Steel Design Guide

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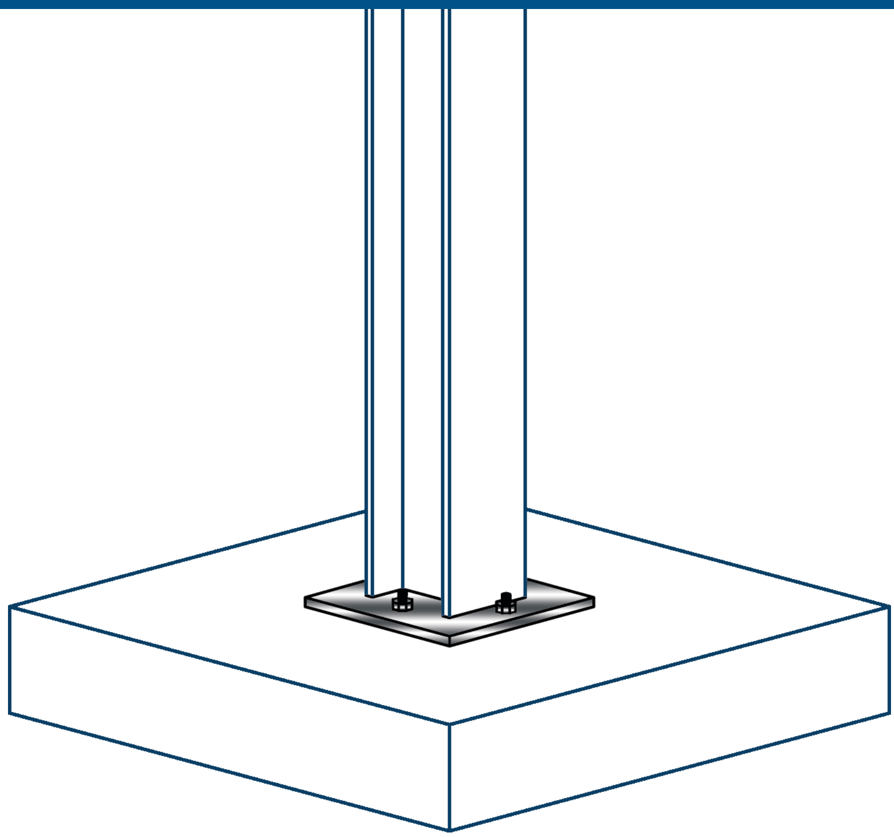
Base Plate and Anchor Rod Design

Second Edition



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1

Base Plate and Anchor Rod Design

Second Edition

JAMES M. FISHER, Ph.D., P.E.
Computerized Structural Design, S.C.
Milwaukee, Wisconsin

and

LAWRENCE A. KLOIBER, P.E.
LeJuene Steel Company
Minneapolis, Minnesota

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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Donald Johnson
Geoffrey L. Kulak
Bill R. Lindley II
David McKenzie
Richard Orr
Davis G. Parsons II
William T. Segui
David F. Sharp
Victor Shneur
Bozidar Stojadinovic
Raymond Tide
Gary C. Violette
Floyd J. Vissat

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1.0 INTRODUCTION

Column base plate connections are the critical interface between the steel structure and the foundation. These connections are used in buildings to support gravity loads and function as part of lateral-load-resisting systems. In addition, they are used for mounting of equipment, and in outdoor support structures, where they may be affected by vibration and fatigue due to wind loads.

Base plates and anchor rods are often the last structural steel items to be designed but are the first items required on the jobsite. The schedule demands along with the problems that can occur at the interface of structural steel and reinforced concrete make it essential that the design details take into account not only structural requirements, but also include consideration of constructability issues, especially anchor rod setting procedures and tolerances. The importance of the accurate placement of anchor rods cannot be overemphasized. This is one of the key components to safely erecting and accurately plumbing the building.

The material in this Guide is intended to provide guidelines for engineers and fabricators to design, detail and specify column-base-plate and anchor rod connections in a manner that avoids common fabrication and erection problems. This Guide is based on the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005a), hereafter referred to as the AISC *Specification*, and includes guidance for designs made in accordance with Load and Resistance Factor Design (LRFD) or Allowable Strength Design (ASD).

This Guide follows the format of the AISC *Specification*, developing strength parameters for foundation system design in generic terms that facilitate either LRFD or ASD. Column bases and portions of the anchorage design generally can be designed in a direct approach based on either LRFD or ASD load combinations. The one area of anchorage design that is not easily designed by ASD is the embedment of anchor rods into concrete. This is due to the common use of ACI 318-08, Appendix D, which is exclusively based on the strength approach (LRFD), for the design of such embedments. ASD and LRFD methods are equally proficient at evaluating other steel elements of the foundation system, including the column base plate and the sizing of anchor diameters. In cases such as anchors subjected to neither tension nor shear, the anchorage development requirement may be a relatively insignificant factor.

The generic approach in development of foundation design parameters taken in this Guide permits the user a choice to develop the loads based on either the LRFD or ASD approach. The derivations of foundation design parameters, as presented herein, are then either multiplied by a *Resistance Factor,* ϕ, or divided by a *Safety Factor,* Ω, based on the appropriate load system utilized in the analysis; consistent with the approach used in the AISC *Specification*. Many of the equations shown herein are independent of the load approach, and thus are applicable to either design methodology. These are shown in singular format. Other derived equations are based on the particular load approach and are presented in a side-by-side format of comparable equations for LRFD or ASD application.

The typical components of a column base are shown in Figure 1.1. This figure shows anchor rods that are threaded and nutted at the embedded end. Anchor rods also may be headed or have hooked ends.

Material selection and design details of base plates can significantly affect the cost of fabrication and erection of steel structures, as well as the performance under load. Relevant aspects of each of these subjects are discussed briefly in the next section. Not only is it important to design the column-base-plate connection for strength requirements, it is also important to recognize these connections affect the behavior of the structure. Assumptions are made in structural analysis about the boundary conditions represented by the connections. Models comprising beam or truss elements typically idealize the column base connection as either a pinned or fixed boundary condition. Improper characterization can lead to error in the computed drifts, leading to unrecognized second-order moments if the stiffness is overestimated, or excessive first-floor column sizes if the stiffness is underestimated. If more accurate analyses are desired, it may be necessary to input the stiffness of the column-base-plate connection in the elastic and plastic ranges, and for seismic loading, possibly even the cyclic force-deformation relations. The forces and deformations from the structural analyses used to design the column-base-plate connection are dependent on the choice of the column-base-plate connection details.

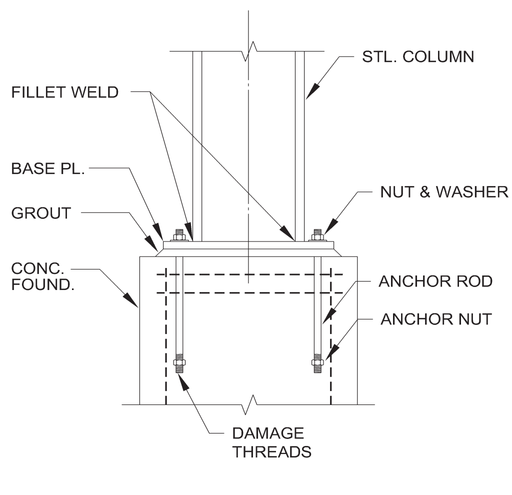


Fig. 1.1. Column base connection components.

The vast majority of building columns are designed for axial compression only with little or no uplift. For such columns, a simple column-base-plate connection detail like that shown in Figure 1.1 is sufficient. The design of column-base-plate connections for axial compression only is presented in Section 3. The design is simple and need not be encumbered with many of the more complex issues discussed in Appendix A, which pertains to special structures. Anchor rods for gravity columns are often not required for the permanent structure and need only be sized to provide for column stability during erection.

Column-base-plate connections are also capable of transmitting uplift forces and can transmit shear, including through the anchor rods if required. If the base plate remains in compression, shear can be transmitted through friction against the grout pad or concrete, thus the anchor rods are not required to be designed for shear. Large shear forces can be resisted by bearing against concrete, either by embedding the column base, or by adding a shear lug under the base plate.

Column-base-plate moment connections can be used to resist wind and seismic loads on the building frame. Moment at the column base can be resisted by development of a force couple between bearing on the concrete and tension in some or all of the anchor rods.

This Guide will enable the designer to design and specify economical column base plate details that perform adequately for the specified demand. The objective of the design process in this Guide is that under service loading, and under extreme loading in excess of the design loads, the behavior of column base plates should be close to that predicted by the approximate mathematical equations in this Design Guide.

Historically, two anchor rods have been used in the area bounded by column flanges and web. Recent regulations of the U.S. Occupational Safety and Health Administration (OSHA)—*Safety Standards for Steel Erection* (OSHA, 2001) (Subpart R of 29 CFR Part 1926)—require four anchor rods in almost all column-base-plate connections, and require all columns to be designed for a specific bending moment to reflect the stability required during erection with an ironworker on the column. This regulation has essentially eliminated the typical detail with two anchor rods except for small post-type structures that weigh less than 300 pounds (e.g., doorway portal frames).

This Guide supersedes the original AISC Design Guide 1—*Column Base Plates*. In addition to the OSHA regulations, there has been significant research and improved design guidelines issued subsequent to the publication of Design Guide 1 in 1990. The ACI *Building Code Requirements for Structural Concrete*, ACI 318-08 (ACI, 2008), has improved provisions for the pullout and breakout strength of anchor rods and other embedded anchors. Design guidance for anchor rods based on the ACI recommendations is included, along with practical suggestions for detailing and installing anchor rod assemblies. These guidelines deal principally with cast-in-place anchors and with their design, installation, inspection and repair in column-base-plate connections.

Table 2.1. Base Plate Materials

Thickness (t_p)	Plate Availability
$t_p \le 4$ in.	ASTM A36[a] ASTM A572 Gr 42 or 50 ASTM A588
4 in. $< t_p \le 6$ in.	ASTM A36[a] ASTM A572 Gr 42 ASTM A588
$t_p > 6$ in.	ASTM A36[a] ASTM A588

[a] Preferred material specification

AISC Design Guide 7, *Industrial Buildings: Roofs to Column Anchorage* (Fisher, 2004), hereafter referred to as AISC *Design Guide 7,* contains additional examples and discussion relative to the design of anchor rods.

