses service level loads, a factor of 0.6 is applied from ASCE/SEI 7-10, ASD Load Combination 7, 0.6D + 0.6W:

$$P_{ha} = 0.6(12.2\text{ kips})$$

$$= 7.32 \text{ kips}$$

$$P_{va} = 0.6(7.63 \text{ kips})$$
$$= 4.58 \text{ kips}$$

From Equation 4-12, with μ = 0.5 and dividing by the safety factor, the nominal horizontal resistance of the footing is:

$$R_n = \frac{\mu(W_d - P_r \sin\theta)}{1.5} \qquad \text{(from Eq. 4-12)}$$
$$= \frac{0.5(21.2 \text{ kips} - 4.58 \text{ kips})}{1.5}$$
$$= 5.54 \text{ kips} < 7.32 \text{ kips} \quad \textbf{n.g.}$$

Check Footing as Short Deadman near Ground Surface

The allowable sliding resistance of a deadman in granular soils is determined using Equation 4-17.

$$T_{all} = 213LH^2 + 15H^3 \qquad (4\text{-}17)$$

where

H = 1.50 ft

L = 6 ft

$$T_{all} = \left[213(6 \text{ ft})(1.50 \text{ ft})^2 + 15(1.50 \text{ ft})^3\right](1 \text{ kip}/1{,}000 \text{ lb})$$
$$= 2.93 \text{ kips} < 7.33 \text{ kips} \quad \textbf{n.g.}$$

Solving Equation 4-17 for the minimum H using the required horizontal force:

$$7{,}300 \text{ lb} = 213(6 \text{ ft})(H)^2 + 15(H)^3$$
$$H \approx 2.36 \text{ ft}$$

Try a 6'-0"×6'-0"×2'-6" footing.

Check Overturning

The anchor is attached to the footing top at the center of the footing, see Figure 4-4.

The overturning moment, M_{OT}, is:

$$M_{OT} = (8.64 \text{ kips})(\sin 32°)(3.00 \text{ ft}) + (8.64 \text{ kips})(\cos 32°)(2.50 \text{ ft})$$
$$= 32.1 \text{ kip-ft}$$

The resisting moment, M_R, is:

$$M_R = (6 \text{ ft})(6 \text{ ft})(2.5 \text{ ft})(150 \text{ lb/ft}^3)(3 \text{ ft})(1 \text{ kip}/1{,}000 \text{ lb}) + (13.1 \text{ kips})(3 \text{ ft})$$
$$= 79.8 \text{ kip-ft}$$

The safety factor for overturning is:

$$\frac{M_R}{M_{OT}} = \frac{79.8 \text{ kip-ft}}{32.1 \text{ kip-ft}}$$
$$= 2.49 > 1.50 \quad \textbf{o.k.}$$

In this example, the size of the footing required to resist the diagonal wire rope 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 temporary braced bays in the project.

In lieu of deadmen, an effective means of anchoring wire rope diagonals is the use of helical piers (also commonly referred to as helical anchors). These devices consist of a shaft to which helical blades are attached. These devices are turned into the soil and their strength is a function of the soil characteristics, number and size of helical blades, and the strength of the shaft. These devices are proprietary to the companies that have developed and manufactured them. Often the companies provide tables that relate installation torques to pull out strengths. If helical piers are considered, both a geotechnical engineer and the manufacturer or specialty contactor need to be involved. Needless to say, all potential conflicts with underground obstructions must be resolved, as well as whether these devices can be left in place or if they must be removed.

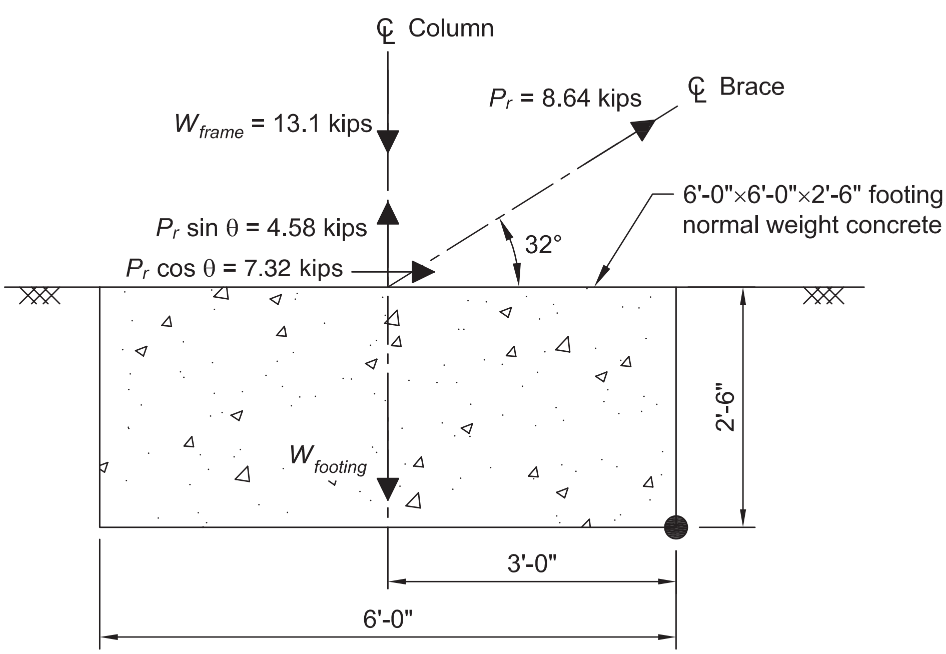


Fig. 4-4. Surface deadman diagram for Example 4.3.2.1.

Chapter 5
Determination of Bracing Requirements Using Prescriptive Requirements

This chapter presents a series of prescriptive requirements, which if followed, eliminates the need to use the calculation methods presented in Chapters 2, 3, and 4, 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.

5.1 PRESCRIPTIVE REQUIREMENTS FOR PERMANENT CONSTRUCTION

Tables 5-1 through 5-24 present prescriptive requirements that limit features of permanent construction. The features that 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 represented—30 ft, 40 ft, and 50 ft. The column heights presented are 15 ft, 30 ft, and 45 ft. Two types of anchor rod 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 that 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 3-14.

If the conditions upon which these tables are based are present in the construction, and the erector follows the requirements for erection sequence and wire rope 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. Wind speeds are those associated with risk category II, using ASCE/SEI 7-10.
2. The design wind pressures are those associated with a 115 mph basic wind speed. The tables are not valid for greater wind speeds. The design wind speed has been reduced for a six-week (or less) exposure duration as described in Section 2.2.1. Also, a design wind speed of 45 mph has been used for elements that are exposed to the wind for a continuously monitored work period of up to one full working day. 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 here for beam/column pairs.
3. In calculating wind forces on frames, 24-in.-deep solid web members and 48-in.-deep open web members were used. Member depths on the frame lines exceeding these maximum depths would invalidate the prescriptive requirements. Also, 12-in.-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 3,000 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 0.5 of 1% 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 overturning 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 in. The weld must be made on both sides of each flange and each side of the web. Lesser weld sizes and/or extents would require calculations as presented in Section 3.2.
6. Headed anchor rods are used in the tables. Washers are required, as discussed in Chapter 3.
7. In the determination of column base flexural 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 wire rope force to be resisted, the degree of base fixity provided by the column bases is considered. This has the effect of reducing the required wire rope force to be developed.
9. The tables require the placement of opposing pair diagonal wire rope 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 wire rope brace required for the one-story frames presented is a ½-in.-diameter wire rope with a minimum nominal breaking strength of 21,000 pounds for the 30- and 40-ft bays (as shown in Figure 5-1), and a ⅝-in.-diameter wire rope with a minimum breaking strength of 35,000 pounds for the 50-ft bays. For the two-story frames, a ⅝-in.-diameter wire rope with a minimum nominal breaking strength of 30,000 pounds is required for the 30- and 40-ft bays, and a ⅝-in.-diameter wire rope with a minimum nominal breaking strength of 35,000 pounds for the 50-ft bays. For the two-story frames, the diagonal wire rope bracing has been determined using the geometry associated with an X in each story. In certain situations erectors may find it advantageous to use a single X from the ground to the roof. In these situations, the table parameters do not apply and substitute calculations would have to be performed based on Chapters 3 and 4 of this Guide.
11. The wire rope diagonals can be anchored to the columns with Type A or Type B anchors as illustrated in Figures 4-1 and 4-2.

