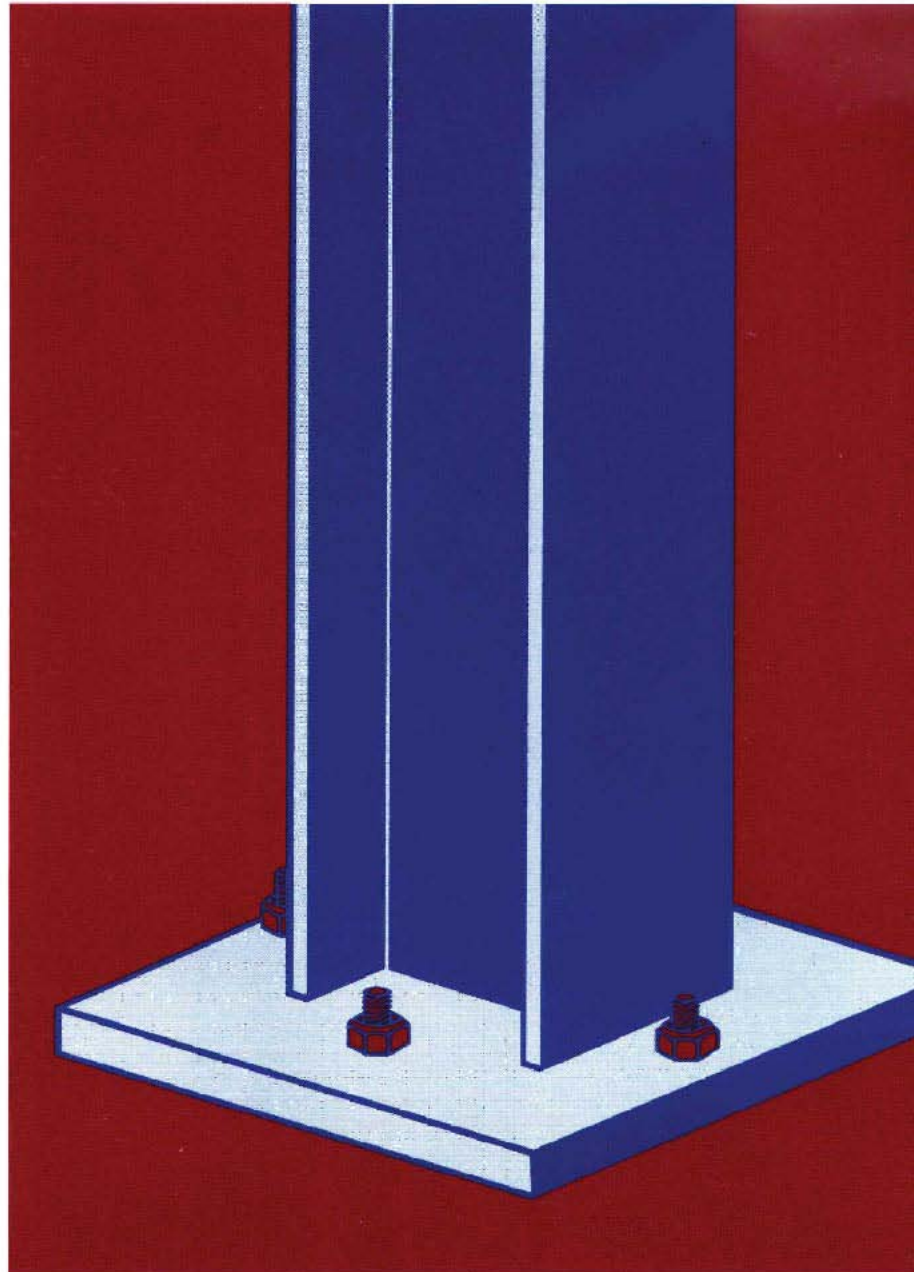




Steel Design Guide Series

Column Base Plates





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Column Base Plates

Design of Column Base Plates

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Some Practical Aspects of Column Base Selection

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A M E R I C A N I N S T I T U T E O F S T E E L C O N S T R U C T I O N

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PREFACE

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

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

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INTRODUCTION

This report contains a compilation of existing information on the design of base plates for steel columns. The material is taken from reports, papers, texts and design guides. The intent is to provide engineers with the research background and an understanding of the behavior of base plates and then to present information and guidelines for their design. The material is intended for the design of column base plates in building frames, though it can be used for related structures. Bearing plates for beams would be based on similar principles.

Three cases are covered, each involving different loads. These are shown in Fig. 1. The first is the axially loaded column, shown in Fig. 1 (a). The load is perpendicular to the plate and through the column centroid. It is used in frames in which the column bases are assumed pinned. A layer of grout is used for leveling of the plate and setting it at the specified elevation. Anchor bolts are also used to stabilize the column during erection, and the fixity which results is neglected in design. The column and base plate are normally centered on the concrete foundation. If the column load is relatively small, the required base plate size determined from the concrete bearing capacity only will be approximately equal to, or smaller, than the actual column size. These base plates are referred to as lightly loaded column base plates, and they require a modified design approach.

The second case, shown in Fig. 1 (b), includes both an axial load and a moment. This kind of connection would be used at the base of moment resistant frames where moment capacity is needed. It is also used where the load is applied eccentrically to the column and the resulting moment must be resisted by the base connection. If the moment is relatively small, the connection can be designed without the use of anchor bolts, other than those provided for stability during construction. The more common case involves the use of one or more bolts to resist the tension resultant from the moment.

The third case, shown in Fig. 1 (c), is a base plate with a horizontal, or shear load. This will occur in rigid frames. Often the shear component is small in relation to the friction developed. Shear is sometimes important when bracing is connected to the base of the column. The shear can be resisted through friction or the development of bearing in the horizontal direction.

Many variables influence the behavior and load resisting capacity of base plates. While some of these have been studied extensively, others have only received nominal study. The following material is based on the best available information, and some of it represents suggested guidelines based on the author's judgement. The design approaches given here are not intended as the only acceptable methods.

Provisions for the design of base plates have generally been developed in the Allowable Stress Design (ASD) format, as reflected in the AISC Manual of Steel Construction (AISC 1989a) and AISC Specification (AISC 1989). The AISC Load and Resistance Factor Design (LRFD) Manual (AISC 1986) and Specification (AISC 1986a) have translated these provisions into equivalent limit state design format which may be used as an alternative to ASD.

Since the provisions were initially developed in the ASD format, the discussions in the design chapters are based on this format, with appropriate references to LRFD design. The design procedures and examples which follow the discussions are given first in ASD format and then repeated in LRFD format.

The design methods chosen for this document are based on the listed references and the author's judgement but they are not the only approaches possible. Other rational design methods based on engineering judgement may also be adequate. See Appendix B for some practical aspects on column base selection.

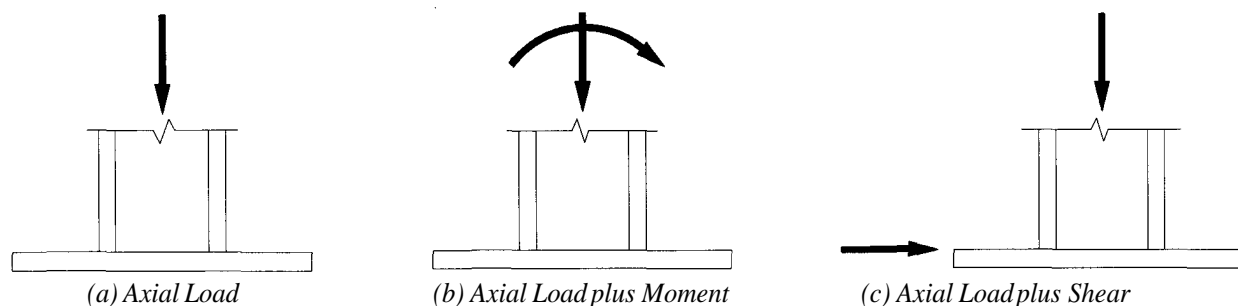


Fig. 1. Base Plate Design Cases

LITERATURE REVIEW - DESIGN PROVISIONS

The review of the research work is given in the Appendix. What follows is a review of applicable design provisions. This includes specification requirements, suggested approaches and design aids. The details are discussed more fully in the subsequent chapters.

Axially Loaded Base Plates

Axially loaded base plates, those with the load applied by a wide flange column to the center of the base plate, are designed according to the method in the ASD Manual of Steel Construction (AISC 1989a). This method is based on an allowable bearing stress defined in the ASD Specification (AISC 1989), which is a function of the concrete compressive strength and the ratio of the concrete to the plate areas. This allowable bearing stress has been increased from that in earlier Specifications to agree with the more liberalized value in the ACT Code (ACI 1983). This had been changed following the results of the studies by Hawkins (1967, 1967a, 1968, 1968a). The allowable bearing stress has been put in terms of load and resistance factor design (AISC 1986, ACI 1983a).

DeWolf (1978) and Narus (1976) have shown that the method in the Manual of Steel Construction is conservative. They have also noted that it does not consider the effects of reinforcement or the relative depth of the concrete foundation, nor does it allow for different plate thicknesses. Thus, plates designed under old specifications cannot be evaluated under the new one.

Design aids for the method in the Manual of Steel Construction have been developed by Blodgett (1966), Sandhu (1973), Dixon (1974), Stockwell (1975), Bird (1976, 1977) and Douty (1976). Good sources of detailing information are the Manual of Steel Construction, Detailing for Steel Construction (AISC 1983), Engineering for Steel Construction (AISC 1984) and Blodgett (1966).

Base plates with especially large loads require more than a simple plate. This may result in a double layer of plates, a grillage system, or the use of stiffeners to reduce the plate thickness. The design of these plates is covered by Blodgett (1966) and noted in Engineering for Steel Construction (AISC 1984).

Lightly Loaded Base Plates in which the plate size is approximately equal to the column size, were initially treated by Fling (1970) using an elastic plate bending approach and the assumption that the full plate is in contact with the concrete. The approach has been used

in the 8th Edition Manual of Steel Construction. This has been shown to be conservative. Stockwell (1975), with modifications by Murray (1983), developed a method which is based on the assumption that bearing occurs only under the column flanges and web. Murray also treats base plates subject to uplift. This approach has been adopted for the 1st Edition Load and Resistance Design Factor Manual (AISC 1986).

Base Plates with Moments

Base plates with both axial loads and moments are not covered in the AISC Specification or the Manual of Steel Construction. Engineers must refer to textbooks for design information, though not all texts cover this case. Two general approaches exist for design, one based on the elastic behavior and one based on the ultimate capacity. For each of these approaches, different assumptions are made.

The elastic approach is covered in the majority of texts which treat moments, including those of Ballio and Mazzolani (1983), Blodgett (1966), Gaylord and Gaylord (1972), McGuire (1968), and Salmon and Johnson (1980). Soifer (1966) has noted that the design can be based on that for reinforced concrete columns. He has stated that the anchor bolt force determination is the most important design element, and that the precise determination of the concrete bearing stress distribution is not essential. He based his discussion on the elastic approach. The approach based on the ultimate capacity, much like that used for the design of reinforced concrete columns today, is based on the study of Salmon, Schenker and Johnston (1957). The method is presented by Gaylord and Gaylord (1972) and McGuire (1968). Both used it to calculate the ultimate load for plates designed by the elastic approach.

DeWolf and Sarisley (1978, 1980) have compared both methods to test data. While they found that either normally provides an adequate factor of safety against collapse, the methods rely on the assumption of some of the variables. Consequently all of the test variables do not usually match with those used in design. They have made suggestions for alterations in the methods and have noted when they are not satisfactory. Thambiratnam and Paramasivam (1986) also conducted tests and compared the results with predictions from the elastic design method.

Maitra (1978, 1978a) has presented a graphical design aid for applying the elastic method.

Anchor Bolts for Tension

For all but the smallest moments, anchor bolts are needed. There are different ways of placing and anchoring these. Lee et. al. (1957) presents designs for placement following setting of the concrete. Others have treated anchorage for machines (Lee 1959, Engineering News Record 1960) and prestressing tendons (Schechter 1960). Hasselwander, Jirsa and Breen (1974) reviewed materials which are suitable for anchor bolts. Details and types of bolts are presented by Fisher (1981), Goldman (1983), Marsh and Burdette (1985, 1985a) as well as in AISC guides for engineering and detailing (AISC 1983, 1984).

The design of anchor bolts is not defined in present codes and specifications for steel construction and is thus left to the discretion of the engineer. Design information is available and has been based on work developed by the American Concrete Institute for nuclear structures (ACI 1978, 1983b). This has been used by Cannon, Godfrey and Moreadith (1981) to write a specification and commentary for anchors not used for nuclear structures. Fisher (1981), Klinger and Mendonca (1982), Shipp and Haninger (1983) and Marsh and Burdette (1985, 1985a) have used the ACI work as a basis for developing guidelines which can be used for the design of anchor bolts for base plates.

Marsh and Burdette also discuss the different types of drilled-in anchors, those that are placed following casting of the concrete foundation. These are not normally used for column base plates and are beyond the scope of this publication. Most of these are proprietary and would be designed according to the manufacturer's specification.

Shear Loads

The design for shear, i.e. loads applied horizontally, is not covered in the ASD Specification (AISC 1989) or in the ASD Manual (AISC 1989a).

Kharod (1980) presents a design based on the capacity of the bolts. It is applicable for smaller shear loads. He incorporates the interaction of shear and tension. Fisher (1981) gives details and general guidelines for using bolts and shear lugs, which he refers to as thrust bars. Cannon, Godfrey and Moreadith (1981) present a specification and commentary for the use of bolts to resist shear. Klinger, Mendonca and Malik (1982) give guidelines for placing reinforcing adjacent to the bolts when they are close to the edges.

Ballio and Mazzolani (1983) discuss the transfer of shear by friction, and the use of bolts and shear lugs. They include the combination of shear and tension. Goldman (1983) discusses the use of friction, bolts and shear lugs. He gives a design example for designing for shear and tension combined. Shipp and Haninger (1983) discuss the design of headed anchor bolts for tension and shear, and give a design example. Tronzo (1983-84) gives a design example using shear lugs to resist the full shear.

DESIGN OF AXIALLY LOADED BASE PLATES

Design Procedure

The method recommended for the design of axially loaded base plates, those assumed to be pinned at the base, is given in the AISC Manual of Steel Construction (AISC 1986, 1989a). The design case is shown in Fig. 2(a). It is assumed that the wide flange column is centered on the plate and that the plate is then centered on the concrete foundation. The AISC method is a two step approach. The required plate area is first determined, based on an assumed uniform allowable bearing stress defined in Section J9 in the AISC Specification (AISC 1989, 1986a). The allowable bearing stress F_p is a function of the concrete strength and the ratio of the concrete to plate areas, as follows:

$$F_p = 0.35 f'_c \sqrt{\frac{A_2}{A_1}} \leq 0.70 f'_c$$

where

f'_c = concrete compressive strength

A_1 = area of the base plate

A_2 = area of the supporting concrete foundation that is geometrically similar to the plate.

The increase in the allowable bearing stress when the concrete area is greater than the plate area accounts for the beneficial effects of confinement. The highest value then occurs when this ratio is equal to or greater than 4.0, and this results in the smallest plate. The full plate area is used to determine the plan dimensions. The loss of area due to anchor bolt holes, even though oversized, and due to holes used for the placement of grout is normally ignored.

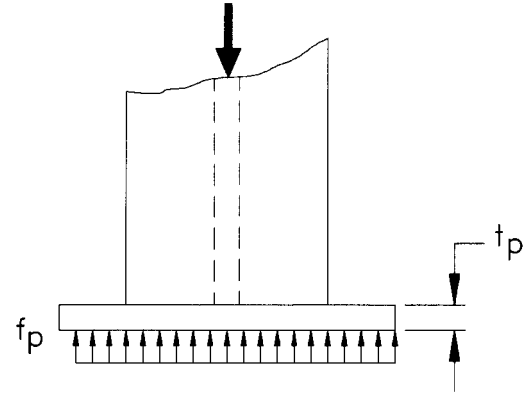
In LRFD format, the factored load on the column P_u should be governed by the following:

$$P_u = \phi_c P_p = 0.85 \phi_c f'_c A_1 \sqrt{\frac{A_2}{A_1}} \leq \phi_c 1.7 f'_c A_1$$

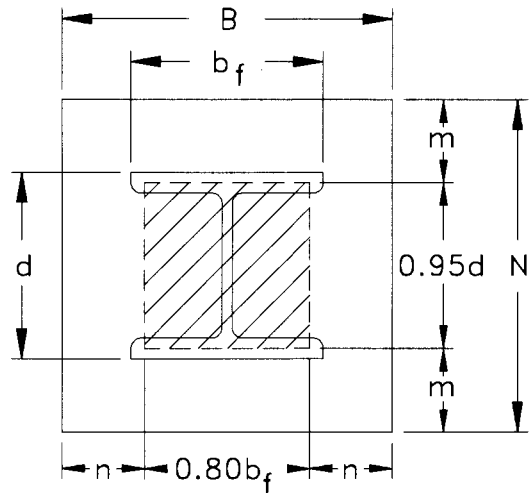
where

ϕ_c = resistance factor for bearing on concrete, equal to 0.60

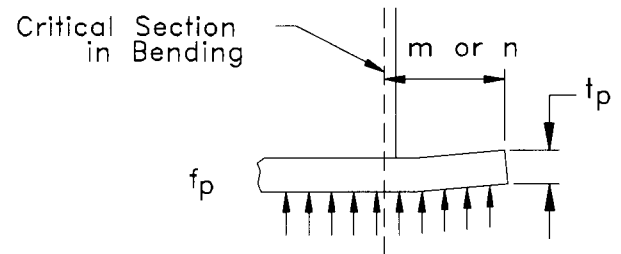
P_p = limit state capacity of the concrete in bearing.



(a) Assumed Bearing Stress



(b) Critical Sections



(c) Determination of Moment

Fig. 2. Design of Plate with Axial Load

The second step then is to determine the plate thickness. The allowable bearing stress has been based on the bearing stress used in the ACI Code (ACI 1983) which is in turn based on the work of Hawkins pertaining to rigid plates (1967, 1968). Thus the plate should be designed to behave like a rigid plate. The plate is assumed to bend about the critical sections as a cantilevered beam, loaded with the uniformly distributed bearing stress. The critical sections are near the edges of the column, shown in Fig. 2 (b), and the cantilever used in design is shown in Fig. 2 (c). The governing bending stress is computed from an elastic analysis based on the larger of m or n . The allowable plate bending stress F_b is equal to $0.75 F_y$ where F_y is the steel yield stress.

In LRFD format, the plastic moment is used for the plate capacity. It is equal to $Z F_y$, where Z is the plastic section modulus, equal to $t_p^2/4$ for a 1 in. wide strip of the plate. The resistance factor for bending ϕ_b is 0.90. The LRFD plastic moment strength is more generous than $0.75 F_y S$ for ASD, and thus LRFD will always result in thinner plates.

Design for Lightest Plate

The most economical plate occurs when m and n , shown in Fig. 2, are equal and the ratio of the concrete to plate area is equal to or greater than 4.0. The first occurs when the difference between B and N , shown in Fig. 2, is equal to the difference between $0.95d$ and $0.80 b_f$, where d is the depth and b_f is the flange width of the wide flange column. The following procedure gives the steps needed for obtaining the lightest plate:

ASD Procedure:

1. The allowable bearing stress F_p is:

$$F_p = 0.70 f'_c$$

2. The required plate area A_1 is:

$$A_1 = \frac{P}{F_p}$$

where P is the axial load.

3. The plate dimensions, B and N , should be determined so that m and n are approximately equal:

$$N = \sqrt{A_1} + \Delta$$

where $\Delta = 0.5 (0.95 d - 0.80 b_f)$

Then:

$$B = \frac{A_1}{N}$$

These values should be rounded up to the nearest half or whole inch.

4. Determine the actual bearing pressure:

$$f_p = \frac{P}{NB}$$

5. Determine m and n .

6. Determine the required plate thickness t_p , based on the larger value m and n :

$$t_p = (m \text{ or } n) \sqrt{\frac{f_p}{0.25 F_y}}$$

7. The pedestal dimensions are then determined. Since the procedure was based on the highest allowable bearing stress, the minimum concrete area should be:

$$A_2 = 4 N B$$

8. Check for plate bending in the portion between the column flanges, using the procedure for lightly loaded plates.

Example 1 (ASD Procedure): A W10 x 100 column, ($d = 11.10$ in., $b_f = 10.34$ in.) has a reaction of 500 kips. The smallest acceptable base plate is desired. $f'_c = 3$ ksi; $F_y = 36$ ksi.

1. The allowable bearing stress is:

$$F_p = 0.70 \times 3 = 2.1 \text{ ksi}$$

2. The required plate area is:

$$A_1 = \frac{500}{2.1} = 238.1 \text{ in.}^2$$

3. $\Delta = 0.5 [0.95 (11.1) - 0.8 (10.34)] = 1.13$ in.


Then N is approximately:

$$N = \sqrt{A_1} + 1.13 = 16.56 \text{ in.}$$

Use 17 in. Then B should be $\frac{238.1}{17}$ or 14 in.

4. The actual bearing stress is then:

$$f_p = \frac{500}{(17 \times 14)} = 2.1 \text{ ksi}$$

5. $m = \frac{[17 - (0.95 \times 11.1)]}{2} = 3.23$ in. 

$$n = \frac{[14 - (0.8 \times 10.34)]}{2} = 2.86 \text{ in.}$$

$$6. \quad t_p = 3.23 \sqrt{\frac{2.10}{(0.25 \times 36)}} = 1.56 \text{ in.}$$

Use 1½ in. thickness.

7. The pedestal area is then:

$$A_2 = 4 \times 17 \times 14 = 952 \text{ in.}^2$$

A 31-in. square pedestal is satisfactory.

8. The portion of the plate between the flanges should be checked, based on the procedure for lightly loaded base plates.

LRFD Procedure:

1. Determine the factored load P_u .
2. The required plate area A_1 is:

$$A_1 = \frac{P_u}{1.7 \phi_c f'_c}$$

where P_u is the factored load.

3. The plate dimensions, B and N , should be determined so that m and n are approximately equal.

$$N = \sqrt{A_1} + \Delta$$

where $\Delta = 0.50 (0.95d - 0.80 b_f)$

Then:

$$B = \frac{A_1}{N}$$

These values should be rounded up to the nearest half or whole inch.

4. Determine m and n .
5. Determine the required plate thickness t_p based on the larger value m and n :

$$t_p = (m \text{ or } n) \sqrt{\frac{2 P_u}{0.90 F_y B N}}$$

6. The pedestal dimensions are then determined. Since the procedure was based on the highest allowable bearing stress, the minimum concrete area should be:

$$A_2 = 4 N B$$

7. Check for plate bending in the portion between the column flanges, using the procedure for lightly loaded base plates.

Example 2 (LRFD Procedure): A W10 x 100 column ($d = 11.10$ in., $b_f = 10.34$ in.) is loaded with 180 kips dead load and 320 kips live load. The smallest acceptable base plate is desired. $f'_c = 3$ ksi; $F_y = 36$ ksi.

1. The factored load is:

$$P_u = 1.2(180) + 1.6(320) = 728 \text{ kips}$$

2. The required plate area is:

$$A_1 = \frac{728}{(1.7 \times 0.60 \times 3)} = 237.9 \text{ in.}^2$$

3. $\Delta = 0.5 [0.95 (11.1) - 0.8 (10.3)] = 1.15$ in.
Then N is approximately:

$$N = \sqrt{A_1} + 1.15 = 16.57 \text{ in.}$$

Use 17 in. then B should be $\frac{237.9}{17}$ or 14 in.

$$4. \quad m = \frac{[17 - (0.95 \times 11.1)]}{2} = 3.23 \text{ in.}$$

$$n = \frac{[14 - (0.80 \times 10.34)]}{2} = 2.86 \text{ in.}$$

$$5. \quad t_p = 3.23 \sqrt{\frac{2 \times 728}{(0.90 \times 36 \times 14 \times 17)}} = 1.40 \text{ in.}$$

Use 1½ in. thickness.

6. The pedestal area is then:

$$A_2 = 4 \times 17 \times 14 = 952 \text{ in.}^2$$

A 31 inch square pedestal is satisfactory.

7. The portion of the plate between the flanges should be checked, based on the procedure for lightly loaded base plates.