2.0 MATERIALS, FABRICATION, INSTALLATION AND REPAIRS

2.1 Material Specifications

The AISC *Specification* lists a number of plate and threaded rod materials that are structurally suitable for use in base plate and anchor rod designs. Based on cost and availability, the materials shown in Tables 2.1 and 2.2 are recommended for typical building design.

2.2 Base Plate Material Selection

Base plates should be designed using ASTM A36 material unless the availability of an alternative grade is confirmed prior to specification. Since ASTM A36 plate is readily available, the plates can often be cut from stock material. There is seldom a reason to use high-strength material, since increasing the thickness will provide increased strength where needed. Plates are available in ⅛-in. increments up to 1¼-in. thickness and in ¼-in. increments above this. The base plate sizes specified should be standardized during design to facilitate purchasing and cutting of the material.

When designing base plate connections, it is important to consider that material is generally less expensive than labor and, where possible, economy may be gained by using thicker plates rather than detailing stiffeners or other reinforcement to achieve the same strength with a thinner base plate. A possible exception to this rule is the case of moment-type bases that resist large moments. For example, in the design

Table 2.2. Anchor Rod Materials						
Material ASTM		Tensile Strength F_u (ksi)	Nominal Tensile Stress[a] $F_{nt} = 0.75F_u$ (ksi)	Nominal Shear Stress (X type)[a, b] $F_{nv} = 0.50F_u$ (ksi)	Nominal Shear Stress (N type)[a, c] $F_{nv} = 0.40F_u$ (ksi)	Maximum Diameter (in.)
F1554	Gr 36[d]	58	43.5	29.0	23.2	4
	Gr 55	75	56.3	37.5	30.0	4
	Gr 105	125	93.8	62.5	50.0	3
A449		120	90.0	60.0	48.0	1
		105	78.8	52.5	42.0	1½
		90	67.5	45.0	36.0	3
A36		58	43.5	29.0	23.2	4
A307 Gr C[e]		58	43.5	29.0	23.2	4
A354 Gr BD		150	112	75.0	60.0	2½
		140	105	70.0	56.0	4

[a] Nominal stress on unthreaded body for cut threads (based on major thread diameter for rolled threads)
[b] Threads excluded from shear plane
[c] Threads included in the shear plane
[d] Preferred material specification
[e] See F1554 Gr 36

of a crane building, the use of a seat or stool at the column base may be more economical, if it eliminates the need for large complete-joint-penetration (CJP) groove welds to heavy plates that require special material specifications.

Most column base plates are designed as square, to match the foundation shape and more readily accommodate square anchor rod patterns. Exceptions to this include moment-resisting bases and columns that are adjacent to walls.

Many structural engineers have established minimum thicknesses for base plates for typical gravity columns. For posts and light HSS columns, the minimum plate thickness is typically ½ in., and for other structural columns a plate thickness of ¾ in. is commonly accepted as the minimum thickness specified.

2.3 Base Plate Fabrication and Finishing

Typically, base plates are thermally cut to size. Anchor rod and grout holes may be either drilled or thermally cut. Section M2.2 of the AISC *Specification* lists requirements for thermal cutting as follows:

"*…thermally cut free edges that will be subject to calculated static tensile stress shall be free of round-bottom gouges greater than 3⁄16-in. (5 mm) deep and sharp V-shaped notches. Gouges deeper than 3⁄16 in. (5 mm) and notches shall be removed by grinding and repaired by welding*."

Because free edges of the base plate are not subject to tensile stress, these requirements are not mandatory for the perimeter edges; however, they provide a workmanship guide that can be used as acceptance criteria. Anchor rod holes, which may be subject to tensile stress, should meet the requirements of Section M2.2. Generally, round-bottom grooves within the limits specified are acceptable, but sharp notches must be repaired. Anchor rod hole sizes and grouting are covered in Sections 2.6 and 2.10 of this Design Guide.

Finishing requirements for column bases that bear on steel plates are covered in Section M2.8 of the AISC *Specification* as follows:

"*Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. (50 mm) but not over 4 in. (100 mm) in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces … to obtain a satisfactory contact bearing. Steel bearing plates over 4 in. (100 mm) in thickness shall be milled for bearing surfaces ….*"

Two exceptions are noted: the bottom surface need not be milled when the base plate is to be grouted, and the top surface need not be milled when CJP groove welds are used to connect the column to the base plate.

AISC *Specification* Section M4.4 defines a satisfactory bearing surface as follows:

"Lack of contact bearing not exceeding a gap of 1⁄16 in. (2 mm) regardless of the type of splice used … is permitted. If the gap exceeds 1⁄16 in. (2 mm), but is less than ¼ in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material."

While the AISC *Specification* requirements for finishing are prescriptive in form, it is important to ensure that a satisfactory contact bearing surface is provided. By applying the provisions of Section M4.4, it may not be necessary to mill plates over 4-in. thick if they are flat enough to meet the gap requirements under the column. Standard practice is to order all plates over approximately 3 in. with an extra ¼ in. to ½ in. over the design thickness to allow for milling. Typically, only the area directly under the column shaft is milled. The base elevation for setting the column is determined in this case by the elevation at the bottom of the column shaft with the grout space and shims adjusted accordingly.

2.4 Base Plate Welding

The structural requirements for column base plate welds may vary greatly between columns loaded in compression only and columns in which moment, shear and/or tension forces are present. Welds attaching base plates to columns are often sized to develop the strength of the anchor rods in tension, which can most often be achieved with a relatively small fillet weld. For example, a 5⁄16-in., 2½-in.-long fillet weld to each column flange will fully develop a 1-in.-diameter ASTM F1554 Grade 36 anchor rod when the directional strength increase for fillet welds loaded transversely is used. Alternative criteria may be advisable when rod diameters are large or material strength levels are high.

A few basic guidelines on base plate welding are as follows:

- Fillet welds are preferred to groove welds for all but large moment-resisting bases.
- The use of the weld-all-around symbol should be avoided, especially on wide-flange shapes, since the small amount of weld across the toes of the flanges and in the radius between the web and flange add very little strength and are very costly.
- For most wide-flange columns subject to axial compression only, welding on one side of each flange (See Figure 2.1) with 5⁄16-in. fillet welds will provide adequate strength and the most economical detail. When these welds are not adequate for columns with moment or axial tension, consider adding fillet welds on all faces up to ¾ in. in size before using groove welds.
- For rectangular HSS columns subject to axial compression only, welding on the flats of the four sides only will avoid having to make an out-of-position weld on the corners. Note, however, that corners must be welded for HSS columns with moment or axial tension, and when anchor rods are located at the corners of the base plate since the critical yield line will form in the plate at the corners of the HSS.
- The minimum fillet weld requirements have been changed in the AISC *Specification*. The minimum-size fillet weld is now based on the thinner of the materials being joined.

Most column base plates are shop welded to the column shaft. In the past it was common to detail heavy base plates for multi-story buildings as loose pieces to be set and grouted before erecting the column shaft. The base plate was detailed with three adjusting screws, as shown in Figure 2.2, and the milled surface was carefully set to elevation.

This approach had the advantage of reducing the weight of heavy members for handling and shipping and provided a fully grouted base plate in place to receive a very heavy column shaft. The column may or may not be welded after erection depending on the structural requirements and the type of erection aid provided. Most erectors now prefer to have the base plate shop welded to the column whenever possible.

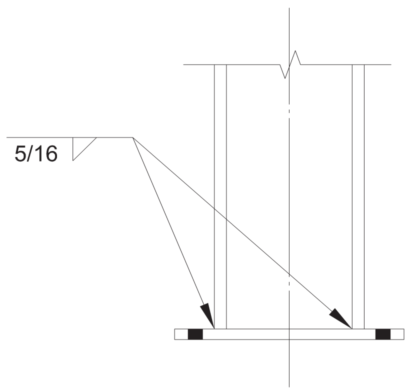


Fig. 2.1. Typical gravity column base plate weld.

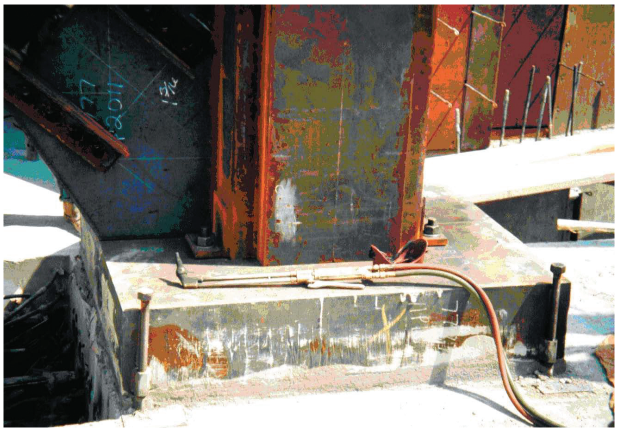


Fig. 2.2. Base plate with adjusting screws.

2.5 Anchor Rod Material

As shown previously in Table 2.2, the preferred specification for anchor rods is ASTM F1554, with Grade 36 being the most common strength level used. The availability of other grades should be confirmed prior to specification.

ASTM F1554 Grade 55 anchor rods are used when there are large tension forces due to moment connections or uplift from overturning. ASTM F1554 Grade 105 is a special high strength rod grade and generally should be used only when it is not possible to develop the required strength using larger Grade 36 or Grade 55 rods.

Unless otherwise specified, anchor rods will be supplied with Unified Coarse (UNC) Threads with a Class 2A tolerance, as permitted in ASTM F1554. While ASTM F1554 permits standard hex nuts, all nuts for anchor rods, especially those used in base plates with large oversize holes, should be furnished as heavy hex nuts, preferably ASTM A563 Grade A or DH for Grade 105.

ASTM F1554 anchor rods are required to be color coded to allow easy identification in the field. The color codes are as follows:

Grade 36……………………………..Blue
Grade 55…………………………… Yellow
Grade 105…………………………Red

In practice, Grade 36 is considered the default grade and often is not color coded.