Anchor required for one-story frames:

Type A:
Plate thickness = ⅜ in.
L = 3 in.
Weld = 3⁄16-in. fillets
Grout thickness = 3 in., maximum

Type B:
Plate thickness = ⅝ in.
B = 4 in.
Grout thickness = 2 in., maximum for ¾-in.-diameter anchor rods
= 3 in., maximum for diameters greater than ¾ 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 ¾-in.-diameter anchor rods
= 3 in., maximum for diameters greater than ¾ in.

Type B:
Plate thickness = ½ in.
B = 5 in.
Grout thickness = 2 in., maximum for ¾-in.-diameter anchor rods
= 3 in., maximum for diameters greater than ¾ in.

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

5.2 PRESCRIPTIVE REQUIREMENTS FOR ERECTION SEQUENCE AND DIAGONAL BRACING

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

Figure 5-1 illustrates an erection plan with diagonal bracing in specific bays. It also identifies an initial box from which the erection is to commence. Figures 5-2 through 5-5 illustrate the build out from the initial box. The pattern of column, girder, column, girder, tie beam, cross-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 rods. This sequence 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 5.1.

The erection plan in Figure 5-1 illustrates columns, girders, tie members, and temporary cross-braces. This plan is divided into four erection sequences. Figure 5-1 contains features that are solely illustrative and others that are prescriptive.

The features are:

1. Proportion of bay: A square bay is shown and is required for use of the tables. The bay dimensions are 30 ft, 40 ft,

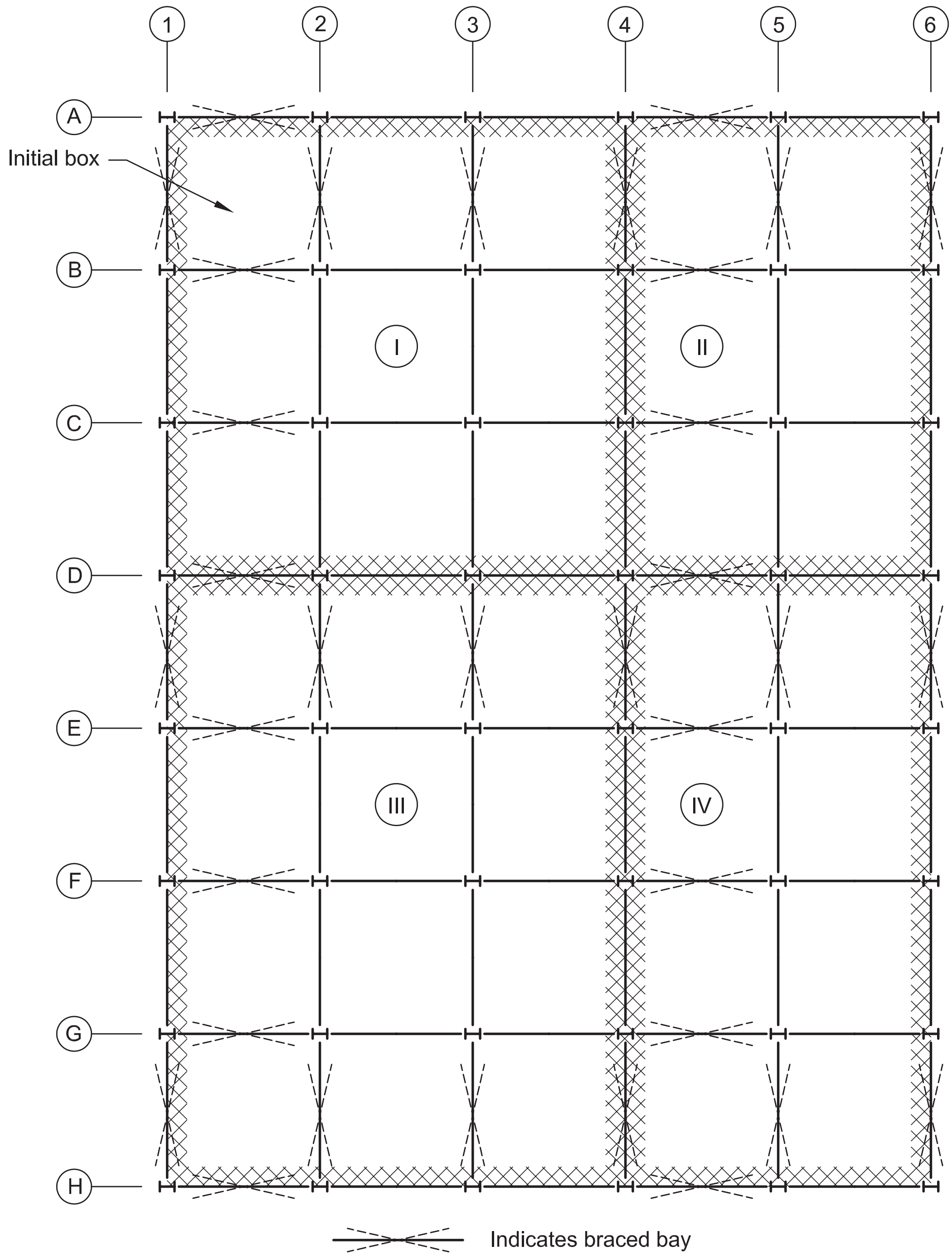


Fig. 5-1. Erection plan.

and 50 ft, as presented in Tables 5-1 through 5-24. Rectangular bays induce a different set of loads, wire rope forces, and angles, and the prescriptive requirements cannot be used. 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 HSS columns may also be used.
4. Erection sequences: Four (I to IV) erection sequences are illustrated. The number and pattern of erection sequences is not limited.
5. 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.
6. Progression from the initial box: The plan (Figure 5-1) and the supplementary figures (Figures 5-2 through 5-5) illustrate a progression from the initial box. This progression follows the 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 cross-brace. This is illustrated in Figure 5-3, which shows a cross-brace installed between columns Cl and C2 before the erection proceeds to grid line D.

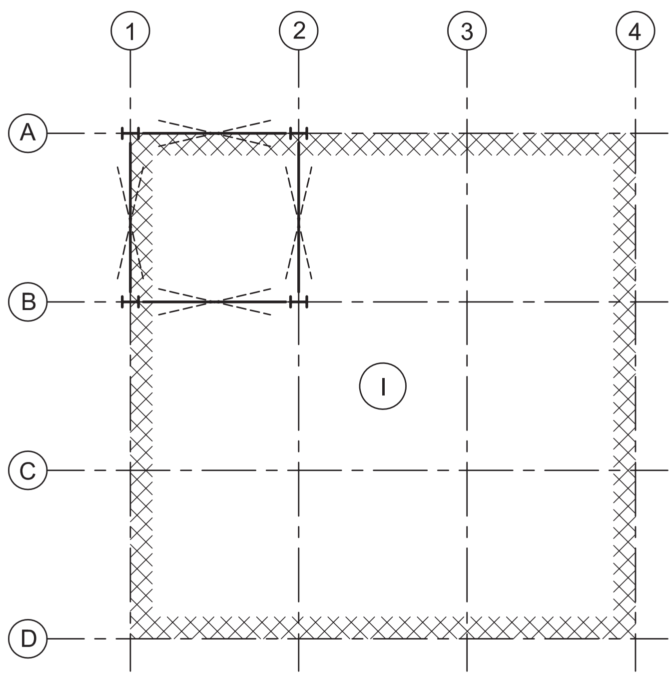


Fig. 5-2. Initial braced box.