General Design Procedure

It is not always possible to have concrete pedestals equal to four times the plate area. If the ratio of the concrete to plate area is determined before designing the plate, the design would follow the previous example, with the appropriate change to the allowable bearing stress.

When the pedestal dimensions are known, it is not possible to calculate the allowable bearing stress directly. The following procedure, from the AISC Manuals should be used:

ASD Procedure:

1. The area of the plate should be equal to the larger of:

$$A_1 = \left(\frac{1}{A_2} \right) \left(\frac{P}{0.35 f'_c} \right)^2$$
$$A_1 = \frac{P}{0.70 f'_c}$$

(If the second equation governs, the concrete area A_2 is equal to or greater than four times the plate area A_1)

2. Same as 3 in previous ASD procedure.
3. Same as 4 previously.
4. Same as 5 previously.
5. Same as 6 previously.
6. Same as 8 previously.

Example 3 (ASD Procedure): Design a plate for a W10 x 100 column supporting a load of 525 kips, bearing on a 25 x 25 in. pedestal. $f'_c = 3$ ksi; $F_y = 36$ ksi.

1. $A_2 = 25 \times 25 = 625 \text{ in.}^2$

$$A_1 = \left(\frac{1}{625} \right) \times \left[\frac{525}{(0.35 \times 3)} \right]^2 = 400 \text{ in.}^2 \quad \leftarrow$$

$$A_1 = \frac{525}{(0.70 \times 3)} = 250 \text{ in.}^2$$

2. $\Delta = 1.13 \text{ in.}$, as determined in previous example.
Then:

$$N = \sqrt{400} + 1.13 = 21.1 \text{ in.}$$

Use 21 in. Then $B = \frac{400}{21} = 19.1 \text{ in.}$ Use 19 in., which is close enough.

3. $f_p = \frac{525}{(21 \times 19)} = 1.32 \text{ ksi}$

4. $m = \frac{[21 - (0.95 \times 11.1)]}{2} = 5.23 \text{ in.}$

$$n = \frac{[19 - (0.80 \times 10.34)]}{2} = 5.36 \text{ in.} \quad \leftarrow$$

5. $t_p = 5.36 \sqrt{\frac{1.32}{(0.25 \times 36)}} = 2.05 \text{ in.}$

Use $2\frac{1}{8} \text{ in.}$

6. The portion of the plate between the flanges should be checked, based on the procedure for lightly loaded base plates.

LRFD Procedure:

1. Determine the factored load P_u .
2. The area of the plate should be equal to the larger of:

$$A_1 = \frac{1}{A_2} \left[\frac{P_u}{(0.60 \times 0.85 f'_c)} \right]^2$$

$$A_1 = \frac{P_u}{0.60 \times 1.7 f'_c}$$

(If the second equation governs, the concrete area A_2 is equal to or greater than 4 times the plate area A_1)

3. Same as previous LRFD procedure.
4. Same as 4 previously.
5. Same as 5 previously.
6. Same as 7 previously.

Example 4 (LRFD Procedure): Design a plate for a W10 x 100 column supporting a dead load of 188 kips and a live load of 338 kips, bearing on a 25 x 25 in. pedestal. $f'_c = 3$ ksi; $F_y = 36$ ksi.

1. $P_u = 1.2(188) + 1.6(338) = 766 \text{ kips}$

2. $A_2 = 25 \times 25 = 625 \text{ in.}^2$

$$A_1 = \frac{1}{625} \left[\frac{766}{(0.60 \times 0.85 \times 3)} \right]^2 = 401 \text{ in.}^2 \quad \leftarrow$$

$$A_1 = \frac{728}{(0.60 \times 1.7 \times 3)} = 238 \text{ in.}^2$$

3. $\Delta = 1.13 \text{ in.}$, as determined in previous example.
Then:

$$N = \sqrt{401} + 1.13 = 21.2 \text{ in.}$$

Use 21 in. Then $B = \frac{401}{21} = 19.1 \text{ in.}$ Use 19 in., which is close enough.

4. $m = \frac{(21 - 0.95 \times 11.1)}{2} = 5.23 \text{ in.}$

$$n = \frac{[19 - (0.8 \times 10.34)]}{2} = 5.36 \text{ in.} \quad \leftarrow$$

5. $t_p = 5.36 \sqrt{\frac{(2 \times 766)}{(0.90 \times 36 \times 19 \times 21)}} = 1.85 \text{ in.}$

Use $1\frac{7}{8} \text{ in.}$

6. The portion of the plate between the flanges should be checked, based on the procedure for lightly loaded base plates.

Design Aid for Axially Loaded Base Plates

Stockwell (1975) prepared a useful allowable stress design aid for determining the values of N and B , the plate dimensions. A set of nomographs allows the engineer to rapidly determine plate sizes for different ratios of A_2/A_1 . This is illustrated in Fig. 3. The two quadrants in each chart have the bearing area A_1 as the common vertical axis. The horizontal axes are the concrete area A_2 and the base plate width B . The range of allowable bearing pressures is shown in the right quadrant. The engineer enters with the value of the load P , then follows horizontally to the zone showing the allowable bearing stresses. The engineer can then choose a value of the allowable stress along a circular segment shown by line 1-2. The two axes then give the areas for the plate and pedestal, A_1 and A_2 . If the area of the concrete is already known, the engineer can get the appropriate allowable stress and the required plate area A_1 from the intersection of the load line and the concrete area. The left quadrant is then used to determine the values of N and B . The curved line represents the values for a square plate. Figures 4 (a) through (f) give the design charts for f'_c equal to 3000 and 4000 psi.

$$A_1 = \frac{1}{A_2} \left[\frac{P}{0.35 f'_c} \right]^2$$

$$A_1 = \frac{1}{A_2} \left[\frac{P_u}{0.60 \times 0.85 f'_c} \right]^2 = \frac{1}{A_2} \left[\frac{P_u}{0.51 f'_c} \right]^2$$

$$= \frac{1}{A_2} \left[\frac{P_u / 1.46}{0.35 f'_c} \right]^2 = \frac{1}{A_2} \left[\frac{\bar{P}}{0.35 f'_c} \right]^2$$

$$\bar{P} = (P_u / 1.46),$$

$$\frac{DL + LL}{0.35} = \frac{1.2 DL + 1.6 LL}{0.60 \times 0.85}$$

$$\frac{DL + 1.8 DL}{0.35} = \frac{1.2 DL + 1.6 \times 1.8 DL}{0.51}$$

$$8DL = 8DL$$

For load ratios other than 1.8, the base plate areas determined by the two methods will differ (larger LRFD areas for greater ratios, smaller LRFD areas for lesser ratios).

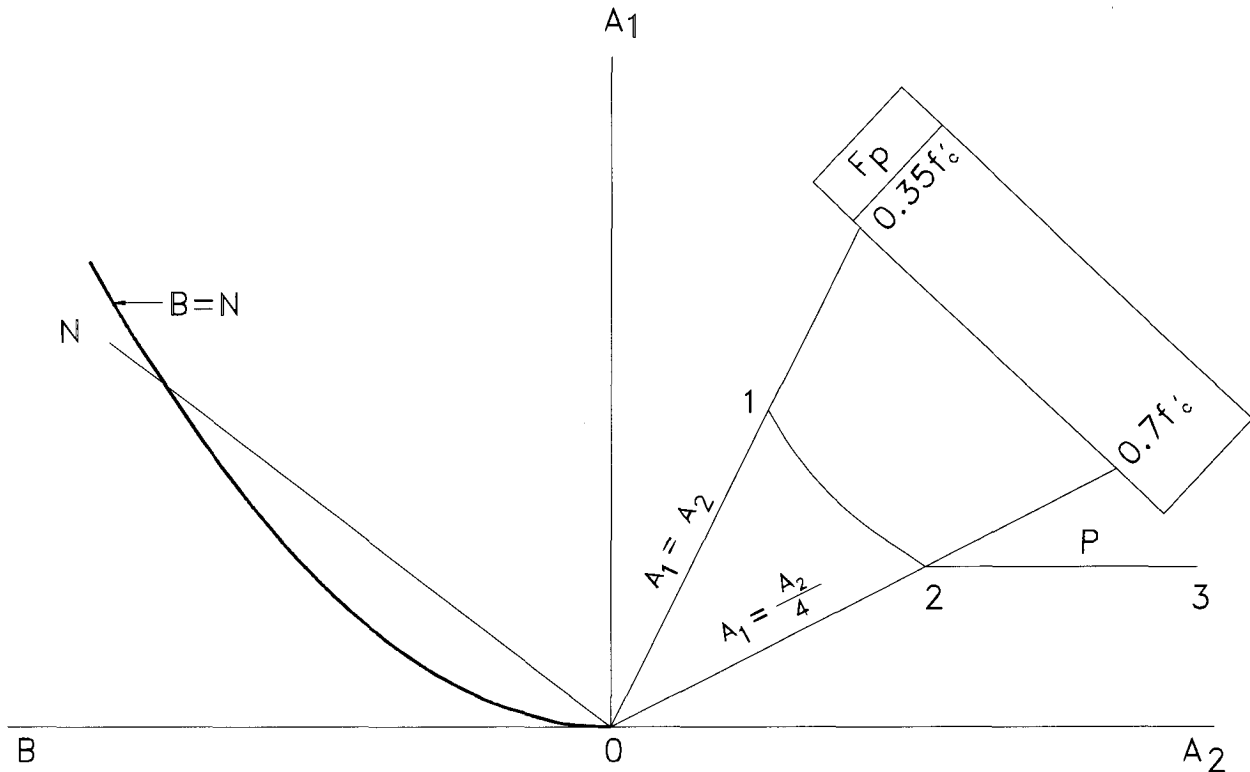


Fig. 3. Stockwell's Design Aid for Axially Loaded Plate.

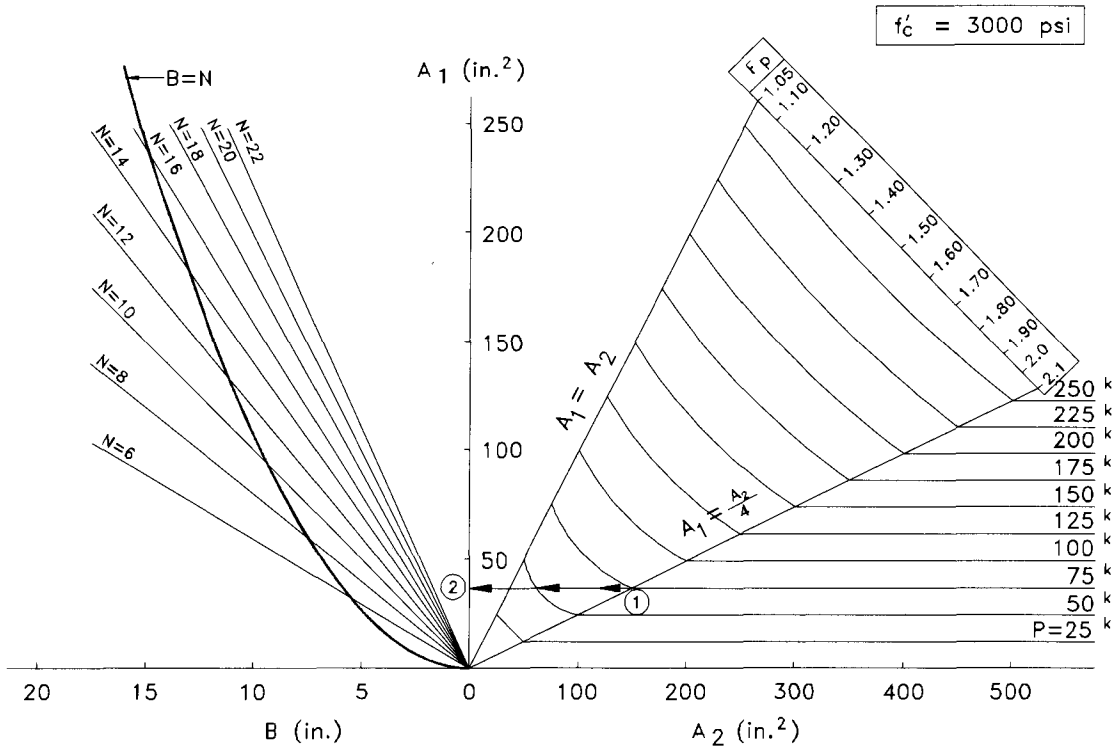


Fig.4a. Design Aid for Axially Loaded Plates (Working Loads)

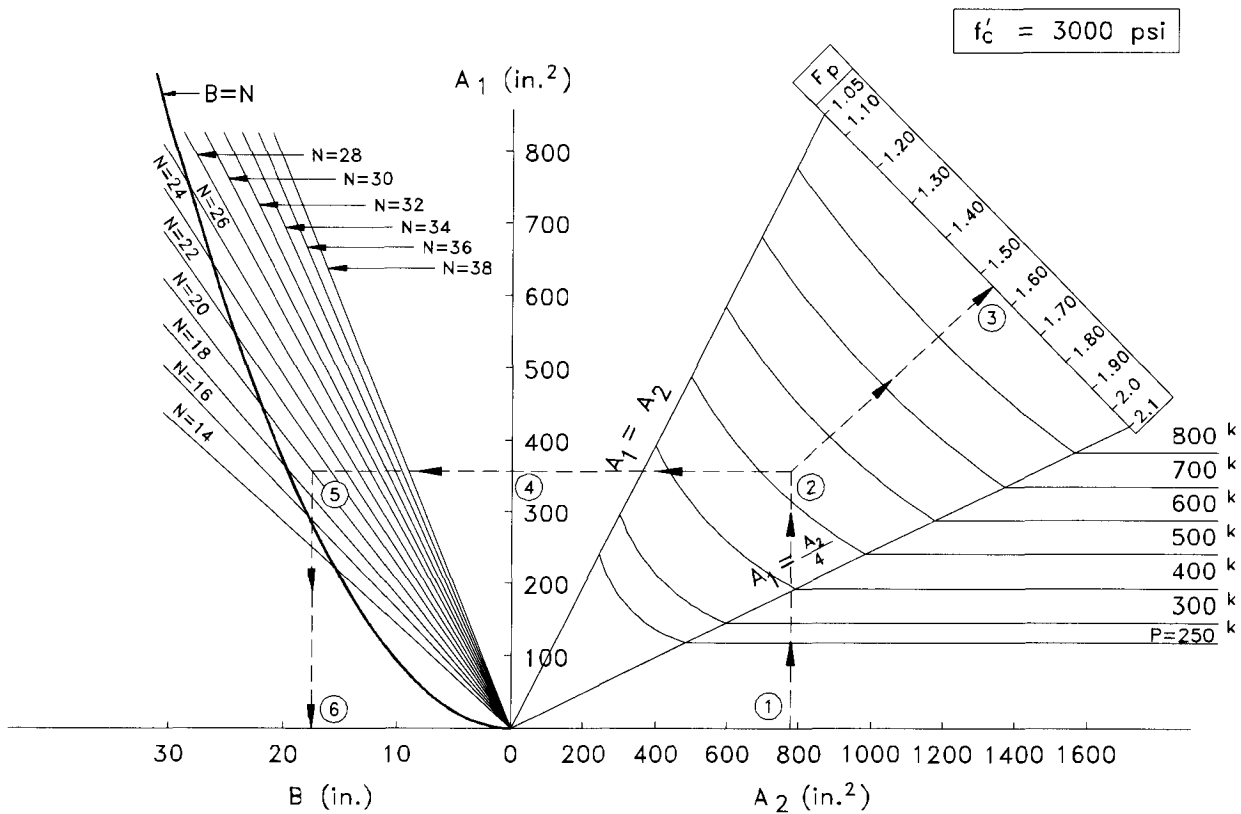


Fig.4b. Design Aid for Axially Loaded Plates (Working Loads)

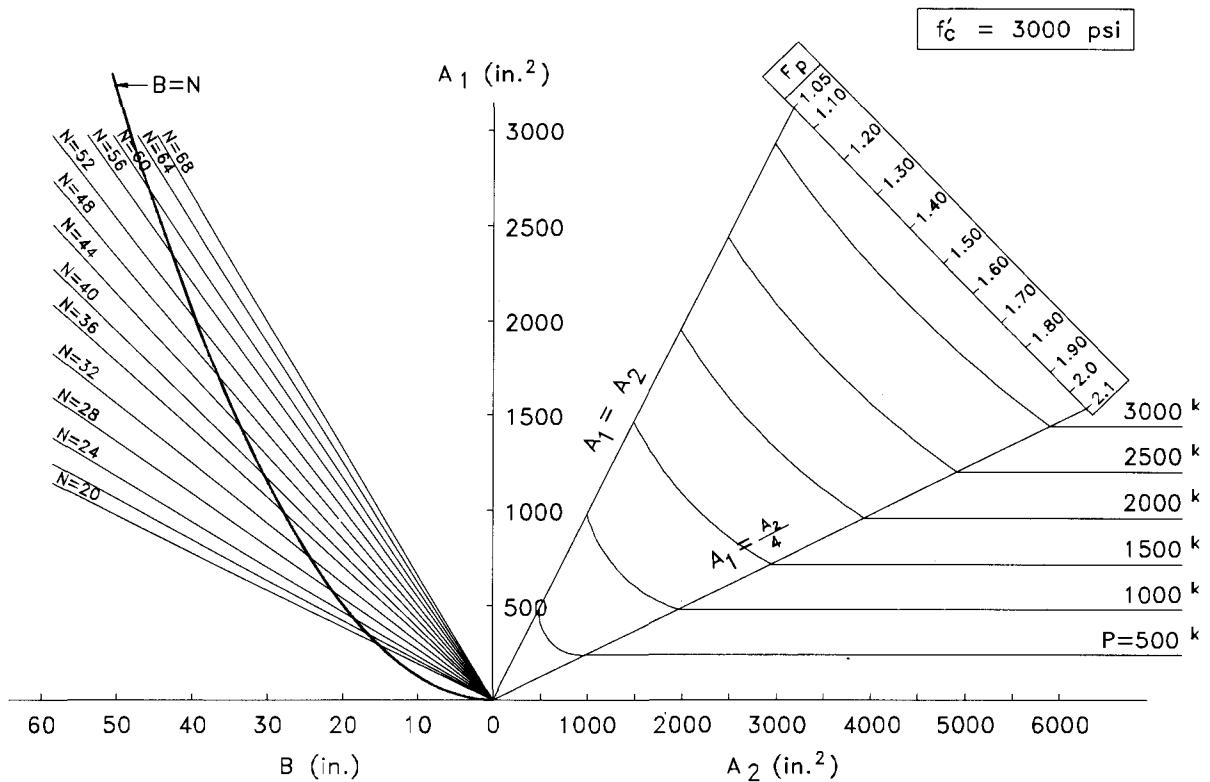


Fig.4c. Design Aid for Axially Loaded Plates (Working Loads)

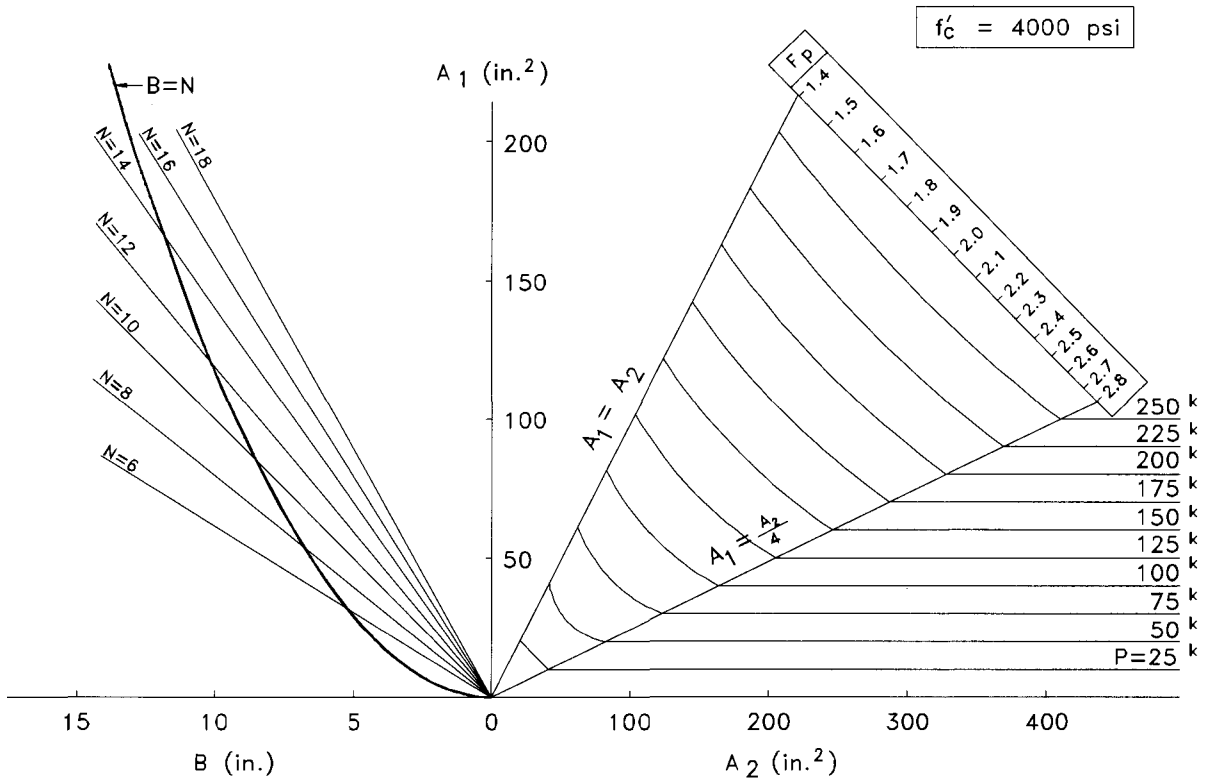
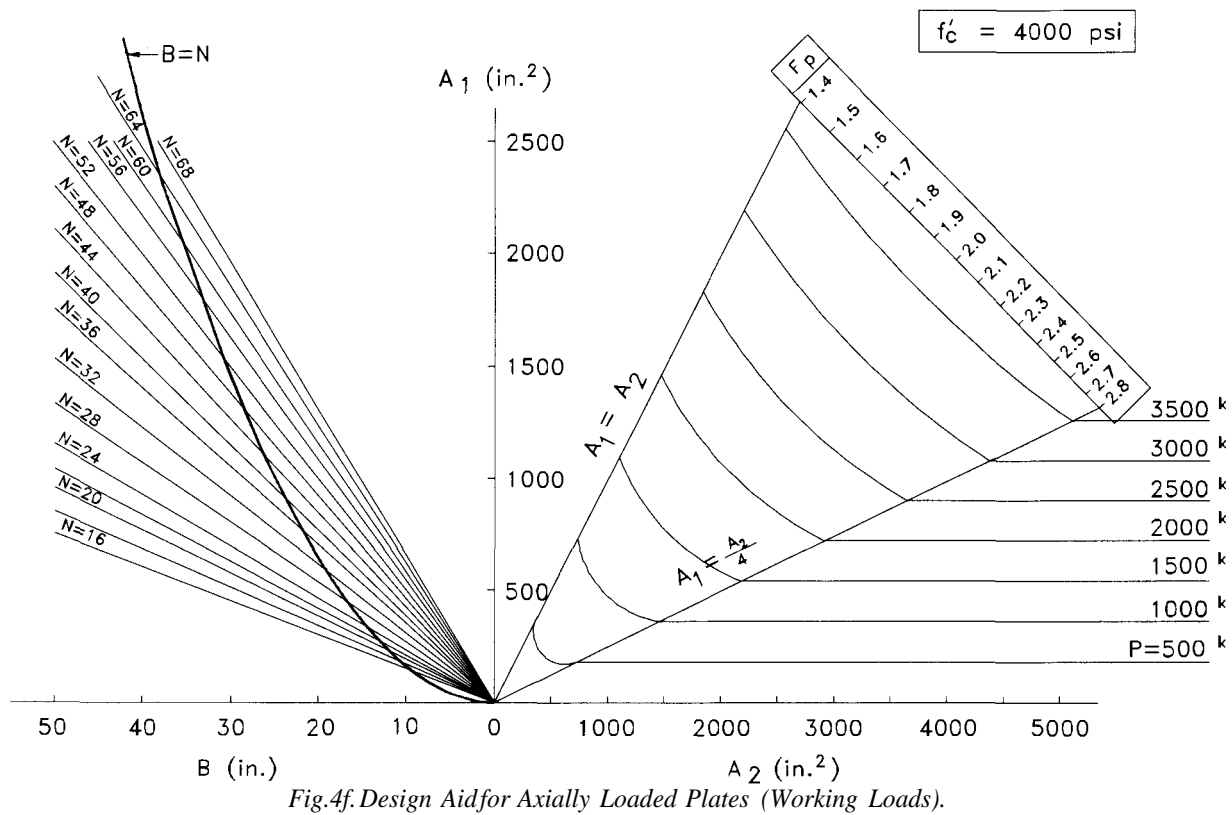
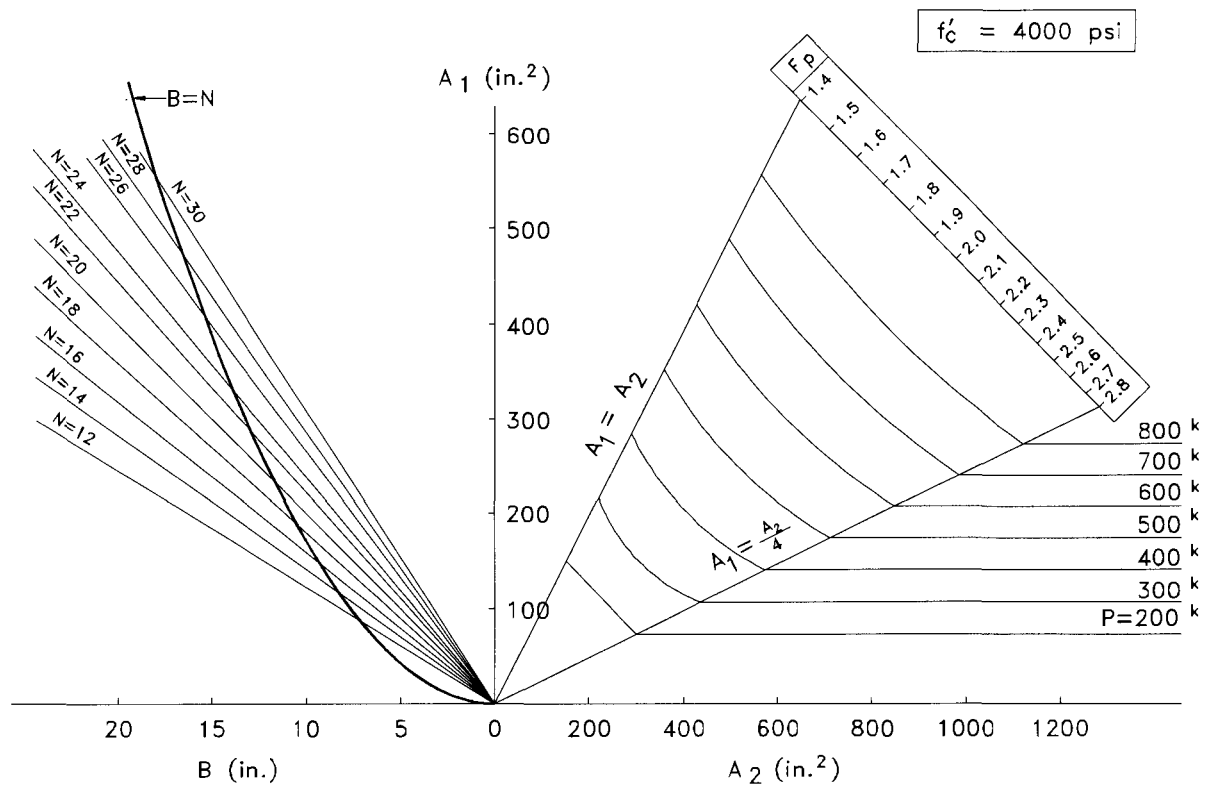