The ASTM *Specification* for F1554 allows anchor rods to be supplied either straight (threaded with nut for anchorage), hooked or headed. Rods up to approximately 1-in. diameter are sometimes supplied with heads hot-forged similar to a structural bolt. Thereafter, it is more common that the rods will be threaded and nutted.

Hooked-type anchor rods have been extensively used in the past. However, hooked rods have a very limited pullout strength compared to that of headed rods or threaded rods with a nut for anchorage. Therefore, current recommended practice is to use headed rods or threaded rods with a nut for anchorage.

The addition of plate washers or other similar devices does not increase the pullout strength of the anchor rod, and can create construction problems by interfering with reinforcing steel placement or concrete consolidation under the plate. Thus, it is recommended that the anchorage device be limited to either a heavy hex nut or a head on the rod. As an exception, the addition of plate washers may be of use when high strength anchor rods are used, or when concrete blowout could occur (see Section 3.2.2 of this Guide). In these cases, calculations should be made to determine if an increase in the bearing area is necessary. Additionally, it should be confirmed that the plate size specified will work with the reinforcing steel and concrete placement requirements.

ASTM F1554 Grade 55 anchor rods can be ordered with a supplementary requirement, S1, which limits the carbon equivalent content to a maximum of 45%, to provide weldability when needed. Adding this supplement is helpful should welding become required for fixes in the field. Grade 36 is typically weldable without supplement.

There are also two supplemental provisions available for Grades 55 and 105 regarding Charpy V-Notch (CVN) toughness. These provide for CVN testing of 15 ft-lbs at either 40 °F (S4) or at −20 °F (S5). Note however that anchor rods typically have sufficient fracture toughness without these supplemental specifications. Additional fracture toughness is expensive and generally does not make much difference in the time to failure for anchor rods subjected to fatigue loading. Although fracture toughness may correspond to a greater crack length at the time of failure (because cracks grow at an exponential rate) 95% of the fatigue life of the anchor rod is consumed when the crack size is less than a few millimeters. This is also the reason why it is not cost effective to perform ultrasonic testing or other nondestructive tests on anchor rods to look for fatigue cracks. There is only a small window between the time cracks are large enough to detect and small enough to not cause fracture. Thus, it generally is more cost effective to design additional redundancy into the anchor rods rather than specifying supplemental CVN properties.

Galvanized anchor rods are often used when the column-base-plate assembly is exposed and subject to corrosion. Either the hot-dip galvanizing process (ASTM A153) or the mechanical galvanizing process (ASTM B695) is allowed in ASTM F1554; however, all threaded components of the fastener assembly must be galvanized by the same process. Mixing of rods galvanized by one process and nuts by another may result in an unworkable assembly. It is recommended that galvanized anchor rods and nuts be purchased from the same supplier and shipped preassembled. Because this is not an ASTM requirement, this should be specified on the contract documents.

Note also that galvanizing increases friction between the nut and the rod and even though the nuts are over tapped, special lubrication may be required.

ASTM A449, A36 and A307 *Specifications* are listed in Table 2.2 for comparison purposes, because some suppliers are more familiar with these *Specifications*. Note that ASTM F1554 grades match up closely with many aspects of these older material specifications. Note also that these older material specifications contain almost none of the anchor-rod specific requirements found in ASTM F1554.

Drilled-in epoxy-type anchor rods are discussed in several places in the Design Guide. This category of anchor rod does not include wedge-type mechanical anchors, which are not recommended for anchor rods because they must be tensioned to securely lock in the wedge device. Column movement during erection can cause wedge-type anchor rods to loosen.

Table 2.3. Recommended Sizes for Anchor Rod Holes in Base Plates			
Anchor Rod Diameter, in.	**Hole Diameter, in.**	**Min. Washer Dimension, in.**	**Min. Washer Thickness, in.**
¾	1$\frac{5}{16}$	2	¼
⅞	1$\frac{9}{16}$	2½	$\frac{5}{16}$
1	1$\frac{13}{16}$	3	⅜
1¼	2$\frac{1}{16}$	3	½
1½	2$\frac{5}{16}$	3½	½
1¾	2¾	4	⅝
2	3¼	5	¾
2½	3¾	5½	⅞

Notes: 1. Circular or square washers meeting the size shown are acceptable.
2. Adequate clearance must be provided for the washer size selected.
3. See discussion in Section 2.6 regarding the use of alternate 1$\frac{1}{16}$-in. hole size for ¾-in.-diameter anchor rods, with plates less than 1¼-in. thick.

2.6 Anchor Rod Holes and Washers

The most common field problem is anchor rod placements that either do not fit within the anchor rod hole pattern or do not allow the column to be properly positioned. Because OSHA requires any modification of anchor rods to be approved by the Engineer of Record, it is important to provide as large a hole as possible to accommodate setting tolerances. The AISC-recommended hole sizes for anchor rods are given in Table 2.3.

These hole sizes originated in the first edition of Design Guide 1, based on field problems in achieving the column setting tolerances required for the previous somewhat smaller recommended sizes. They were later included in the AISC *Steel Construction Manual* (AISC, 2005d).

The washer diameters shown in Table 2.3 are sized to cover the entire hole when the anchor rod is located at the edge of the hole. Plate washers are usually custom fabricated by thermal cutting the shape and holes from plate or bar stock. The washer may be either a plain circular washer or a rectangular plate washer as long as the thickness is adequate to prevent pulling through the hole. The plate washer thicknesses shown in the table are similar to the recommendation in Section 4.2.5 of AISC Design Guide 10, that the minimum washer thickness be approximately 3 the anchor rod diameter. The same thickness is adequate for all grades of ASTM F1554, since the pull-through criterion requires appropriate stiffness as well as strength.

For anchor rods for columns designed for axial compression only, the designer may consider using a smaller hole diameter of 1$\frac{1}{16}$ in. with ¾-in.-diameter rods and base plates less than 1¼-in. thick, as allowed in Footnote 3 in Table 2.3. This will allow the holes to be punched up to this plate thickness, and the use of ASTM F844 (USS Standard) wash-ers in lieu of the custom washers of dimensions shown in the table. This potential fabrication savings must be weighed against possible problems with placement of anchor rods out of tolerance.

For anchor rods designed to resist moment or axial tension, the hole and washer sizes recommended in Table 2.3 should be used. The added setting tolerance is especially important when the full or near-full strength of the rod in tension is needed for design purposes, because almost any field fix in this case will be very difficult.

Additional recommendations regarding washers and anchor rod holes are as follows:

- Washers should not be welded to the base plate, except when the anchor rods are designed to resist shear at the column base (see Section 3.5).
- ASTM F436 washers are not used on anchor rods because they generally are of insufficient size.
- Washers for anchor rods are not, and do not need to be, hardened.

2.7 Anchor Rod Sizing and Layout

Use ¾-in.-diameter ASTM F1554 Grade 36 rod material whenever possible. Where more strength is required, consider increasing rod diameter up to about 2 in. in ASTM F1554 Grade 36 material before switching to a higher-strength material grade.

Anchor rod details should always specify ample threaded length. Whenever possible, threaded lengths should be specified at least 3-in. greater than required, to allow for variations in setting elevation.

Anchor rod layouts should, where possible, use a symmetrical pattern in both directions and as few different layouts as possible. Thus, the typical layout should have four anchor rods in a square pattern.

Anchor rod layouts should provide ample clearance distance for the washer from the column shaft and its weld, as well as a reasonable edge distance. When the hole edge is not subject to a lateral force, even an edge distance that provides a clear dimension as small as ½ in. of material from the edge of the hole to the edge of the plate will normally suffice, though field issues with anchor rod placement may necessitate a larger dimension to allow some slotting of the base plate holes. When the hole edge is subject to a lateral force, the edge distance provided must be large enough for the necessary force transfer.

Keep the construction sequence in mind when laying out anchor rods adjacent to walls and other obstructions. Make sure the erector will have the access necessary to set the column and tighten the nuts on the anchor rods. Where special settings are required at exterior walls, moment bases and other locations, clearly identify these settings on both the column schedule and foundation drawings.

Anchor rod layouts must be coordinated with the reinforcing steel to ensure that the rods can be installed in the proper location and alignment. This is especially critical in concrete piers and walls where there is less room for adjustment in the field. Anchor rods in piers should never extend below the bottom of the pier into the footing because this would require that the anchor rods be partially embedded prior to forming the pier, which makes it almost impossible to maintain alignment. When the pier height is less than the required anchor rod embedment length, the pier should be eliminated and the column extended to set the base plate on the footing.

2.8 Anchor Rod Placement and Tolerances

Proper placement of anchor rods provides for the safe, fast and economical erection of the structural steel frame.

The placement process begins with the preparation of an anchor rod layout drawing. While it is possible to lay out anchor rods using the foundation design drawings and the column schedule, there will be fewer problems if the structural steel detailer coordinates all anchor rod details with the column-base-plate assembly. The anchor rod layout drawing will show all anchor rod marks along with layout dimensions and elevation requirements. Because of schedule pressures, there is sometimes a rush to set anchor rods using a drawing submitted for approval. This should be avoided; only placement drawings that have been designated as "Released for Construction" should be used for this important work.

Layout (and after-placement surveying) of all anchor rods should be done by an experienced construction surveyor. The surveyor should be able to read structural drawings and be knowledgeable of construction practices. A typical licensed land surveyor may or may not have the necessary knowledge and experience for this type of work.

Templates should be made for each anchor rod setting pattern. Typically, templates are made of plywood on site. The advantage of plywood templates is they are relatively inexpensive to make and are easy to fasten in place to the wood foundation forms. The anchor rods can be held securely in place and relatively straight by using a nut on each side of the template. Steel templates consisting of flat plates or angle-type frames are sometimes used for very large anchor rod assemblies requiring close setting tolerances. Provisions should be made to secure the template in place, such as with nailing holes provided in the steel plate. Steel plate templates can also be reused as setting plates.