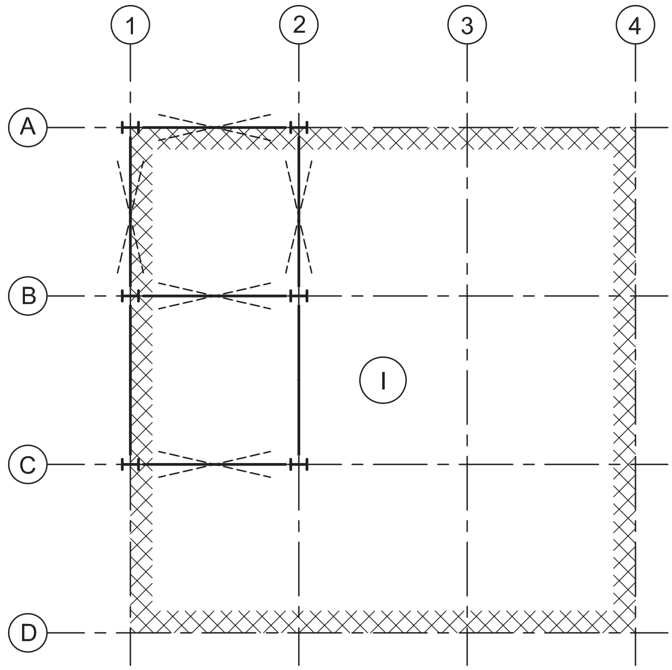


Fig. 5-3. Build out from initial box.

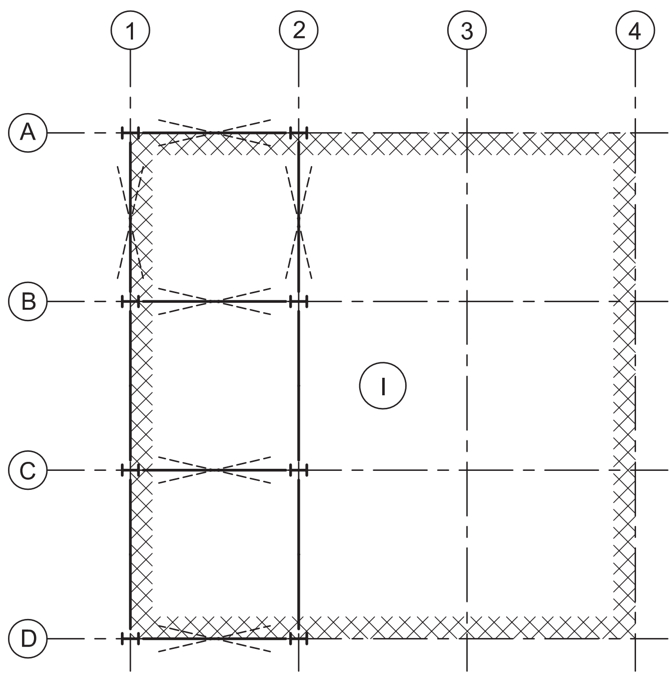


Fig. 5-4. Build out from initial box (continued).

7. Location of cross-braces: The plan shows cross-braces in the exterior bay 1-2. It is not required that cross-braces be located in exterior bays unless it is necessary to meet the prescriptive requirements. Cross-braces must be located according to 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 5-2 through 5-5).
8. Use of cross-braces: Each opposing wire rope pair is shown as a cross-brace. The opposing wire rope 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 5-2 and 5-3).
9. Use of temporary bracing: Figures 5-1 through 5-5 show the use of temporary bracing only. Permanent bracing may be used; however, this requires evaluation by the calculation methods shown previously to properly determine the interaction of permanent and temporary bracing.

For the analysis, the following criteria were used:

1. Joists were used as the filler beams and were spaced at 6.25 ft.
2. The three bays had a total of 25 fillers.
3. No deck weight was applied because the lack of this counterbalancing load is conservative.
4. Second-order analysis was done using the direct analysis method.
5. No construction live load was applied.
6. The wire rope used in the analysis had a modulus of E =13,000 ksi
7. Simple base fixity was used.

Lastly, temporary bracing must remain in place until its removal is permitted in accordance with the AISC *Code of Standard Practice*.

These notes apply to Tables 5-1 through 5-24:

- Footing thicknesses given are a minimum, which must be increased to match embedment length plus cover in some cases.
- Pier sizes given are the minimum size required for strength. A larger pier may be required to match the column provided.
- The anchor rod parameters given are minimums.

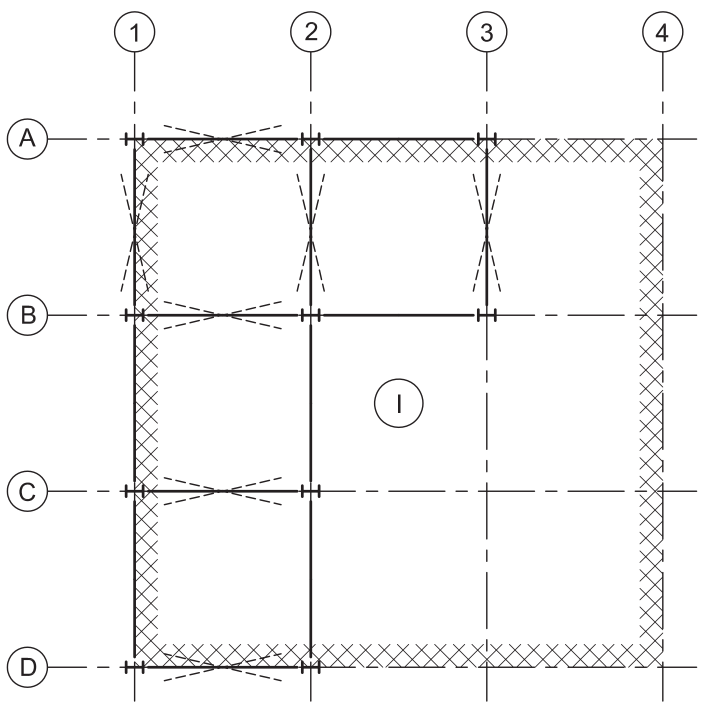


Fig. 5-5. Build out from initial box (continued).

Table 5-1. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4×4×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-2. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4½×4½×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-3. Prescriptive Requirements for Exposure B, 30-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	B
Bay size, ft	30
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1.0
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5½×5½×13
Anchor rods with leveling nuts	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	5×5
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	5×5
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-4. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4×4×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-5. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-6. Prescriptive Requirements for Exposure B, 40-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	B
Bay size, ft	40
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1.0
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5½×5½×17
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	5×5
Headed	4-in. hook
Embedment, in.	6
Cover below anchor, in.	9
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	5×5
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-7. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4×4×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-8. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	18×18
Footing size, ft × ft × in.	5×5×13
Anchor rods with leveling nuts	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Table 5-9. Prescriptive Requirements for Exposure B, 50-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	B
Bay size, ft	50
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1
Pier size, in. × in.	22×22
Footing size, ft × ft × in.	5½×5½×17
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	6
Cover below anchor, in.	3
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-10. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4×4×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-11. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Table 5-12. Prescriptive Requirements for Exposure C, 30-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	C
Bay size, ft	30
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	6×6×15
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Table 5-13. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4×4×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-14. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5½×5½×13
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	9
Cover below anchor, in.	6
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	3

Table 5-15. Prescriptive Requirements for Exposure C, 40-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	C
Bay size, ft	40
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1¼
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	6×6×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	9
Cover below anchor, in.	6
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	15×15
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Table 5-16. 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	15
Stories	1
Column size	W8×24
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	4½×4½×12
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-17. 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	30
Stories	1
Column size	W8×31
Base plate thickness, in.	⅞
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5½×5½×17
Anchor rods with leveling nuts	
Anchor rod, diameter, in.	¾
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods, base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	3

Table 5-18. Prescriptive Requirements for Exposure C, 50-ft Bays, 45-ft Column Height, One-Story Frame	
Exposure Category	C
Bay size, ft	50
Column height, ft	45
Stories	1
Column size	W12×65
Base plate thickness, in.	1¼
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	6½×6½×16
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	15×15
Headed	4-in. hook
Embedment, in.	6
Cover below anchor, in.	9
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	15×15
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	3

Table 5-19. Prescriptive Requirements for Exposure B, 30-ft Bays, 20-ft Column Height, Two-Story Frame	
Exposure Category	B
Bay size, ft	30
Column height, ft	20
Stories	2
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-20. Prescriptive Requirements for Exposure B, 40-ft Bays, 30-ft Column Height, Two-Story Frame	
Exposure Category	B
Bay size, ft	40
Column height, ft	30
Stories	2
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	6
Cover below anchor, in.	3