Fig.4d. Design Aid for Axially Loaded Plates (Working Loads)



The example, taken from Stockwell's paper, follows:

Example 5 (ASD Procedure): A base plate is to be selected for a W12 x 106 column supporting a load of 560 kips, bearing on a 28 x 28-in. pier. The concrete compressive strength f'_c is 3 ksi.

1. In Fig. 4(b), enter the first quadrant at $A_2 = 28 \times 28 = 784 \text{ in.}^2$
2. Proceed vertically to $P = 560$ kips.
3. Draw a line from the origin in the right quadrant through point 2 and find that the allowable bearing pressure is $F_p = 1.54$ ksi.
4. From point 2, proceed horizontally to $A_1 = 365$ in.
5. Continue horizontally to $N = 20$ in.
6. Proceed vertically to $B = 18.4$ in.
7. Use a 19 by 20 in. plate and determine the thickness as in the previous examples.

Example 6 (LRFD Procedure): A base plate is to be selected for a W12 x 106 column with a dead load of 200 kips and a live load of 360 kips. The pier is 28 in. square and f'_c is 3 ksi.

1. Determine the factored load:
 $P_u = 1.2(200) + 1.6(360) = 816$ kips
2. Determine \bar{P} (equal to P_u divided by 1.46):
 $\bar{P} = \frac{816}{1.46} = 559$ kips
3. Follow steps 3 through 6 in the previous ASD Example 5.
4. Use P_u to compute the required plate thickness by the LRFD equation.

Base Plates Placed Eccentrically on the Concrete Foundations

When the plate is axially loaded, but placed eccentrically on the concrete foundation, Hawkins (1967,1968) found that the design could be conservatively based on using that portion of the concrete which is concentric around the plate. This is recommended for use here. The design approach already given can then be used, with the appropriate A_2 .

Determining the Design Load for Existing Base Plates

The method in the AISC Manual for designing base plates results in a unique plate thickness. It is not directly applicable for determining the allowable axial column load for an existing base plate that has been designed under different conditions. An example is a plate designed according to earlier AISC Specifications which specified lower allowable bearing stresses. It will have a greater area A_1 , with greater values of m and n . The resulting thickness, which was determined for the lower allowable bearing stress, will not allow evaluation of the plate with the new allowable bearing stresses. As an approximation, the following procedure can be used. It is based on assuming that only a portion of the plate is effective.

Example 7 (ASD Procedure): A plate has been designed for a W14 x 95 column ($d = 14.12$ in., $b_f = 14.545$ in.). $f'_c = 3$ ksi; $F_y = 36$ ksi. The plate was designed for a load of 480 kips using an older edition of the AISC Specification, with an allowable bearing stress

of $0.25 f'_c$. $\frac{A_2}{A_1}$ is equal to 1.0. The building is being remodeled, and it is desired to find out if an additional load of 70 kips can be applied without exceeding the allowable bearing stress in the present edition of the AISC Specification. The plate size is 25 x 1 5/8 in. x 2 ft 2 in.

The allowable plate bending stress is the same as used in the original design. It is thus not possible to use the new bearing stress based on the original values of A_1 and A_2 , without exceeding the allowable bending stress. Instead only a portion of the plate will be used. The two requirements are that the allowable bearing stress should not exceed $F_p = 0.35 f'_c \sqrt{\frac{A_2}{A_1}}$ and that the bending stress

F_b should not exceed $0.75 F_y$. The design aid prepared by Stockwell will be used.

1. For a load of 550 kips and $A_2 = 25 \times 26 = 650 \text{ in.}^2$, A_1 is found equal to approximately 425 in.^2 from Fig. 4(b).
2. $\Delta = 0.50 (0.95 \times 14.12 - 0.80 \times 14.545) = 0.89$ in.
 $N = \sqrt{425 + .89} = 20.6$ in. Use 22 in.
 $B = \frac{425}{22} = 19.3$ in. Use 20 in.
3. $f_p = \frac{550}{(22 \times 20)} = 1.25$ ksi.

$$4. \quad m = \frac{[22 - (0.95 \times 14.12)]}{2} = 4.29 \text{ in.}$$

$$n = \frac{[20 - (0.80 \times 14.545)]}{2} = 4.18 \text{ in.}$$

$$5. \quad t_p = 4.29 \sqrt{\frac{1.25}{(0.25 \times 36)}} = 1.60 \text{ in.}$$

This thickness is smaller than the actual value, so that it is possible to increase the allowable load to 550 kips from the original design value of 480 kips. A trial and error approach could be used to determine how much load could actually be added to the column without exceeding the new allowable bearing stress and the allowable bending stress.

Example 8 (LRFD Procedure): A plate has been designed with ASD provisions for a W 14 x 95 column ($d = 14.12$ in. $b_f = 14.545$ in. $f'_c = 3$ ksi; $F_y = 36$ ksi). The plate was designed for a dead load of 170 kips and a live load of 310 kips. An older edition of the AISC Specification was used with an allowable bearing stress of $0.25 f'_c \cdot \frac{A_2}{A_1}$ is equal to 1.0. It is desired to find out if an additional live load of 64 kips can be applied without exceeding the allowable bearing stress in the LRFD provisions. The plate size is 25 x 1 5/8 in. x 2 ft 2 in.

Only a portion of the plate will be assumed to be effective, as done with the ASD example. The two requirements are that the factored load on the column P_u should not exceed $0.85 \phi_c f'_c A_1 \sqrt{\frac{A_2}{A_1}}$ and that the required plate

effective thickness, equal to $(m \text{ or } n) \sqrt{\frac{2 P_u}{(0.90 F_y B N)}}$, should be equal to or smaller than the actual thickness.

1. Determine the factored load:

$$P_u = 1.2(170) + 1.6(310 + 64) = 802 \text{ kips}$$

2. Determine the equivalent ASD design load:

$$\bar{P} = \frac{802}{1.46} = 550 \text{ kips}$$

3. From Stockwell's design aid with a load of 550 kips and $A_2 = 650 \text{ in.}^2$, find $A_1 = 425 \text{ in.}^2$
4. Same as 2 in ASD sample.
5. Same as 4 in ASD example.

$$6. \quad t_p = 4.29 \sqrt{\frac{(2 \times 802)}{(0.90 \times 36 \times 20 \times 22)}} = 1.44 \text{ in.}$$

This is smaller than the actual thickness, and thus the additional live load can be added to the base plate.

Design of Lightly Loaded Base Plates

Lightly loaded base plates are those in which the plate size is equal to or slightly larger than the column dimensions. For these m and n are approximately zero, and the critical portion of the plate for bending is between the column flanges, adjacent to the web. As noted, the 8th Edition Manual of Steel Construction contains a procedure for this plate thickness determination. Recent work by Murray (1983) has been used for the new Load and Resistance Factor Design Manual (AISC 1986) and 9th Edition. It is based on the approach of Stockwell (1975) and is more realistic than the method in the 8th Edition Manual. The Murray/Stockwell method has been modified to allowable stress design format in the following procedure. The plate allowable bending stress partially reflects its compact bending capacity, $F_b = 0.75 F_y$.

Note: The following procedure includes both the check for the area between the column flanges and the general design procedure previously discussed. All plates can be fully designed with this procedure. If m or n is greater than either $b_f/2$ or $d/2$ (then c will not govern), the procedure for the area between the plates is unnecessary. Thus the earlier design procedures would then be sufficient.

An improved approach to the design of lightly loaded base plates is given in Appendix C. This new approach is included in the revised printing of the 9th Ed. Manual.

ASD Procedure:

1. The area of the plate A_1 should be equal to the larger of:

$$A_1 = \left(\frac{1}{A_2} \right) \left(\frac{P}{0.35 f'_c} \right)^2$$

$$A_1 = \frac{P}{0.70 f'_c}$$

$$A_1 = b_f d$$

2. The bearing area is based on the assumption that it is H-shaped and located just below the column elements. This is shown in Fig. 5. It is the larger of:

$$A_H = \frac{P}{0.35 f'_c \sqrt{\frac{A_2}{b_f d}}}$$

$$A_H = \frac{P}{0.70 f'_c}$$

3. The distance for the cantilevered section used to figure the bending stress is shown in Fig. 5 and can be calculated from:

$$2(d + b - 2c)c = \frac{P}{F_p} = A_H, \text{ therefore}$$

$$c = 0.25 \left[d + b_f - \sqrt{(d + b_f)^2 - 4 A_H} \right]$$

The thickness is then determined as previously, assuming elastic behavior:

$$t_p = c \sqrt{\frac{P}{0.25 F_y A_H}}$$

- (Optional) If $(A_2 / db_f) \leq 4$ refine design by using previous A_H for db_f in step 2 and continue through step 4.

Example 9 (ASD Procedure): A W 10 x 100 column has a reaction of 200 kips. Design the base plate for a concrete pedestal 25 in. square. $f'_c = 3$ ksi and $F_y = 36$ ksi.

$$1. A_1 = \left(\frac{1}{625} \right) \left[\frac{200}{(0.35 \times 3)} \right]^2 = 58.0 \text{ in.}^2$$

$$A_1 = \frac{200}{(0.70 \times 3)} = 95.2 \text{ in.}^2$$

$$A_1 = 11.1 \times 10.34 = 114.8 \text{ in.}^2$$

The plate size is governed by the column size. The plate size is then set equal to $A_1 = 11 \times 11.5 = 126.5 \text{ in.}^2$

$$2. A_H = \frac{200}{(0.35 \times 3) \sqrt{\frac{625}{(10.34 \times 11.1)}}} = 81.5 \text{ in.}^2$$

$$A_H = \frac{200}{(0.70 \times 3)} = 95.2 \text{ in.}^2$$

$$3. d + b_f = 11.1 + 10.34 = 21.44 \text{ in.}$$

$$c = .25[21.44 - \sqrt{(21.44)^2 - 4(95.2)}] = 3.14 \text{ in.}$$

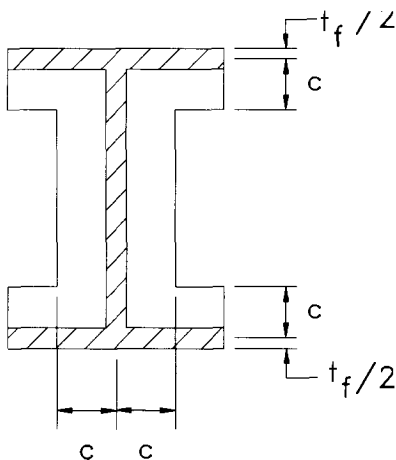


Fig. 5. Bearing Area for Lightly Loaded Base Plate

$$4. t_p = 3.14 \sqrt{\frac{200}{(0.25 \times 36 \times 95.2)}} = 1.52 \text{ in.}$$

- Since $625/114.8 = 5.44 > 4$, cannot further refine design.

LRFD Procedure:

- The area of the plate A_1 should be equal to the larger of:

$$A_1 = \frac{1}{A_2} \left[\frac{P_u}{(0.60 \times 0.85 f'_c)} \right]^2$$

$$A_1 = \frac{P_u}{0.60 \times 1.7 f'_c}$$

$$A_1 = b_f d$$

- The bearing area is based on the assumption that it is H-shaped and located just below the column elements. It is the larger of:

$$A_H = \frac{P_u}{\left(0.60 \times 0.85 f'_c \sqrt{\frac{A_2}{b_f d}} \right)}$$

$$A_H = \frac{P_u}{(0.6 \times 1.7 f'_c)}$$

- The distance for the cantilevered section used to figure the bending stress is shown in Fig. 5. It can be calculated from:

$$c = 0.25 \left[d + b_f - \sqrt{(d + b_f)^2 - 4 A_H} \right]$$

- The thickness is then determined as previously, assuming elastic behavior:

$$t_p = c \sqrt{\frac{2 P_u}{0.90 F_y A_H}}$$

- (Optional) If $A_2/b_f d \leq 4$, refine design by using previous A_H for $b_f d$ in step 2 and continue through step 4.

Example 10 (LRFD Procedure): A W 10 x 100 column has a dead load of 72 kips and a live load of 128 kips. Design the base plate for a concrete pedestal 25 in. square. $f'_c = 3$ ksi and $F_y = 36$ ksi.

- The factored load is:

$$P_u = 1.2(72) + 1.6(128) = 291 \text{ kips}$$

$$A_1 = \left(\frac{1}{625} \right) \left[\frac{291}{(0.60 \times 0.85 \times 3)} \right]^2 = 57.9 \text{ in.}^2$$

$$A_1 = \frac{291}{(0.60 \times 1.7 \times 3)} = 95.1 \text{ in.}^2$$

$$A_1 = 11.1 \times 10.34 = 114.8 \text{ in.}^2 \quad \leftarrow$$

The plate size is governed by the column size.
The plate size is then set equal to
 $A_1 = 11 \times 11.5 = 126.5 \text{ in.}^2$

$$2. \quad A_H = \frac{291}{(0.60 \times 0.85 \times 3) \sqrt{\frac{625}{(10.34 \times 11.1)}}} = 81.46 \text{ in.}^2$$

$$A_H = \frac{291}{(0.60 \times 1.7 \times 3)} = 95.1 \text{ in.}^2 \quad \leftarrow$$

$$3. \quad d + b_f = 11.1 + 10.34 = 21.44 \text{ in.}$$

$$c = 0.25[21.44 - \sqrt{21.44^2 - 4(95.1)}] = 3.13 \text{ in.}$$

$$4. \quad t_p = 3.13 \sqrt{\frac{(2 \times 291)}{(0.90 \times 36 \times 95.1)}} = 1.36 \text{ in.}$$

5. Since $625/114.8 = 5.44 > 4.0$, cannot further refine design.

This procedure for lightly loaded base plates, or that in the 8th Ed. AISC Manual based on determining n' should be used for all plates in order to check the portion of the plate between the column flanges. Thus, the previous examples should also include this check.

Base Plates for Uplift Loading

Under certain conditions, base plates are subject to concentric axial tension, or uplift. These plates need to be checked for bending when the design results in a relatively flexible plate which is approximately the same size as the column, i.e. a lightly loaded plate. A typical design is shown in Fig. 6. It is necessary to use properly embedded anchor bolts for these with a plate that can resist bending in the area between the flanges, adjacent to the web.

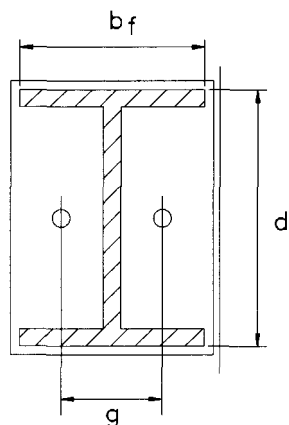


Fig. 6. Base Plate for Uplift Loading

Murray (1983), gives design equations for this case, based on a yield line method and the work of Blodgett (1966) and Stockwell (1975). This is adopted here, with the addition of the bending resistance factor used in the LRFD Manual:

When $\sqrt{2}b_f \leq d$, the required plate thickness is:

$$t_p = \sqrt{\frac{\sqrt{2}P_u g}{4\phi_b b_f F_y}}$$

When $\sqrt{2}b_f > d$, the required plate thickness is:

$$t_p = \sqrt{\frac{P_u g d}{\phi_b F_y (d^2 + 2b_f^2)}}$$

where g is the gage, shown in Fig. 6; ϕ_b is the bending resistance factor, equal to 0.9; and, P_u is the factored load. This applies to the LRFD method.

It can be modified for the ASD method by applying a factor of safety equal to 2.0 and using the applied service load P . Thus, when $\sqrt{2}b_f \leq d$, the required plate thickness is:

$$t_p = \sqrt{\frac{\sqrt{2}Pg}{2b_f F_y}}$$

and when $\sqrt{2}b_f > d$, the required plate thickness is:

$$t_p = \sqrt{\frac{Pg d}{0.50F_y (d^2 + 2b_f^2)}}$$

The design of the anchor bolts, necessary to resist the uplift is treated in the section on the design of anchor bolts for tension.

Example 11 (ASD Procedure): Determine the plate thickness for an uplift load of 25 kips due to wind and a column with $d = 10.24$ in. and $b_f = 4.02$ in. The anchor bolt gage g is 4 in. $F_y = 36$ ksi.

- $\sqrt{2} \times 4.02 = 5.69 \text{ in.} < 10.24 \text{ in.}$
- Since the load is due to wind, the allowable stress may be increased by 33%, which is equivalent to reducing the uplift force to $0.75 \times 25 = 18.75$ kips.

$$t_p = \sqrt{\frac{(\sqrt{2} \times 18.75 \times 4)}{(2 \times 4.02 \times 36)}} = 0.61 \text{ in.}$$

Example 12 (LRFD Procedure):

- $\sqrt{2} \times 4.02 = 5.69 \text{ in.} < 10.24 \text{ in.}$
- The load factor for wind is 1.3, so $P_u = 1.3 \times 25 = 32.5$ kips

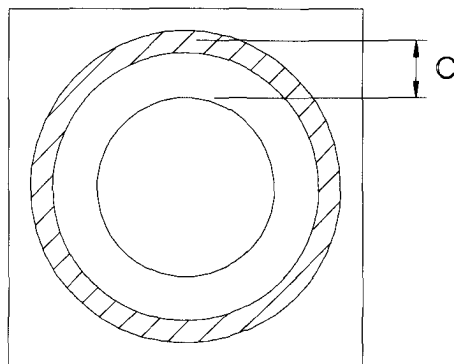
$$t_p = \sqrt{\frac{(\sqrt{2} \times 32.5 \times 4)}{(4 \times 0.90 \times 4.02 \times 36)}} = 0.59 \text{ in.}$$

Base Plates for Tube and Pipe Columns

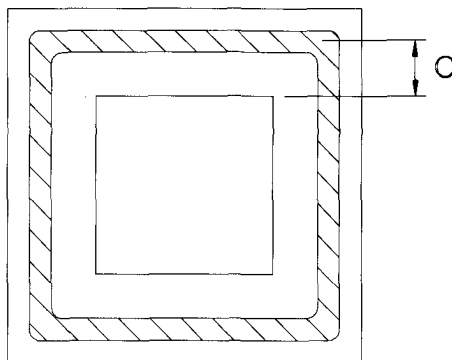
Base plates for rectangular and round pipe columns can be designed with the previous provisions, which have been developed for wide flange shaped columns.

The critical section used to determine the plate thickness should be based on 0.95 times the outside column dimension for rectangular tubes and 0.80 times the outside dimension for round pipes. These correspond to the values for wide flange columns and have been chosen here to conservatively approximate the critical sections for bending.

For lightly loaded plates, the procedure in the LRFD Manual (AISC 1986) can conservatively be applied to both shapes. The inside area in the enclosed area for tubes and pipes is stiffer than that between the flanges of a wide flange column. The dimension c , used in determining the



(a) Pipe Columns



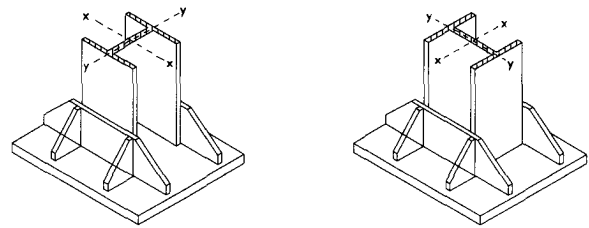
(b) Tubular Columns

Fig.7. Bearing Arm for Lightly Loaded Pipe & Tubular Columns.

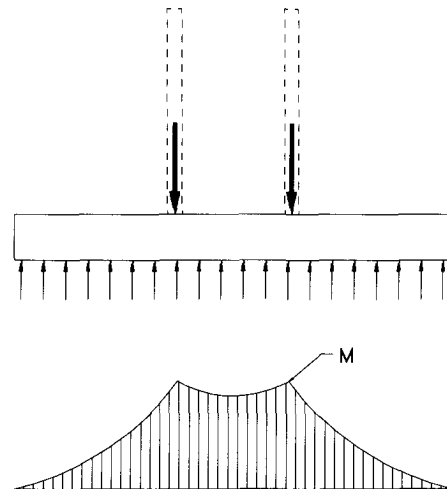
plate thickness, then is based on the bearing area A_H . The distance from the center of the tube wall to the edge of this bearing area should be equal at all points. This is shown in Fig. 7.

Base Plates with Large Loads

For column bases subject to heavier loads, plate thicknesses can become excessive. An alternative is to attach brackets to the column as shown in Fig. 8 (a) (Blodgett 1966). These brackets act with the plate to resist bending. The plate is then designed as a continuous beam perpendicular to the brackets, with supports at the centers of the two brackets, shown in Fig. 8 (b). The brackets are sized with the portion of the plate between the outer faces of the brackets to resist bending and shear. A full design example is given by Blodgett (1966).



