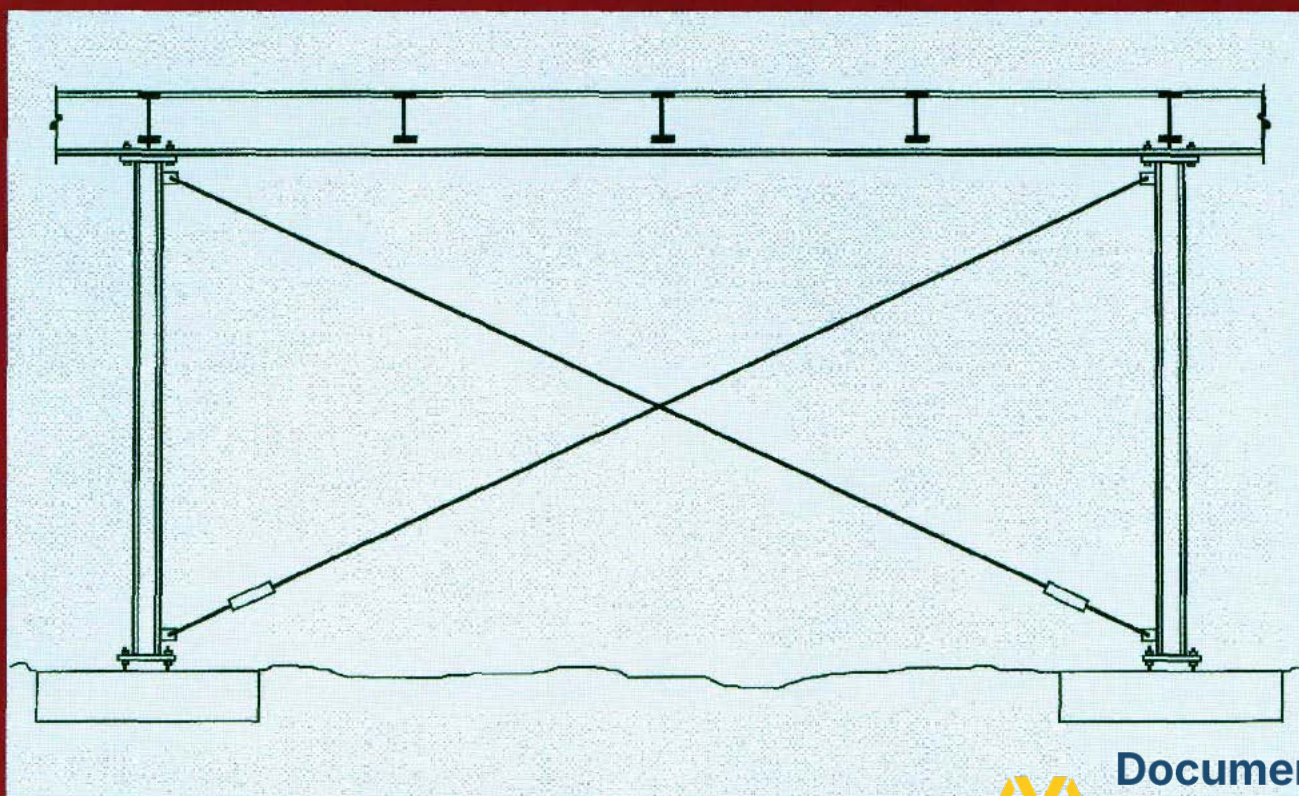




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Erection Bracing of Low-Rise Structural Steel Buildings



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Erection Bracing of Low-Rise Structured Steel Buildings

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ERECTION BRACING OF LOW RISE STRUCTURAL STEEL BUILDINGS

1. INTRODUCTION

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

- (1) Function: general purpose structures for such uses as light manufacturing, crane buildings, warehousing, offices, and other commercial and institutional buildings.
- (2) Proportions:
 - (a) height: 60 feet tall or less.
 - (b) stories: a maximum of two stories.

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

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

1.1 Types of Systems

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

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

such as a roof deck diaphragm which would change the frame to a non-self-supporting type.

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

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

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

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

1.2 Current State of the Art

In high-rise construction and bridge construction the need for predetermined erection procedures and temporary support systems has long been established in the industry. Low-rise construction does not command a comparable respect or attention because of the low heights and relatively simple framing involved. Also the structures are relatively lightly loaded and the fram-

ing members are relatively light. This has led to a number of common fallacies which are supported by anecdotal evidence.

1.3 Common Fallacies

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

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

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

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

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

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

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

designed in anticipation of continuity, then the bottom chords must not be welded.

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

1.4 Use of This Guide

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

PART 1

DETERMINATION OF BRACING REQUIREMENTS BY CALCULATION METHOD

2. INTRODUCTION TO PART 1

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

3. CONSTRUCTION PHASE LOADS FOR TEMPORARY SUPPORTS

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

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

Environmental loads

Wind
Seismic

Stability loads

Erection operation

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

3.1 Gravity Loads

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

Dead loads can be determined using the unit density and unit weights provided in the AISC Manual of Steel Construction, (LRFD Part 7, ASD Part 6) and ASCE 7-93, Tables C1 and C2. Dead loads can also be obtained from manufacturers and suppliers.

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

3.2 Environmental Loads

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

3.2.1 Wind Loads

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

is a discussion of the procedure provided in ASCE 7-93 (1) which will illustrate the basic concept.