Embedded templates are sometimes used with large anchor rod assemblies to help maintain alignment of the rods during concrete placement. The template should be kept as small as possible to avoid interference with the reinforcing steel and concrete placement. When using a single exposed template, the reinforcing steel can be placed before positioning the anchor rods in the form. With the embedded template, the anchor rod assembly must be placed first and the reinforcing steel placed around or through the template. Care must be taken to consolidate the concrete around the template to eliminate voids. This is especially important if the template serves as part of the anchorage.

When the templates are removed, the anchor rods should be surveyed and grid lines marked on each setting. The anchor rods should then be cleaned and checked to make sure the nuts can be easily turned and that the vertical alignment is proper. If necessary, the threads should be lubricated. OSHA requires the contractor to review the settings and notify the Engineer of Record of any anchor rods that will not meet the tolerance required for the hole size specified.

As exceptions to the foregoing recommendations, fast-track projects and projects with complex layouts may require spe-cial considerations. In a fast-track project, the steel design and detailing may lag behind the initial foundation work and the structural layout changed as the job progresses. A project with complex layouts may be such that even the most ac-curate placement possible of anchor rods in concrete forms does not facilitate proper fit-up. On these projects, it may be better to use special drilled-in epoxy-type anchor rods rather than cast-in-place rods.

For fast-track projects, this has the advantage of allowing the foundation work to start without waiting for anchor rods and anchor rod layout drawings. For complex layouts, this has the advantage of providing easier and more accurate anchor-rod layout, for more accurate column erection.

Coordination of AISC anchor rod setting tolerances and ACI tolerances for embedded items can be an issue. ACI 117-06 (ACI, 2006), Section 2.3, *Placement of embedded items, excluding dowels in slabs-on-ground,* includes the following tolerance provisions:

- Centerline of assembly from specified location:
  Horizontal deviation …±1 in.
  Vertical deviation …±1 in.
- Anchor rods in concrete
  Top of anchor rod from specified elevation:
  Vertical deviation … ±½ in.
- Centerline of individual anchor rods from specified location:
  Horizontal deviation
    for ¾-in. and ⅞-in. bolts… ±¼ in.
    for 1-in., 1¼-in. and 1½-in. bolts… ±⅜ in.
    for 1¾- in., 2 in. and 2½ in. bolts… ±½ in.

AISC *Code of Standard Practice* Section 7.5.1 lists the following tolerances:

- The variation in dimension between the centers of any two Anchor Rods within an Anchor-Rod Group shall be equal to or less than ⅛ in.
- The variation in dimension between the centers of adjacent Anchor-Rod Groups shall be equal to or less than ¼ in.
- The variation in elevation of the tops of Anchor Rods shall be equal to or less than plus or minus ½ in.
- The accumulated variation in dimension between centers of Anchor-Rod Groups along the Established Column Line through multiple Anchor-Rod Groups shall be equal to or less than ¼ in. per 100 ft, but not to exceed a total of 1 in.
- The variation in dimension from the center of any Anchor-Rod Group to the Established Column Line through that group shall be equal to or less than ¼ in.

Thus, ACI 117-06 provisions are somewhat more generous than the AISC *Code of Standard Practice* for anchor rod tolerances. Furthermore, since each trade will work to their own industry standard unless the contract documents require otherwise, it is recommended that the project specifications, typically the Construction Specifications Institute (CSI, 2004) Division 3, require that the anchor rods be set in accordance with the AISC *Code of Standard Practice,* (AISC, 2005c) tolerance requirements, in order to clearly establish a basis for acceptance of the anchor rods. It may be helpful to actually list the tolerance requirements instead of simply providing a reference.

2.9 Column Erection Procedures

Occupational Safety and Health Administration (OSHA) requires the general contractor to notify the erector in writing that the anchor rods are ready for start of steel erection. This notice is intended to ensure that the layout has been checked, any required repairs have been made and the concrete has achieved the required strength. The erector then, depending on project requirements, rechecks the layout and sets elevations for each column base.

There are three common methods of setting elevations: setting nuts and washers, setting plates, and shim stacks. Project requirements and local custom generally determine which of these methods is used. It is important in all methods that the erector tightens all of the anchor rods before removing the erection load line, so that the nut and washer are tight against the base plate. This is not intended to induce any level of pretension, but rather to ensure that the anchor rod assembly is firm enough to prevent column base movement during erection. If it is necessary to loosen the nuts to adjust column plumbness, care should be taken to adequately brace the column while the adjustment is made.

2.9.1. Setting Nut and Washer Method

The use of four anchor rods has made the setting nut and washer method of column erection very popular, as it is easy and cost-effective. Once the setting nuts and washers are set to elevation, there is little chance they will be disturbed. The four-rod layout provides a stable condition for erection, especially if the anchor rods are located outside of the column area. The elevation and plumbness of the column can be adjusted using the nuts. When designing anchor rods using setting nuts and washers, it is important to remember these rods are also loaded in compression and their strength should be checked for push out at the bottom of the footing. It is recommended that use of the setting nut and washer method be limited to columns that are relatively lightly loaded during erection. Even after the base plate is grouted, the setting nut will transfer load to the anchor rod and this should be considered when selecting the method to set the column elevation.

2.9.2. Setting Plate Method

Setting plates (sometimes called leveling plates) are a very positive method for setting column base elevations, but are somewhat more costly than setting nuts and washers.

Setting plates are usually about ¼-in. thick and slightly larger than the base plate. Because a plate this thin has a tendency to warp when fabricated, setting plates are typically limited to a maximum dimension of about 24 in.

If the setting plate is also to be used as a template, the holes are made ¹⁄₁₆-in. larger than the anchor rod diameter. Otherwise, standard anchor rod hole sizes are used.

After the anchor rods have been set, the setting plate is removed and the anchor rods are checked as noted above. The bearing area is then cleaned and the elevations are set using either jam nuts or shims. Grout is spread over the area and the setting plate tapped down to elevation. The elevation should be rechecked after the plate is set to verify that it is correct. If necessary, the plate and grout can be removed and the process started over.

One problem with using setting plates is that warping in either the setting plate or the base plate, or column movement during “bolt-up” may result in gaps between the setting plate and base plate. Generally there will still be adequate bearing and the amount of column settlement required to close the gap will not be detrimental to the structure. The acceptability of any gaps can be determined using the provisions in AISC *Specification* Section M4.4.

Setting plates provide a positive check on anchor rod settings prior to the start of erection and provide the most stable erection base for the column. The use of setting plates should be considered when the column is being erected in an excavation where water and soil may wash under the base plate and make cleaning and grouting difficult after the column is erected.

2.9.3. Shim Stack Method

Column erection on steel shim stacks is a traditional method for setting base plate elevations that has the advantage that all compression is transferred from the base plate to the foundation without involving the anchor rods. Steel shim packs approximately 4 in. wide are set at the four edges of the base plate. The areas of the shim stacks are typically large enough to carry substantial dead load prior to grouting of the base plate.

2.9.4. Setting Large Base Plates

Base plate size and weight may be such that the base plate must be preset to receive the column. When crane capacities or handling requirements make it advantageous to set the plate in advance of the column, the plates are furnished with either wedge-type shims or leveling or adjusting screws to allow them to be set to elevation and grouted before the column is set, as illustrated in Figure 2.2. Leveling-screw assemblies consist of sleeve nuts welded to the sides of the plate and a threaded rod screw that can be adjusted. These plates should be furnished with hole sizes as shown in Table 2.3. The column shaft should be detailed with stools or erection aids, as required. Where possible, the column attachment to the base plate should avoid field welding because of the difficulty in preheating a heavy base plate for welding.

2.10 Grouting Requirements

Grout serves as the connection between the steel base plate and the concrete foundation to transfer compression loads. Accordingly, it is important that the grout be properly designed, and placed in a proper and timely manner.

Grout should have a design compressive strength at least twice the strength of the foundation concrete. This will be adequate to transfer the maximum steel bearing pressure to the foundation. The design thickness of the grout space will depend on how fluid the grout is and how accurate the elevation of the top of concrete is placed. If the column is set on a finished floor, a 1-in. space may be adequate, while on the top of a footing or pier, normally the space should be 1½ to 2 in. Large base plates and plates with shear lugs may require more space.

Grout holes are not required for most base plates. For plates 24 in. or less in width, a form can be set up and the grout can be forced in from one side until it flows out the opposite side. When plates become larger or when shear lugs are used, it is recommended that one or two grout holes be provided. Grout holes are typically 2 to 3 in. in diameter and are typically thermally cut in the base plate. A form should be provided around the edge and some sort of filling device should be used to provide enough head pressure to cause the grout to flow out to all of the sides.

It is important to follow the manufacturer’s recommendations for mixing and curing times. When placing grout in cold weather, make sure protection is provided per the manufacturer’s specification.

Grouting is an interface between trades that provides a challenge for the specification writer. Typically, the grout is furnished by the concrete or general contractor, but the timing is essential to the work of the steel erector. Because of this, specification writers sometimes place grouting in the steel section. This only confuses the issue because the erector then has to make arrangements with the concrete contractor to do the grouting. Grouting should be the responsibility of the concrete contractor and there should be a requirement to grout column bases promptly when notified by the erector that the column is in its final location.

2.11 Anchor Rod Repairs

Anchor rods may require repair or modification during installation or later on in service. OSHA requires that any modification of anchor rods during construction be reviewed and approved by the Engineer of Record. On a case-by-case basis, the Engineer of Record must evaluate the relative merits of a proposed repair as opposed to rejecting the foundation and requiring the contractor to replace part of the foundation with new anchor rods per the original design.

Records should be kept of the repair procedure and the results. The Engineer of Record may require special inspection or testing deemed necessary to verify the repair.

Most of these repairs are standard simple modifications that do not require calculations. The most common anchor rod problems are addressed in the following sections.

2.11.1 Anchor Rods in the Wrong Position

For anchor rods in the wrong position, the repair method depends on the nature of the problem and when in the construction process it is first noted. Is the repair required for only one rod, or for the entire pattern of rods? How far out of position is the rod or pattern, and what are the required strengths of the rods?

If the error is discovered before the column base plate has been fabricated, it might be possible to use a different pattern or even a different base plate. If the rod positions interfere with the column shaft, it may be necessary to modify the column shaft by cutting and reinforcing sections of the flange or web.