(a) Details



(b) Moment Diagram for Thickness

Fig. 8. Column with Brackets

For extremely heavy loads, the force may be distributed by a grillage (AISC 1984), shown in Fig. 9. The grillage consists of one or more layers of closely spaced beams, usually S-shapes because of the thicker webs. The entire grillage is then encased in the concrete foundation. The plate then rests on the steel beams, with a resulting increase in the ultimate bearing pressure. Guidelines are not available for the allowable bearing stresses, though as a conservative estimate, the engineer can assume the entire load is transferred from the plate to the beams, neglecting the concrete. Design of the plate would be based on distributing the load over the beam webs, based on the AISC allowable bearing stress for steel. The load at the base of the grillage can then be assumed as a uniformly distributed load.

Details for Base Plates

Typical details for axially loaded base plates are shown in the AISC ASD Manual (1989a) and Engineering for Steel Construction (AISC 1984). For smaller loads, the plates are usually welded to the base of the column in the shop, while for larger loads, the plates are shipped to the field separately. The surface preparation is governed by Section M2.8 in the ASD Specification (1989).

Section M4.1 in the ASD Specification and M4.1 in the LRFD Specification specify that the plates should be set level at the correct elevation with full bearing on the foundation. The normal procedure is to maintain the top of the rough concrete footing 1-in. or so below the bottom

of the base plate. Grout is then worked under the plate. This allows for field adjustment.

Normally four anchor bolts and a minimum thickness (values of 0.50 to 0.75 in. have been suggested) have been used for concentrically loaded base plates to provide stability against column overturning during erection. The design of these anchor bolts should follow the strength provisions stated in the section on the design of anchor bolts and be evaluated by the erector for the estimated construction loads and conditions.

DeWolf and Sarisley (1978a, 1978b, 1982) have demonstrated that the ultimate load carrying capacity is reduced when the concrete pedestal has a depth greater than the plan dimensions. This is based on tests with unreinforced specimens; all tests used for the development of the allowable bearing stresses in the ACI Code (1983) and the AISC Specification (1989) involved unreinforced specimens. When the depth is large, the concrete is unconstrained for lateral movement in the vicinity of the apex of the pyramid which forms at failure. For cubes of concrete, the attachment to the base of the testing machine provides the necessary confinement. DeWolf (1982) has recommended that for depths greater than the plan dimensions, the pedestal should be reinforced as if it is a column. A minimum of four bars should be placed at the corners of the pedestal. Ties should begin just below the base plate, subject to the minimum cover requirements. This reinforcing should be used in all pedestals, regardless of height.

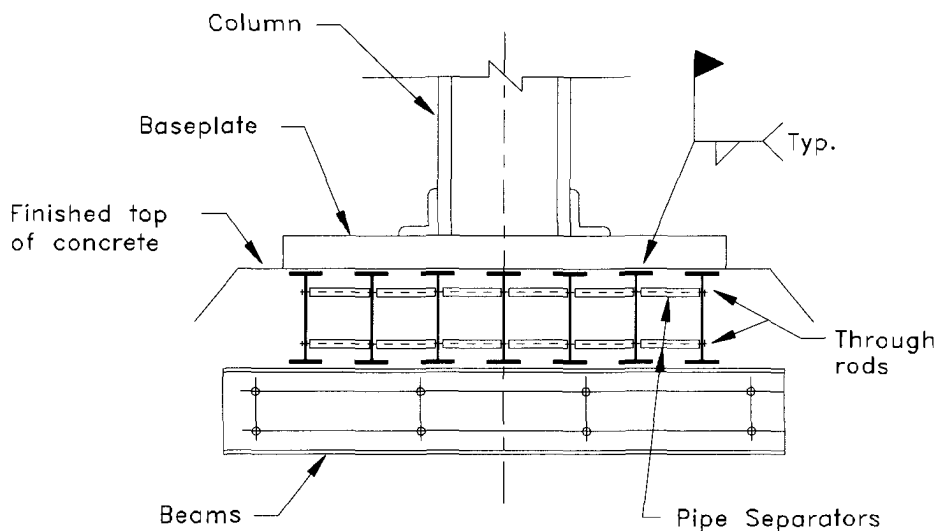


Fig. 9. Grillage Footing

DESIGN OF BASE PLATES WITH MOMENTS

General Behavior

As noted in the literature review, two general approaches exist for the design of base plates subject to an axial load plus a moment. One is based on elastic behavior and the other is based on the loads at failure. The first is generally covered in texts and design references which deal with base plates subject to moments. The second has been referred to in the texts as a means of determining the actual factor of safety against collapse.

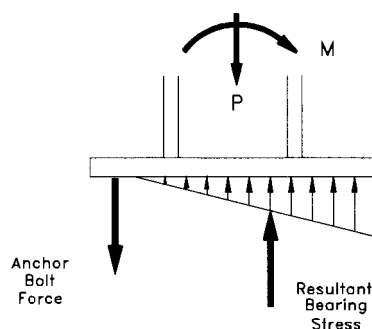
Only two sets of tests have been conducted for base plates subject to moments and axial loads, those by DeWolf and Sarisley (1978b, 1980) and those by Thambiratnam and Paramasivan (1986). DeWolf and Sarisley compared their experimental data to both approaches, and concluded that either could be used satisfactorily in design for their limited range of tests. Thambiratnam and Paramasivan compared their tests only to the elastic approach. Their plate thicknesses were substantially smaller than required by the elastic approach however, being equal to or less than two thirds the required thickness. Consequently failure generally occurred by plate bending, with relatively small factors of safety.

Both approaches are based on assuming a plate size and then making assumptions on the magnitude and distribution of the bearing stress in the concrete foundation and the stress or force in the anchor bolts. It is generally assumed that the concrete and the anchor bolt reach failure simultaneously, though in actuality this may not be true. At this time, there are insufficient guidelines to ascertain that these assumptions are correct, and consequently, it is necessary that the designer be cognizant of the general behavior.

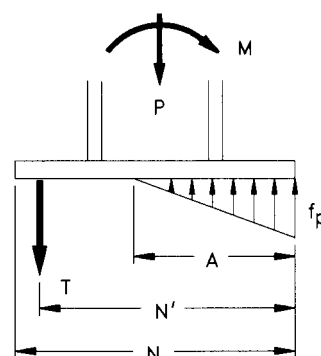
While the ultimate strength approach is more consistent with the trend to limit state design, more experimental evidence is needed before it can be applied to design. As an example, DeWolf and Sarisley (1978b, 1980) have demonstrated that the effect of confinement increases the bearing stress beyond that shown in texts, though to properly account for it, test data must be developed. They also demonstrated that if the plate is too thick, the actual failure may be by bearing at the compression edge of the plate, caused by the plate rotating on end. Thus in the following, elastic behavior is assumed, and design is based on ascertaining that the stresses from the design or working load do not exceed the values determined from the appropriate specification.

There are three different variations of the elastic method. One involves the assumption that the resultant compressive bearing stress distribution in the concrete foundation is directly under the column compression flange (Blodgett 1966, Salmon and Johnson 1980). This is shown in Fig. 10(a). The resulting bearing area is generally large, extending to the vicinity of the anchor bolt. If this occurs, it is unlikely that the anchor bolt is effective. This method is limited and not widely applicable.

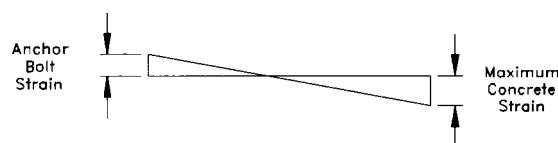
The second variation involves the assumption that at the junction between the plate and the concrete foundation



(a) Resultant Compressive Bearing Stress Under Column Flange



(b) General Case



(c) Strain Distribution

Fig. 10. Elastic Analysis for Axial Load Plus Moment

plane sections remain plane (Blodgett 1966, McGuire 1968). The stress distribution is shown in Fig. 10 (b), and the strain distribution is shown in Fig. 10 (c). The strain distribution is linear, and the result is that the strain in the anchor bolt is dependent on the strain distribution in the bearing area. This is not consistent with actual behavior, since there is no reason to assume that the plate and concrete remain in contact everywhere and that the vertical deformation of the plate is linear on the tension side.

The third variation is to neglect the assumption that plane sections remain plane (Ballio and Mazzolani 1983; DeWolf and Sarisley 1978a, 1978b; Gaylord and Gaylord 1972; Maitra 1978; 1978a; Thambiratnam and Paramasivam 1986). Thus the force in the bolt and the bearing in the concrete are independent. The assumed linear elastic stress distribution is shown in Fig. 10 (b). This approach is adopted here because it is more readily applied and more consistent with the actual behavior. DeWolf and Sarisley (1978b, 1980) were able to get good results when this approach was compared to tests.

The design is related to the equivalent eccentricity e , equal to the moment M divided by the axial force P . The moment and axial force are then replaced by an equivalent axial force at a distance e from the center of the column. For small eccentricities, the equivalent axial force is resisted by bearing only. For larger eccentricities, it is necessary to use an anchor bolt. It is necessary to assume plate dimensions to determine whether an anchor bolt is needed.

Design for Small and Moderate Eccentricities

If the equivalent eccentricity e is equal to or smaller than $N/6$, compressive bearing exists everywhere. This linear compression bearing stress distribution is shown in Fig. 11. The bearing stresses are calculated as if the plate represents the cross section of a beam. At the edges of the plate they are;

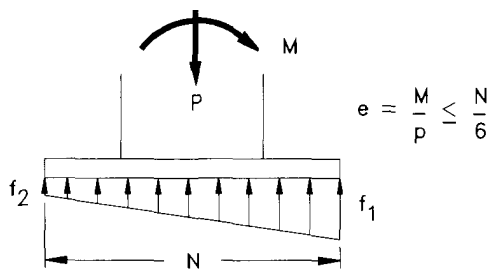
$$f_{1,2} = \frac{P}{BN} \pm \frac{Mc}{I}$$


Fig. 11. Small Eccentricity - Bearing on Full Plate

where

B and N are the plate dimensions

c is $N/2$ and I is the moment of inertia, $BN^3 / 12$

For ASD the maximum stress f_1 must not exceed the allowable bearing stress F_p determined from the AISC Specification (1989). At $e = N/6$, f_2 equals 0 for the limiting case of this model.

When LRFD is used, the design will be based on the assumptions of elastic behavior, with appropriate modifications to the load and stress. Thus the load P and moment M should be the factored values P_u and M_u , and the maximum stress f_1 should not exceed:

$$0.85\phi_c f'_c \sqrt{\frac{A_2}{A_1}} \leq 1.7\phi_c f'_c$$

If the equivalent eccentricity e is between $N/6$ and $N/2$ (moderate), bearing occurs only over a portion of the plate, shown in Fig. 12. For equilibrium, the resultant for the triangular bearing stress distribution must be equal to the axial load and located at a distance e from the plate center. Thus the maximum stress f_1 is then:

$$f_1 = \frac{2P}{AB}$$

where A is the distance over which bearing occurs, determined from $A = 3(N/2 - e)$. It is clear that the range of applicability of this moderate eccentricity model is $N/(6/2)$, since A equals N and 0 , respectively, for these limiting e values.

ASD Procedure:

1. Determine the maximum allowable bearing stress:

$$F_p = 0.35f'_c \sqrt{A_2/A_1} \leq 0.70f'_c$$

2. Pick a trial plate size, N by B .

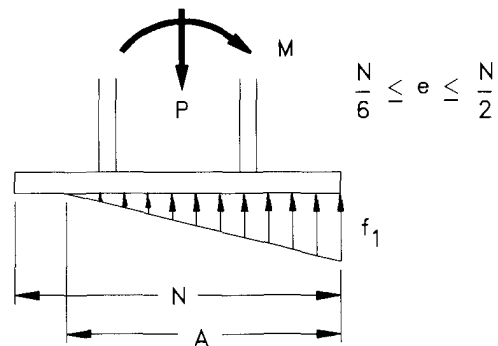


Fig. 12. Moderate Eccentricity - Bearing on Partial Plate

3. Determine the equivalent eccentricity, $e = M/P$, and the maximum bearing stress from the load. If this stress is acceptable when compared with the allowable, go to next step; otherwise return to step 2.
4. Determine the plate thickness, based on the elastic bearing stress distribution, using the critical section as determined for axially loaded plates and the following:

$$t_p = \sqrt{\frac{6M_{pl}}{F_B}}$$

where M_{pl} is the moment for a 1 in. wide strip and F_B is the allowable bending stress, equal to $0.75F_y$.

Example 13 (ASD Procedure): Design a base plate for an axial load of 140 kips and a moment of 280 in.-kips. Bending is about the strong axis for the wide flange column with depth d equal to 11.1 in. The ratio of the concrete to plate areas A_2/A_1 is unity. F_y is 36 ksi, and f'_c is 3 ksi.

1. $F_p = 0.35 \times 3\sqrt{1} = 1.05$ ksi.
2. Assume $N = 16$ in. and $B = 12$ in.
3. $e = 280/140 = 2$ in. This is smaller than $16/6 = 2.67$ in., so that bearing occurs across the full plate, as shown in Fig. 13.

$$f_{1,2} = \left[\frac{140}{(12 \times 16)} \right] \pm \left[\frac{(280 \times 8)}{\left(\frac{12 \times (16)^3}{12} \right)} \right] = 1.28, 0.18 \text{ ksi}$$

The allowable stress is exceeded, and a larger plate should be used.

2. Assume $N = 17$ in. and $B = 14$ in.
3. $17/6 = 2.83$ in. and bearing occurs across the full plate.

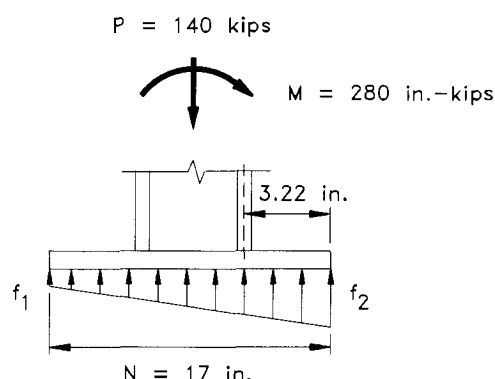


Fig. 13. Design Example with Small Eccentricity

$$f_{1,2} = \left[\frac{140}{(14 \times 17)} \right] \pm \left[\frac{280 \times 8.5}{\left(\frac{(14 \times (17)^3)}{12} \right)} \right] = 1.01, 0.17 \text{ ksi.}$$

The dimensions are satisfactory.

4. The critical section is at $(17 - 0.95 \times 11.1)/2 = 3.22$ in. from the edge. The bearing stress at this location is 0.85 ksi. The moment, M_{pl} , for a 1 in. strip, determined from the bearing stress distribution shown in Fig. 13 is:

$$M_{pl} = \left[\frac{0.85 \times (3.22)^2}{2} \right] + \left[\frac{0.16 \times (3.22)^2 \times 0.67}{2} \right] = 4.96 \text{ in.-kips/in.}$$

and then:

$$t_p = \sqrt{\frac{(6 \times 4.96)}{(0.75 \times 36)}} = 1.05 \text{ in.}$$

Use a 14 in. x 1 1/8 x 1 ft 5 in. plate.

LRFD Procedure:

1. Determine the factored load and moment.
2. Determine the maximum design bearing stress F_p :
$$F_p = 0.85\phi_c f'_c \sqrt{A_2/A_1} \leq 1.7 f'_c$$
3. Pick a trial plate size, N by B .
4. Determine the equivalent eccentricity, $e = M/P$, and the maximum bearing stress from the load. If this stress is acceptable when compared with the maximum design value, go to next step; otherwise return to step 3.
5. Determine the plate thickness, based on the elastic bearing stress distribution, using the critical section as determined for axially loaded plates and the following:

$$t_p = \sqrt{\frac{4M_{plu}}{0.90F_y}}$$

where M_{plu} is the moment for a 1-in. wide strip.

Example 14 (LRFD Procedure): Design a base plate for an axial dead and live load equal to 50 and 90 kips, respectively, and a moment from the dead and live loads equal to 100 and 180 in.-kips, respectively. Bending is about the strong axis for the wide flange column with depth of 11.1 in. The ratio of the concrete to plate areas is unity. F_y is 36 ksi, and f'_c is 3 ksi.

1. $P_u = 1.2(50) + 1.6(90) = 204$ kips

$$M_u = 1.2(100) + 1.6(180) = 408 \text{ in.-kips}$$

2. The maximum bearing stress is:
 $0.85 \times 0.60 \times 3\sqrt{1} = 1.53 \text{ ksi.}$
3. Assume $N = 16 \text{ in.}$ and $B = 12 \text{ in.}$
4. $e = 280/140 = 2 \text{ in.}$ This is smaller than $16/6 = 2.67 \text{ in.}$, so that bearing occurs across the full plate, as shown in Fig. 13.

$$f_{1,2} = \left[\frac{204}{(12 \times 16)} \right] \pm \left[\frac{(408 \times 8)}{\left(\frac{12 \times (16)^3}{12} \right)} \right] = 1.86, 0.27 \text{ ksi}$$

The allowable stress is exceeded.

3. Assume $N = 17 \text{ in.}$ and $B = 14 \text{ in.}$
4. $17/6 = 2.83 \text{ in.}$ and bearing occurs across the full plate.

$$f_{1,2} = \left[\frac{204}{(14 \times 17)} \right] \pm \left[\frac{(408 \times 8.5)}{\left(\frac{(14 \times (17)^3)}{12} \right)} \right] = 1.46, 0.25 \text{ ksi.}$$

The dimensions are satisfactory.

5. The critical section is at $(17 - 0.95 \times 11.1)/2 = 3.22 \text{ in.}$ from the edge. The factored moment, M_{plu} , for a 1 in. strip, determined from the bearing stress distribution shown in Fig. 13 with $f_1 = 1.46$, $f_2 = 0.25$ and the stress at the critical section equal to 1.24 ksi is:

$$M_{plu} = \left[\frac{(1.24 \times (3.22)^2)}{2} \right] + \left[\frac{(0.22 \times (3.22)^2 \times 0.67)}{2} \right]$$

$$= 7.19 \text{ in.-kips/in.}$$

and then:

$$t_p = \sqrt{\frac{(4 \times 7.19)}{(0.90 \times 36)}} = 0.94 \text{ in.}$$

Use a 14 in. x 15/16 x 1 ft 5 in. plate.

Design for Large Eccentricities

When the effective eccentricity is large, it is necessary to use one or more anchor bolts to resist the tensile component resulting from the moment. This is shown in Fig. 10(b).

For a plate size chosen so that the resulting bearing stress does not exceed the maximum value from the Specification, the unknowns are the magnitude of the anchor bolt force T and the length of bearing A . The maximum bearing stress is assumed equal to the allowable value.

Two equilibrium equations are then used to determine the unknowns. The sum of the forces yields:

$$T + P = \frac{f_p A B}{2}$$

and the sum of moments about the resultant bolt force yields:

$$PA' + M = \frac{f_p A B}{2} \left(N' - \frac{A}{3} \right)$$

where A' is the distance between the anchor bolt and the column center.

The second equation gives the bearing distance A :

$$A = \frac{f' \pm \sqrt{f'^2 - 4 \left(\frac{f_p B}{6} \right) (PA' + M)}}{\frac{f_p B}{3}}$$

where $f' = f_p B N' / 2$. The first equation then gives the resultant force T in the anchor bolt or bolts:

$$T = \frac{f_p A B}{2} - P$$

ASD Procedure:

1. Determine the allowable bearing stress:

$$F_p = 0.35 f'_c \sqrt{\frac{A_2}{A_1}} \leq 0.70 f'_c$$

2. Assume a plate size, $N \times B$.
3. Determine the length of bearing A , equal to the smallest positive value from the above equation. If this value is reasonable, go to the next step. If it is close to the value of N' , the solution is not practical since this implies that bearing extends to the vicinity of the anchor bolt. If this were so, the anchor bolt could not develop its full tensile capacity. It is then necessary to return to step 2 and pick another, larger plate.
4. Determine the resultant anchor bolt force T from the above equation. If it is reasonable go to the next step. Otherwise return to step 2. (The design of the anchor bolt is covered in the following section.)
5. Determine the plate thickness from the following:

$$t_p = \sqrt{\frac{6M_{pl}}{F_b}}$$

where F_b is the allowable bending stress, equal to $0.75F_y$.

Example 15 (ASD Procedure): Design a base plate for an axial load of 60 kips and a moment of 480 in.-kips. Bending is about the strong axis and the column depth is 8 in. The ratio of the concrete to plate area is four, F_y for the plate and the anchor bolts is 36 ksi, and f'_c is 3 ksi. (See Fig. 14)

1. $F_p = 0.35 \times 3\sqrt{4} = 2.1$ ksi.
2. Assume 14 x 14 in. plate. The effective eccentricity is $e = 480/60 = 8$ in., which is greater than half the plate width. Thus an anchor bolt is required. It is assumed at 1.5 in. from the plate edge.

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$$3. f' = \frac{2.1 \times 14 \times 12.5}{2} = 183.8 \text{ [kips]}$$

and then:

$$A = \frac{183.8 - \sqrt{(183.8)^2 - 4 \left(\frac{2.1 \times 14}{6} \right) [(60 \times 5.5) + 480]}}{\left[\frac{(2.1 \times 14)}{3} \right]}$$

=5.1 in.

This is reasonable when compared to N' which is 12.5 in.

$$4. T = \left[\frac{(2.1 \times 5.1 \times 14)}{2} \right] - 60 = 14.96 \text{ kips}$$

This is reasonable for the bar sizes available.

5. The critical section is at $[(14 - 0.95 \times 8)]/2 = 3.2$ in.

The moment, M_{pl} , for a 1 in. strip, determined from the bearing stress distribution in Fig. 14, is:

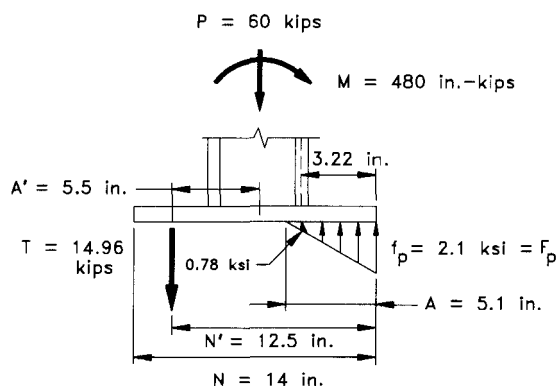


Fig. 14. Design Example with Large Eccentricity

$$M_{pl} = \left[\frac{(0.78 \times (3.2)^2)}{2} \right] + \left[\frac{(1.3 \times (3.2)^2 \times 0.67)}{2} \right]$$

$$= 8.44 \text{ in. - kips / in.}$$

The moment M_{pl} based on the critical section on the anchor bolt side is determined as follows. The full plate width is not always available. It is assumed that the critical plate width is based on the load spreading out at 45 degrees, shown in Fig. 15. This width is then equal to twice the distance from the bolt to the critical section for each bolt, provided that the critical section does not intersect with the edge of the plate. The moment M_{pl} , for a 1 in. strip, is then:

$$M_{pl} = \frac{[7.48](3.2 - 1.5)}{[2(3.2 - 1.5)]} = 3.74 \text{ in.-kips/in.}$$

The moment from the bearing stress distribution governs, and the required plate thickness is then:

$$t_p = \sqrt{\frac{(6 \times 8.44)}{(0.75 \times 36)}} = 1.37 \text{ in.}$$

Use a 14 x 1 3/8 in. x 1 ft 2 in. plate

LRFD Procedure:

1. Determine the factored load and factored moment.
2. Determine the allowable bearing stress:

$$F_p = 0.85\phi_c f'_c \sqrt{A_2/A_1} \leq 1.7\phi_c f'_c$$

3. Assume a plate size, $N \times B$.
4. Use the factored loads to determine the length of bearing A , equal to the smallest positive value from the equation for A . If this value is reasonable, go to the next step. If it is close to the value of N' , the solution is not practical since this implies that bearing extends to the vicinity of the anchor bolt. If this were so, the anchor bolt could not develop its full tensile capacity. It is then necessary to return to step 3 and pick another, larger plate.