Table 5-21. Prescriptive Requirements for Exposure B, 50-ft Bays, 30-ft Column Height, Two-Story Frame	
Exposure Category	B
Bay size, ft	50
Column height, ft	30
Stories	2
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods base plate shimmed or pregrouted	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Table 5-22. Prescriptive Requirements for Exposure C, 30-ft Bays, 30-ft Column Height, Two-Story Frame	
Exposure Category	C
Bay size, ft	30
Column height, ft	30
Stories	2
Column size	W8×31
Base plate thickness, in.	¾
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	3

Table 5-23. Prescriptive Requirements for Exposure C, 40-ft Bays, 30-ft Column Height, Two-Story Frame	
Exposure Category	C
Bay size, ft	40
Column height, ft	30
Stories	2
Column size	W8×31
Base plate thickness, in.	1
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	5×5×18
Anchor rods with leveling nuts	
Anchor rod diameter, in.	¾
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	6
Cover below anchor, in.	6
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	1
Anchor pattern, in. × in.	4×4
Headed	4-in. hook
Embedment, in.	9
Cover below anchor, in.	3

Table 5-24. Prescriptive Requirements for Exposure C, 50-ft Bays, 30-ft Column Height, Two-Story Frame	
Exposure Category	C
Bay size, ft	50
Column height, ft	30
Stories	2
Column size	W8×31
Base plate thickness, in.	1
Pier size, in. × in.	12×12
Footing size, ft × ft × in.	6×6×14
Anchor rods with leveling nuts	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	9
Cover below anchor, in.	9
Anchor rods and base plate shimmed or pregrouted	
Anchor rod diameter, in.	⅞
Anchor pattern, in. × in.	11×11
Headed	—
Embedment, in.	9
Cover below anchor, in.	3

Appendix

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Table A-l—Available Flexural Strength of Base Plates with Inset Anchor Rods Based on Weld Strength

For the column sizes indicated, the minor-axis flexural strength for the column-to-base plate connection is provided. The failure mode is shown in Figure 3-1. The available strengths are based on Equation 3-2 using a 5⁄16-in. fillet weld pattern as shown in Figure 3-19.

Table A-2—Available Flexural Strength of Base Plates with Inset Anchor Rods Based on Plate Strength

Available strengths are provided for the parameters as shown in the table. The table is based on Equations 3-4 and 3-5.

Tables A-3a (LRFD) and A-3b (ASD) —Available Flexural Strength of Base Plates with Outset Anchor Rods

For the column sizes indicated, the available flexural strength 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 available strengths are applicable to loads applied about either axis of the column. The available strength is based on Equation 3-9 using 5⁄16-in. fillet welds 2 in. long and the anchor rod offset from the flange tip by 2 in. in each direction.

Table A-4 —Available Flexural Strength of HSS Column Base Plates

For the HSS column size and anchor rod spacing shown, the available flexural strength of the column base plate is provided. The table is based on 4-in.-long 5⁄16-in. fillet welds and E70XX electrodes with the anchor rod offset from the flange tip by 2 in. in each direction. The columns are assumed to be welded all around. The available strength is based on Equation 3-9.

Table A-5 —Available Tensile Strength of Anchor Rods Based on Anchor Rod Strength

Table A-5 provides the available tensile strengths for ASTM F1554 Grade 36 anchor rods. The values provided in the table are taken directly from the AISC *Manual*. The failure mode associated with the table values is that of anchor rod rupture as shown in Figure 3-7.

Table A-6a (LRFD) and A-6b (ASD) —Available Tensile Strength of Hooked Anchor Rods Based on Hook Length, $f_c' = 3{,}000$ psi

The values provided in Table A-6 are derived from Equation 3-17. The values are somewhat conservative in that no allowance for strength is provided for any bond between the anchor rod and the concrete. Inclusion of the bond strength can be unconservative because anchor rods are often oily after the threads are cut. The values provided in Table A-6 are based on a concrete strength of 3,000 psi.

Table A-7a (LRFD) and A-7b (ASD) —Available Tensile Strength of Hooked Anchor Rods Based on Hook Length, $f_c' = 4{,}000$ psi

Table A-7 is identical to Table A-6 except that a concrete strength of 4,000 psi is used to determine the provided values.

Table A-8—Design Tensile Strength of Single Anchor Rods Based on Concrete Pullout Capacity

Presented in Table A-8 are breakout resistances for the anchor rod sizes and embedment depths shown. The concrete strength used for the calculations is 3,000 psi. The values are for single anchor rods—that is, no group action. The values are based on the breakout strength of the concrete truncated pyramid.

Tables A-9, A-10, A-11, and A-12—Concrete Breakout Strength of Two Anchor Rods Based on Spacing and Embedment

Breakout resistances for a group of two anchor rods are presented. The tables are based on a concrete strength of 3,000 psi and embedment depths equal to 9, 12, 15, and 18 in. Equation 3-23 is used to determine the values.

Table A-13—Pushout Strength of Two Anchor Rods

Pushout values for a group of two anchor rods are provided. Equation 3-17 was used to determine the values shown. A concrete strength of 3,000 psi is used. A clear cover of 3, 6, 9, and 12 in. under the nut or hook of the anchor rod were used to determine the values shown.

Table A-14—Available Flexural Strength of Single Square Concrete Piers

Available flexural strengths are provided for the data shown in the table. Equation 3-28 was used with a concrete strength of 3,000 psi to determine the listed values.

Table A-15—Concrete Footing Overturning Resistance

Overturning resistances are provided for the footing sizes shown. The values are based on Equation 3-31. Only the dead weight of the footing is used in determining the values.

Table A-16—Reinforcing Bars Required Development Length, $f'_c = 3{,}000$ psi

The required development length for Grade 60 hooked and straight reinforcing bars is shown in Table A-16. Equations 3-29, 3-30a, and 3-30b are used to determine the development lengths with a concrete strength of 3,000 psi.

Table A-17—Reinforcing Bars Required Development Length, $f'_c = 4{,}000$ psi

Table A-17 is identical to Table A-16 with the exception that $f'_c = 4{,}000$ psi.

Table A-18—Type A Anchor Plate Dimensions and Weld Sizes

This table provides plate height, plate thickness, and fillet weld size for an ASTM A36 plate Type A for the wire rope type and slopes presented. A plate of this geometry and attachment will develop the wire rope design force using a minimum safety factor of 3 in selecting the wire rope. The Type A plate is shown in Figure 4-1. The table data was determined using the calculation method in Example 4.2.1.1.

Table A-19—Allowable Wire Rope Force, Type A Plate Anchor Limited by Anchor Rod Strength

This table provides the maximum unfactored wire rope force for the parameters presented based on the calculation method and material strengths in Example 4.2.3.1.

Table A-20—Type B Anchor Plates Dimensions and Weld Sizes

This table provides the plate width and thickness for an ASTM A36 plate Type B for the wire rope types and slopes presented. A plate of this geometry will develop the wire rope design force using a minimum safety factor of 3 in selecting the wire rope. The Type B plate is shown in Figure 4-2. The table data was determined using the calculation method in Example 4.2.2.1.

Table A-21—Allowable Wire Rope Force, Type B Plate Anchor Limited by Anchor Rod Strength

This table provides the maximum unfactored wire rope force for the parameters presented based on the calculation method and material strengths in Example 4.2.3.2.