If one or two rods in a pattern are misplaced after the column has been fabricated and shipped, the most common repair is to slot the base plate, and use a plate washer to span the slot. If the entire pattern is off uniformly, it might be possible to cut the base plate off and offset the base plate to accommodate the out of tolerance. It is necessary to check the base plate design for this eccentricity. When removing the base plate, it may be required to turn the plate over to have a clean surface on which to weld the column shaft.

If the anchor rod or rods are more than a couple of inches out of position, the best solution may be to cut off the existing rods and install new drilled-in epoxy-type anchor rods. When using such rods, carefully follow the manufacturer's recommendations, and provide inspection as required in the applicable building code. Locate the holes to avoid reinforcing steel in the foundation. If any reinforcing steel is cut, a check of the effect on foundation strength should be made.

2.11.2 Anchor Rods Bent or Not Vertical

Care should be taken when setting anchor rods to ensure they are plumb. If the rods are not properly secured in the template, or if there is reinforcing steel interference, the rods may end up at an angle to the vertical that will not allow the base plate to be fit over the rods.

Rods can also be damaged in the field by equipment, such as when backfilling foundations or performing snow removal. Anchor rod locations should be clearly flagged so that they are visible to equipment operators working in the area. The anchor rods shown in Figure 2.3 were damaged because they were covered with snow and the crane operator could not see them.

Fig. 2.3. Anchor rods run over by a crane.

ASTM F1554 permits both cold and hot bending of anchor rods to form hooks; however, bending in the threaded area can be a problem. It is recommended that only Grade 36 rods be bent in the field and the bend limited to 45° or less. Rods up to about 1 in. in diameter can be cold bent. Rods over 1 in. can be heated up to 1,200 °F to make bending easier. It is recommended that bending be done using a rod bending device called a hickey. After bending, the rods should be visually inspected for cracks. If there is concern about the tensile strength of the anchor rod, the rod can be load tested.

2.11.3 Anchor Rod Projection Too Long or Too Short

Anchor rod projections that are too short or too long must be investigated to determine if the correct anchor rods were installed. If the anchor rod is too short, the anchor rod may be projecting below the foundation. If the rod projection is too long, the embedment may not be adequate to develop the required tensile strength.

Often, when the anchor rod is short, it may be possible to partially engage the nut. A conservative estimate of the resulting nut strength can be made based on the percentage of threads engaged, as long as at least half of the threads in the nut are engaged. Welding the nut to the anchor rod is not a prequalified welded joint and is not recommended.

If the anchor rod is too short and the rods are used only for column erection, then the most expedient solution may be to cut or drill another hole in the base plate and install a drilled-in epoxy-type anchor rod. When the rods are designed for tension, the repair may require extending the anchor rod by using a coupling nut or welding on a piece of threaded rod. Figure 2.4 shows a detail of how a coupling nut can be used to extend an anchor rod. This fix will require enlarging the anchor rod hole to accommodate the coupling nut along with using oversize shims to allow the plate washer and nut to clear the coupling nut. Table 2.4 lists the dimensions of typical coupling nuts that can be used to detail the required hole size and plate fillers.

ASTM F1554 Grade 36 anchor rods and ASTM F1554 Grade 55 with supplement S1 anchor rods can be extended by welding on a threaded rod. Butt welding two round rods together requires special detailing that uses a run out tab in order to make a proper groove weld. Figure 2.5a shows a recommended detail for butt welding. The run out tab can be trimmed off after welding, if necessary, and the rod can even be ground flush if required. For more information on welding to anchor rods, see AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2006).

Table 2.4 Hex Coupling Nut Dimensions			
Diameter of Rod, in.	Width Across Flats, in.	Width Across Corners, in.	Height of Nut, in.
¾	1⅛	1⁵⁄₁₆	2¼
⅞	1⁵⁄₁₆	1½	2⅝
1	1½	1¾	3
1¼	1⅞	2³⁄₁₆	3¾
1½	2¼	3⅛	4½
1¾	2¾	3³⁄₁₆	5¼
2	3⅛	3⅝	6
2½	3⅞	4½	7½
Dimensions based on IFI #128 of Industrial Fastener Institute. Material conforms to ASTM A563 Grade A.			

It is also possible to extend an anchor by using splice bars to connect a threaded rod extension. Details similar to that shown in Figure 2.5b will require enlarging the anchor rod hole similar to what is required for the threaded coupler. Either of these welded details can be designed to develop a full-strength splice of the anchor rod.

When anchor rods are too long, it is easy to add plate washers to attain an adequate thread length to run the nut down to the base plate. As noted previously, anchor rod details should always include an extra 3 in. or more of thread beyond what the detail dimension requires to compensate for some variation in anchor rod projection.

2.11.4 Anchor Rod Pattern Rotated 90°

Non-symmetrical anchor rod patterns rotated 90° are very difficult to repair. In special cases, it may be possible to remove the base plate and rotate it to accommodate the anchor rod placement. In most cases, this will require cutting off the anchor rods and installing drilled-in epoxy-type anchors.

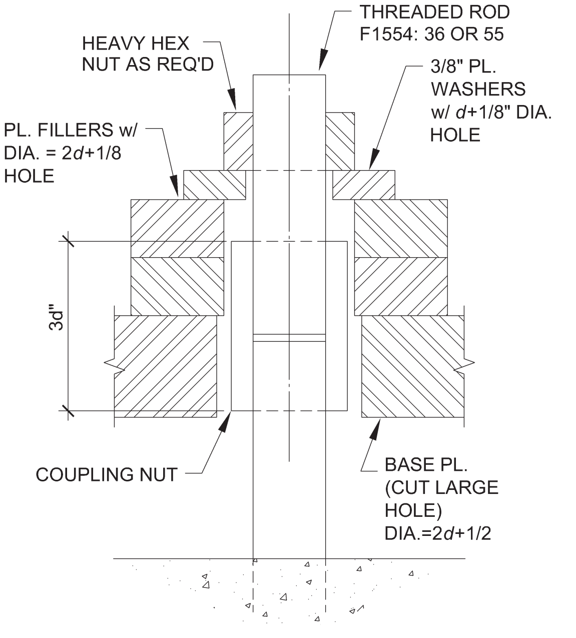


Fig. 2.4. Coupling nut detail for extending anchor rod.

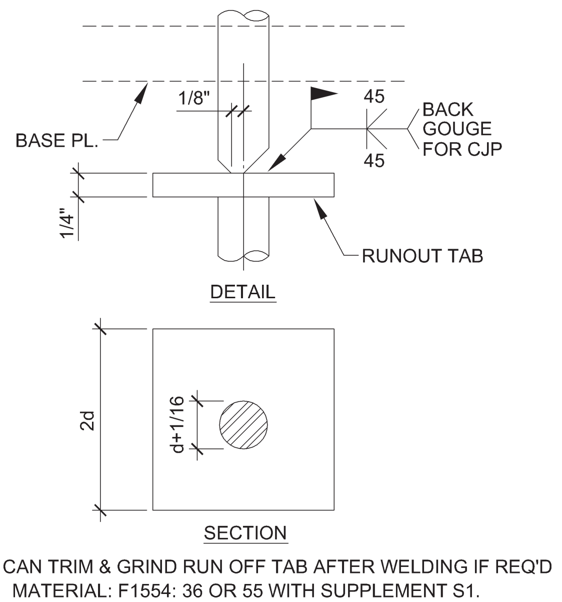


Fig. 2.5a. Groove weld splice.

2.12 Details for Seismic Design

The 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005b), hereafter referred to as the AISC *Seismic Provisions*, govern the design of structural steel members and connections in the seismic load resisting system (SLRS) for buildings and other structures where the seismic response modification coefficient, R, is taken greater than 3, regardless of the seismic design category.

The base plate and anchor rod details for columns that are part of the SLRS must have adequate strength to achieve the required ductile behavior of the frame. Column base strength requirements for columns that are part of the SLRS are given in Section 8.5 of the AISC *Seismic Provisions*. Seismic shear forces are sometimes resisted by embedding the column base and providing for shear transfer into the floor system. Reinforcing steel should be provided around the column to help distribute this horizontal force into the concrete.

The available strength for the concrete elements of column base connections is given in ACI 318-08 (ACI, 2008), Appendix D, except that the special requirements for "regions of moderate or high seismic risk or for structures assigned to intermediate or high seismic performance or design categories" need not be applied. The AISC *Seismic Provisions* Commentary explains that these "special requirements" are not necessary because the required strengths in Sections 8.5a and 8.5b of the AISC *Seismic Provisions* are calculated at higher force levels. The AISC *Seismic Provisions* Commentary Section 8.5 is a recommended source for information on the design of column bases in the SLRS.

Braced frame bases must be designed for the required strength of the elements connected to the base. The column base connection must be designed not only for the required tension and compression strengths of the column, but also for the required strength of the brace connection and base fixity or bending resistance for moments that would occur at the design story drift (inelastic drifts as predicted by the building code). Alternatively, where permitted, the column base may be designed for the amplified forces derived from the load combinations of the applicable building code, including the amplified seismic load.

Moment frame bases can be designed as rigid "fully-restrained (FR) moment connections," true "pinned bases" or, more accurately, as "partially-restrained (PR) moment connections." The intent of the discussion provided in the AISC *Seismic Provisions* regarding this issue is to design this connection consistent with the expected behavior of the joint, accounting for the relative stiffness and strain capability of all elements of the connection (the column, anchor rods, base plate, grout and concrete). Depending on the connection type, the column base either must have adequate strength to maintain the assumed degree of fixity or must be able to provide the required shear strength while allowing the expected rotation to occur. Moment base details shown in Figures 2.6 and 2.7 are from the Commentary to the AISC *Seismic Provisions*.