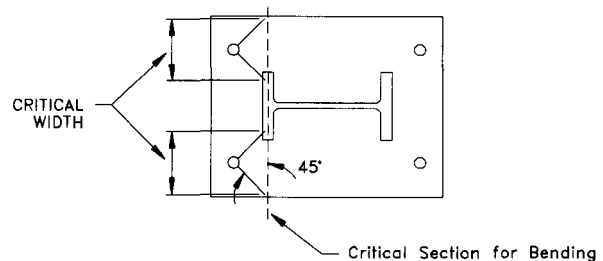


Fig. 15. Critical Plate Width for Anchor Bolt (Tension Side)

- Determine the resultant anchor bolt force T from the above equation. If it is reasonable go to the next step. Otherwise return to step 3. (The design of the anchor bolt is covered in the following section.)
- Determine the plate thickness from the following:

$$t_p = \sqrt{\frac{4M_{plu}}{0.9F_y}}$$

Example 16 (LRFD Procedure): Design a base plate for axial dead and live loads of 21 and 39 kips, respectively, and dead and live load moments of 171 and 309 in.-kips, respectively. Bending is about the strong axis and the column depth is 8 in. The ratio of the concrete to plate area A_2/A_1 is 4.0. F_y for the plate and the anchor bolts is 36 ksi, and f'_c is 3 ksi.

- $P_u = 1.2(21) + 1.6(39) = 88$ kips

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$$M_u = 1.2(171) + 1.6(309) = 700 \text{ in.-kips}$$

- $F_p = 0.85 \times 0.60 \times 3\sqrt{4} = 3.06$ ksi.
- Assume 14 x 14 in. plate. The effective eccentricity is $e = 700/88 = 7.95$ in., which is greater than half the plate width. Thus an anchor bolt is required. It is assumed at 1.5 in. from the plate edge.
- $f' = \frac{(3.06 \times 14 \times 12.5)}{2} = 267.8$ ksi

and then:

$$A = \frac{267.8 - \sqrt{267.8^2 - 4 \left(\frac{(3.06 \times 14)}{6} \right) [(88 \times 5.5) + 700]}}{\left[\frac{(3.06 \times 14)}{3} \right]}$$

$$= 5.1 \text{ in.}$$

This is reasonable when compared to N' which is 12.5 in.

- $T = \frac{(3.06 \times 5.1 \times 14)}{2} - 88 = 21.2$ kips

This is reasonable for the bar sizes available.

- The critical section is at $(14 - 0.95 \times 8)/2 = 3.2$ in. The moment, M_{plu} , for a 1 in. strip, determined from the bearing stress distribution in Fig. 14 with 3.06 for 2.1 ksi, 1.14 for .78 ksi and 21.2 kips for T is:

$$M_{plu} = \left[\frac{(1.14 \times (3.2)^2)}{2} \right] + \left[\frac{(1.92 \times (3.2)^2 \times 0.67)}{2} \right]$$

$$M_{plu} = 12.42 \text{ in.-kips/in.}$$

The moment M_{plu} based on the critical section on the anchor bolt side is determined as follows. The full plate width is not always available. It is assumed that

the critical plate width is based on the load spreading out at 45 degrees, shown in Fig. 15. This width is then equal to twice the distance from the bolt to the critical section for each bolt, provided that the critical section does not intersect with the edge of the plate. The moment M_{plu} , for a 1 in. strip, is then:

$$M_{plu} = \frac{10.6(3.2 - 1.5)}{2(3.2 - 1.5)} = 5.3 \text{ in.-kips/in.}$$

The moment from the bearing stress distribution governs, and the required plate thickness is then:

$$t_p = \sqrt{\frac{(4 \times 12.42)}{(0.90 \times 36)}} = 1.24 \text{ in.}$$

Use a 14 x 1 1/4 x 1 ft 2 in. plate

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Design Aid for Plates with Large Eccentricities:

Maitra (1978,1978a) has developed a graphical solution for the case with anchor bolts that eliminates the need to solve for A and T using the previous equations. It can be applied to either ASD or LRFD design. The procedure follows:

ASD Procedure:

- Determine the maximum allowable bearing stress:

$$F_p = 0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$$

- Assume a plate size, $N \times B$

- Calculate β : $\beta = \frac{(M + PA')}{F_p B N'^2}$

- From the graph in Fig. 16, determine the value of A/N' and then calculate A . If the value of A is reasonable, go to the next step. Otherwise, return to step 2 and try a new plate size.

- From the graph, determine the value of α . Then get the anchor bolt force from the following:

$$T = \left[\frac{(M + PA')}{\alpha N'} \right] - P$$

- Determine the thickness as before.

Example 17 (ASD Procedure): Use the Maitra's graphical solution to solve the previous ASD problem, with an $P = 60$ kips, $M = 480$ in.-kips, the column depth $d = 8$ in., $F_y = 36$ ksi, and $f'_c = 3$ ksi.

- $F_p = 2.1$ ksi.
- Assume a 14 x 14 in. plate.
- $M + PA' = 480 + (60 \times 5.5) = 810$ in.-kips

$$F_p B N'^2 = (2.1 \times 14 \times 12.5^2) = 4594 \text{ in.-kips}$$

$$\beta = \frac{810}{4594} = 0.176$$

4. $A/N' = 0.40$ and $A = 0.40 \times 12.5 = 5.0 \text{ in.}$

This is reasonable when compared to N' .

5. $\alpha = 0.87$

$$T = \left[\frac{810}{(0.87 \times 12.5)} \right] - 60 = 14.5 \text{ kips}$$

6. The thickness is then determined as before. As noted in the procedure, it is necessary to compare the value of A , the distance over which bearing occurs, to the distance to the anchor bolt. With the elastic approach which utilizes the plane section assumption, this is automatically done.

LRFD Procedure:

1. Determine the factored load and factored moment.
2. Determine the maximum bearing stress:

$$F_p = 0.87 \phi_c f'_c \sqrt{A_2/A_1} \leq 1.7 \phi_c f'_c$$

3. Assume a plate size, $N \times B$.
4. Calculate:

$$\beta = \left[\frac{(M_u + P_u A')}{F_p B N'^2} \right]$$

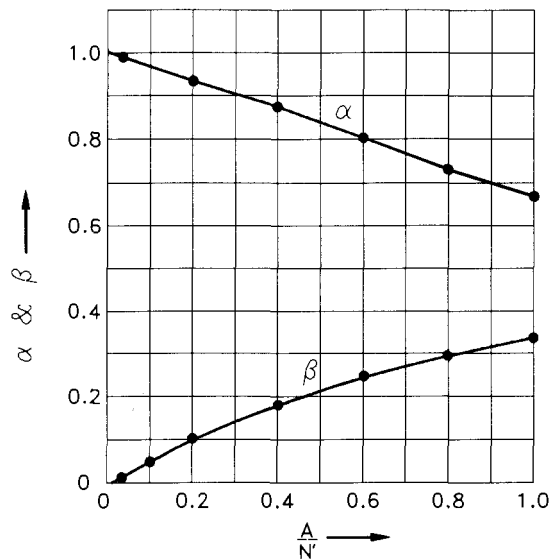


Fig. 16. Design Aid for Axial Load Plus Moment

5. From the graph in Fig. 16, determine the value of A/N' and then calculate A . If the value of A is reasonable, go to the next step. Otherwise, return to step 2 and try a new plate size.

6. From the graph, determine the value of α . Then get the anchor bolt force from the following:

$$T = \left[\frac{(M_u + P_u A')}{\alpha N'} \right] - P_u$$

7. Determine the thickness as before.

Example 18 (LRFD Procedure): Use the Maitra's graphical solution to solve the previous LRFD problem, with dead and live loads equal to 21 and 39 kips, respectively, dead and live load moments equal to 171 and 309 in.-kips, respectively, the column depth $d = 8 \text{ in.}$, $F_y = 36 \text{ ksi}$, and $f'_c = 3 \text{ ksi}$.

1. $P_u = 88 \text{ kips}$ and $M_u = 700 \text{ in.-kips}$
2. $F_p = 3.06 \text{ ksi}$.
3. Assume a $14 \times 14 \text{ in.}$ plate.
4. $M_u + P_u A' = 700 + (88 \times 5.5) = 1184 \text{ in.-kips}$

$$F_p B N'^2 = (3.06 \times 14 \times 12.5^2) = 6694 \text{ in.-kips}$$

$$\beta = \frac{1184}{6694} = 0.177$$

5. $A/N' = 0.40$ and $A = 0.40 \times 12.5 = 5.0 \text{ in.}$

This is reasonable when compared to N' .

6. $\alpha = 0.87$

$$T = \left[\frac{1184}{(0.87 \times 12.5)} \right] - 88 = 21.0 \text{ kips}$$

7. The thickness is then determined as before. As noted in the procedure, it is necessary to compare the value of A , the distance over which bearing occurs, to the distance to the anchor bolt. With the elastic approach which utilizes the plane section assumption, this is automatically done.

DESIGN OF ANCHOR BOLTS FOR TENSION

General Behavior

Anchor bolts are needed for all base plates. First they are used to safely anchor all plates to prevent column overturning during construction. They are also necessary when the plate is subject to large design moments or uplift.

There are two general types of anchor bolts, cast-in-place and drilled-in bolts. The drilled-in bolts are placed after the concrete sets. They are not normally used for base plates and their design is governed by the manufacturer's specifications, with additional information in the guide by Cannon, Godfrey and Moreadith (1981).

Different types of cast-in-place bolts are shown in Fig. 17. These are generally made from either bolts or bar stock, referred to as a rod. The commonly used hooked rod is made from a round shape and shown in Fig. 17 (a). The tensile load is resisted through bond developed along the length and by the hook. Smooth rods do not always form reliable bond however, due to oil, etc. Those with hooks may fail by straightening and pulling out of the concrete. A more positive anchorage is often preferred (Fisher 1981, Marsh and Burdette 1985a). Unless precautions are taken with hooked rods to assure proper anchorage, the hooked rod should be used only for axial-

ly loaded columns, where the development of any fixity at the base is not required, except during erection.

A more positive anchorage is formed when bolts or rods with threads and a nut are used, as shown in Fig. 17 (b) and (c). Marsh and Burdette have noted that the bolt head or a simple nut is all that is necessary. The anchorage is then developed by bearing on the head or nut. It is only necessary to provide for adequate embedment depth and edge distance. The failure mechanism is the pull-out of a cone of concrete radiating outward from the head of the bolt or nut. The use of a washer or plate only spreads out the cone and does not add significantly to the anchorage potential. In fact, the edge distance could be decreased by these, leading to earlier failure. Since headed bolts are not often available in lengths and diameters required for base plates, the designer should generally specify the rod with threaded ends and the provision of a nut for anchorage, as shown in Fig. 17 (c). The lower nut should be welded to the rod so that the rod does not turn out when the top nut is tightened.

Minimum Bolt Lengths and Edge Distances

Shipp and Haninger (1983) have presented minimum guidelines for bolt embedment and edge distance, adopted from ACI 349. These are presented for use in designing anchor bolts for tension as follows:

Bolt Type, Material	Minimum Embedded Length	Minimum Embedded Edge Distance
A307, A36	$12 d$	$5 d > 4 \text{ in.}$
A325, A449	$17 d$	$7 d > 4 \text{ in.}$

where d is the nominal diameter of the bolt or rod. The use of the above embedment lengths are conservative. The minimum edge distance is necessary to prevent blow-out. The failure associated with this involves the development of a conical failure surface between the anchor and the edge of the concrete, similar to the pull-out of a cone of concrete due to the direct tension. The minimum edge distance is an important consideration in determining pedestal sizes.

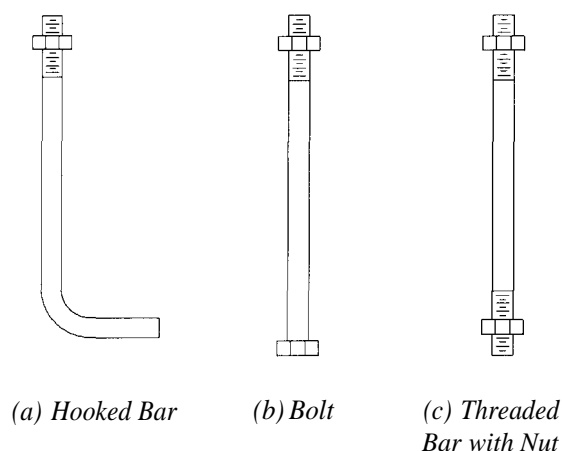


Fig. 17. Anchor Bolts

Design of Hooked Bolts

The design of hooked anchors should be based on the anchorage provided by the hook only. Fisher (1981) recommends that the hook be designed for bearing, with the tensile capacity T_h given by the following:

$$T_h = 0.7 f'_c d L_h$$

where d is the bolt diameter and L_h is the hook length. This is based on the ultimate capacity, and he recommends that a load factor of 1.7 be applied to this. For ASD, a factor of safety equal to 1.7 is then used, and thus the right side of the equation should be divided by 1.7. For LRFD, the equation for T_h should be used as it is. It is recommended that hooked anchors should only be used for axially loaded base plates where there is no specified design force. Since failure can occur by straightening and pulling out, it is recommended that the hook be designed to develop a minimum force equal to half the tensile capacity of the bolt. The erection procedure might require the need for an anchorage force greater than half the tensile capacity. Some bond is developed along the vertical part of the bolt, and this with the hook should prevent pull out of the bolt when the nuts are tightened. The hooked portion should be pointed diagonally inward toward the center of the foundation. It is necessary that the engineer use judgment to pick a suitable bolt diameter. The design of the hook should be based on the following procedure.

ASD Procedure:

1. Determine the allowable bolt tensile load T :

$$T = A_g F_t$$

where A_g is the gross area of the bolt and F_t is the allowable tensile stress, equal to $0.33 F_u$

2. The required hook length to develop half of T is then:

$$L_h = \frac{\left(\frac{T}{2}\right)}{\left(\frac{0.70 f'_c d}{1.7}\right)}$$

where d is the diameter of the bolt. This is based on developing half the bolt tensile capacity.

3. The total length of the bolt should be equal to the hook length plus the length taken from the previous table.

Example 19 (ASD Procedure): Determine the hook length for a $\frac{3}{4}$ in. diameter anchor bolt made from A36 stock with a minimum tensile strength $F_u = 58$ ksi, f'_c is 3 ksi.

$$1. T = \left(\frac{3.14 \times (0.75)^2}{4}\right) \times 0.33 \times 58 = 8.46 \text{ kips}$$

$$2. L_h = \frac{\left(\frac{8.46}{2}\right)}{\left(\frac{0.7 \times 3 \times 0.75}{1.7}\right)} = 4.6 \text{ in.}$$

3. The total bolt length should be 4.6 plus the minimum embedment length, equal to $12 \times 0.75 = 8$ in. 13 in. is thus acceptable. The bolt should not be closer than $5 \times 0.75 = 3.75$, but a minimum of 4 in., from the edge of the concrete.

LRFD Procedure:

1. Determine the bolt tensile capacity T_u :

$$T_u = 0.75 \phi_t F_u A_g$$

where ϕ_t is the tensile resistance factor, equal to 0.75, F_u is the specified minimum tensile strength, and A_g is the gross area of the bolt.

2. The required hook length to develop half T_u is then:

$$L_h = \frac{\left(\frac{T_u}{2}\right)}{0.7 f'_c d}$$

3. The total length of the bar should be equal to the hook length plus the length taken from the previous table.

Example 20 (LRFD Procedure): Determine the hook length for a $\frac{3}{4}$ in. diameter anchor bolt made from A36 bar stock with $F_u = 58$ ksi, $f'_c = 3$ ksi

$$1. T_u = 0.75 \times 0.75 \times 58 \left[\frac{(3.14 \times (0.75)^2)}{4}\right] = 14.4 \text{ kips}$$

$$2. L_h = \frac{\left(\frac{14.4}{2}\right)}{(0.70 \times 3 \times 0.75)} = 4.6 \text{ in}$$

3. The total bolt length should be 4.6 plus the minimum embedment length, equal to $12 \times 0.75 = 8$ in., 13 in. is thus acceptable. The bar should not be closer than $5 \times 0.75 = 3.75$, but a minimum of 4 in., from the edge of the concrete.

Design of Bolts and Rods with a Nut

Anchor bolts which are needed to develop a tensile capacity to resist design moments or uplift should normally be either bolts or threaded rods with a nut provided for the anchorage. The design procedure, presented by Marsh and Burdette (1985a), should be used. Failure

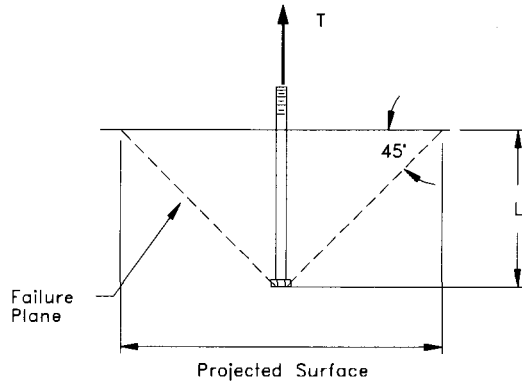
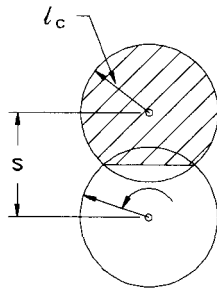


Fig. 18. Failure Cone for Anchor

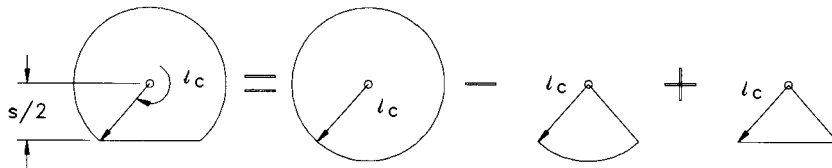
occurs when either the bolt fails or when a cone of concrete surrounding the bolt separates from the foundation, shown in Fig. 18. The cone radiates at an assumed angle of 45 degrees and tensile failure occurs along the surface of the cone, at an average stress of $4\sqrt{f'_c}$, where f'_c is in psi. Some references assume the capacity is equal to this average stress times the surface area of the cone, while others assume that the failure stress should be perpendicular to the surface of the cone, so that the capacity is equal to the component of the stress in the direction of the load multiplied by the surface area (Klinger, Mendonca and Malik 1982). Fisher (1981) presents an example, based on the PCI Handbook, which follows the first. Marsh and Burdette (1985a) recommend the second, which is conservative. The second is adopted here. This approach can be simplified by using the projected area, the circular surface area determined by the failure plane in Fig. 18, and applying the full average stress of $4\sqrt{f'_c}$ to this area. This is equivalent to using the component of the stress and the surface area of the cone.

The use of sleeves for bolts and threaded rods to allow for adjusting the embedded bolt with respect to the hole



$$\text{Shaded Area} = \pi l_c^2 - \frac{2 \cos^{-1} \left\{ \frac{s}{2l_c} \right\} \pi l_c^2}{360^\circ} + \frac{s}{2} \sqrt{l_c^2 - \frac{s^2}{4}}$$

(a). Overlapping Cones.



$$\text{Area} = \underbrace{\pi l_c^2}_{\text{Circle}} - \underbrace{\frac{2 \cos^{-1} \left\{ \frac{s}{2l_c} \right\} \pi l_c^2}{360^\circ}}_{\text{Sector}} + \underbrace{\frac{s}{2} \sqrt{l_c^2 - \frac{s^2}{4}}}_{\text{Triangle}}$$

(b). Cone at Edge of Pedestal.

Fig. 19. Calculation of Equivalent Areas.

in the plate should not reduce the anchorage capacity based on the failure cone, provided that the sleeve does not extend to the vicinity of the bolt head or nut.

For multiple anchorages, the separate failure cones may overlap. The effective area of the group should then be used. Figure 19, taken from the paper by Marsh and Burdette (1985), shows how to calculate the effective area for two bolts with overlapping cones and how to calculate the area when a cone intersects a pedestal edge.

It is also necessary to keep the anchor at sufficient distance from any edge to prevent a blow-out failure, where a cone of concrete splits out horizontally. The values given in the previous table should be used.

ASD Procedure:

1. Determine the gross bolt size A_g based on the allowable tensile stress, equal to $0.33 \times F_u$:

$$A_g = \frac{T}{0.33F_u}$$

where T is the required bolt tensile force.

2. Determine the required projected surface area:

$$A_{psf} = \frac{T}{2\sqrt{f'_c}}$$

This is based on an assumed factor of safety equal to 2.0, with f'_c in psi, T in pounds and A_{psf} in in.

3. Determine the required bolt length and concrete edge distance from this projected surface area. As a simplification for a single anchor not near a pedestal edge, if the area of the nut is discounted, the length is equal to the radius of the projected surface area:

$$L = \sqrt{\frac{A_{psf}}{3.14}}$$

If the cone intersects the side of the pedestal, the projected area should be reduced accordingly. Modification is also needed when more than one bolt is used. Additionally, the bolt length and edge distance should be no smaller than the values in the previous table. When a single bolt is used and when the cone does not intersect with the projected surface area, the minimum length from the table will govern. The requirement for the edge distance should be considered when the pedestal dimensions are set; it usually precludes the use of pedestals equal in size to the plate.

Example 21 (ASD Procedure): Design a single anchor bolt to resist a tensile force of 15 kips. It is to be made from a round A36 bar with F_u equal to 58 ksi. f'_c is 3 ksi. (Note that the minimum length from the previous table

will govern; the example is presented to demonstrate the general approach for multiple anchors or anchors near the edge of the concrete pedestal.)

$$1. A_g = \frac{15}{(0.33 \times 58)} = 0.78 \text{ in.}^2$$

A 1 in. diameter bolt will be used

$$(A_g = \pi/4 = 0.785 \text{ in.}^2)$$

$$2. A_{psf} = \frac{(15 \times 1000)}{2\sqrt{3000}} = 136.9 \text{ in.}^2$$

3. For a single bolt, with the full cone, the required length is:

$$L = \sqrt{\frac{136.9}{3.14}} = 6.6 \text{ in.}$$

The minimum length, taken from the table is $12 \times 1.0 = 12 \text{ in.}$, and this governs as expected. The minimum edge distance is $5 \times 1.0 = 5.0 \text{ in.} > 4 \text{ in.}$ Therefore use 6.6 in. which is needed for A_{psf} .

LRFD Procedure:

1. Determine the gross bolt area A_g based on tensile fracture:

$$A_g = \frac{T_u}{0.75 \phi_t F_u}$$

where T_u is the required bolt tensile force, F_u is the minimum tensile strength and ϕ_t is the resistance factor for tension, equal to 0.75.

2. Determine the required surface area:

$$A_{psf} = \frac{T_u}{4 \phi_t \sqrt{f'_c}}$$

The resistance factor ϕ_t is assumed equal to 0.75, with f'_c in psi, T_u in pounds and A_{psf} in in². With this value, the resulting area will be approximately equal to that for the ASD procedure when the ratio of live to dead load is 2.0 - 3.0.

3. Determine the required bolt length and concrete edge distance from this surface area. As a simplification for a single anchor not near a pedestal edge, if the area of the nut is discounted, the length is equal to the radius of the circular projected area:

$$L = \sqrt{\frac{A_{psf}}{3.14}}$$

If the cone intersects the side of the pedestal, the effective area should be reduced accordingly. Modification is also needed when more than one bolt is used. Additionally, the bolt length and edge distance should be no smaller

than the values in the previous table. The requirement for the edge distance should be considered when the pedestal dimensions are set; it usually precludes the use of pedestals equal in size to the plate.

Example 22 (LRFD Procedure): Design an anchor bolt to resist a factored tensile force of 21.9 kips. It is to be made from a round A36 bar with $F_u = 58$ ksi, f'_c is 3 ksi. (Note that the minimum length from the previous table will govern; the example is presented to demonstrate the general approach for multiple anchors or anchors near the edge of the concrete pedestal.)

$$1. A_g = \frac{21.9}{(0.75 \times 0.75 \times 58)} = 0.67 \text{ in.}^2$$

A 1 in. diameter bolt will be used ($A_g = 0.785 \text{ in.}^2$)

$$2. A_{psf} = \frac{(21.9 \times 1000)}{(4 \times 0.75 \sqrt{3000})} = 133.3 \text{ in.}^2$$

3. For a single bolt, with the full cone, the required length is:

$$L = \sqrt{\frac{133.3}{3.14}} = 6.5 \text{ in.}$$

The minimum length, taken from the table is $12 \times 1.0 = 12 \text{ in.}$, and this governs as expected. The minimum edge distance is $5 \times 1.0 = 5.0 > 4 \text{ in.}$ Therefore use 6.5 in. which is needed for A_{psf} .

Fisher (1981) states that the anchor bolt force should be transferred to the reinforcing bars, and that these should be extended into the failure cone to do this. Since the design has been based on preventing pull-out of the cone of concrete without reinforcing, the added requirement for developing these reinforcing bars in the cone is conservative. Nevertheless, the reinforcing bars in pedestals supporting a base plate with a moment and anchor bolt in tension should be extended into the area of the failure cone.