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

$$p = qG_h C_p - qh(GC_{pi}) \quad \text{Eq. 3-1}$$

where

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

K = velocity pressure coefficient varying with height and exposure

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

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

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

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

C_p = a factor accounting for the shape of the structure

q_h = q taken at height, h

GC_{pi} = a factor accounting for internal pressure

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

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

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

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

$$p = q_z G_h C_f \quad \text{Eq. 3-3}$$

where

q_z = q evaluated at height z

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

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

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

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

This topic has been treated in the following documents:

1. Part A4.3.3 of the "Low Rise Building Systems Manual" (12) published by the Metal Building Manufacturers Association.
2. "Wind forces on Structures" (18), Paper No. 3269, ASCE Transactions, published by the American Society of Civil Engineers.
3. "Standards for Load Assumptions, Acceptance and Inspection of Structures" (16), No. 160, published by the Swiss Association of Engineers and Architects.
4. "Design Loads for Buildings" (5), German Industrial Standard (DIN) 1055, published by the German Institute for Standards.

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

"6.1.2. Frameworks without Cladding

Structures shall resist the effect of wind acting upon successive unenclosed components.

Staging, shoring, and falsework with regular rectangular plan dimensions may be treated as trussed towers in accordance with ASCE 7. Unless detailed analyses are performed to show that lower loads may be used, no allowance shall be given for shielding of successive rows or towers.

For unenclosed frames and structural elements, wind loads shall be calculated for each element. Unless detailed analyses are performed, load reductions due to shielding of elements in such structures with repetitive patterns of elements shall be as follows:

1. The loads on the first three rows of elements along the direction parallel to the wind shall not be reduced for shielding.
2. The loads on the fourth and subsequent rows shall be permitted to be reduced by 15 percent.

Wind load allowances shall be calculated for all exposed interior partitions, walls, temporary enclosures, signs, construction materials, and equipment on or supported by the structure. These loads shall be added to the loads on structural elements.

Calculations shall be performed for each primary axis of the structure. For each calculation, 50% of the wind load calculated for the perpendicular direction shall be assumed to act simultaneously."

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

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

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

where

A = total factored area

η = a stacking factor taken from Figure 3.2.

n = the total number of parallel elements

A_1 = the projected area of one element

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

3.2.2 Seismic Loads

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

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

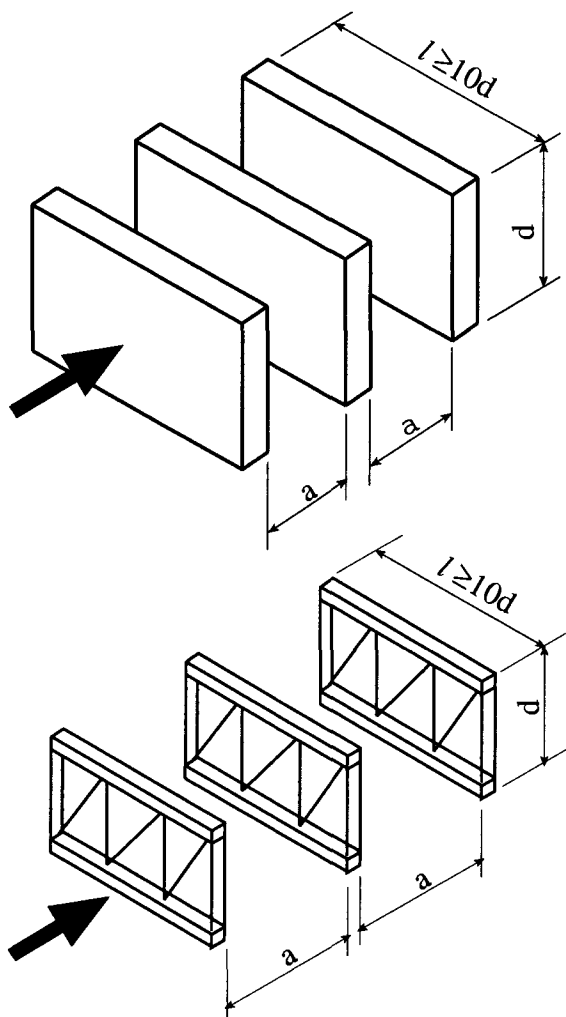


Fig. 3.1 Parameters for Use with Fig. 3.2

2. the use of the structure,
3. the geometry and framing system type of the structure,
4. the geological nature of the building site, and
5. the mass, i.e. self-weight of the structure.