Table A-1. Available Flexural Strength of Base Plates with Inset Anchor Rods Based on Weld Strength 5⁄16-in. Fillet Welds; E70XX Electrodes						
Shape	***d*, in.**	**b_f, in.**	**ϕM_{nx}, kip-ft**	**ϕM_{ny}, kip-ft**	**M_{nx}/Ω, kip-ft**	**M_{ny}/Ω, kip-ft**
W8×24	7⅞	6½	47.2	12.3	31.5	8.17
W8×28	8	6½	47.5	12.3	31.7	8.17
W8×31	8	8	57.5	18.6	38.4	12.4
W8×35	8⅛	8	57.9	18.6	38.6	12.4
W8×40	8¼	8⅛	59.1	19.1	39.4	12.8
W8×48	8½	8⅛	60.0	19.1	40.0	12.8
W10×33	9¾	8	72.4	18.6	48.3	12.4
W10×39	9⅞	8	72.6	18.6	48.4	12.4
W10×45	10⅛	8	73.6	18.6	49.1	12.4
W10×49	10	10	89.6	29.0	59.8	19.3
W10×54	10⅛	10	90.1	29.0	60.1	19.3
W10×60	10¼	10⅛	91.7	29.7	61.1	19.8
W10×68	10⅜	10⅛	92.0	29.7	61.3	19.8
W12×40	12	8	92.5	18.6	61.7	12.4
W12×45	12	8	91.7	18.6	61.1	12.4
W12×50	12¼	8⅛	94.5	19.1	63.0	12.8
W12×53	12	10	111	29.0	73.9	19.3
W12×58	12¼	10	113	29.0	75.2	19.3
W12×65	12⅛	12	132	41.8	87.8	27.8
W12×72	12¼	12	132	41.8	88.1	27.8
W12×79	12⅜	12⅛	134	42.6	89.4	28.4
W12×87	12½	12⅛	135	42.6	89.7	28.4
W14×61	13⅞	10	132	29.0	87.6	19.3
W14×68	14	10	131	29.0	87.6	19.3
W14×74	14⅛	10⅛	133	29.7	88.8	19.8
W14×82	14¼	10⅛	134	29.7	89.1	19.8
W14×90	14	14½	184	61.0	123	40.7
W14×99	14⅛	14⅝	186	62.0	124	41.3
W14×109	14⅜	14⅝	188	62.0	125	41.3
W14×120	14½	14⅝	189	62.0	126	41.3
W14×132	14⅝	14¾	191	63.1	127	42.1

Table A-2. Available Flexural Strength of Base Plates with Inset Anchor Rods Based on Plate Strength ASTM A36 Plate (F_y = 36 ksi)

Shape	Anchor Rod Spacing, in.	Base Plate Plan Size, in.	ϕM_{nx}, kip-ft			ϕM_{ny}, kip-ft			M_{nx}/Ω, kip-ft			M_{ny}/Ω, kip-ft		
			Base Plate Thickness, in.											
		$L \times W$	¾	1	1¼	¾	1	1¼	¾	1	1¼	¾	1	1¼
W8×24	4	9 × 7	11.3	20.1	31.4	11.3	20.1	31.4	7.53	13.4	20.9	7.53	13.4	20.9
W8×28	4	9 × 7	11.3	20.1	31.4	11.3	20.1	31.4	7.51	13.4	20.9	7.51	13.4	20.9
W8×31	4	9 × 9	12.6	22.3	34.9	12.6	22.3	34.9	8.36	14.9	23.2	8.36	14.9	23.2
W8×35	4	9 × 9	12.5	22.2	34.8	12.5	22.2	34.8	8.32	14.8	23.1	8.32	14.8	23.1
W8×40	4	9 × 9	12.5	22.2	34.7	12.5	22.2	34.7	8.31	14.8	23.1	8.31	14.8	23.1
W8×48	4	9 × 9	12.4	22.0	34.4	12.4	22.0	34.4	8.24	14.7	22.9	8.24	14.7	22.9
W10×33	4	11 × 9	10.6	18.9	29.6	10.6	18.9	29.6	7.08	12.6	19.7	7.08	12.6	19.7
	5	11 × 9	14.1	25.1	39.2	14.1	25.1	39.2	9.38	16.7	26.1	9.38	16.7	26.1
W10×39	4	11 × 9	10.6	18.8	29.4	10.6	18.8	29.4	7.04	12.5	19.5	7.04	12.5	19.5
	5	11 × 9	14.0	25.0	39.0	14.0	25.0	39.0	9.34	16.6	25.9	9.34	16.6	25.9
W10×45	4	11 × 9	10.5	18.7	29.2	10.5	18.7	29.2	7.00	12.4	19.4	7.00	12.4	19.4
	5	11 × 9	14.0	24.9	38.9	14.0	24.9	38.9	9.31	16.6	25.9	9.31	16.6	25.9
W10×49	4	11 × 11	11.7	20.7	32.4	11.7	20.7	32.4	7.76	13.8	21.5	7.76	13.8	21.5
	5	11 × 11	15.7	27.9	43.7	15.7	27.9	43.7	10.5	18.6	29.1	10.5	18.6	29.1
	6	11 × 11	20.4	36.3	56.7	20.4	36.3	56.7	13.6	24.1	37.7	13.6	24.1	37.7
W10×54	4	11 × 11	11.6	20.6	32.3	11.6	20.6	32.3	7.73	13.7	21.5	7.73	13.7	21.5
	5	11 × 11	15.7	27.9	43.5	15.7	27.9	43.5	10.4	18.5	29.0	10.4	18.5	29.0
	6	11 × 11	20.3	36.0	56.3	20.3	36.0	56.3	13.5	24.0	37.5	13.5	24.0	37.5
W10×60	4	11 × 11	11.6	20.7	32.3	11.6	20.7	32.3	7.74	13.8	21.5	7.74	13.8	21.5
	5	11 × 11	15.7	27.9	43.6	15.7	27.9	43.6	10.5	18.6	29.0	10.5	18.6	29.0
	6	11 × 11	20.3	36.1	56.4	20.3	36.1	56.4	13.5	24.0	37.5	13.5	24.0	37.5
W10×68	4	11 × 11	11.5	20.5	32.0	11.5	20.5	32.0	7.67	13.6	21.3	7.67	13.6	21.3
	5	11 × 11	15.6	27.7	43.3	15.6	27.7	43.3	10.4	18.4	28.8	10.4	18.4	28.8
	6	11 × 11	20.0	35.5	55.5	20.0	35.5	55.5	13.3	23.6	36.9	13.3	23.6	36.9
W12×40	4	13 × 9	9.37	16.7	26.0	9.37	16.7	26.0	6.23	11.1	17.3	6.23	11.1	17.3
	5	13 × 9	12.4	22.0	34.3	12.4	22.0	34.3	8.22	14.6	22.8	8.22	14.6	22.8
W12×45	4	13 × 9	9.32	16.6	25.9	9.32	16.6	25.9	6.20	11.0	17.2	6.20	11.0	17.2
	5	13 × 9	12.3	21.8	34.1	12.3	21.8	34.1	8.17	14.5	22.7	8.17	14.5	22.7
W12×50	4	13 × 9	9.32	16.6	25.9	9.32	16.6	25.9	6.20	11.0	17.2	6.20	11.0	17.2
	5	13 × 9	12.3	21.8	34.1	12.3	21.8	34.1	8.16	14.5	22.7	8.16	14.5	22.7
W12×53	4	13 × 11	10.1	18.0	28.1	10.1	18.0	28.1	6.73	12.0	18.7	6.73	12.0	18.7
	5	13 × 11	13.4	23.8	37.3	13.4	23.8	37.3	8.92	15.9	24.8	8.92	15.9	24.8
	6	13 × 11	17.0	30.2	47.2	17.0	30.2	47.2	11.3	20.1	31.4	11.3	20.1	31.4
W12×58	4	13 × 11	10.1	17.9	28.0	10.1	17.9	28.0	6.72	11.9	18.7	6.72	11.9	18.7
	5	13 × 11	13.4	23.8	37.2	13.4	23.8	37.2	8.90	15.8	24.7	8.90	15.8	24.7
	6	13 × 11	17.0	30.2	47.1	17.0	30.2	47.1	11.3	20.1	31.4	11.3	20.1	31.4
W12×65	4	13 × 13	10.9	19.4	30.4	10.9	19.4	30.4	7.28	12.9	20.2	7.28	12.9	20.2
	5	13 × 13	14.6	26.0	40.6	14.6	26.0	40.6	9.72	17.3	27.0	9.72	17.3	27.0
	6	13 × 13	18.7	33.2	51.9	18.7	33.2	51.9	12.4	22.1	34.5	12.4	22.1	34.5

Table A-2 (cont'd). Available Flexural Strength of Base Plates with Inset Anchor Rods Based on Plate Strength
ASTM A36 Plate (F_y = 36 ksi)