DESIGN FOR SHEAR

General Behavior

Normally, the column base shear forces are adequately resisted by friction, due to the axial compressive load. Consequently, it is usually not necessary to provide additional base shear strength. There are cases however, i.e. rigid frames, when bracing connections exist or when uplift occurs, that the applied frictional resistance is exceeded and the shear force must be transferred to the foundation in another manner.

Four ways exist for resisting the shear forces: the development of friction; bolt shear/bearing; the use of shear lugs; and, the embedment of the column base into the foundation.

Friction is due to the axial compression load. The LRFD Specification (AISC 1986a) states that the coefficient of friction μ should be 0.90 for concrete placed against as-rolled steel with the contact plane a full plate thickness below the concrete surface; 0.70 for concrete or grout placed against as-rolled steel with the contact plane coincidental with the concrete surface; 0.55 for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface. These values are for limit state conditions. For ASD, these values of μ should be used with a factor of safety equal to 2.0.

Goldman (1983) notes that the axial compressive load should not be overestimated, and he thus recommends that 75% of the dead load be used in calculating the frictional resistance.

Shear can also be resisted with anchor bolts. This involves either the development of shear friction or bearing. Shear friction necessitates that a clamping force exist. This occurs when the anchor bolt nuts are tightened against the plate. Marsh and Burdette (1985a) note that this clamping force can also occur in base plate anchor bolts which are not adequately tightened. Shear will cause wedge failure in the concrete adjacent to the bolt, and this wedge will push up on the plate, developing the clamping force. However, specific guidelines are not available on what is necessary to actually develop this clamping force and maintain it with time.

The bolts can also resist shear through bearing between the bolt and the plate. Since the bolt holes are often oversized to allow for placement in the field, it is improbable that all of the anchor bolts are in bearing against the steel base plate. Fisher (1981) states that shear transfer through bearing on bolts should be limited to no more than 2 bolts unless the bolts are leaded in because of this. Others (Goldman

1983, Kharod 1980) recommend shear transfer through bolts in bearing only for small shear loads. Goldman (1983) says the design shear should be no greater than 2 kips per bolt. Bolt shear failure is primarily related to the edge distance, since failure occurs when a semi-cone of concrete breaks away horizontally.

As an alternative, bolts could be more reliably used for shear resistance if the nuts are welded to the base plate after installation. An alternative would be to provide special washers without oversize holes and then weld these washers to the base plate. This may be necessary because of poor welding which could occur in using hardened nuts. Then the shear capacity could be developed without the need for bearing between the base plate and the bolt shaft. The design capacity would then be a function only of the concrete edge distance, with provisions in the guide by Cannon, Godfrey and Moreadith(1981).

As noted, shear friction is dependent on the clamping effect, and this is in turn influenced by the bolt tension. This clamping effect may not always be present in actual practice. Also, it is not likely that bearing will occur in a majority of the anchor bolts. Leading in the bolts is also risky since in normal practice the grout may work its way up between the bolt and the plate, and its bearing capacity is much smaller than the bearing capacity for steel.

The designer nevertheless has the option of specifying that bolts be used to resist shear. Provision must then be made to develop shear transfer, either through clamping, bearing or welding.

The third way of resisting shear is to use shear lugs. An example is shown in Fig. 20. The shear lug is a plate welded perpendicular to the bottom of the base plate. The shear force is then transferred through this plate acting

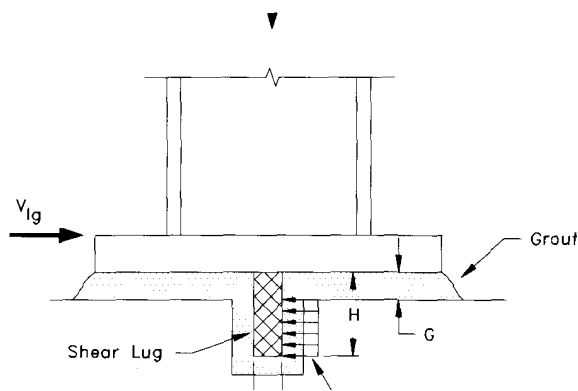


Fig. 20. Shear Lug

as a cantilever. Designs for shear lugs are presented by Fisher (1981), Goldman (1983) and Tronzo (1983-84). Failure occurs when a wedge of concrete shears off. The design approaches involve treating the failure as a bearing problem. Fisher refers to a PCI Handbook to calculate the allowable bearing stress in the concrete, based on limit state design concepts. Tronzo used the allowable bearing stress given in the AISC Specification for base plates. He assumed full confinement, provided by the concrete foundation and the base plate above, and thus used $F_p = 0.7 f'_c$. Goldman used an allowable value of

1 ksi to account for the grout. However, he does not account for the grout's full bearing potential. It should be as strong horizontally as it is vertically, where it provides for full development of the concrete foundation in bearing. A conservative choice would be to use the unconfined allowable bearing stress, equal to $0.35 f'_c$. All three assume bearing on the portion of the plate adjacent to the concrete foundation, i.e. they neglect the upper part in the grout under the plate. Bearing is assumed to be uniformly distributed through this height. The plate is then sized for bearing and bending as a cantilevered beam. Lug shear strength has been traditionally ignored, as in base plate design, since it will not govern.

The shear lug design approach will be detailed for the following example. The shear lug should be designed for the applied shear portion not resisted by friction between the base plate and the concrete foundation. Adequate grout must be placed around the shear lug. With stiffer grouts, it will be necessary to use grout holes and/or to extend the notch in the concrete support beyond the plate edge to allow for placement from the side.

The fourth way to resist shear is to embed the column in the foundation. The shear is transferred through the grout horizontally, and the required depth of embedment depends on the bearing capacity of the grout. Goldman used a value of 0.5 ksi, which is conservative.

Proposed Design Approach

The design approach favored by the author is based on the utilization of friction and shear lugs. The portion developed by friction is computed with the coefficient of friction from the LRFD Specification, with a factor of safety of 2.0 for the ASD procedure. For LRFD, the appropriate load factors should be used. The load used should be the dead load and any portion of the live load which generates the shear force. The shear lug design is based on the allowable bearing stress recommended by Tronzo, but conservatively neglecting confinement. Thus, $F_p = 0.35 f'_c$ for ASD and $0.85 \phi_c f'_c$ for LRFD,

with $\phi_c = 0.60$. The shear lug thickness should be no larger than the base plate thickness.

ASD Procedure:

1. Determine the portion of the shear which can be transferred by friction equal to half of the dead load plus that portion of the live load which generates the shear force. The portion to be resisted by the shear lug, is then the difference between the applied shear and this frictional resistance.
2. The required bearing area for the shear lug, or lugs, is:

$$A_{lg} = \frac{V_{lg}}{0.35 f'_c}$$

3. Determine the shear lug dimensions assuming that bearing occurs on the portion of the lug below the concrete foundation ($H-G$).
4. The cantilever end moment M_{lg} acting on a unit length of the shear lug is:

$$M_{lg} = \left(\frac{V_{lg}}{W} \right) \left[\frac{(H+G)}{2} \right]$$

where W is the total horizontal width of the lug or lugs, H is the vertical height of the lug and G is the grout thickness (see Fig. 20).

5. The shear lug thickness, based on bending and $F_b = 0.75 \times F_y$, is:

$$t_{lg} = \sqrt{\frac{6 M_{lg}}{0.75 F_y}}$$

Example 23 (ASD Procedure): Design a shear lug for a 14 in. square base plate, subject to an axial dead load of 120 kips, live load of 150 kips, and shear of 55 kips. The base plate and shear lug have $F_y = 36$ ksi, and $f'_c = 3$ ksi.

1. The contact plane between the grout and base plate is assumed above the concrete surface so that $\mu = 0.55$. Thus:

$$V_{lg} = 55 - (1/2)(0.55 \times 120) = 22.0 \text{ kips.}$$

$$2. A_{lg} = \frac{22}{(0.35 \times 3)} = 21.0 \text{ in.}^2$$

3. Assume the shear lug width W is 12 in. The height of the bearing portion is then:

$$H - G = \frac{21.0}{12} = 1.75 \text{ in.}$$

The required depth H is then 1.75 in. plus the grout depth, assumed equal to 1 in. This is then rounded to 3.0 in.

$$4. M_{lg} = \left(\frac{22.0}{12} \right) \times \left(\frac{3.0 + 1}{2} \right) = 3.67 \text{ in.-kips/in.}$$

$$5. t_{lg} = \sqrt{\frac{(6 \times 3.67)}{(0.75 \times 36)}} = 0.90 \text{ in.}$$

Use a shear lug, 12 in. long x 3 in. high x 1 in thick.

LRFD Procedure:

1. Determine the portion of the shear which can be transferred by friction equal to μ multiplied by the factored dead load V_{lgu} , plus the appropriate portion of the live load which generates the shear force. The portion to be resisted by the shear lug, is then the difference between the factored shear load and this force.
2. The required bearing area for the shear lug, or lugs, is:

$$A_{lgu} = \frac{V_{lgu}}{0.85\phi_c f'_c}$$

3. Determine the shear lug dimensions assuming that bearing occurs on the portion of the lug below the concrete foundation.
4. The factored cantilever end moment M_{lgu} acting on a unit length of the shear lug is:

$$M_{lgu} = \left(\frac{V_{lgu}}{W} \right) \left[\frac{(H + G)}{2} \right]$$

where W is the total horizontal width of the lug or lugs, H is the vertical height of the lug and G is the grout thickness (see Fig. 20).

5. The shear lug thickness is then determined. It is based on the LRFD expression used for plates subject to moments:

$$t_{lg} = \sqrt{\frac{4M_{lgu}}{0.90F_y}}$$

Example 24 (LRFD Procedure): Design a shear lug for a 14 in. square base plate, subject to an axial dead load of 120 kips, live load of 150 kips, and shear of 55 kips resulting from wind loading. The base plate and shear lug have $F_y = 36$ ksi, and $f'_c = 3$ ksi.

1. The load factors are 0.9 for dead load and 1.3 for wind load. The contact plane between the grout and base plate is assumed above the concrete surface so that $\mu = 0.55$.

$$V_{lgu} = 1.3(55) - 0.55[0.90(120)] = 12.1 \text{ kips.}$$

$$2. A_{lg} = \frac{12.1}{(0.85 \times 0.60 \times 3)} = 7.91 \text{ in.}^2$$

3. Assume the shear lug width W is 8 in. The height of the bearing portion is then:

$$H - G = \frac{7.9}{8} = 0.99 \text{ in}$$

The required depth H is then 0.99 in. plus the grout depth, assumed equal to 1 in. This is then rounded to 2.0 in.

$$4. M_{lg} = \left(\frac{12.1}{8} \right) \left(\frac{2.0 + 1}{2} \right) = 2.30 \text{ in.-kips/in.}$$

$$5. t_{lg} = \sqrt{\frac{(4 \times 2.3)}{(0.9 \times 36)}} = 0.53 \text{ in.}$$

Use a shear lug, 8 in. long x 2 in. high 9/16 in. thick.

The LRFD procedure results in smaller shear lugs than the ASD procedure because of the difference in the relative magnitudes of the design loads and the treatment of bending. Also note that the load for the LRFD example was assumed due to wind while that for the ASD example was not.

APPENDIX A - RESEARCH REVIEW

This appendix contains the review of column base plate research studies. It is divided according to the different cases of interest, i.e. axially loaded columns, those with moments and those developing shear. The design of anchor bolts follows the review of base plates with moments. Within sections, the work is organized chronologically.

Axially Loaded Columns

Fully Loaded Base Plates - The earliest studies involved concrete loaded by a punch or through a plate in which the full plate was loaded, unlike base plate foundations in which the load is applied by the column cross section to only a portion of the plate. Plate bending is thus precluded, and the assumed uniform bearing stress distribution in the concrete foundation is not necessarily equivalent to that in the column base plate foundations.

Meyerhof (1953) was interested in the effect of confinement, which occurs when the area of the concrete foundation is larger than the base plate area. He found that the surrounding concrete confines the concrete directly below the plate and can result in greater load carrying capacity. His test variables were the ratio of the concrete to plate area, the concrete strength and the depth of the concrete foundation, measured perpendicular to the plate. The depth was equal to or smaller than the horizontal concrete dimensions. Failure occurred when an inverted cone (apex pointing downward) of concrete formed directly under the plate and pushed downwards, splitting the concrete block outwards from the line of loading. He attributed the failure along the cone to shear failure. When the concrete depth was small, a cone also formed at the base of the concrete block with its apex near the upper inverted cone under the plate. He found that the bearing capacity decreased as the ratio of the concrete depth to plate width decreased, for depths smaller than the other concrete support dimensions. The results were not sufficient to establish guidelines. He also conducted a limited number of tests with large circular reinforced concrete footings. Two test specimens had reinforcing, different from that for normal concrete columns and pedestals, and these had significantly larger bearing capacities than the unreinforced specimens.

Au and Baird (1960) tested concrete blocks loaded through plates with variable ratios of concrete to plate area, variable concrete strengths and concrete depths equal to or smaller than the horizontal dimensions. Failure was due to the formation of an inverted pyramid

under the plate, similar to that of Meyerhof. In the tests with smaller depths, the concrete split radially without the formation of a clear-cut pyramid. The retardation of the formation of the pyramid resulted in increased failure loads, which is opposite to Meyerhof's test results. The authors state that factors such as friction at the base of the concrete block, i.e. on the face opposite to that with the plate, caused erratic results.

Kriz and Rath (1963) tested both plain and reinforced concrete column connections. The loads were applied near the edges of the concrete support, and the results demonstrated that lateral reinforcement was needed to prevent early failure. This was one of the few tests conducted with reinforcement, though again the results were not sufficient to draw quantitative conclusions.

Chen and Drucker (1969) used a plasticity analysis to study concrete blocks with strip loading and circular and square punches. They obtained both upper and lower bounds for the blocks, where the ratio of concrete to plate area was greater than unity.

Hawkins (1967, 1968) tested concrete blocks with the load applied over the full plate area. He varied the relative sizes of the concrete block and the plate area, the concrete strength and the position of the plate on the block. In the concentrically loaded specimens, failure occurred when an inverted pyramid formed under the plate, as with previous investigations. For eccentrically loaded concrete blocks, the failure was similar, though at different loads and with different ratios of load at first cracking to failure load. He proposed an equation for the ultimate bearing strength. It is a function of the ratio of the effective concrete area to the plate area, where the effective concrete area is based on the portion of the concrete around the plate which is concentric. For edge loading the equation is conservative, and he gave an additional equation for modifying this predicted capacity.

Hyland and Chen (1970) tested cylindrical blocks to study the effects of block height, base friction and the placement of a hole directly under the plate. They found that the presence of the center hole did not have a significant effect on the bearing strength, while the effect of the block height had an effect. Nevertheless, there is insufficient data to directly translate these conclusions to the design of base plate foundations.

In a series of papers, Niyogi reports on tests of concrete foundations with fully loaded plates. In the first (Niyogi 1973), the variables included the ratio of concrete to plate

area, the relative concrete depth, and the location of the plate on the concrete foundation. Failure was similar to that in previous work. The bearing strength decreased with increasing height, where the depth was greater than the horizontal dimensions, particularly for lower ratios of concrete to plate areas. The bearing strength also decreased with increasing eccentricity of the plate with respect to the concrete block. In the second paper (1974), the variables included the type of bed at the base of the concrete block, the relative sizes of the plate and the concrete foundations and the type of mix. In nearly all tests, the ultimate bearing strength was greater than the concrete compressive strength. The ratio of bearing strength to compressive strength decreased with increasing compressive strength. In a study of the effects of reinforcing (1975), he found that reinforcing could increase the ultimate bearing strength to as much as three times that for unreinforced specimens. In another study (1978), he found that outward sloping concrete foundations placed directly under the plate increased the bearing strength over that for specimens with vertical edges. While Niyogi has given some expressions to cover most of the variables, they are limited and not directly applicable to column base plate design because of the different loading conditions.

The results from these studies of foundations with punches and fully loaded plates offer some qualitative information for the behavior of base plate foundations in which the plate is only partially loaded by the column, the case of interest here. Failure occurs when an inverted pyramid forms under the plate for all but the shallowest foundations. It is clear that confinement offered by the surrounding concrete can increase the bearing strength. Also reinforcement is beneficial. For depths smaller than the plate's horizontal dimensions, the bearing strength may be increased, while for depths greater than the horizontal dimensions, the bearing strength may be decreased. Increases in the concrete strength lead to increases in the bearing strength.

Centrally Loaded Plates - Fewer studies have been made of plates in which the load is applied over only a portion of the plate. The earliest was an experimental investigation by Shelson (1957) and Avetal (1958). He tested concrete blocks loaded through plates with different ratios of concrete to plate area and in which the portion of the plate subject to loading varied. Failure occurred when an inverted wedge below the plate split the block, as in previously discussed investigations. He proposed an equation for the maximum bearing strength which was a function of the ratio of concrete to plate area and the concrete strength.

Hawkins in a study related to his previous one (1976a, 1968a) conducted tests in which the load was applied only to the central area of the plate. The variables were the plate thickness, the plate yield stress and the ratio of concrete to plate area. For the thinner, more flexible plates, he found that the bearing capacity increased linearly for increases on the plate thickness, the concrete strength and the plate yield stress. As the plate thickness increases, there is a point at which a maximum is reached, and that for this, the behavior corresponds to that noted in Hawkins' previous study of fully loaded plates. He developed analytical methods for predicting the bearing strength, but these are too complicated for routine design.

In a discussion of Hawkins' work, Taylor (1969) reports on tests he conducted. He found that the plate's yield stress had only a negligible effect on the bearing capacity, and he questioned Hawkins' analytical assumptions. Taylor's tests however involved concrete specimens with depths of three times the dimensions parallel to the plate and with a central hole for a prestressing tendon, and they are thus not equivalent Hawkins' tests.

DeWolf (1978, 1982) and Narus (1976) conducted tests of cubes of concrete in which the variables were ratio of the concrete to plate area, the relative plate thickness and the size of the concrete cube. As with the previous work, failure occurred when an inverted pyramid of concrete caused the block to split apart. They developed an equation for the bearing capacity which was based on the ratio of the concrete to plate area, the concrete strength, and the plate's flexibility, related to the thickness and the overhang from the edge of the column. They gave limits, partly based on the work of Hawkins, for the range of applicability. When compared with other test results, they found that the plate yield stress was not a significant variable. The plate's flexibility is what is important, and for the usual range of variables, this is related to the modulus of elasticity rather than the yield stress.

DeWolf and Sarisley (1978a, 1978b) found that the previous work by DeWolf and Narus does not properly account for the depth of the concrete, i.e. the dimension of the concrete perpendicular to the plate. They noted that when the depth of the concrete is increased so that it is greater than the other dimensions, the bearing capacity may be decreased. This is related to the lack of confinement at the base of the concrete block. With greater depths, the concrete is free to expand horizontally, and this allows the concrete block to split apart at lower loads. As noted in previous work, when the depth of the concrete is less than the dimensions parallel to the plate, the bearing strength can actually increase. In this case, the concrete foundation confines the concrete from splitting

outwards. Except where noted, all previous tests were based on unreinforced concrete blocks. DeWolf and Sarisley note that reinforcing, i.e. longitudinal bars in the direction of the load along with ties, should supply the necessary confinement so that increasing the depth beyond that for the cube should not lead to decreased capacities.

It has been established from experimental studies that the onset of cracking does not mean that the maximum load has been reached. DeWolf (1988) and Kou (1983) conducted a three dimensional finite element analysis of concrete cubes loaded through base plates to study this post-cracking behavior. The development of the cracks matched that from the tests. They did not model the nonlinear material behavior however and thus were not able to get failure loads that matched those from the tests.

The studies of partially loaded plates have shown that the bearing capacity is related to the concrete strength, the ratio of the area of the concrete to that of the plate, the relative plate thickness, the relative concrete depth, the amount of reinforcement, and perhaps nominally to the plate's yield stress.

Related Studies - Most analytical work for concrete subject to bearing loads has treated prestressed tendon anchorages. The dissimilarities between these and column base plates involve the way in which the load is applied, usually with multiple tendons which result in negligible bending in the bearing plate, and the prestressing tendon holes. However, the general conclusions and analytical approaches apply to both base plates and prestressed tendon anchorages.

Guyon (1955) presents analytical stress distributions for the anchorage zones of prestressed tendon anchorages. The analysis is based on elastic behavior and does not consider cracking, which occurs in the tension areas.

Ban, Magurama and Ogaki (1957) conducted two and three dimensional tests to determine the anchorage zone stresses in post-tensioned concrete members and then compared the strain distributions with those from previously developed theoretical solutions. The load was applied by a bolt through a center hole. They found that the load at which cracking occurred and the failure load were related linearly to the plate thickness and the concrete strength. They also found that spiral reinforcement increased both of these loads.

Zielinski and Rowe (1960) conducted tests and compared the behavior to existing analytical theories. Their variables included the ratio of the concrete to plate area, the type of anchorage, the use of ducts for prestressing tendons, and the amount and type of reinforcement. They

found that the duct did not have a significant effect. The main factor was the ratio of concrete to plate area. The reinforcing substantially increased the bearing capacity. They found that the existing theories did not give satisfactory predictions of the stresses.

Douglas and Trahair (1960) studied circular cylinders with central holes. They found from their experimental work that the central hole for the prestressing tendons significantly reduced the failure load.

Iyengar and Yogananda (1966) used a three dimensional elasticity solution for a circular concrete member with a coaxial duct and compared their analytical results with tests conducted by others. In a discussion, Taylor (1967a) questioned the validity of applying elastic principles to these and stated that design should be based on empirical results rather than just analytical results which involve assumptions of elasticity, etc. He also referred to tests he conducted with square blocks with holes. In their closure, the authors stated that good agreement could not be expected between their circular blocks and Taylor's square ones.

Taylor in a study of anchorage bearing stresses (1967) referred again to the large discrepancies between theories and experimental evidence. He conducted tests of anchorage devices and plain concrete. He found that horizontal compression, applied perpendicular to the load, increased the load carrying capacity; this would be similar to the favorable influence of confinement. His failure was by wedging action, similar to that noted in studies of base plate foundations.

In three papers, Yettram and Robbins (1969, 1970, 1971) used a finite element analysis to study anchorage zone stresses. The plate was fully loaded, and their elastic analysis was limited to precracking stresses.

The studies of prestressed tendon anchorages demonstrate that analytical results do not generally compare with tests. This is due to the need to develop a post-cracking analysis and the complexity of the many variables. While failure was similar to that for base plates, differences were noted due to the shape of the block and the method for loading. A central prestressing hole can substantially reduce the failure load. Reinforcing generally increased the load carrying capacity.

Studies of Fixity for Axially Designed Base Plates -

Other researchers have reviewed the amount of fixity that exists at base plates which are designed as pinned connections. Voce (1958) questions the assumption of a uniform bearing stress distribution. If the plate remains flat, he notes that the end of the column would then be vertical, i.e. no rotation occurs as expected in a pinned

end. For this to happen, there must be a moment. He proposed an equation for determining a lateral load which gives an equivalent bending moment.

Stephenson and Tarpy (1981) also looked at this discrepancy between the assumed pinned base behavior and the actual development of moment capacity. They state the discrepancy is of little consequence under ordinary circumstances. It can be important however for evaluating existing structures for modification necessary to meet recently developed seismic design provisions. Based on studies of assumed pin ended plates with properly embedded anchor bolts, they found that a high degree of residual stiffness exists and that this can be used to avoid unwarranted and expensive modifications for seismic design. This study indicates that plates designed for axial loads only are safe, and this contradicts Voce's suggestion to design for some moment even when there are only axial loads.

Picard and Beaulieu (1985) also looked at the fixity that exists in axially designed base plates. Their base plates had anchor bolts which developed a significant tensile force. From the moment-rotation curves, they found that the axial compression force adds to the fixity at the base. This leads to decreases in the lateral displacement of frames which are subject to sway. This could be used in design to reduce member sizes.

Lightly Loaded Base Plates - In all the previous studies of base plates subject to bending, the load was applied to the base plate through rectangular plates. When the load is applied by a wide flange column, the areas between the flanges and adjacent to the webs is also subject to bending. While this is often not critical in the design, the exception is lightly loaded columns. In these the required plate dimensions, based on the concrete bearing capacity, are approximately equal to or smaller than the column's cross-sectional dimensions. The bending in the region between the flanges thus governs.