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

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

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

where

C_s = the seismic design coefficient

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

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

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

$$C_s = \frac{1.2A_v S}{RT^{2/3}} \quad \text{Eq. 3-6}$$

where

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

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

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

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

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

$$C_s = \frac{2.5A_a}{R} \quad \text{Eq. 3-7}$$

where

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

R = the response modification factor described above

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

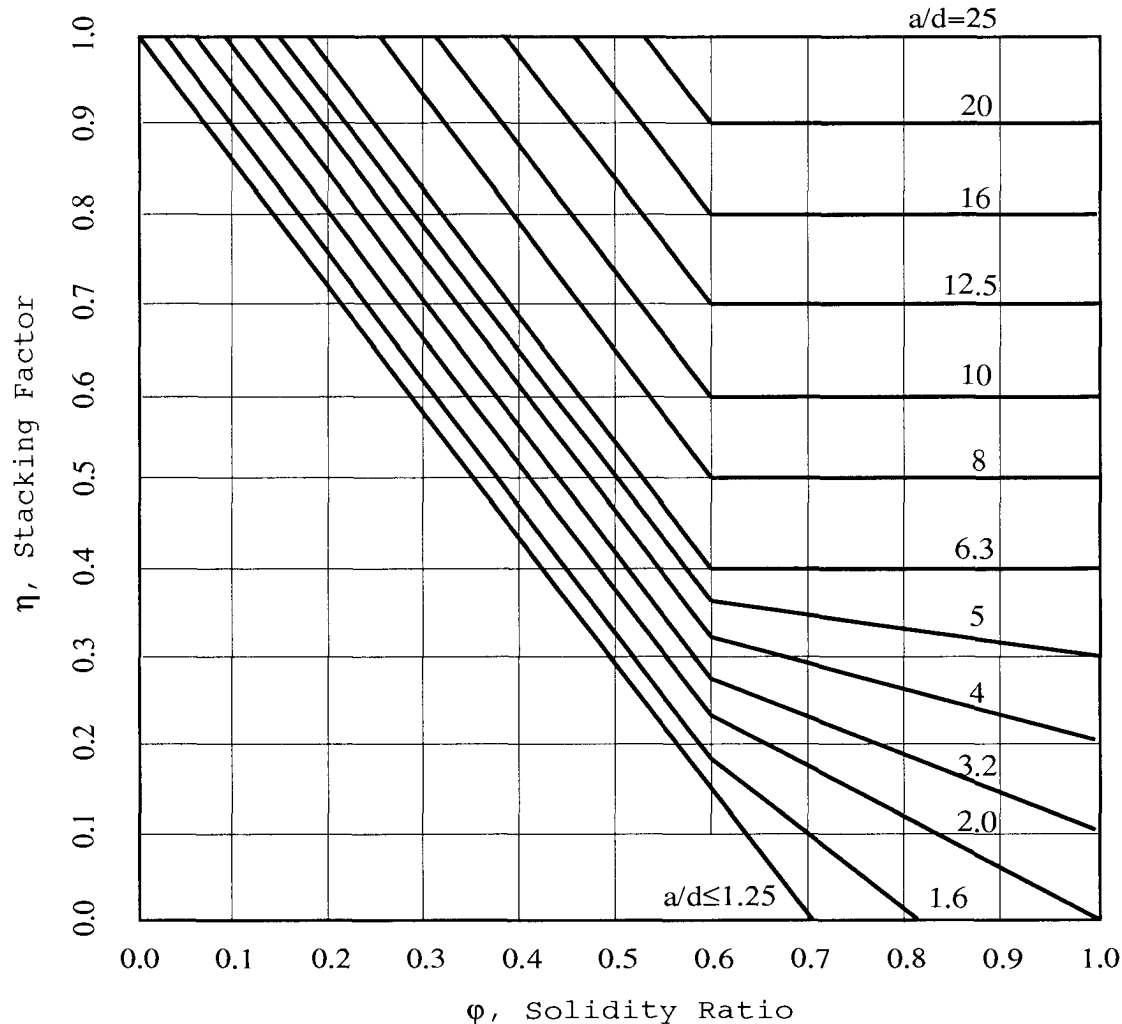


Fig. 3.2 Stacking Factor vs. Solidity Ratio

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

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

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

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

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

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

horizontal distribution would consist of applying the dead load, W, which is tributary to each frame.

3.3 Stability Loads

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

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

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

3.4 Erection Operation Loads

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

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

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

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

3.5 Load Combinations

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

$$1.4D$$

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

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

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

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

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

The nominal loads to be considered are:

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

L: live load due to occupancy and moveable equipment

L_r : roof live load

W: wind load

S: snow load

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

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

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

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

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

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

These equations are the same as Equations 5 and 6 in ASCE 7, paragraph 2.4.2. It should be noted that E is not

the exact effect of the seismic force due to the seismic base shear but must be modified by the following equations taken from ASCE 7, paragraph 9.3.7:

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

where

- E = the effect of horizontal and vertical earthquake-induced forces
- A_v = the coefficient representing effective peak velocity-related acceleration from ASCE 7
- D = the effect of dead load, D
- Q_E = the effect of horizontal seismic (earthquake-induced) forces

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

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

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

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

- a. Stability loading
- b. 0.75 (stability loading plus wind loading)

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

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

4. RESISTANCE TO CONSTRUCTION PHASE LOADS BY THE PERMANENT STRUCTURE

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

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

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

Columns

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

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

Columns supported on leveling nuts or shims prior to grouting.

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

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

Column Bases

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

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

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

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

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

1. The use of leveling nuts and, in some cases, washers on the anchor rods beneath the base plates.
2. The use of shim stacks between the base plate bottoms and top of concrete supports.
3. The use of 1/4" steel leveling plates which are set to elevation and grouted prior to the setting of columns.

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

used the compressive components of the column base reaction are carried by the shims and the tension and shear components are carried by the anchor rods.

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

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

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

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

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

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

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

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

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

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

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

Beams and Joists

Framing members on the column center lines act as tie members and struts during erection. As such they are subject to axial forces as well as gravity load bending. In

most cases the axial compression strength of tie members and struts will be limited by their unbraced length in the absence of the flange bracing. The resistance of strut and tie members must be evaluated with the lateral bracing in place at the time of load application.

Diagonal Bracing

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

Connections

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

Diaphragms

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

4.1 Columns

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

Specific guidelines for this evaluation are not presented here, because of the many variables that can oc-

cur. Basic structural engineering principles must be applied to each situation.

4.2 Column Bases

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

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

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

Modes of Failure:

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

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

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

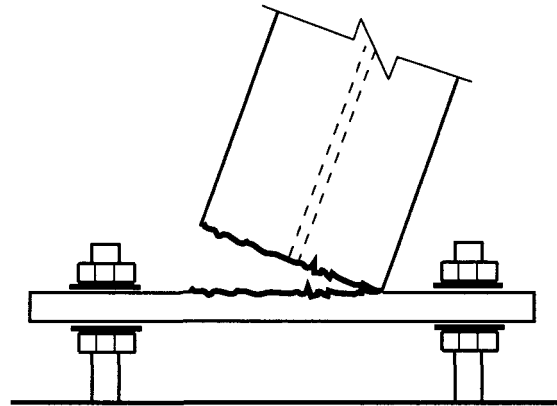


Fig. 4.1 Fracture of Weld

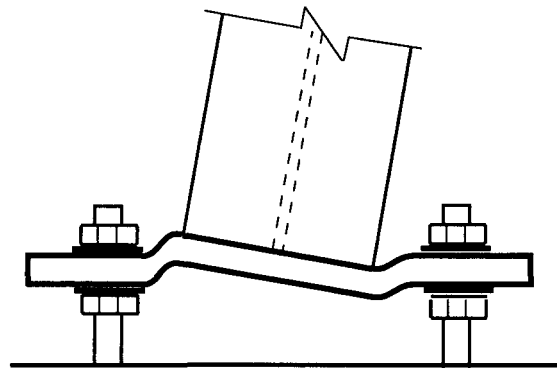


Fig. 4.2 Bending Failure of Base Plate

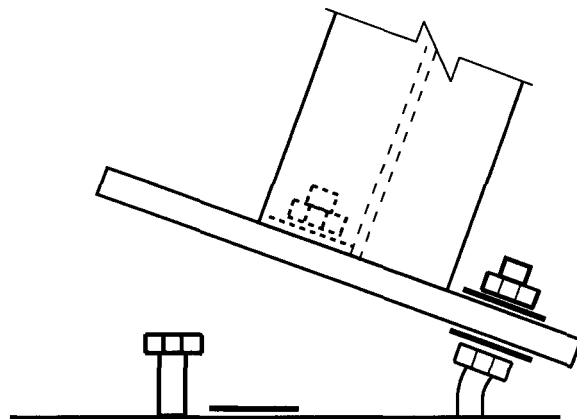


Fig. 4.3 Rupture of Anchor Rods

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

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

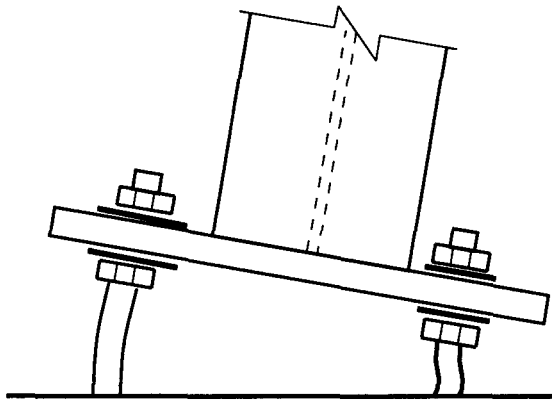


Fig. 4.4 Anchor Rod Buckling

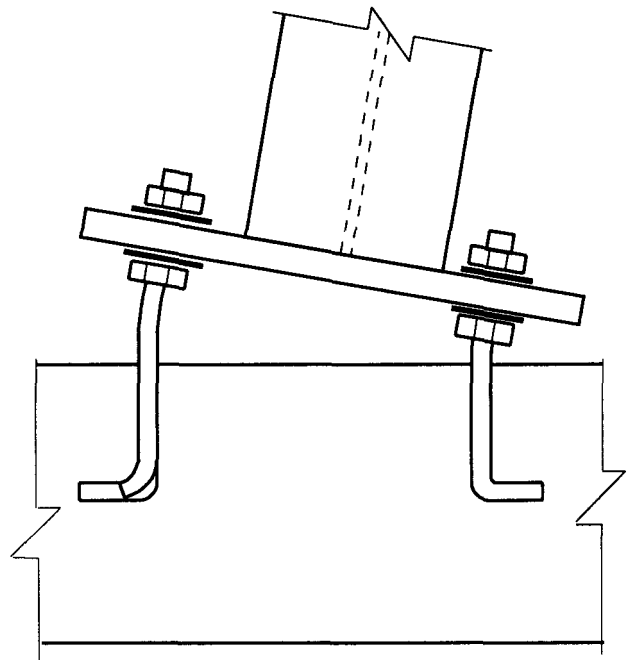


Fig. 4.7 Anchor Rod Straightening

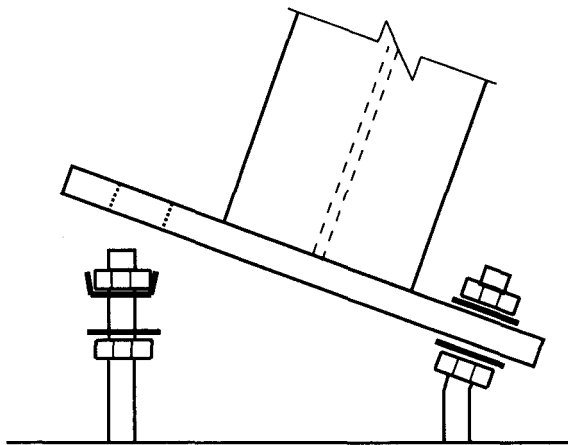


Fig. 4.5 Anchor Rod Pull Through

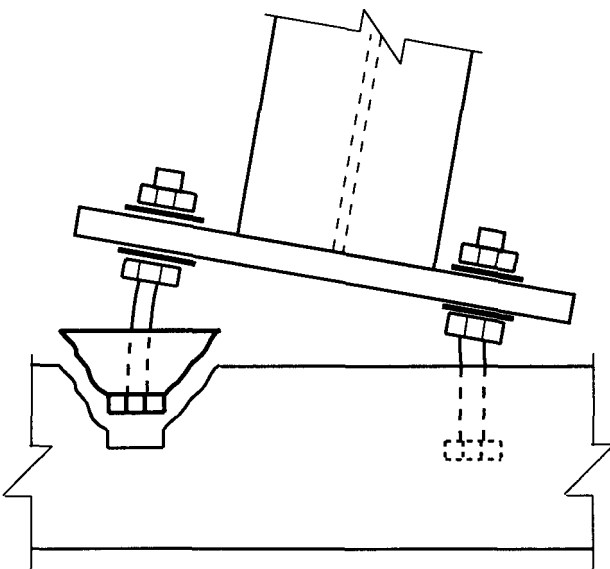


Fig. 4.6 Anchor Rod Pull Out

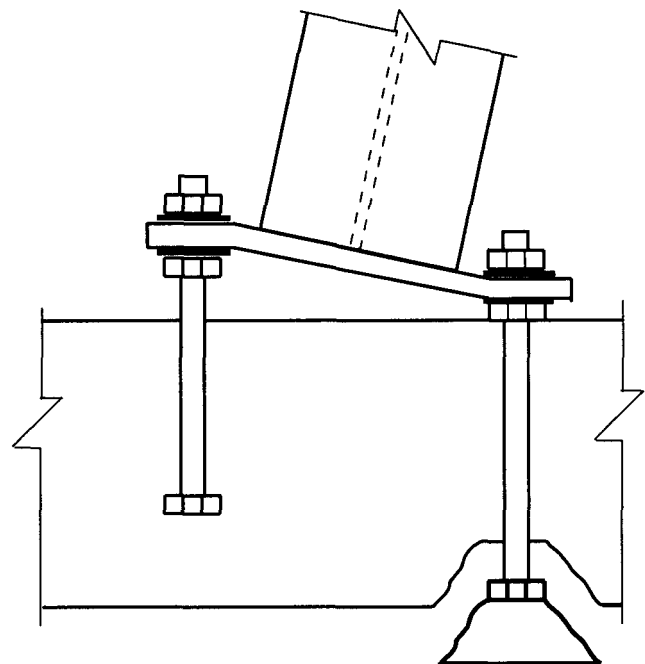


Fig. 4.8 Anchor Rod Push Out

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

weld group is weaker about the weak axis, and because the wind forces are greater when acting against the weak axis, as explained earlier.