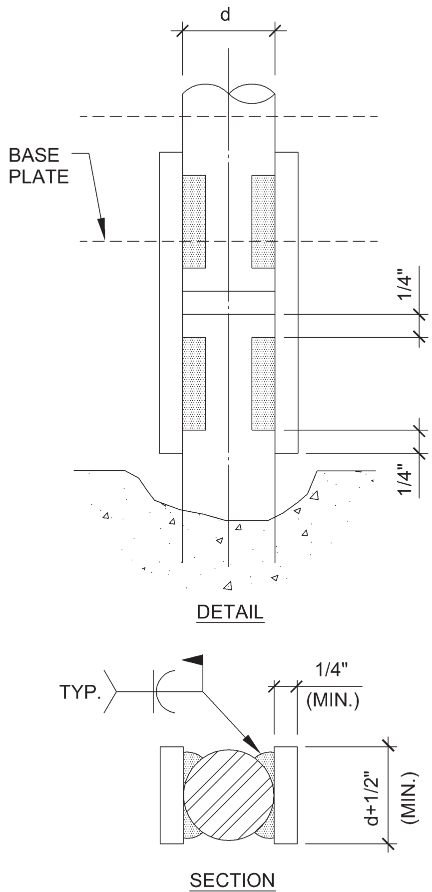


Fig. 2.5b. Lap plate splice.

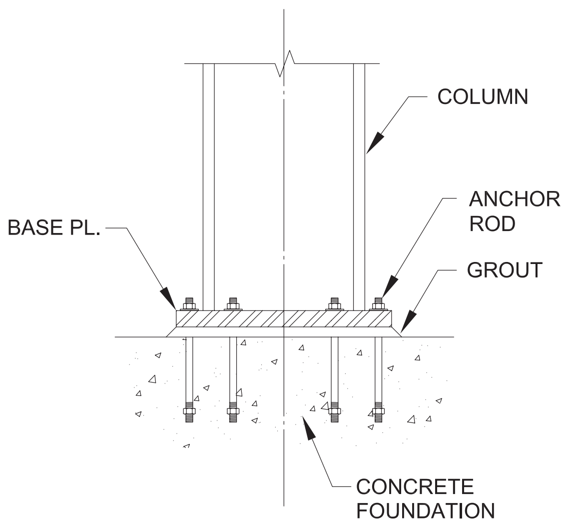


Fig. 2.6. Typical moment base detail.

The base plate connection can be designed using concepts similar to beam-to-column connections. However, the Commentary to the AISC *Seismic Provisions* notes some significant differences:

- Long anchor rods embedded in concrete will strain much more than high-strength bolts or welds in beam-to-column connections.
- Column base plates are bearing on grout and concrete, which is more compressible than the column flanges of the beam-to-column connections.
- Column base connections have significantly more longitudinal load in the plane of the flanges and less transverse load when compared to beam-to-column connections.
- The shear mechanism between the column base and the grout or concrete is different from the shear mechanism between the beam end plate and the column flange.
- AISC-standard hole diameters for column base anchor rods are different than AISC-standard holes for high-strength bolts.
- Foundation rocking and rotation may be an issue, especially on isolated column footings.

As the Commentary to the AISC *Seismic Provisions* suggests, research is lacking regarding the performance and design of base details for high-seismic loading. However, the Commentary also acknowledges that these details are very important to the overall performance of the SLRS. Therefore, careful consideration must be given to the design of these details.

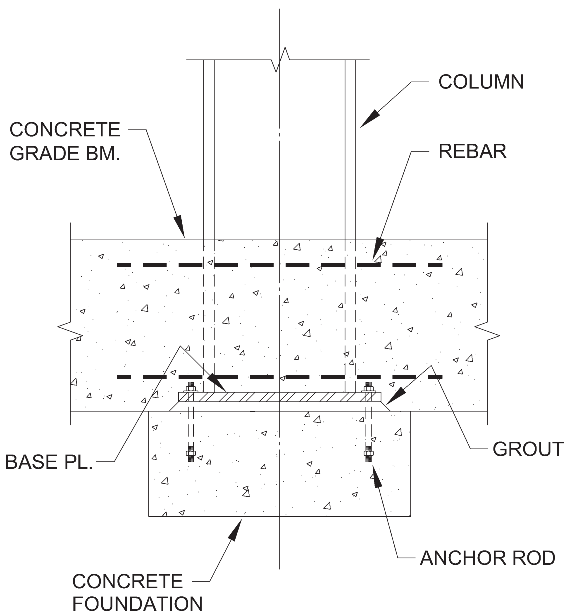


Fig. 2.7. Embedded moment base detail.

3.0 DESIGN OF COLUMN BASE PLATE CONNECTIONS

This section of the Design Guide provides the design requirements for typical column base plate connections in buildings, such as the one shown in Figure 1.1.

Five different design load cases in column base plate connections are discussed:

- Section 3.1 Concentric Compressive Axial Loads
- Section 3.2 Tensile Axial Loads
- Section 3.3 Design of Column Base Plates with Small Moments
- Section 3.4 Design of Base Plates with Large Moments
- Section 3.5 Design for Shear

In column base connections, the design for shear and the design for moment are often performed independently. This assumes there is no significant interaction between them. Several design examples are provided in the following sections for each loading case.

The general behavior and distribution of forces for a column base plate connection with anchor rods will be elastic until either a plastic hinge forms in the column, a plastic mechanism forms in the base plate, the concrete in bearing crushes, the anchor rods yield in tension, or the concrete pullout strength of the anchor-rod group is reached. If the concrete pullout strength of the anchor-rod group is larger than the lowest of the other aforementioned limit states, the behavior generally will be ductile. However, it is not always necessary or even possible to design a foundation that prevents concrete failure.

For example, in statically loaded structures, if the strength is much larger than the demand, the ductility is not necessary and it is acceptable to design with the limit state of tensile or shear strength of the anchor-rod group governing the design. However, frames designed for seismic lateral load resistance are expected to behave in a ductile manner and, in this case, it may be necessary to design the foundation and the column-base-plate connection so that the concrete limit states of tensile or shear strength of the anchor-rod group do not govern the design. See ACI 318-08, Appendix D, Section D3.3.4.

OSHA Requirements

The regulations of the Occupational Safety and Health Administration (OSHA) *Safety Standards for Steel Erection* (OSHA, 2001) require a minimum of four anchor rods in column-base-plate connections. The requirements exclude post-type columns that weigh less than 300 pounds. Columns, base plates, and their foundations must have sufficient moment strength to resist a minimum eccentric gravity load of 300 pounds located 18 in. from the extreme outer face of the column in each direction.

The OSHA criteria can be met with even the smallest of anchor rods on a 4-in. by 4-in. pattern. If one considers only the moments from the eccentric loads (since including the gravity loads results in no tensile force in the anchor rods), and the resisting force couple is taken as the design force of the two bolts times a 4-in. lever arm, the LRFD flexural strength for two ¾-in. A36 anchor rods equals (2)(14.4 kips)(4 in.) = 115 kip-in. For a 14-in.-deep column, the OSHA required moment strength is only (1.6)(0.300 kip)(18 in. + 7 in.) = 12.0 kip-in.

3.1 Concentric Compressive Axial Loads

When a column base resists only compressive column axial loads, the base plate must be large enough to resist the bearing forces transferred from the base plate (Concrete Bearing Limit), and the base plate must be of sufficient thickness (Base Plate Yielding Limit).

3.1.1 Concrete Bearing Limit

The design bearing strength on concrete is defined in ACI 318-08, Section 10.14 as $\phi\left(0.85 f_c' A_1\right)$ when the supporting surface is not larger than the base plate. When the supporting surface is wider on all sides than the loaded area, the design bearing strength above is permitted to be multiplied by $\sqrt{A_2/A_1} \le 2$.

where

A_1 = area of the base plate, in.^{2}

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

AISC *Specification* Section J8 provides the nominal bearing strength, P_p, as follows:

Equation J8-1:

$$P_p = 0.85 f_c' A_1 \text{ on the full area of a concrete support}$$

Equation J8-2:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \text{ on less than the full area of a concrete support}$$

These equations are multiplied by the resistance factor, ϕ_c, for LRFD or divided by the safety factor, Ω_c, for ASD. Section J8 stipulates the ϕ_c and Ω_c factors (in the absence of code regulations) for bearing on concrete as follows:

$$\phi_c = 0.60 \text{ (LRFD)}$$

$$\Omega_c = 2.50 \text{ (ASD)}$$

Alternatively, ACI 318-08 stipulates a resistance factor of 0.65 for bearing on concrete. This apparent conflict exists due to an oversight in the AISC *Specification* development process. The authors recommend the use of the ACI-specified resistance factor in designing column base plates.

The nominal bearing strength can be converted to a stress format by dividing out the area term such that:

On the full area of a concrete support:

$$f_{p(max)} = 0.85 f_c'$$

When the concrete base is larger than the loaded area on all four sides:

$$f_{p(max)} = 0.85 f_c' \sqrt{A_2/A_1} \le 1.7 f_c'$$

The conversion of the generic nominal pressure to an LRFD or ASD available bearing stress is:

$$f_{pu(max)} = \phi_c f_{p(max)} \quad \text{(LRFD)}$$

$$f_{pa(max)} = \frac{f_{p(max)}}{\Omega_c} \quad \text{(ASD)}$$

The concrete bearing strength is a function of the concrete compressive strength, and the ratio of geometrically similar concrete area to base plate area, as indicated in Section 10.14 of ACI 318-08 (ACI, 2008) as follows:

$$f_{p(max)} = \phi\left(0.85 f_c'\right)\sqrt{A_2/A_1}$$

$$\sqrt{A_2/A_1} \le 2$$

where

ϕ = strength reduction factor for bearing, 0.65 per Section 9.3, ACI 318-08

f_c' = specified compressive strength of concrete, ksi

$f_{p(max)}$ = maximum concrete bearing stress, ksi

The increase of the concrete bearing capacity associated with the term $\sqrt{A_2/A_1}$ accounts for the beneficial effects of the concrete confinement. Note that A_2 is the largest area that is geometrically similar to (having the same aspect ratio as) the base plate, and can be inscribed on the horizontal top surface of the concrete footing, pier or beam without going beyond the edges of the concrete.

There is a limit to the beneficial effects of confinement, which is reflected by the limit on A_2 (to a maximum of four times A_1) or by the inequality limit. Thus, for a column base plate bearing on a footing far from edges or openings, $\sqrt{A_2/A_1} = 2$.