Fling (1970) proposed the use of yield line theory for these columns. He assumed that the plate bending is elastic and stated that the deflection between the plate and the concrete foundation should be limited to a prescribed value. He conceded that his method is conservative because of his assumptions. In a discussion, Gogate (1970) noted that Fling neglected the favorable influence that anchor bolts could have in these plates. In reply, Fling stated it would be difficult to account for the anchor bolts due to the variances in actual designs.

Stockwell (1975) in discussing base plates for lightly loaded columns notes that the flexibility of the plate and the subsequent redistribution of bearing is not consistent with analytical solutions which assume uniform bearing

stresses below the plate. He proposed that only the portions of the plate directly under the column elements should be assumed effective and that the area of these portions should be based on engineering judgment.

Murray (1983) conducted an analytical and experimental study. He used an elastic finite element analysis to model the base plate. Springs were used to connect the plate to a rigid foundation, and they were then disconnected when uplift occurred. Based on his analytical study and his test results, he proposed that Stockwell's approach should be used, and he defined the effective bearing portion of the plate. He also developed a design approach for plates subject to uplift, based on a yield line analysis.

Axial Load Plus Moment

Limited work has been done for base plates which are designed to resist both an axial load and a moment, such as would occur at fixed bases or where there is an eccentrically applied load on the column. What has been done has treated plates with anchor bolts which are properly embedded, to develop the tensile component from the moment.

Salmon, Schenker and Johnston (1957) estimated upper and lower bound loads for columns with moments. They did not conduct tests and stated that their work could serve as a first approximation until experimental data became available.

LaFraugh and Magura (1966) conducted tests for base plate connections for precast concrete structures. Their variables included the plate dimensions, the anchor bolt size and the load eccentricity. Their tests were not equivalent to base plates for steel columns due to the shape of the column and the lack of significant overhang from the column, which results in plate bending.

DeWolf and Sarisley (1978b, 1980, 1982) conducted tests of base plates with moments and compared the results to the present design methods. Their variables included the plate thickness, the anchor bolt size and the eccentricity for the equivalent axial load. They found that the behavior at failure was not always consistent with the assumption used in present design approaches.

Thambiratnam and Paramasivam (1986) conducted a study very similar to that of DeWolf and Sarisley. Their test variables included the plate thickness and the equivalent load eccentricity, and they determined the strains in the base plate.

Anchor Bolts for Tension

A number of studies have been concerned with anchor bolts and their design. Anchorage for tension is dependent on the amount of bond developed along the length and what is done at the end of the bolt embedded in the concrete. The normal approaches are to hook the bolt or to use a bolt head or nut, with or without a plate or washer. With proper design, the anchor bolt can then develop its full tensile capacity.

Breen (1966) tested anchor bolts embedded into drilled shafts. He subjected these to tensile loading and varied the bolt size. He used a standard nut or standard nut plus a washer at the bottom of the bolt. The amount of bond developed for the smooth bar was minor, and anchorage was due to the nut, with or without the washer.

Conrad (1969) studied different types of grouted anchor bolts subject to tension and shear. These were grouted into holes drilled in the concrete and the variables were the type of grout and the bolt size. He noted that only nonshrink grout is suitable for developing tensile loads.

Lee and Breen (1970) conducted a model study based on Breen's work. They found that reduced scale models can be used in anchorage studies when they are used in combination with some full-scale tests.

Cannon, with Burdette and Funk (1975) looked at the anchorage requirements for concrete inserts, anchor bolts, welded studs and expansion anchors for both tension and shear loads. They found that the anchorage is a function of the concrete's tensile strength, the size, strength and number of anchors, and the proximity of the edges. They state that the use of bearing plates in addition to the bolt head or nut at the interior end of the bolt is neither necessary nor helpful in developing the tensile capacity.

Hasselwander, Jirsa, Breen and Lo (1977) evaluated the effects of bolt diameter, embedment length, clear cover and bearing area on the behavior of high-strength anchor bolts. They found that the tensile capacity is a function of the bearing area of the washer placed at the interior end of the bolt, the concrete compressive strength, and the distance between the edge of the bolt and the edge of the concrete, provided that tensile failure of the bolt is prevented. This applies to single bolts. They found that there is a significant reduction in capacity for multiple bolts.

Adihardjo and Soltis (1979) studied the effect of the grout placed between the base plate and concrete foundation for bolts subject to tension and shear. They found that the grout did not reduce the tensile capacities.

Powell and Bryant (1983) were interested in the behavior of anchor bolts subject to earthquake loading. They found that structural grade mild steel bar is a suitable material and provides sufficient ductility and strength for these loads.

Shear Loads

Shear can be resisted by friction between the plate and the foundation, embedment of the column into the concrete foundation, anchor bolts and shear lugs, which are attached to the bottom of the plate. The research efforts in this area have been limited to determining the shear capacity of anchor bolts.

Conrad (1969) conducted tests of different grouted anchor bolts. He found that all were satisfactory for shear loads.

Cannon, Burdette and Funk (1975) found that the shear strength is a function of the bolt strength, the amount of pretightening and the position of the bolt with respect to the edge. Preloading embedded bolts increases the shear capacity.

Bailey and Burdette (1977) studied the effects of reinforcing on the shear capacity of anchor bolts located near the edge of the concrete and loaded with large shear forces directed toward the edge. They found in tests with small edge distances there are hazards when attempting to reinforce for shear. They note the need for more tests in this area.

Adihardjo and Soltis (1979) found in their study of grouted plates that the shear capacity for anchor bolts is reduced over that for plates without grout. They do not give a design approach to account for this.

Klinger, Mendonca and Malik (1982) studied anchor bolts subject to monotonic and reversed cyclic shear loads. The bolts were embedded sufficiently to develop the tensile strength. They give expressions for calculating the minimum edge distance to develop the full shear strength of the bolt, and they review design information for using reinforcing, which is needed when the bolts are close to the edge.

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NOMENCLATURE

A	Length of base plate bearing region in concrete	M	Moment, ASD
A	Distance between anchor bolt and column center	M_u	Factored moment, LRFD
A_1	Area of base plate	M_{pl}	Moment at critical section, per inch width, ASD
A_2	Area of concrete support	M_{plu}	Factored moment at critical section, per inch width, LRFD
A_H	Area of H-shaped region for lightly loaded plates	m, n	Cantilever dimensions for determining moment in base plate
A_g	Gross cross-sectional area of anchor bolt	N	Base plate depth
A_{lg}	Bearing area of shear lug	N'	Distance from anchor bolt to far edge of plate
A_{psf}	Projected surface area for anchor bolt failure	n'	Equivalent cantilever dimension for lightly loaded plate in AISC Manual, ASD
B	Base plate width	P	Service load, ASD
b_f	Column flange width	\bar{P}	$P_u / 1.46$
c	Effective cantilever dimension for lightly loaded base plate; distance to maximum bearing stress for base plate with a moment	P_o	Portion of load resisted by area enclosed by column
d	Anchor bolt nominal diameter; column depth	P_p	Limit state capacity, LRFD
DL, LL	Dead, live service loads	P_u	Factored load, LRFD
e	Equivalent eccentricity, equal to M/P	T	Force in anchor bolt or bolts
F_b	Allowable bending stress, ASD	T_h	Anchor bolt tensile capacity
F_t	Allowable anchor bolt tensile stress, ASD	t_f	Thickness of column flange
F_p	Allowable bearing stress, ASD	t_{lg}	Thickness of shear lug
F_u	Steel tensile strength	t_p	Plate thickness
F_y	Steel yield stress	V	Shear, or horizontal load, on base plate
f'_c	Concrete compressive strength	V_{lg}	Shear load resisted by shear lug, ASD
f_p	Applied concrete bearing stress, ASD	V_{lgu}	Shear load resisted by shear lug, LRFD
f'	Average bearing stress for plate with a moment	W	Width of shear lug
$f_{1,2}$	Bearing stresses at edge of plate with a moment	Z	Plastic section modulus
G	Grout thickness	α	Coefficient for distance of anchor bolts from centroid of bearing
g	Gage used in designing plate for uplift	β	Coefficient for loads and bearing
H	Vertical height of shear lug	Δ	Approximate difference between B and N
L	Anchor bolt embedment length or concrete edge distance	μ	LRFD base coefficient of friction
L_h	Anchor bolt hook length	ϕ_b	Resistance factor for bending, LRFD
M_{lg}	Cantilever end moment of shear lug, ASD	ϕ_c	Resistance factor for bearing on concrete, LRFD
M_{lgu}	Factored cantilever end moment of shear lug, LRFD	ϕ_t	Resistance factor for tension, LRFD

the leveling nut method is not recommended if the bolts must be closely spaced in one direction, as on a narrow wall. The bolts will not develop good restraint in that one direction. In this situation it would be better to use a leveling plate or to design the wall with a pilaster so as to utilize a more nearly square base plate.

When the leveling nut method is used, usually one of the lower nuts is brought to the proper elevation, allowing for the thickness of the heavy washer which must be placed on top of the nut and below the base plate. This bolt and nut is then spray painted to identify it as the nut with the proper elevation. The other nuts are brought to the same elevation. If someone bent on mischief attempts to change the elevation of the key nut, the broken paint will expose the misdeed and help to re-establish the proper elevation.

When it comes time to erect the column, it can be dropped into place very quickly and efficiently and the upper washers and nuts installed. One of the major advantages of the leveling nut method is that it can accommodate a base plate slightly out of level or a base plate curled by the heat of welding. Leveling nuts are best used for base plates ranging up to about 36 in. in size. Beyond this size, bending of the base plate may become a problem, and shipping the base plate separately should be considered.

PRESET BASE PLATES

Large-sized base plates (36 in. and larger) are often shipped to the job site and set in advance of the start of erection. This is done because these large plates are often so heavy and cumbersome that they make shipping and handling of the column very difficult if not impossible.

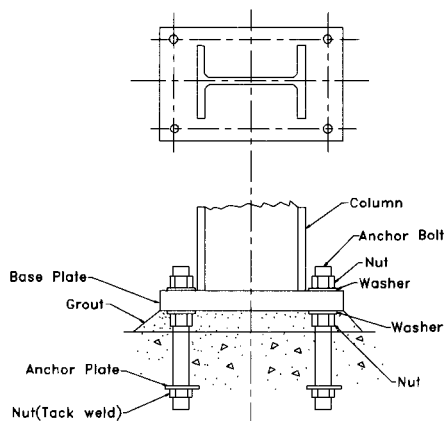


Fig. 2 Column base with leveling nuts

These large base plates are usually furnished with some kind of leveling devices in the form of bolts or threaded rods. Shims and wedges can safely be used in this situation because there is not an attached column shaft waving around in the sky. A three-point support (like a milking stool) is satisfactory. If leveling bolts are provided, small steel plates must be placed under the points of the bolts so they won't dig into the concrete.

When colossal-sized (say over four tons in weight) base plates are required, an angle frame is often supplied in advance. This angle frame is carefully leveled and filed with concrete which is screeded off accurately and results in a level concrete pad of proper elevation on which the column base plate is directly placed (see Fig. 4).

ANCHOR BOLTS

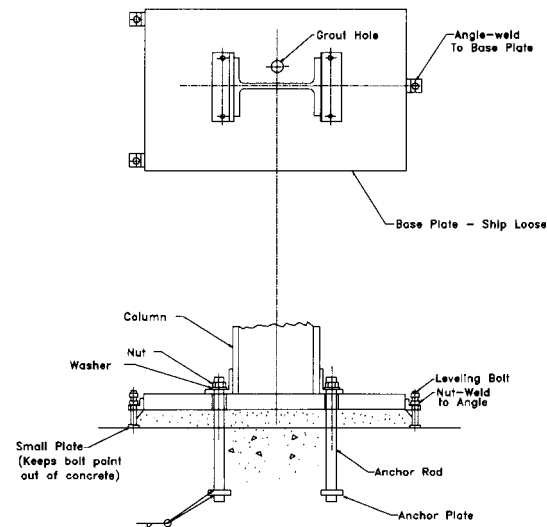


Fig. 3 Heavy column base

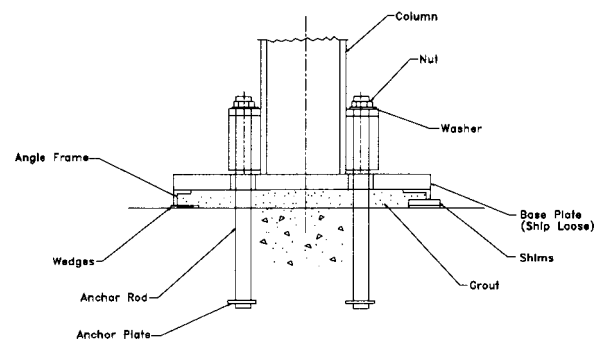


Fig. 4:

The selection of the column base type is determined primarily by the geometry of the foundation and the nature of the loads which influence the base. The geometry consists of the shape and location of the foundation—whether it is a square or rectangular footing, pile cap, a narrow wall, a pier or a pilaster, isolated or part of a wall, or at a corner of a wall. The loads may consist of vertical gravity loads, uplift, shear, moment, or combinations of any of these. Erection loads, for example, may be a combination of gravity load and moment (see Fig. 14).

Columns subject to gravity loading alone, theoretically, would not need any base anchorage. During the erecting of a column, however, there is a brief period of time, before the column is stabilized with beams or guys, when a column must stand on its own. For example, a 14 in. wide flange freestanding column 31 ft long, being scaled by an erector on a breezy day, will require a resisting base moment of approximately 5 ft kips. Some kind of anchorage is required to hold the base plate to the foundation—usually anchor bolts or rods. Anchor bolts and

anchor rod mean the same thing in this text, and the terms are used interchangeably as they are in the trade. Anchor bolts are primarily a tension device. To prevent the anchors from pulling out of the concrete—should the bond stresses be exceeded—hooks, plates, or other shapes are added to the embedded portion of the anchors (see Fig. 10).

Anchor bolts vary in size from approximately $\frac{3}{4}$ in. diameter to 2 $\frac{1}{2}$ in. diameter with $\frac{3}{4}$ in., $\frac{7}{8}$ in., 1 in., 1 $\frac{1}{4}$ in., and 1 $\frac{1}{2}$ in. being the most common diameters. Avoid specifying bolt diameters in sixteenths and eighths (except $\frac{7}{8}$ in. and 1 $\frac{1}{8}$ in.) as these sizes may not be readily available. Anchor bolts less than $\frac{3}{4}$ in. diameter may lose section due to corrosion and result in less than anticipated service life. Anchor bolts greater than 2 $\frac{1}{2}$ in. diameter may be difficult to find nuts for and wrenches to fit.

Anchor bolts, subject to corrosive conditions, may be galvanized. When ordering galvanized bolts, specify that the threads be "chased" so the nuts will work freely. If

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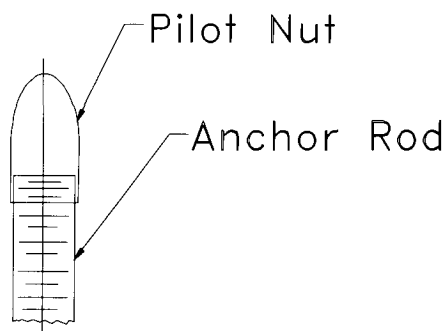


Figure 5

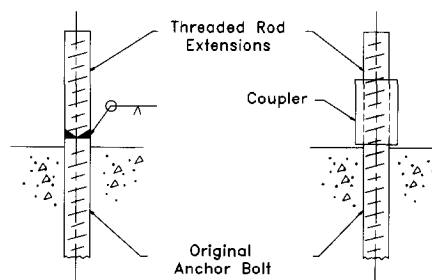


Figure 7

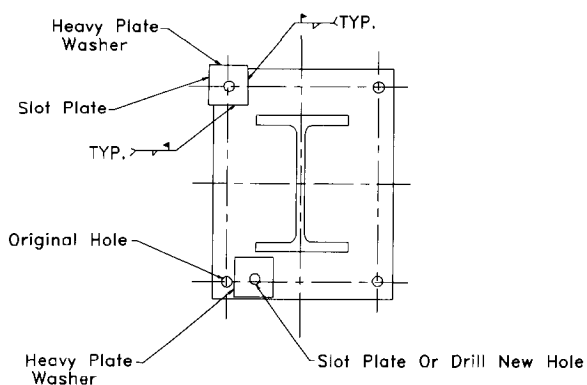


Figure 6

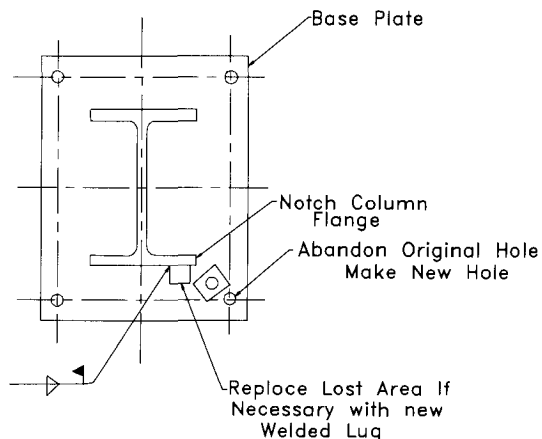


Figure 8

anchor bolts must be galvanized, it is best to specify A307 and A36 material to avoid the embrittlement sometimes resulting when high-strength steels are galvanized. Weathering steels may also be used where anchor bolts are exposed to corrosive atmospheres, but with the understanding that they will rust and stain the foundation concrete if so exposed.

Most anchor bolts are made from A36 material. Other materials used are A307, A325, A572, and A588. When higher strength is required, 4140 steel is sometimes used. If it is necessary to identify the exact type of steel used, certified mill test reports should be requested from the supplier. Normally this is not required unless the anchors are subject to significant tension. See Table 1C, page 4-4, AISC Manual of Steel Construction, 9th ed.¹ for other rod steels.

Most anchor bolts come with hex nuts. Occasionally a supplier may furnish square nuts. This is of little consequence. In tight situations where bolt projection is critical a half nut may be utilized. In rare cases, where the anchor bolt is not allowed to project above the top of the base plate, the plate can be counterbored so the nut will set into the depression. This is expensive and should not be used unless absolutely necessary. Of course, the plate must be thick enough to permit the required depth of counterboring. Generally, because most base plates will be provided with oversized holes, it is necessary to provide thick erection washers under the nuts. These are usually $\frac{5}{16}$ to $\frac{1}{2}$ in. thick and must be accounted for when the bolt projection is established. The use of lock washers on anchor bolts is seldom justified. Split-ring type lock washers have a relatively small outside diameter and have been known to suck down into the oversized holes resulting in an ineffective anchor bolt. It is better to spike the threads or use double nuts than to use lock washers.

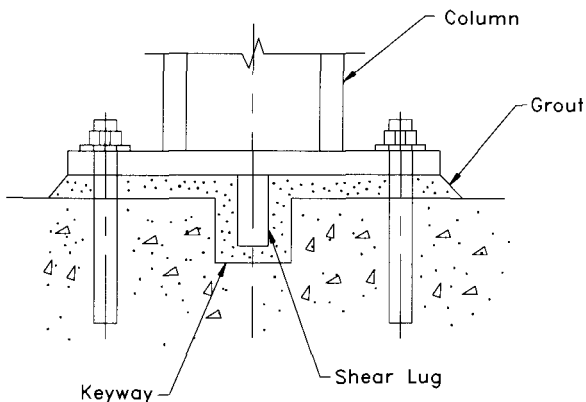


Figure 9

Anchor bolt projection is shown in one of two ways, depending on the custom of the fabricator-erector. The top end of the anchor bolt may be dimensioned either down to the underside of the base plate, or down to the top of the concrete, as in Fig. 11. During the concreting operation, it is very difficult to get the concrete to the exact elevation. This must be recognized by those who choose to dimension to the theoretical top of concrete. (Theoretical and as-built usually differ.) However, both methods are used extensively and both seem to work.

Provide plenty of extra threads on anchor bolts. It is very frustrating for an erector to have a nut "shank out" (run out of threads). In such an event the nut must be backed off and extra washers added. Stacked washers are not a problem on anchor bolts and special restrictions should not be imposed.

When anchorage is required in concrete which is already poured, a hole is core drilled in the concrete and a straight anchor bolt with a swaged shank may be grouted in place. The swaging consists of dents pounded randomly in the shank. A similar result can be obtained by depositing little blobs of weld metal on the shank (see Fig. 12). Concrete reinforcing rods with threaded ends have been successfully used for this application. In any event such bolts should be used with caution. If the dents or blobs are a bit sparse or the concrete is not properly cured, then the anchors may pull out of the concrete. Anchor bolts of any type should have proper concrete embedment.

Avoid anchor bolts with upset threads unless absolutely necessary. The source of these rods are scarce and delivery times are long. Avoid specifying unusual threading requirements such as NF threads or left-hand threads.

Anchor bolts, when used with the leveling nut method of setting column bases, should have a nut and heavy erec-

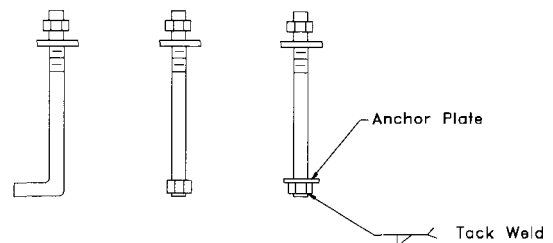


Figure 10

tion washer both above and below the base plate. The washers are used to prevent the nuts from attempting to push through the oversized holes. Extra thread should be supplied for lower nut adjustment (see Fig. 2).

Rods with full length threading and nuts on each end can be used for anchor bolts. The lower nut must be welded to the rod to prevent it unwinding when the top nut is tightened (see Fig. 10C).

Because of long-term relaxation of concrete, prestressing of anchor bolts is unreliable and hardly ever justified.

Anchor bolts are the link between foundation and column. When a column base is subject to uplift, the uplift force must be resisted by a weight greater than the uplift force or the column base must be connected to the basic rock. Anchor bolts transmit this tension force. Several characteristics in anchor bolt design can be varied to suit the load requirements: the quantity of bolts, the diameter, the length, and type of material.

Anchor bolts should not be used to resist shear forces in a column base. Shear forces can be resisted by some device attached to the underside of the base plate and inserted into a groove or keyway in the concrete foundation (see Fig. 9). The shear forces can also be resisted by the encasement of concrete around a column base. More commonly, the shear resistance is developed by the

friction developed between the bottom of the base plate and the top of the concrete foundation. (If the leveling plate method of setting column bases is used, the coefficient of friction used should be that of steel-to-steel.)

The main reason anchor bolts should not be used to resist shear forces is that they have a relatively low bending resistance. If a base plate were to ease sideways into bearing against an anchor bolt, the load (discounting the grout) would be delivered an inch or so above the concrete top. The anchor bolt would act like a vertical cantilevered member. A $\frac{3}{4}$ in. diameter A36 anchor bolt resists less than a 5 kip lateral load delivered 2 in. above the top of concrete. Also the random location of anchor bolts as they are located in the oversized base plate holes results in a situation where seldom does more than one bolt of a multi-bolt group go into bearing at any one time.

Large uplift forces are resisted primarily by the anchor bolts going into tension. Significant uplift forces must be resisted by a greater weight anchored to the lower end of the anchor bolts or by drilling direct into basic rock. For larger uplift forces, it may not be adequate to rely merely on the attachment of the base plate to the bottom of the column shaft or to hope that the base plate will not bend. For these conditions, it may be required to extend the anchor bolt up the side of the column flange to provide a shear connection to the face of the flange, thus bypassing the base plate altogether (see Fig. 4).