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

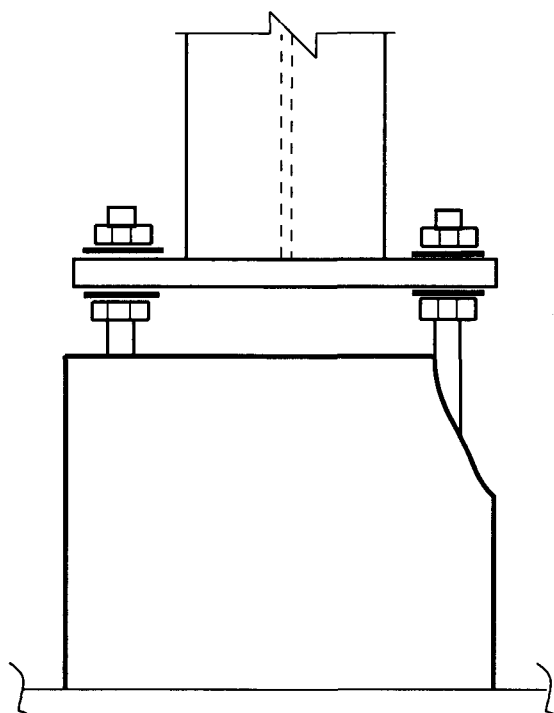


Fig. 4.9 Pier Spalling

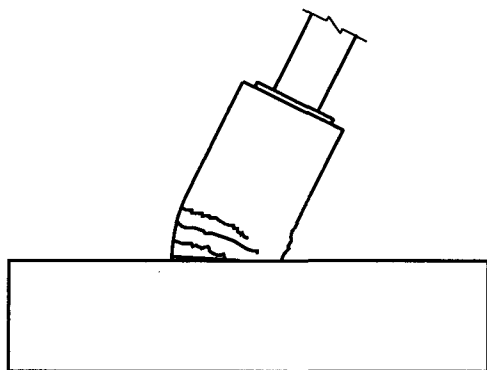


Fig. 4.10 Pier Bending Failure

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

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

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

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

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

where

$$\phi = 0.75$$

F_w = the nominal weld stress, ksi

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

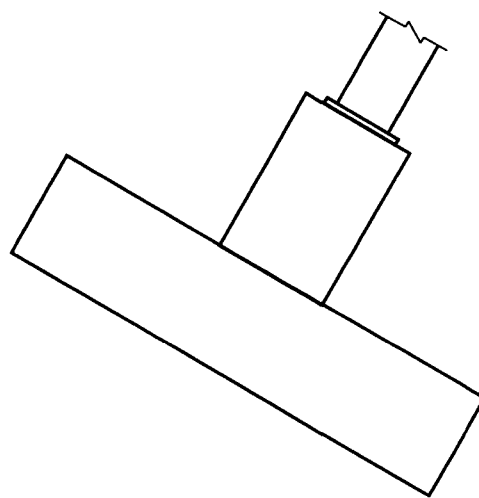


Fig. 4.11 Footing Overturning

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

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

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

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

4.2.2 Bending Failure of the Base Plate.

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

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

Case A: Inset Anchor Rods - Wide Flange Columns.