The bearing stress on the concrete must not be greater than $f_{p(max)}$:

$$\frac{P_u}{A_1} \le f_{pu(max)} \quad \text{(LRFD)}$$

$$\frac{P_a}{A_1} \le f_{pa(max)} \quad \text{(ASD)}$$

Thus,

$$A_{1(req)} = \frac{P_u}{f_{pu(max)}} \quad \text{(LRFD)}$$

$$A_{1(req)} = \frac{P_a}{f_{pa(max)}} \quad \text{(ASD)}$$

When $A_2 = A_1$, the required minimum base plate area can be determined as:

$$A_{1(req)} = \frac{P_u}{\phi_c 0.85 f_c'} \quad \text{(LRFD)}$$

$$A_{1(req)} = \frac{\Omega_c P_a}{0.85 f_c'} \quad \text{(ASD)}$$

When $A_2 \geq 4A_1$, the required minimum base plate area can be determined as:

$$A_{1(req)} = \frac{1}{2}\left(\frac{P_u}{\phi_c 0.85 f_c'}\right) \quad \text{(LRFD)}$$

$$A_{1(req)} = \frac{1}{2}\left(\frac{\Omega_c P_a}{0.85 f_c'}\right) \quad \text{(ASD)}$$

Many column base plates bear directly on a layer of grout. Because the grout compressive strength is always specified higher than the concrete strength—the authors recommend that the grout strength be specified as two times the concrete strength—it is conservative to use the concrete compressive strength for f_c' in the above equations.

The important dimensions of the column-base plate connection are shown in Figure 3.1.1.

3.1.2 Base Plate Yielding Limit (W-Shapes)

For axially loaded base plates, the required bearing stress under the base plate is assumed uniformly distributed and can be expressed as:

$$f_{pu} = \frac{P_u}{BN} \quad \text{(LRFD)}$$

$$f_{pa} = \frac{P_a}{BN} \quad \text{(ASD)}$$

This bearing pressure causes bending in the base plate at the assumed critical sections shown in Figure 3.1.1(b). This bearing pressure also causes bending in the base plate in the area between the column flanges (Thornton, 1990; Drake and Elkin, 1999). One procedure is presented here to determine the base plate thickness for both situations.

The required strength per inch of the base plate can be determined as:

$$M_{pl} = f_{pu}\left(\frac{l^2}{2}\right) \quad \text{(LRFD)}$$

$$M_{pl} = f_{pa}\left(\frac{l^2}{2}\right) \quad \text{(ASD)}$$

Where the critical base plate cantilever dimension, l, is the larger of m, n, and $\lambda n'$.

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$

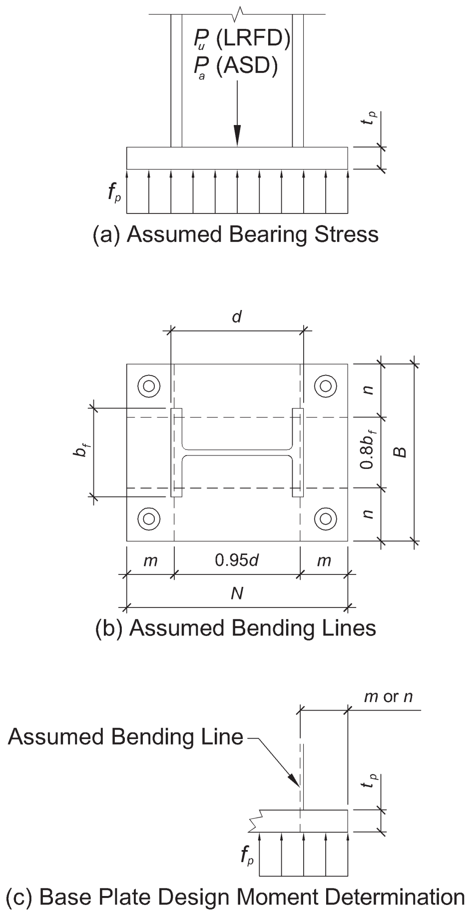


Fig. 3.1.1. Design of base plate with axial compressive load.

where

b_f = column flange width, in.
d = overall column depth, in.
n' = yield-line theory cantilever distance from column web or column flange, in.
B = base plate width, in.
N = base plate length, in.

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \leq 1$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi_c P_p} \qquad \text{(LRFD)}$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{\Omega_c P_a}{P_p} \qquad \text{(ASD)}$$

where

ϕ_c = 0.65
Ω_c = 2.50
P_a = required axial compressive strength (ASD), kips
P_p = nominal strength of concrete under the base plate, kips
= $0.85 f_c' A_1 \sqrt{A_2/A_1}$
P_u = required axial compressive strength (LRFD), kips

It is conservative to take λ as 1.0.

For the yielding limit state, the required minimum thickness of the base plate can be calculated as follows (Thornton, 1990; AISC, 2005d):

$$t_{min} = l\sqrt{\frac{2P_u}{\phi_b F_y BN}} \qquad \text{(LRFD)}$$

$$t_{min} = l\sqrt{\frac{2\Omega_b P_a}{F_y BN}} \qquad \text{(ASD)}$$

where

ϕ_b = resistance factor for bending in LRFD, 0.90
Ω_b = safety factor for bending in ASD, 1.67
F_y = specified minimum yield stress of the base plate, ksi

Since l is the maximum value of m, n and $\lambda n'$, the thinnest base plate can be found by minimizing m, n and λ. This is usually accomplished by proportioning the base plate dimensions so that m and n are approximately equal.

3.1.3 Base Plate Yielding Limit (HSS and Pipe)

For HSS columns, adjustments for m and n must be made (DeWolf and Ricker, 1990). For rectangular HSS, both m and n are calculated using yield lines at 0.95 times the depth and width of the HSS. For round HSS and Pipe, both m and n are calculated using yield lines at 0.8 times the diameter. The $\lambda n'$ cantilever distance is not used for HSS and Pipe.

3.1.4 General Design Procedure

Three general cases exist for the design of base plates subject to axial compressive loads only:

Case I: $A_2 = A_1$
Case II: $A_2 \geq 4A_1$
Case III: $A_1 < A_2 < 4A_1$

The most direct approach is to conservatively set A_2 equal to A_1 (Case I); however, this generally results in the largest base plate plan dimensions. The smallest base plate plan dimensions occur when the ratio of the concrete to base plate area is larger than or equal to 4, i.e., $A_2 \geq 4A_1$ (Case II). Base plates resting on piers often meet the case that A_2 is larger than A_1 but less than $4A_1$, which leads to Case III.

When a base plate bears on a concrete pedestal larger than the base plate dimension, the required minimum base plate area cannot be directly determined. This is because both A_1 and A_2 are unknown.

As mentioned before, the most economical base plates usually occur when m and n, shown in Figure 3.1.1(b), are equal. This situation occurs when the difference between B and N is equal to the difference between $0.95d$ and $0.8b_f$.

In selecting the base plate size from a strength viewpoint the designer must consider the location of the anchor rods within the plate and the clearances required to tighten the nuts on the anchor rods.

Steps for obtaining base plate sizes for the above cases are suggested below. Anchor rod design is covered in Section 3.2.

Case I: $A_2 = A_1$

The largest base plate is obtained when $A_2 = A_1$.

1. Calculate the required axial compressive strength, P_u (LRFD) or P_a (ASD).

2. Calculate the required base plate area.

$$A_{1(req)} = \frac{P_u}{\phi_c 0.85 f_c'} \qquad \text{(LRFD)}$$

$$A_{1(req)} = \frac{\Omega_c P_a}{0.85 f_c'} \qquad \text{(ASD)}$$

3. Optimize the base plate dimensions, N and B.

$$N \approx \sqrt{A_{1(req)}} + \Delta$$

where

$$\Delta = \frac{0.95d - 0.8b_f}{2}$$

then

$$B = \frac{A_{1(req)}}{N}$$

Note that the base plate holes are not deducted from the base plate area when determining the required base plate area. As mentioned earlier in the Guide, from a practical view point select N equal to B.

4. Calculate the required base plate thickness.

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$

where

$\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$

b_f = column flange width, in.
d = overall column depth, in.
n' = yield-line theory cantilever distance from column web or column flange, in.
B = base plate width, in.
N = base plate length, in.

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{P_u}{\phi_c P_p} \quad \text{(LRFD)}$$

$$X = \left(\frac{4db_f}{(d+b_f)^2}\right)\frac{\Omega_c P_a}{P_p} \quad \text{(ASD)}$$

where

$$\phi_c P_p = \phi_c 0.85 f_c' A_1 \quad \text{(LRFD)}$$

$$\frac{P_p}{\Omega_c} = \frac{0.85 f_c' A_1}{\Omega_c} \quad \text{(ASD)}$$

The critical base plate cantilever dimension, l, is the larger of m, n and $\lambda n'$, and the required thickness, t_{min}, is:

$$t_{min} = l\sqrt{\frac{2P_u}{\phi_b F_y BN}} \quad \text{(LRFD)}$$

$$t_{min} = l\sqrt{\frac{2P_a \Omega_b}{F_y BN}} \quad \text{(ASD)}$$

5. Determine the anchor-rod size, and the location of the anchor rods. Anchor rods for gravity columns are generally not required for the permanent structure, and need only to be sized for OSHA requirements, and practical considerations.

Case II: $A_2 \ge 4A_1$

The smallest base plate is obtained when $A_2 \ge 4A_1$ for this case.

1. Calculate the required axial compressive strength, P_u (LRFD) or P_a (ASD).

2. Calculate the required base plate area.

$$A_{1(req)} = \frac{P_u}{2\phi_c 0.85 f_c'} \quad \text{(LRFD)}$$

$$A_{1(req)} = \frac{\Omega_c P_a}{2\left(0.85 f_c'\right)} \quad \text{(ASD)}$$

3. Optimize the base plate dimensions, N and B.

   Use the same procedure as in Step 3 from Case I.