Pure moment forces are rare in column bases. They are usually accompanied by gravity loads and also shear. The moment couple consists of a downward force near one edge of the base plate and an opposite force upward at the anchor bolts on the far side of the plate. Naturally, the further apart these forces are, the greater the moment resistance. Modest moment can be developed by means of a base plate welded to the bottom of column. However, significant base moments may require an arrangement

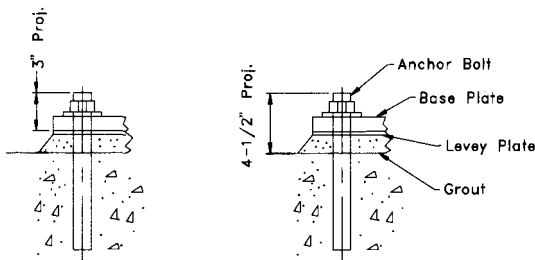


Figure 11

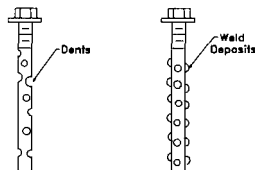


Figure 12

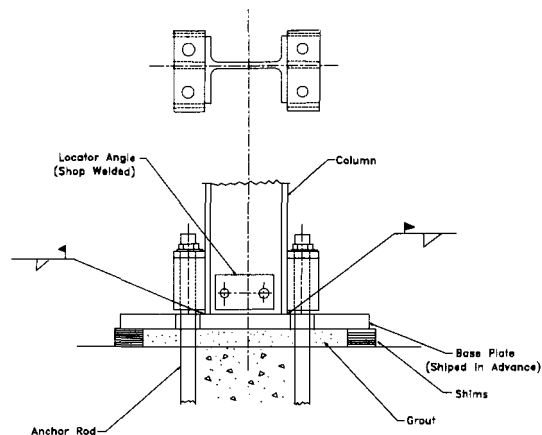


Figure 13

such as shown in Figs. 4 or 13. This relieves the base plate of bending and the base plate weld of over stressing and may allow a thinner base plate to be used.

Pilot nuts are sometimes used on top of the anchor rods to facilitate the entry of the rods into the base plate holes. This is an option of the erector (see Fig. 5). The base plate holes must be made large enough to accommodate the pilot nuts. After the column is set, the pilot nuts are removed and replaced by the desired washer and nut.

The design of the actual base plate itself is well covered elsewhere. (See John T. DeWolf, Ref. 3.) Base plates are square more often than not. However, some foundation geometry may dictate that the plate be rectangular, such as a plate on a narrow wall. Sometimes the shape of the base plate is dictated by where the anchor bolts are located and the number of bolts required. A pair of bolts may be located close into the column web, as in Fig. 14A, or there may be four bolts located near the outer corners of the plate or any of a member of similar patterns (see Fig. 14).

Anchor bolts exert relatively small lateral load on base plates, and the edge distance rules listed in AISC Table J3.5¹ do not apply since "tear-out" is not a problem. The only edge requirement is: enough steel is left between the edge of the plate and the closest edge of the hole so that the drill or punch will not drift as the hole is made. One-quarter in. is usually enough for this. (See the discussion on misplaced anchor bolts.)

Burning of base plate holes is an accepted procedure, especially when applied to field repair situations. Most base plates over 1 in. thick are cut to size by the burning process. The slight hardening exhibited at the burned edge is not important since bearing of the anchor bolt

against the side of the hole is not a factor; it makes little difference how the hole is made.

When sizing the area of a base plate, the loss of area at the anchor bolt holes (and grout holes) is generally ignored.

AISC Manual of Steel Construction, 9th ed., pp. 4-130, lists suggestions for oversizing holes for anchor bolts. Based on the trend toward foundation inaccuracy, these allowances are very often not enough. It is suggested that an additional quarter inch over the hole diameter listed be used. A heavy plate washer should be used over the holes ($\frac{5}{16}$ to $\frac{1}{2}$ in. thick).

New Suggested Sizes for Oversized Holes in Base Plates			
Bolt Diameter	Hole Diameter	Bolt Diameter	Hole Diameter
$\frac{3}{4}$	$1\frac{5}{16}$	$1\frac{1}{2}$	$2\frac{5}{16}$
$\frac{7}{8}$	$1\frac{9}{16}$	$1\frac{3}{4}$	$2\frac{3}{4}$
1	$1\frac{13}{16}$	2	$3\frac{1}{4}$
$1\frac{1}{4}$	$2\frac{1}{16}$	$2\frac{1}{2}$	$3\frac{3}{4}$

The finishing requirements of column base plates given in AISC Spec. M2.8¹ were adopted many years ago when plate rolling and temperature control were not as sophisticated as they are today. Consequently, much costly and unnecessary plate milling is paid for each year by unsuspecting clients. Many base plates are received from the mill flatter than after they are welded to the column. Welding, especially large groove welding, tends to curl the plates upon cooling. It is frustrating for a fabricator to mill an already flat plate and then watch it curl up after

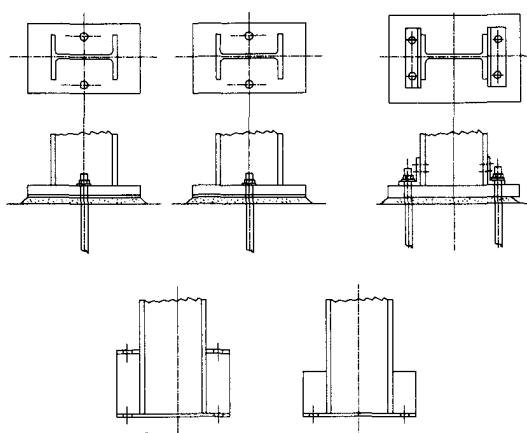


Figure 14

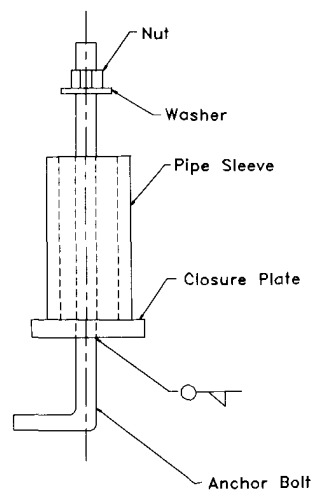


Figure 15

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welding. A sensible approach used by some engineers is to not require milling unless the out-of-flatness exceeds allowable tolerances, regardless of the thickness.

To prevent the curling mentioned above, it is suggested that base plate welds be kept as small in size and quantity as the strength requirements and good practice allow, and that fillet welds be given preference over full and partial penetration welds. However, the minimum fillet weld sizes listed in the AISC Table J2.4¹ must be observed.

Base plates in contact with other steel members, such as leveling plates or steel beams, should have any spatter left from the burning process removed to ensure good surface contact. Burrs resulting from shearing are of little consequence and need not be removed. Bearing plate surfaces in direct contact with concrete need not be ground smooth.

It is not necessary that base plate material match the column shaft material. It is, however, most important that the material be weldable if welding is used. Some commercial grades of steel have the necessary strength but contain too much carbon to permit reliable welds. Most base plates can be ASTM A36 steel.

Base plates near the edges of walls, piers, foundations, etc., should be held back an inch or so from the edge, if possible, to prevent spalling at the free edge of the concrete.

GROUTING

The grouting of base or leveling plates is governed primarily by common sense. When leveling plates are used, the grout is plopped in place off the end of a shovel or trowel and the plate is laid on top of the grout pile and tapped to the proper elevation and made horizontal. The grout does not have to flow except toward the free edges of the plate, and hence the theoretical grout thickness can be established at any figure that will accommodate comfortable construction tolerances. Three-fourths to 1 ½ in. are common grout thicknesses, the lesser figure being common for smaller plates while the higher figure favors larger leveling plates.

When the leveling nut system is used or when large base plates are shipped loose to the job and preset on leveling devices, the grout must be worked under the plate so that there are no resulting air pockets or other non-bearing areas. For small to medium-sized base plates (say, to 36 in. square) a common method to ensure total bearing is to start the grout in one side of the plate, continuing the process until it comes out the far side. For the grout to flow laterally for any distance, there must be ample space between the bottom of the steel plate and the top of the

concrete. An inch and a half would be a minimum space. Also, it is suggested that the concrete foundation be dampened prior to grouting—dampened but not puddled. This helps reduce the absorption of water from the grout by the otherwise dry concrete. However, check the directions on the grout bag prior to use.

Regardless of the grout space allowed, grout will flow only so far laterally, even though it is prodded with a blunt board. For large base plates (say, over 36 in.) it may be necessary to drill a hole in the base plate near the center but not so as to foul the column section. The grout is fed through this hole and urged with the handy blunt board to fill the void beneath the base plate. This grout hole should be approximately 3 in. diameter. For very large plates or long rectangular plates, two grout holes may be required.

The more space left between the bottom of the base plate and the top of the concrete, the easier will be the lateral flow of grout. Spaces 3 to 4 inches are not uncommon for large plates.

Large base plate size is not the only determining factor for grout holes. If a column base plate must be set in a depression in a concrete foundation, with no side access for poking the grout along using the ubiquitous blunt board, a grout hole near the center of the base plate will allow the grout to enter and flow outward toward the plate edges. Grout is usually trimmed neatly at the edges of the base plates—either squared off or at an approximate 45 degree angle. There are many good grouts on the market today of varying strengths. They should be of the non-shrink variety.

When column bases are subject to shear and when the base is designed to resist the shear by means of a shear lug fitted into a concrete keyway, access must be provided to ensure that grout is distributed throughout the cavity to provide the proper bearing area (see Fig. 9).

General contractors should be aware that grouting must be performed before too much load is applied to the column bases which could collapse the anchor bolts or bend the base plates.

In summary, the grout thickness for the leveling plate method should be in the ¾ to 1 ½ in. range. For the leveling nut method, allow between 1 ½ and 3 in. of grout, depending on anchor bolt diameter and the corresponding nut height. Extra-large base plates require more grout thickness in general. In establishing the grout thickness, the anticipated degree of accuracy of the elevation of the concrete pour should be considered. More grout space should be allowed for a foundation contractor known for inaccurate work.

In certain rare circumstances, the concrete foundation may be finished to such a precise level and elevation that the column base plate is placed directly upon it without grout. Such precision usually requires the use of mechanical finishing devices, and these are difficult to operate in the presence of anchor bolts protruding from the concrete. This method is more often associated with the setting of heavy machinery bases and is rarely used in building construction.

FOUNDATION INACCURACIES

The concrete chemists who do such an admirable job of transforming sand, stone, water, and cement into concrete foundations do not always understand the full significance of the little lines between the inch marks on their measuring tapes. Often the result is mislocated anchor bolts. In addition to being out of place, the bolts may be tilted, have too much or too little projection above the top of the concrete, or be at the wrong elevation. The entire bolt group may be rotated 90 degrees from its proper orientation, or the wrong diameter bolts may be used. Sometimes only one bolt of a four-bolt cluster will be misplaced. It seems at times as if anchor bolts are set with about as much finesse as a brain surgeon operating with a garden hoe and with callous disregard for the other trades which must build upon the foundation.

Thanks to alignment tolerances (see Ref. 1, Code of Standard Practice, par. 7.5 regarding the accurate setting of anchor bolts and bearing devices) and the fact that most column base plates are furnished with oversized holes, small mislocations of anchor bolts can usually be tolerated. Bolts that are tilted (not vertical) can sometimes be straightened with a rod bending device called a "hickey." For anchor bolts mislocated up to about $\frac{3}{4}$ in., the concrete may be chipped away to a depth of a few inches and the "hickey" used to bend the bolt into the proper position.

Anchor bolts mislocated by over $\frac{3}{4}$ in. usually require that the base plate be slotted. Severe error may locate the bolt outside or near the edge of the base plate. Edge distance is usually not a problem even if the base plate hole needs to be slotted clear through and out to the edge of the plate as long as the plate is not weakened. Heavy plate washers with offset holes are used to cover the slots. These are welded to the top of the base plate in the field (see Fig. 6). Anchor bolts may also be mislocated toward the interior of the base plate. Large errors may result in the bolts fouling the web or flanges of the column. There are several remedies to this situation, all costly. The errant bolt can be burned off at the surface of the concrete and a new expansion bolt drilled into the proper location

if room permits. Swaged bolts set into core drilled holes with epoxy grout can also be used for this purpose. Such drilling, however, is often complicated by the presence of reinforcing rods, and it must be determined by the engineer of record if it is permissible to sacrifice the reinforcing should one or more rods be encountered.

If an entire bolt group is misplaced but the individual bolts otherwise are in proper relationship with one another, it may be possible to offset the column base plate from its intended location. There are benefits to having the anchor bolt survey performed before fabrication. Likewise, if a bolt group is set in place 90 degrees to what it should be, very often the base plate can be turned on the column with no ill effects.

Anchor bolts which are accurately located but do not project far enough above the top of the concrete to allow a washer and nut to be installed are a real problem. Threaded rod extensions can be welded to the top of the bolt. A sleeve coupling can be used with a short threaded rod extension, but the hole in the base plate must be enlarged to ensure that it clears the coupling (see Fig. 7).

Sometimes a bolt group is so poorly placed, out of alignment and with the individual bolts tilted and offline, that a new base plate must be fabricated with holes drilled to suit the errant locations.

Bolts that are mislocated too near the edge of the column flange may require the flange to be notched somewhat to allow the nut to turn. This slight loss of column bearing area is usually not a problem but should be investigated. If the area loss should prove to be a problem, the deleted area can usually be added nearby in the form of a bearing stiffener with only a small ripple of inconvenience (see Fig. 8).

Anchor bolts set with too much projection are less troublesome. If the threads are insufficient to allow tightening of the nut, then washers must be stacked under the nut. This is not detrimental to performance but is an advertisement of sloppy work on the part of the foundation contractor. If the bolts are so long that they would extend above the floor line, they must be cut off. This can usually be done after the column is tied into the structure with beams. The anchor bolts, one by one, can be denutted and the shank welded directly to the base plate or to a heavy washer which is in turn welded to the base plate. The offending rod is then burned off somewhere below the floor line. Field alterations of this nature may affect the tension capacity of the anchor bolts, and this should be checked by the engineer. Any contemplated column base alteration should be brought to the attention of the engineer so that he may pass judgment as to its

effectiveness. This is especially true of bases subject to uplift, moment and/or shear forces.

Anchor bolts with sleeves provide a greater latitude of adjustment (see Fig. 15). Oddly enough, this device is declining in popularity at a time when the incidence of anchor bolt error appears to be on the rise.

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APPENDIX C

Design of Small Base Plates for Wide Flange Columns

W. A. THORNTON

The 9th Edition¹ of the AISC Manual of Steel Construction uses the Murray-Stockwell² method for analysis of small base plates, i.e., plates that are only slightly larger than the column depth d and width b_f . It combines this method with the cantilever method of the 8th³ and earlier editions for large base plates. The Murray-Stockwell method assumes a bearing pressure of F_p , the maximum permitted, over an H-shaped contact area under the column cross-section between the plate and the concrete. The cantilever method, on the other hand, assumes a uniform bearing pressure, $f_p \leq F_p$, over the entire base plate surface of area $B \times N$ (Fig. 1). Thus, the two methods assume very different bearing pressure distributions and are difficult to combine into a single method.

A solution to this dilemma is to return to the 8th Edition assumption of uniform pressure between the base plate and the concrete. This assumption is conservative with respect to the base plate thickness determination because the true pressure distribution will be less near the plate edges and more under the column cross-section, which cross-section also provides support for the plate at its top surface. Since the plate is assumed more heavily loaded distant from its

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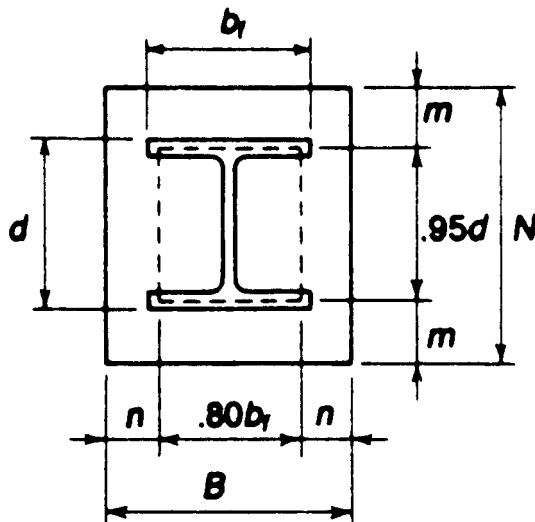


Fig. 1. Column base plate geometry and symbols (from AISC⁴).

supports than it will be, a plate thickness determined under this load will be thicker than it needs to be.

To supplement the cantilever method for large base plates, which is actually a yield line method, it is consistent again to use yield line theory applied to the portion of the base plate contained within the column depth and width. Happily, exact solutions to this problem are available in the literature.⁴ Consider Fig. 2, which shows a plate supported on three edges and free on the fourth. The dimensions of the plate are taken as the column depth d and the half column width $b_f/2$, rather than the more correct $d - 2t_f$ and $(b_f - t_w)/2$. This is done for simplicity and is conservative. If the three supported edges are taken as completely fixed, i.e., no displacement and no rotation about an axis parallel to each edge, the required base plate thickness with a factor of safety of 2 is

$$t_p = \alpha b_f \sqrt{\frac{f_p}{F_y}} \quad (1)$$

where

f_p = uniform pressure between base plate and concrete
= $P/B \times N$, ksi

F_y = yield stress of base plate, ksi

$$\alpha = \sqrt{\frac{1}{3} \left(\frac{3\eta^2 - \sqrt{1+6\eta^2} + 1}{\eta^2 + \sqrt{1+6\eta^2} - 1} \right)} \quad (2)$$

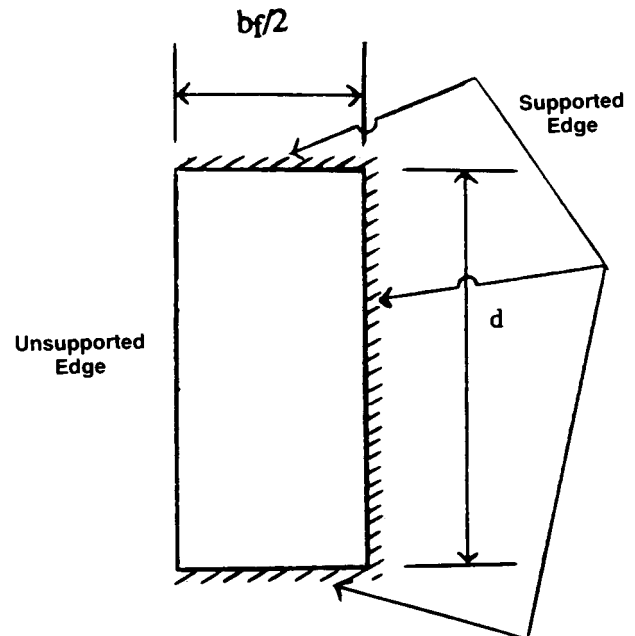


Fig. 2. Small base plate geometry and support conditions.

Table 1.
Examples To Compare Methods ($F_y = 36$ ksi for all cases)

Example	Source	Data									n'/t_p (in./in.)			
		Col. Sect.	P (kips)	d (in.)	b_f (in.)	N (in.)	B (in.)	f_p (ksi)	m (in.)	n (in.)	Mod. 1	Mod. 2	Ahmed & Kreps	AISC 8th Ed.
1.	AISC Des. Guide ^a	W10 x 100	200	11.10	10.34	11.5	11	1.58	.48	1.36	2.14 .90	2.68 1.12	2.33 .98	3.92 1.64
2.	Ahmed & Kreps ^b	W12 x 106	331	12.89	12.22	14	13	1.82	.88	1.61	2.51 1.13	3.14 1.41	2.71 1.22	4.77 2.15
3.	—	W12 x 106	300	12.89	12.22	14	13	1.65	.88	1.61	2.51 1.07	3.14 1.34	2.71 1.16	4.77 2.04
4.	—	W12 x 106	300	12.89	12.22	16	16	1.17	1.88	3.11	2.51 1.12	3.14 1.13	2.71 1.12	4.77 1.72
5.	AISC 8th Ed.	W10 x 100	525	11.10	10.34	19	17	1.63	4.23	4.36	2.14 1.86	2.68 1.86	2.33 1.86	3.92 1.86
6.	AISC 8th Ed.	W12 x 106	600	12.89	12.22	18	16	2.08	2.88	3.11	2.51 1.50	3.14 1.51	2.71 1.50	4.77 2.29
7.	Fling ^c	14 x 8WF	—	14	8	—	—	.75	—	—	2.12 .61	2.65 .77	2.94 .85	3.68 1.06
8.	—	W24 x 68	450	23.73	8.965	24	9	2.08	—	—	2.92 1.41	3.65 1.76	4.98 2.40	4.23 2.04
9.	—	W36 x 160	1000	36.01	12.00	38	14	1.88	1.90	2.20	4.16 1.90	5.20 2.38	7.56 3.46	5.63 2.57

a. See Ref. 6
b. See Ref. 5
c. See Ref. 7, Fling gets $t_p = 0.711$ in. for this example

where $\eta = d/b_f$

The expression for α given in Eq. 2 can be approximated by

$$\alpha = \sqrt{\frac{\eta}{6}} \quad (3)$$

with an error of -2.97% (unconservative) to +6.00% (conservative) in the range of η from % to 3. Then, Eq. 1 becomes with Eq. 3

$$t_p = 2 \frac{\sqrt{db_f}}{2\sqrt{6}} \sqrt{\frac{f_p}{F_y}} \approx 2(\frac{1}{5}\sqrt{db_f}) \sqrt{\frac{f_p}{F_y}} \quad (4)$$

where $\frac{1}{2\sqrt{6}}$ has been replaced by $\frac{1}{5}$ with an error of 2 %.

Combining Eq. 4 with the cantilever method for large base plates, let

$$n' = \frac{1}{5}\sqrt{db_f} \quad (5)$$

and

$$l = \max(m, n, n') \quad (6)$$

where m and n are defined in Fig. 1. Then the required plate thickness is

$$t_p = 2l \sqrt{\frac{f_p}{F_y}} \quad (7)$$

If the base plate is small with $N \approx d$, it may be unconservative to assume complete fixity of the base plate to the column flanges. If the plate of Fig. 2 is completely fixed to the column web along the side of length d but simply supported, i.e., no displacement but rotation unrestrained, along the sides of length $b_f/2$, the required base plate thickness with a factor of safety of 2 is given by Eq. 1, with

$$\alpha = \sqrt[1/3]{\frac{6\eta^2 - \sqrt{1+12\eta^2} + 1}{2\eta^2 + \sqrt{1+12\eta^2} - 1}} \quad (8)$$

This expression for α can be approximated by

$$\alpha = \frac{1}{2}\sqrt{\eta} \quad (9)$$

with an error of -0% (unconservative) and +17.7% (conservative) in the range of η from $\frac{3}{4}$ to 3. In the more common range of $\frac{3}{4} \leq \eta \leq 2$, the error is only +8.00% (conservative). Using Eq. 9 in Eq. 1,

$$t_p = 2(\frac{1}{4}\sqrt{db_f}) \sqrt{\frac{f_p}{F_y}} \quad (10)$$

Combining Eq. 10 with the cantilever method for large base plates, let

$$n' = \frac{1}{4}\sqrt{db_f} \quad (11)$$

$$l = \max(m, n, n') \quad (12)$$

$$t_p = 2l\sqrt{\frac{f_p}{F_y}} \quad (13)$$

The formulation for the two models just discussed can be seen to be exactly the same except for n' . Let the first formulation, for which $n' = \frac{1}{5}\sqrt{db_f}$ be referred to as Model 1 and the second, with $n' = \frac{1}{4}\sqrt{db_f}$ be referred to as Model 2. It will be instructive to see how these two models compare with a method suggested by Ahmed and Kreps⁵ and the method of the AISC 8th Edition Manual. To this end, consider Table 1. The nine examples of this table show that both Models 1 and 2 produce plate thicknesses less than or equal to the method of the AISC 8th Edition. The method of Ahmed and Kreps produces plate thicknesses between Models 1 and 2 for small base plates of square columns, but tends to produce plates too thick for nonsquare columns ($\eta > 1$), as seen from Examples 7, 8 and 9. In the case of Examples 8 and 9, it produces plates thicker than the 8th Edition method.

Considering the results shown in Table 1, and recognizing that Model 2 is clearly conservative while still producing plates thinner or at most as thick as the method of the AISC 8th Edition Manual, it is recommended that Model 2, i.e.,

$$\begin{aligned} n' &= \frac{1}{4}\sqrt{db_f} \\ l &= \max(m, n') \\ t_p &= 2l\sqrt{\frac{f_p}{F_y}} \end{aligned}$$

be used to replace the current AISC 9th Edition Manual base plate design method for axial load.

The equivalent Load and Resistance Factor Design (LRFD) equation for base plate thickness is:

$$t_p = l\sqrt{\frac{2P_u}{0.9F_yBN}} \quad (14)$$

where

P_u = total factored column load

NOTATION

The symbols used in this paper follow the usage of the AISC Manual, 8th or 9th Edition.

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