Shape	Anchor Rod Spacing, in.	Base Plate Plan Size, in.	ϕM_{nx}, kip-ft			ϕM_{ny}, kip-ft			M_{nx}/Ω, kip-ft			M_{ny}/Ω, kip-ft		
			Base Plate Thickness, in.											
		$L \times W$	¾	1	1¼	¾	1	1¼	¾	1	1¼	¾	1	1¼
W12×72	4	13 × 13	10.9	19.3	30.1	10.9	19.3	30.1	7.22	12.8	20.1	7.22	12.8	20.1
	5	13 × 13	14.5	25.7	40.2	14.5	25.7	40.2	9.62	17.1	26.7	9.62	17.1	26.7
	6	13 × 13	18.5	33.0	51.5	18.5	33.0	51.5	12.3	21.9	34.3	12.3	21.9	34.3
W12×79	4	13 × 13	10.9	19.3	30.2	10.9	19.3	30.2	7.23	12.9	20.1	7.23	12.9	20.1
	5	13 × 13	14.5	25.8	40.2	14.5	25.8	40.2	9.64	17.1	26.8	9.64	17.1	26.8
	6	13 × 13	18.6	33.0	51.6	18.6	33.0	51.6	12.4	22.0	34.4	12.4	22.0	34.4
W12×87	4	13 × 13	10.9	19.3	30.2	10.9	19.3	30.2	7.22	12.8	20.1	7.22	12.8	20.1
	5	13 × 13	14.5	25.7	40.2	14.5	25.7	40.2	9.62	17.1	26.7	9.62	17.1	26.7
	6	13 × 13	18.6	33.0	51.6	18.6	33.0	51.6	12.4	22.0	34.3	12.4	22.0	34.3
W14×61	4	15 × 11	9.36	16.6	26.0	9.36	16.6	26.0	6.23	11.1	17.3	6.23	11.1	17.3
	5	15 × 11	12.2	21.7	33.9	12.2	21.7	33.9	8.11	14.4	22.5	8.11	14.4	22.5
	6	15 × 11	15.4	27.4	42.8	15.4	27.4	42.8	10.2	18.2	28.4	10.2	18.2	28.4
W14×68	4	15 × 11	9.35	16.6	26.0	9.35	16.6	26.0	6.22	11.1	17.3	6.22	11.1	17.3
	5	15 × 11	12.2	21.7	33.8	12.2	21.7	33.8	8.10	14.4	22.5	8.10	14.4	22.5
	6	15 × 11	15.4	27.3	42.7	15.4	27.3	42.7	10.2	18.2	28.4	10.2	18.2	28.4
W14×74	4	15 × 11	9.33	16.6	25.9	9.33	16.6	25.9	6.21	11.0	17.2	6.21	11.0	17.2
	5	15 × 11	12.2	21.6	33.8	12.2	21.6	33.8	8.08	14.4	22.5	8.08	14.4	22.5
	6	15 × 11	15.3	27.2	42.6	15.3	27.2	42.6	10.2	18.1	28.3	10.2	18.1	28.3
W14×82	4	15 × 11	9.32	16.6	25.9	9.32	16.6	25.9	6.20	11.0	17.2	6.20	11.0	17.2
	5	15 × 11	12.1	21.6	33.7	12.1	21.6	33.7	8.07	14.4	22.4	8.07	14.4	22.4
	6	15 × 11	15.3	27.2	42.5	15.3	27.2	42.5	10.2	18.1	28.3	10.2	18.1	28.3
W14×90	4	15 × 15	10.8	19.2	30.0	10.8	19.2	30.0	7.20	12.8	20.0	7.20	12.8	20.0
	5	15 × 15	14.2	25.3	39.5	14.2	25.3	39.5	9.47	16.8	26.3	9.47	16.8	26.3
	6	15 × 15	18.2	32.3	50.5	18.2	32.3	50.5	12.1	21.5	33.6	12.1	21.5	33.6
	8	15 × 15	26.7	47.5	74.3	26.7	47.5	74.3	17.8	31.6	49.4	17.8	31.6	49.4
W14×99	4	15 × 15	10.8	19.2	30.0	10.8	19.2	30.0	7.17	12.8	19.9	7.17	12.8	19.9
	5	15 × 15	14.2	25.2	39.4	14.2	25.2	39.4	9.43	16.8	26.2	9.43	16.8	26.2
	6	15 × 15	18.1	32.1	50.2	18.1	32.1	50.2	12.0	21.4	33.4	12.0	21.4	33.4
	8	15 × 15	26.6	47.2	73.8	26.6	47.2	73.8	17.7	31.4	49.1	17.7	31.4	49.1
W14×109	4	15 × 15	10.8	19.2	29.9	10.8	19.2	29.9	7.17	12.7	19.9	7.17	12.7	19.9
	5	15 × 15	14.2	25.2	39.3	14.2	25.2	39.3	9.42	16.8	26.2	9.42	16.8	26.2
	6	15 × 15	18.1	32.1	50.1	18.1	32.1	50.1	12.0	21.4	33.4	12.0	21.4	33.4
	8	15 × 15	26.5	47.1	73.6	26.5	47.1	73.6	17.6	31.4	49.0	17.6	31.4	49.0
W14×120	4	15 × 15	10.7	19.1	29.8	10.7	19.1	29.8	7.15	12.7	19.9	7.15	12.7	19.9
	5	15 × 15	14.1	25.1	39.2	14.1	25.1	39.2	9.39	16.7	26.1	9.39	16.7	26.1
	6	15 × 15	18.0	32.0	49.9	18.0	32.0	49.9	12.0	21.3	33.2	12.0	21.3	33.2
	8	15 × 15	26.4	46.9	73.2	26.4	46.9	73.2	17.5	31.2	48.7	17.5	31.2	48.7
W14×132	4	15 × 15	10.7	19.0	29.7	10.7	19.0	29.7	7.11	12.6	19.8	7.11	12.6	19.8
	5	15 × 15	14.0	24.9	39.0	14.0	24.9	39.0	9.34	16.6	25.9	9.34	16.6	25.9
	6	15 × 15	17.8	31.7	49.6	17.8	31.7	49.6	11.9	21.1	33.0	11.9	21.1	33.0
	8	15 ×15	26.1	46.5	72.6	26.1	46.5	72.6	17.4	30.9	48.3	17.4	30.9	48.3

Table A-3a. Available Flexural Strength of Base Plates with Outset Anchor Rods (LRFD) 5⁄16-in. Fillet Welds; E70XX Electrode										
			Rod Pattern		ϕM_{nx}, kip-ft			ϕM_{ny}, kip-ft		
					Base Plate Thickness, in.			Base Plate Thickness, in.		
Shape	***d*, in.**	**b_f, in.**	***L*, in.**	***W*, in.**	**¾**	**1**	**1¼**	**¾**	**1**	**1¼**
W8×24	7⅞	6½	12	10½	12.9	19.9	27.8	11.3	17.4	24.3
W8×28	8	6½	12	10½	12.9	19.9	27.8	11.3	17.4	24.3
W8×31	8	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8×35	8⅛	8	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8×40	8¼	8⅛	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W8×48	8½	8⅛	12	12	12.9	19.9	27.8	12.9	19.9	27.8
W10×33	9¾	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10×39	9⅞	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10×45	10⅛	8	14	12	15.1	23.2	32.4	12.9	19.9	27.8
W10×49	10	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10×54	10⅛	10	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10×60	10¼	10⅛	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W10×68	10⅜	10⅛	14	14	15.1	23.2	32.4	15.1	23.2	32.4
W12×40	12	8	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12×45	12	8	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12×50	12¼	8⅛	16	12	17.2	26.5	37.1	12.9	19.9	27.8
W12×53	12	10	16	14	17.2	26.5	37.1	15.1	23.2	32.4
W12×58	12¼	10	16	14	17.2	26.5	37.1	15.1	23.2	32.4
W12×65	12⅛	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12×72	12¼	12	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12×79	12⅜	12⅛	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W12×87	12½	12⅛	16	16	17.2	26.5	37.1	17.2	26.5	37.1
W14×61	13⅞	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14×68	14	10	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14×74	14⅛	10⅛	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14×82	14¼	10⅛	18	14	19.4	29.8	41.7	15.1	23.2	32.4
W14×90	14	14½	18	18½	19.4	29.8	41.7	19.9	30.7	42.8
W14×99	14⅛	14⅝	18	18⅝	19.4	29.8	41.7	20.0	30.9	43.1
W14×109	14⅜	14⅝	18	18⅝	19.4	29.8	41.7	20.0	30.9	43.1
W14×120	14½	14⅝	18	18⅝	19.4	29.8	41.7	20.0	30.9	43.1
W14×132	14⅝	14¾	18	18¾	19.4	29.8	41.7	20.2	31.1	43.4