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

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

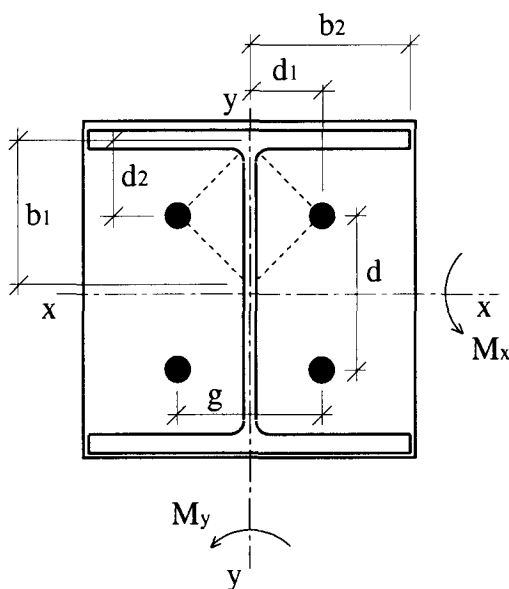


Fig. 4.12 Base Plate Dimensions

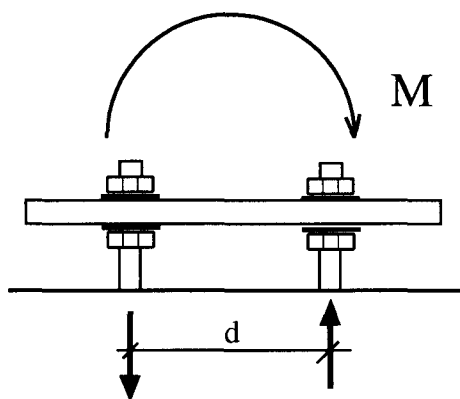


Fig. 4.13 Base Plate with Leveling Nuts

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

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

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

where

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

M_n^1 = the plastic moment resistance based on b_1 , in.-kips

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

$$\phi = 0.90$$

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

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

$$\phi M_n = 2\phi P d \text{ or } 2\phi P g \quad \text{Eq. 4-4}$$

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

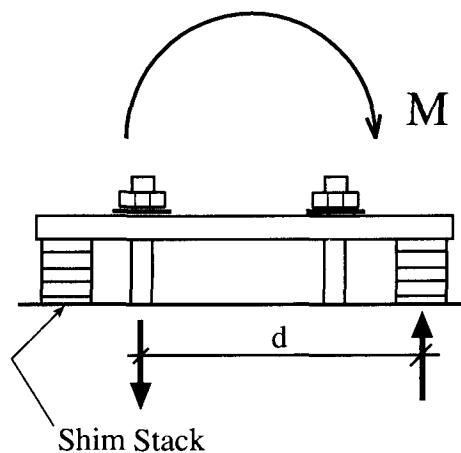


Fig. 4.14 Base Plate with Shim Stacks

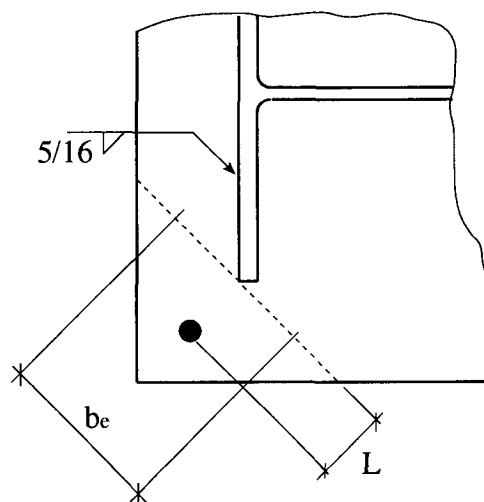


Fig. 4.15 Effective Width

Currently the AISC standard detail illustrates weld only along the flanges, unless shown otherwise on the contract drawings. The addition of a fillet weld along one side of the web adds considerable strength to the

connection. Without the web weld only the length b_e would be used in the strength calculations.

Case B: Outset Rods - Wide Flange Columns

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

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

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

Based on the plate effective width:

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

Based on weld strength:

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

Based on weld strain:

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

where

$$\phi_p = 0.90$$

$$\phi_w = 0.75$$

$$b_e = \text{the effective plate width, in.}$$

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

$$L = \text{the distance of the anchor rod to the flange tip, in.}$$

$$t = \text{the throat width of the weld, in.}$$

$$t_p = \text{the base plate thickness, in.}$$

$$F_y = \text{the specified yield strength for the base plate, ksi}$$

$$F_w = \text{the nominal weld stress, ksi}$$

$$= 0.9 F_{EXX}, \text{ ksi (90° loading)}$$

$$F_{EXX} = \text{electrode classification number, ksi}$$

Using the controlling value for ϕP_n and d :

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

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

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

$$\phi P_n = \phi F_w t (4") \quad \text{Eq. 4-9}$$

4.2.3 Rupture of Anchor Rods

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

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

where

$$\phi = 0.75 \text{ (Table J3.2)}$$

$$\phi P_n = \text{the tension rod design strength, kips}$$

$$F_n = \text{nominal tensile strength of the rod } F_t, \text{ ksi}$$

$$F_t = 0.75 F_u \text{ (Table J3.2)}$$

$$F_u = \text{specified minimum tensile strength, ksi}$$

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

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

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

4.2.4 Buckling of the Anchor Rods

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

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

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

4.2.5 Anchor Rod Pull or Push Through

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

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

4.2.6 Anchor Rod Pull Out

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

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

The failure occurs in the concrete when the tensile stresses along the surface of a stress cone surrounding the anchor rod exceed the tensile strength of the concrete. The extent of the stress cone is a function of the embedment depth, the thickness of the concrete, the spacing between the adjacent anchors, and the location of free edges of in the concrete. This failure mode is presented in detail in Appendix B of ACI 349-90(4). The tensile strength of the concrete, in ultimate strength terms, is represented as a uniform tensile stress of $4\phi\sqrt{f'_c}$ over the surface area of these cones. By examining the geometry, it is evident that the pull out strength of a cone is equal to $4\phi\sqrt{f'_c}$ times the projected area, A_e , of the cone at the surface of the concrete, excluding the

area of the anchor head, or for the case of hooked rods the projected area of the hook.

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

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

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

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

where

L_d = the embedment depth, in.

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

s = the rod spacing, in.