Table A-3b. Available Flexural Strength of Base Plates with Outset Anchor Rods (ASD) ⁵⁄₁₆-in. Fillet Welds; E70XX Electrode										
			Rod Pattern		M_{nx}/Ω, kip-ft			M_{ny}/Ω, kip-ft		
					Base Plate Thickness, in.			Plate Thickness, in.		
Shape	*d*, in.	b_f, in.	*L*, in.	*W*, in.	¾	1	1¼	¾	1	1¼
W8×24	7⅞	6½	12	10½	8.59	13.2	18.5	7.52	11.6	16.2
W8×28	8	6½	12	10½	8.59	13.2	18.5	7.52	11.6	16.2
W8×31	8	8	12	12	8.59	13.2	18.5	8.59	13.2	18.5
W8×35	8⅛	8	12	12	8.59	13.2	18.5	8.59	13.2	18.5
W8×40	8¼	8⅛	12	12	8.59	13.2	18.5	8.59	13.2	18.5
W8×48	8½	8⅛	12	12	8.59	13.2	18.5	8.59	13.2	18.5
W10×33	9¾	8	14	12	10.0	15.4	21.6	8.59	13.2	18.5
W10×39	9⅞	8	14	12	10.0	15.4	21.6	8.59	13.2	18.5
W10×45	10⅛	8	14	12	10.0	15.4	21.6	8.59	13.2	18.5
W10×49	10	10	14	14	10.0	15.4	21.6	10.0	15.4	21.6
W10×54	10⅛	10	14	14	10.0	15.4	21.6	10.0	15.4	21.6
W10×60	10¼	10⅛	14	14	10.0	15.4	21.6	10.0	15.4	21.6
W10×68	10⅜	10⅛	14	14	10.0	15.4	21.6	10.0	15.4	21.6
W12×40	12	8	16	12	11.5	17.6	24.7	8.59	13.2	18.5
W12×45	12	8	16	12	11.5	17.6	24.7	8.59	13.2	18.5
W12×50	12¼	8⅛	16	12	11.5	17.6	24.7	8.59	13.2	18.5
W12×53	12	10	16	14	11.5	17.6	24.7	10.0	15.4	21.6
W12×58	12¼	10	16	14	11.5	17.6	24.7	10.0	15.4	21.6
W12×65	12⅛	12	16	16	11.5	17.6	24.7	11.5	17.6	24.7
W12×72	12¼	12	16	16	11.5	17.6	24.7	11.5	17.6	24.7
W12×79	12⅜	12⅛	16	16	11.5	17.6	24.7	11.5	17.6	24.7
W12×87	12½	12⅛	16	16	11.5	17.6	24.7	11.5	17.6	24.7
W14×61	14	10	18	14	12.9	19.8	27.7	10.0	15.4	21.6
W14×68	14	10	18	14	12.9	19.8	27.7	10.0	15.4	21.6
W14×74	14	10	18	14	12.9	19.8	27.7	10.0	15.4	21.6
W14×82	14	10	18	14	12.9	19.8	27.7	10.0	15.4	21.6
W14×90	14	14½	18	18½	12.9	19.8	27.7	13.2	20.4	28.5
W14×99	14	14⅝	18	18⅝	12.9	19.8	27.7	13.3	20.5	28.7
W14×109	14	14⅝	18	18⅝	12.9	19.8	27.7	13.3	20.5	28.7
W14×120	14	14⅝	18	18⅝	12.9	19.8	27.7	13.3	20.5	28.7
W14×132	14	14¾	18	18¾	12.9	19.8	27.7	13.4	20.7	28.9

Table A-4. Available Flexural Strength of HSS Column Base Plates 5⁄16-in. Fillet Welds, E70XX Electrode							
Nominal HSS Size	Anchor Rod Spacing, in.	ϕM_n, kip-ft			M_n/Ω, kip-ft		
		Base Plate Thickness, in.			Base Plate Thickness, in.		
		¾	1	1¼	¾	1	1¼
4×4	8	8.61	13.3	18.5	5.73	8.82	12.3
5×5	9	9.69	14.9	20.8	6.45	9.92	13.9
6×6	10	10.8	16.6	23.2	7.16	11.0	15.4
8×8	12	12.9	19.9	27.8	8.59	13.2	18.5
10×10	14	15.1	23.2	32.4	10.0	15.4	21.6
12×12	16	17.2	26.5	37.1	11.5	17.6	24.7

Table A-5. Available Tensile Strength of ASTM F1554 Grade 36 Anchor Rods Based on Anchor Rod Strength				
Rod Diameter, in.				
¾	⅞	1	1⅛	1¼
Tension Area, in.^{2}				
0.442	0.601	0.785	0.994	1.23
LRFD: ϕP_n, kips/rod				
14.4	19.6	25.6	32.4	40.0
ASD: P_n/Ω, kips/rod				
9.60	13.1	17.1	17.1	26.7

Table A-6a. Available Tensile Strength of Hooked Anchor Rods Based on Hook Length (LRFD) ASTM F1554 Grade 36 Rods, $f'_c = 3{,}000$ psi					
Rod Diameter, in.	¾	⅞	1	1⅛	1¼
Tension Area, in.^{2}	**0.442**	**0.601**	**0.785**	**0.994**	**1.23**
Hook Length, in.	ϕP_n, kips/rod				
3	5.95	6.95	7.94	—	—
4	6.70	9.12	10.6	11.9	13.2
5	6.70	9.12	11.9	14.9	16.5
6	6.70	9.12	11.9	15.1	18.6

Table A-6b. Available Tensile Strength of Hooked Anchor Rods Based on Hook Length (ASD) ASTM F1554 Grade 36, $f'_c = 3{,}000$ psi					
Rod Diameter, in.	**¾**	**⅞**	**1**	**1⅛**	**1¼**
Tension Area, in.²	**0.442**	**0.601**	**0.785**	**0.994**	**1.23**
Hook Length, in.	**P_n/Ω, kips/rod**				
3	3.97	4.63	5.30	—	—
4	4.47	6.08	7.06	7.94	8.83
5	4.47	6.08	7.94	9.93	11.0
6	4.47	6.08	7.94	10.1	12.4

Table A-7a. Available Tensile Strength of Hooked Anchor Rods Based on Hook Length (LRFD) ASTM F1554 Grade 36 Rods, $f'_c = 4{,}000$ psi					
Rod Diameter, in.	**¾**	**⅞**	**1**	**1⅛**	**1¼**
Tension Area, in.²	**0.442**	**0.601**	**0.785**	**0.994**	**1.23**
Hook Length, in.	**ϕP_n, kips/rod**				
3	7.94	9.26	10.6	—	—
4	8.93	12.2	14.1	15.9	17.6
5	8.93	12.2	15.9	19.8	22.1
6	8.93	12.2	15.9	20.1	24.8

Table A-7b. Available Tensile Strength of Hooked Anchor Rods Based on Hook Length (ASD) ASTM F1554 Grade 36 Rods, $f'_c = 4{,}000$ psi					
Rod Diameter, in.	**¾**	**⅞**	**1**	**1⅛**	**1¼**
Tension Area, in.²	**0.442**	**0.601**	**0.785**	**0.994**	**1.23**
Hook Length, in.	**P_n/Ω, kips/rod**				
3	5.29	6.17	7.06	—	—
4	5.95	8.10	9.41	10.6	11.8
5	5.95	8.10	10.6	13.2	14.7
6	5.95	8.10	10.6	13.4	16.5

Table A-8. Design Tensile Strength of Single Anchor Rods Based on Concrete Pullout Capacity, ϕP_n, kips/rod					
Embedment Depth, in.	Anchor Rod Diameter, in.				
	¾	⅞	1	1⅛	1¼
9	54.0	55.0	55.9	56.9	57.9
12	93.0	94.3	95.7	97.0	98.3
15	143	144	146	148	149
18	203	205	207	209	211

Table A-9. Concrete Breakout Strength of Two Anchor Rods Based on Spacing and Embedment, ϕP_n, kips/rod Anchor Rods in Groups, Embedment Depth = 9 in.					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	¾	⅞	1	1⅛	1¼
	Area, in.^{2}				
	0.442	**0.601**	**0.785**	**0.994**	**1.23**
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 A10. Concrete Breakout Strength of Two Anchor Rods Based on Spacing and Embedment, ϕP_n, kips/rod Anchor Rods in Groups, Embedment Depth = 12 in.					
Anchor Rod Spacing, in.	**Anchor Rod Diameter, in.**				
	¾	**⅞**	**1**	**1⅛**	**1¼**
	Area, in.^{2}				
	0.442	**0.601**	**0.785**	**0.994**	**1.23**
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-11. Concrete Breakout Strength of Two Anchor Rods Based on Spacing and Embedment, ϕP_n, kips/rod Anchor Rods in Groups, Embedment Depth = 15 in.					
Anchor Rod Spacing, in.	**Anchor Rod Diameter, in.**				
	¾	**⅞**	**1**	**1⅛**	**1¼**
	Area, in.^{2}				
	0.442	**0.601**	**0.785**	**0.994**	**1.23**
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	105	104	104	104	103
14	110	110	110	109	109
16	116	116	115	115	114
18	122	121	120	120	119
20	127	127	126	125	125

Table A-12. Concrete Breakout Strength of Two Anchor Rods Based on Spacing and Embedment, ϕP_n, kips/rod Anchor Rods in Groups, Embedment Depth = 18 in.					
Anchor Rod Spacing, in.	Anchor Rod Diameter, in.				
	¾	⅞	1	1⅛	1¼
	Area, in.^{2}				
	0.442	**0.601**	**0.785**	**0.994**	**1.23**
4	115	115	115	114	114
5	118	118	118	118	118
6	122	121	121	121	121
8	128	128	128	127	127
12	142	141	141	140	140
14	148	148	147	147	147
16	155	155	154	154	153
18	162	161	161	160	159
20	168	168	167	167	166

Table A-13. Pushout Strength, ϕP_n, kips Group of Two Anchor Rods				
Anchor Rod Spacing, in.	Clear Cover, in.			
	3	**6**	**9**	**12**
4	8.63	20.7	36.7	53.6
5	9.30	21.6	36.8	54.9
6	9.96	22.5	38.0	56.3
8	11.3	24.4	40.3	59.0
12	14.0	28.2	44.9	64.3
14	15.3	30.1	47.2	67.0
16	16.6	31.9	49.5	69.7
18	17.9	33.8	51.8	72.4
20	19.3	35.7	54.1	75.0

Table A-14. Available Flexural Strength Single Square Concrete Piers				
	Reinforcing			
	0.5%		1.0%	
Size, in.	**Bars**	**ϕM_n, kip-ft**	**Bars**	**ϕM_n, kip-ft**
12×12	4 - #5	25.3	4 - #6	34.7
14×14	4 - #5	31.2	4 - #7	57.0
16×16	4 - #6	51.4	4 - #8	87.7
18×18	4 - #6	59.6	4 - #9	127
20×20	4 - #7	91.0	4 - #9	146
22×22	4 - #8	133	8 - #8	195
24×24	4 - #8	147	8 - #8	217

Table A-15. Concrete Footing Overturning Resistance Single Square Footings				
Size	**Footing Thickness, in.**	**M_o, kip-ft**	**Footing Thickness, in.**	**M_o, kip-ft**
4'-0"	12	4.80	13	5.20
4'-6"	12	6.83	13	7.40
5'-0"	12	9.38	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
9'-6"	22	118	28	150
10'-0"	24	150	30	188

Table A-16. Reinforcing Bars Required Development Length, in. Grade 60 Bars, $f'_c = 3000$ psi			
Bar Size	Standard Hooked Bar	Straight Bars	
		#6 and Smaller	#7 and Larger
3	6	17	—
4	6	22	—
5	8	28	—
6	11	33	—
7	14	—	48
8	16	—	55
9	20	—	62
10	23	—	70
11	27	—	78

Table A-17. Reinforcing Bars Required Development Length, in. Grade 60 Bars, $f'_c = 4000$ psi			
Bar Size	Standard Hooked Bar	Straight Bars	
		#6 and Smaller	#7 and Larger
3	6	15	—
4	6	19	—
5	8	24	—
6	10	29	—
7	13	—	42
8	15	—	48
9	18	—	54
10	22	—	61
11	26	—	67

Table A-18. Type A Anchor Plate Dimensions and Weld Sizes (Plate in Tension and Bending)									
Wire Rope, 6×7 FC (IPS)	Slope, degrees from horizontal								
	30°			45°			60°		
Diameter, in.	*L*, in.	*t*, in.	Fillet Weld	*L*, in.	*t*, in.	Fillet Weld	*L*, in.	*t*, in.	Fillet Weld
½	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
¾	5	5⁄16	3⁄16	5	5⁄16	3⁄16	5	3⁄8	3⁄16
Wire Rope, 6×7 IWRC (IPS)	Slope, degrees from horizontal								
	30°			45°			60°		
Diameter, in.	*L*, in.	*t*, in.	Fillet Weld	*L*, in.	*t*, in.	Fillet Weld	*L*, in.	*t*, in.	Fillet Weld
½	3	5⁄16	3⁄16	3	3⁄8	3⁄16	3	½	3⁄16
5⁄8	4	5⁄16	3⁄16	4	3⁄8	3⁄16	4	3⁄8	3⁄16
¾	5	5⁄16	3⁄16	5	3⁄8	3⁄16	5	3⁄8	3⁄16

Table A-19. Allowable Wire Rope Force, *P*, lb Type A Plate Anchor Limited by Anchor Rod Strength (Anchor Rods in Tension and Bending)									
Grout Depth, in.	Anchor Rod Diameter, in.								
	¾			1			1¼		
	Slope			Slope			Slope		
	30°	45°	60°	30°	45°	60°	30°	45°	60°
2	13,700	15,100	18,100	34,300	36,200	41,400	71,100	71,900	78,100
3	7,420	8,560	11,000	18,100	20,500	25,500	36,500	40,300	48,800
4	5,090	5,980	7,900	12,300	14,300	18,400	24,500	28,000	35,400
5	3,870	4,590	6,170	9,320	10,900	14,400	18,500	21,500	27,800
6	3,120	3,730	5,050	7,500	8,870	11,900	14,800	17,400	22,900
7	2,620	3,140	4,280	6,270	7,460	10,000	12,400	14,600	19,500
8	2,250	2,710	3,720	5,390	6,440	8,740	10,600	12,600	16,900

Table A-20. Type B Anchor Plate Dimensions and Weld Sizes (Plate in Tension and Bending)						
Wire rope, 6×7 FC, (IPS)	Slope, degrees from horizontal					
	30°		45°		60°	
Diameter, in.	*B*, in.	*t*, in.	*B*, in.	*t*, in.	*B*, in.	*t*, in.
½	4	½	4	½	4	⅝
⅝	5	½	5	⅝	5	⅝
¾	6	½	6	⅝	6	⅝
Wire Rope, 6×7 IWRC, (IPS)	Slope, degrees from horizontal					
	30°		45°		60°	
Diameter, in.	*B*, in.	*t*, in.	*B*, in.	*t*, in.	*B*, in.	*t*, in.
½	4	½	4	½	4	⅝
⅝	5	½	5	⅝	5	⅝
¾	6	⅝	6	⅝	6	⅝

Table A-21. Allowable Wire Rope Force, *P*, lb Type B Plate Anchor Limited by Anchor Rod Strength (Anchor Rods in Tension and Bending)									
Grout Depth, in.	Anchor Rod Diameter, in.								
	¾			1			1¼		
	Slope			Slope			Slope		
	30°	45°	60°	30°	45°	60°	30°	45°	60°
2	9,360	8,760	8,790	20,800	18,400	17,500	38,100	32,100	29,500
3	5,930	6,070	6,690	13,500	13,200	13,900	25,300	23,800	24,000
4	4,340	4,650	5,400	9,970	10,300	11,500	18,900	18,900	20,300
5	3,420	3,770	4,530	7,910	8,450	9,780	15,100	15,700	17,500
6	2,830	3,170	3,900	6,560	7,160	8,530	12,600	13,400	15,500
7	2,410	2,730	3,420	5,600	6,220	7,560	10,800	11,700	13,800
8	2,090	2,400	3,050	4,890	5,490	6,790	9,410	10,400	12,500

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