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

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

where

ϕ = 0.85

f'_c = the specified concrete strength, psi

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

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

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

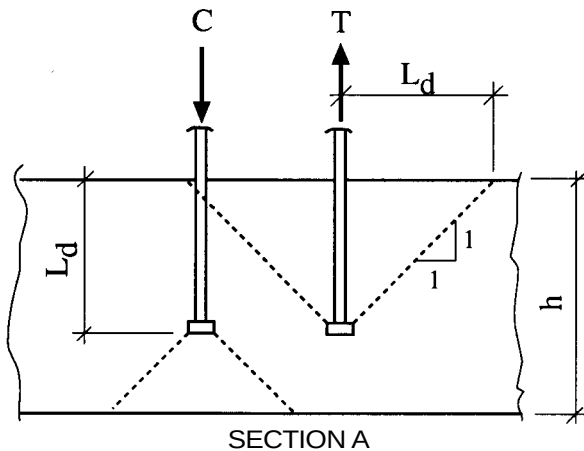
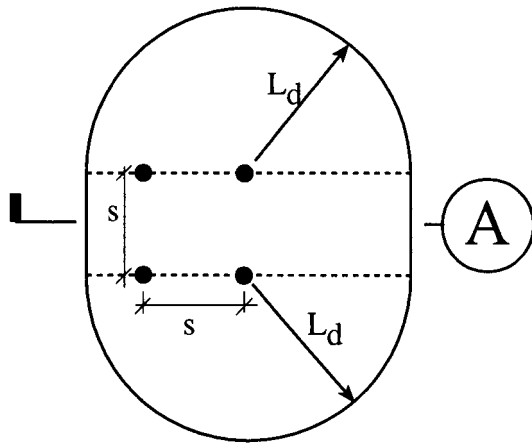


Fig. 4.16 Failure Cones

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

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

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

where

ϕT_n = Hook Bearing Design Strength, kips

f'_c = the concrete compressive strength, psi

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

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

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

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

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

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

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

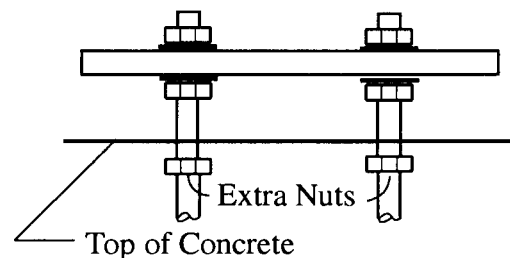


Fig. 4.17 Prevention of Push Out

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

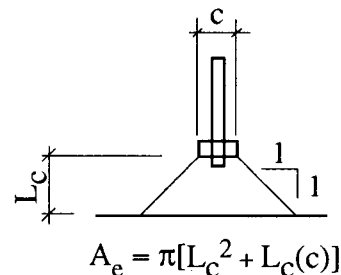


Fig. 4.18 Push Out Cones

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

where

ϕ = .75

f'_c = the concrete compressive strength, psi

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

4.2.8 Pier Bending Failure

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

Determine the depth of the compression area.

$$C = T$$

$$0.85f'_c b a = F_y A_s$$

$$a = F_y A_s / 0.85f'_c b$$

$$C = 0.85f'_c b a$$

$$d = \text{the effective depth of the tension reinforcing} \\ = \text{pier depth} - \text{cover} - 1/2 \text{ of the bar diameter}$$

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

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

For the hooked bar in the footing:

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

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

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

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

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

where

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

$$l_d = \text{development length, in.}$$

$$f'_c = \text{specified concrete strength, psi}$$

$$d_b = \text{the bar diameter, in.}$$

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

4.2.9 Footing Over Turning

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

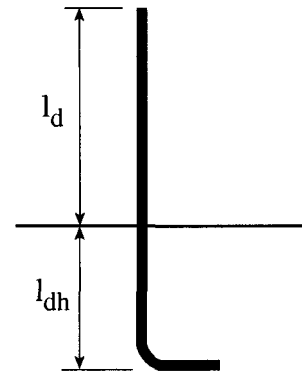


Fig. 4.19 Development Lengths

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

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

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

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

where

$$\phi = 0.9$$

$$W = P_1 + P_2 + P_3$$

$$P_1 = \text{the weight of any superimposed loads, kips}$$

$$P_2 = \text{the weight of the pier, if any, kips}$$

$$P_3 = \text{the weight of the footing, kips}$$

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

Example 4-1:

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

Given:

Leveling Nuts and Washers

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

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

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



Failure Mode 6: Anchor Rod Pullout

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

$$\begin{aligned} A_e &= \pi(L_d + c/2)^2 \\ &\quad + 2(L_d + c/2)(s+c) - \pi(c)^2/2 \quad (\text{Eq. 4-12}) \\ &= [3.14(12+0.75/2)^2 \\ &\quad + 2(12+0.75/2)(5+0.75) + 3.14(.75)^2/2] \\ &= 628 \text{ in.}^2 \end{aligned}$$

Check Pier Area:

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

Note that edge distance will not control.

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

Check Hook Bearing Strength:

$$\begin{aligned} \phi T_n &= 2(0.7)(0.85)f'_c d_{\text{hook}} L_{\text{hook}} \quad (\text{Eq. 4-14}) \\ &= 2(0.7)(0.85)(3000)(0.75)(4) \\ &= 10.7 \text{ kips} \\ &= 21.4 \text{ kips for two rods (Controls)} \end{aligned}$$

$$\begin{aligned} \phi M_n &= \phi T_n d = (21.4)(5) = 107 \text{ in.-kips} \quad (\text{Eq. 4-15}) \\ &= 8.9 \text{ ft.-kips} \end{aligned}$$

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

Failure Mode 8 : Pier Bending Resistance

Determine the depth of the compression area:

$$\begin{aligned} a &= F_y A_s / 0.85 f'_c b \\ &= 60,000(2)(0.44) / 0.85(3000)(16) \\ &= 1.294 \text{ in.} \end{aligned}$$

$$\begin{aligned} C &= 0.85 f'_c a \\ &= 0.85(3000)(16)(1.294) / 1000 \\ &= 52.8 \text{ kips} \end{aligned}$$

$$\begin{aligned} \phi M_n &= C(d-a/2) = 692 \text{ in.-kips} \quad (\text{Eq. 4-17}) \\ &= 52.8(13.75-1.294/2) \\ &= 58 \text{ ft.-kips} \end{aligned}$$

Check Reinforcing Development length:

Req'd length in footing:

$$\begin{aligned} l_{dh} &= 840 d_b / \sqrt{f'_c} \geq 8 d_b \text{ or } 6 \text{ in.} \quad (\text{Eq. 4-18}) \\ &= 840(.75) / \sqrt{3000} = 11.5 \text{ in.} < 12 \text{ in. o.k.} \end{aligned}$$

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

$$\begin{aligned} l_d &= 2400 d_b / \sqrt{f'_c} \quad (\text{Eq. 4-19}) \\ &= 2400(.75) / \sqrt{3000} = 32.9 \text{ in.} < 33 \text{ in. o.k.} \end{aligned}$$

Failure Mode 9: Footing Overturning

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

where

$$\phi = 0.9$$

$$W = P_1 + P_2 + P_3$$

$$P_1 = 65(40) / 71000 = 2.6 \text{ kips (Column)}$$

$$P_2 = 0.15(1.33)1.33(3) = 0.8 \text{ kips (Pier)}$$

$$P_3 = 0.15(1.25)6(6) = 6.75 \text{ kips (Footing)}$$

$$W = 10.15 \text{ kips, } L = 6 \text{ ft.}$$

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

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

Example 4-2:

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

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

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

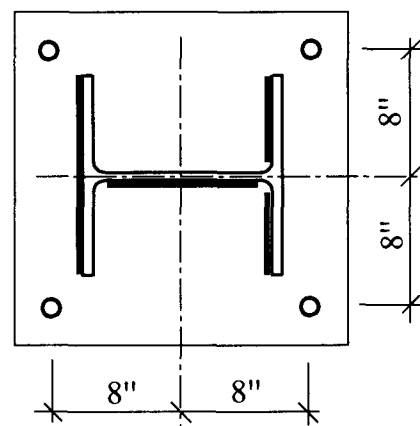


Fig. 4.23 Base Plate Detail

Solution:

Failure Mode 1: Weld Design Strength

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

Failure Mode 2: Base Plate Failure

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

$$\phi P_n = \phi b_e t_p^2 F_y / 4L$$



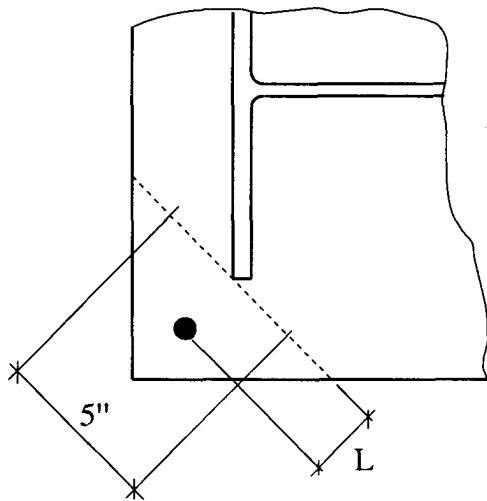


Fig. 4.24 Base Plate Yield Line

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

$$= 16.2 \text{ kips}$$

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

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

$$= 20.9 \text{ kips}$$

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

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

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

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

$$= 2(9.94)(16) = 318 \text{ in.-kips}$$

$$= 26.5 \text{ ft.-kips}$$

Failure Mode 3: Rupture of Anchor Rods

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

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

$$= 2(14.4)(16) = 461 \text{ in.-kips}$$

$$= 38.4 \text{ ft.-kips}$$

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

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

Failure Mode 6: Anchor Rod Pull Out

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

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

By inspection the pier area will control.

Check Pier Area:

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

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

$$\begin{aligned} \phi M_n &= 2\phi T_n d = 2(65.7)(16) \\ &= 2102 \text{ in.-kips} \end{aligned} \quad (\text{Eq. 4-15})$$

$$= 175 \text{ ft.-kips}$$

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

Failure Mode 8: Pier Bending Resistance

Determine the depth of the compression area:

$$\begin{aligned} a &= F_y A_s / .85 f'_c b \\ &= 60,000(2)(0.44) / 0.85(3000)(24) \\ &= 0.863 \text{ in.} \end{aligned}$$

$$\begin{aligned} C &= 0.85 f'_c ab \\ &= 0.85(3000)(0.863)(24) / 1000 \\ &= 52.8 \text{ kips} \end{aligned}$$

$$\begin{aligned} \phi M_n &= C(d - a/2) \quad (\text{Eq. 4-17}) \\ &= 52.8(21.75 - 0.863/2) \\ &= 1126 \text{ in.-kips} \\ &= 94 \text{ ft.-kips} \end{aligned}$$

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

Failure Mode 9: Footing Overturning:

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

where

$$\phi = 0.9$$

$$W = P_1 + P_2 + P_3$$

$$P_1 = 65(40) / 1000 = 2.6 \text{ kips (Column)}$$

$$P_2 = 0.15(2)(2)(3) = 1.8 \text{ kips (Pier)}$$

$$P_3 = 0.15(1.25)(6)(6) = 6.75 \text{ kips (Footing)}$$

$$W = 11.15 \text{ kips}$$

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

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