4. Check if sufficient area, A_2 exists for Case II applicability ($A_2 \ge 4A_1$).

   Based on the pier or footing size, it will often be obvious, if the condition is satisfied. If it is not obvious, calculate A_2 geometrically similar to A_1. With new dimensions N_2 and B_2, A_2 then equals N_2B_2. If $A_2 \ge 4A_1$, calculate the required thickness using the procedure shown in Step 4 of Case I, except that

$$\phi_c P_p = 2\phi_c 0.85 f_c' A_1 \quad \text{(LRFD)}$$

$$\frac{P_p}{\Omega_c} = \frac{2(0.85 f_c')A_1}{\Omega_c} \quad \text{(ASD)}$$

5. Determine the anchor rod size and location.

Case III: $A_1 < A_2 < 4A_1$

1. Calculate the required axial compressive strength, P_u(LRFD) or P_a(ASD).

2. Calculate the approximate base plate area based on the assumption of Case III.

$$A_{1(req)} = \frac{P_u}{2\phi_c 0.85 f_c'} \quad \text{(LRFD)}$$

$$A_{1(req)} = \frac{\Omega_c P_a}{2\left(0.85 f_c'\right)} \quad \text{(ASD)}$$

3. Optimize the base plate dimensions, N and B.
   Use the same procedure as in Step 3 from Case I.

4. Calculate A_2, geometrically similar to A_1.

5. Determine whether

$$P_u \leq \phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \qquad \text{(LRFD)}$$

$$P_a \leq \frac{P_p}{\Omega_c} = \left(\frac{0.85 f_c' A_1}{\Omega_c} \right) \sqrt{\frac{A_2}{A_1}} \qquad \text{(ASD)}$$

   If the condition is not satisfied, revise N and B, and retry until criterion is satisfied.

6. Determine the base plate thickness using Step 4, as shown in Case I.

7. Determine the anchor rod size and location.

3.2 Tensile Axial Loads

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

1. Determine the maximum net uplift for the column.

   The maximum net uplift for the column is obtained from the structural analysis of the building for the prescribed building loads. When the uplift due to wind exceeds the dead load of a roof, the supporting columns are subjected to net uplift forces. In addition, columns in rigid bents or braced bays may be subjected to net uplift forces due to overturning.

2. Select the anchor rod material and the number and size of anchor rods required to resist uplift.

   Anchor rods should be specified to conform to the material discussed in Section 2.5. The number of anchor rods required is a function of the maximum net uplift on the column and the strength per rod for the anchor rod material chosen.

   Prying forces in anchor rods are typically neglected. This is usually justified when the base plate thickness is calculated assuming cantilever bending about the web and/or flange of the column section (as described in Step 3 following), and because the length of the rods result in larger deflections than for steel to steel connections. The procedure to determine the required size of the anchor rods is discussed in Section 3.2.1 below.

3. Determine the appropriate base plate size, thickness and welding to transfer the uplift forces.

   Base plate thickness may be governed by bending associated with compressive or tensile loads.

   For tensile loads, a simple approach is to assume the anchor rod loads generate bending moments in the base plate consistent with cantilever action about the web or flanges of the column section (one-way bending). See Figure 3.1.1. If the web is taking the anchor load from the base plate, the web and its attachment to the base plate should be checked. Alternatively, a more refined base plate analysis for anchor rods positioned inside the column flanges can be used to consider bending about both the web and the column flanges (two-way bending). For the two-way bending approach, the derived bending moments should be consistent with compatibility requirements for deformations in the base plate. In either case, the effective bending width for the base plate can be conservatively approximated using a 45° distribution from the centerline of the anchor rod to the face of the column flange or web.

4. Determine the method for developing the strength of the anchor rod in the concrete (i.e., transferring the tension force from the anchor rod to the concrete foundation).

   Methods of determining the required concrete anchorage are treated in Section 3.2.2.

3.2.1 Anchor Rod Tension

The tensile strength of an anchor rod is equal to the strength of the concrete anchorage of the anchor-rod group (or those anchor rods participating in tension in the case of tension due to moment), or the sum of the steel tensile strengths of the contributing anchor rods.

For anchor-rod connections in tension, the design tensile strength of contributing anchor rods is taken as the smallest of the sum of the steel tensile strengths of the contributing individual anchor rods or the concrete tensile strength of the anchor group. Concrete tensile strength and the development length of deformed bars are calculated in accordance with ACI 318 (ACI, 2008).

The limiting tension on a rod is based on the minimum area along the maximum stressed length of that rod. For an anchor rod, this is typically within the threaded portion (except when upset rods are used). ANSI / ASME B1.1 defines this threaded area as:

$$A_{ts} = 0.785 \left(D - \frac{0.974}{n} \right)^2$$

where

n = number of threads per inch
D = major diameter, in.

Table 3.1. ASTM F1554 Anchor Rod (rod only) Available Tensile Strength, kips							
Rod Diameter, in.	Rod Area, A_b, in.^{2}	LRFD ϕR_n $\phi = 0.75$			ASD R_n/Ω $\Omega = 2.00$		
		Grade 36 kips	Grade 55 kips	Grade 105 kips	Grade 36 kips	Grade 55 kips	Grade 105 kips
⅝	0.307	10.0	12.9	21.6	6.68	8.63	14.4
¾	0.442	14.4	18.6	31.1	9.60	12.4	20.7
⅞	0.601	19.6	25.4	42.3	13.1	16.9	28.2
1	0.785	25.6	33.1	55.2	17.1	22.1	36.8
1⅛	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1¼	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1½	1.77	57.7	74.6	124	38.4	49.7	82.8
1¾	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2¼	3.98	130	168	280	86.5	112	186
2½	4.91	160	207	345	107	138	230
2¾	5.94	194	251	418	129	167	278
3	7.07	231	298	497	154	199	331
3¼	8.30	271	350	583	180	233	389
3½	9.62	314	406	677	209	271	451
3¾	11.0	360	466	777	240	311	518
4	12.6	410	530	884	273	353	589

Table 7-18 in the AISC *Steel Construction Manual* (AISC, 2005d) lists the net tensile stress area for diameters between ¼ in. and 4 in.

Two methods of determining the required tensile stress area are commonly used. One is based directly on the ANSI/ASME-stipulated tensile stress area as described above. The other is to add a modifying factor that relates the tensile stress area directly to the unthreaded area as a means of simplifying the design process. The latter method is stipulated in the AISC *Specification*.

The strength of structural fasteners in AISC documents has historically been based on a modifying factor and the nominal bolt diameter, while the direct tensile stress area approach is stipulated in ACI 318-08, Appendix D. The designer should be aware of the differences in these design approaches and stay consistent within one system when determining the required anchor area. However, the calculated strength of a particular anchor analyzed by either method will produce a similar end result.

Strength tables for commonly used anchor rod materials and sizes are easily developed by the procedures that follow, for either design method. Table 3.1 included herein has been developed for ASTM F1554 rods based on the nominal bolt diameter approach of AISC. (Note: ASTM F1554 is the suggested standard and preferred anchor rod material.)

The AISC *Specification* stipulates the nominal tensile strength of an anchor rod as:

$$R_n = 0.75 F_u A_b$$

To obtain the design tensile strength for LRFD, use $\phi = 0.75$, thus

$$\phi R_n = (0.75)(0.75) F_u A_b = 0.563 F_u A_b$$

To obtain the allowable tensile strength for ASD use $\Omega = 2.00$, thus,

$$\frac{R_n}{\Omega} = \frac{0.75}{2.00} F_u A_b = 0.375 F_u A_b$$

ACI 318-08, Appendix D stipulates the design tensile strength of an anchor as:

$$\phi R_n = \phi F_u A_{ts} = 0.75 F_{uta} A_{ts}$$

Table 3.2. Anchor Rod Concrete Pullout Strength, kips					
Rod Diameter, in.	Rod Area, A_b, in.^{2}	Bearing Area, A_{brg}, in.^{2}	Concrete Pullout Strength, ϕN_p		
			f'_c = 3,000 psi	f'_c = 4,000 psi	f'_c = 5,000 psi
⅝	0.307	0.689	11.6	15.4	19.3
¾	0.442	0.906	15.2	20.3	25.4
⅞	0.601	1.22	20.5	27.3	34.1
1	0.785	1.50	25.2	33.6	42.0
1⅛	0.994	1.81	30.4	40.5	50.7
1¼	1.23	2.24	37.7	50.2	62.8
1½	1.77	3.13	52.6	70.1	87.7
1¾	2.41	4.17	70.0	93.4	117
2	3.14	5.35	90.0	120	150
2¼	3.98	6.69	112	150	187
2½	4.91	8.17	137	183	229
2¾	5.94	9.80	165	220	274
3	7.07	11.4	191	254	318
3¼	8.30	13.3	223	297	372
3½	9.62	15.3	257	343	429
3¾	11.0	17.5	294	393	491
4	12.6	19.9	334	445	557

where

ϕ = 0.75 for an anchor governed by strength of a ductile steel element
A_b = nominal bolt area, in.^{2}
A_{ts} = tensile stress area, in.^{2}
F_{uta} = lesser of F_u, $1.9F_y$ and 125 ksi

Shown in Table 3.1 are the design and allowable strengths for various anchor rods based on the 2005 AISC *Specification*.

3.2.2 Concrete Anchorage for Tensile Forces

It is presumed that ASCE 7 (ASCE, 2005) load factors are employed in this Guide. The ϕ factors used herein correspond to those in Appendix D4.4 and Section 9.3 of ACI 318-08.

Appendix D of ACI 318-08 (ACI, 2008) addresses the anchoring to concrete of cast-in or post-installed expansion or undercut anchors. The provisions include limit states for Concrete Pullout, and Breakout Strength following the Concrete Capacity Design (CCD) Method.

Concrete Pullout Strength

ACI concrete pullout strength is based on the ACI 318-08, Appendix D provisions (Section D5.3).

$$\phi N_p = \phi \psi_4 A_{brg} 8 f'_c$$

where

ϕ = 0.70
ψ_4 = 1.4 if the anchor is located in a region of a concrete member where analysis indicates no cracking at service levels, otherwise $\psi_4 = 1.0$
f'_c = specified compressive strength of concrete, psi
A_{brg} = net bearing area of the anchor rod head or nut, in.^{2}
N_p = nominal pullout strength, in.

Shown in Table 3.2 are design pullout strengths for anchor rods with heavy hex heads and nuts. The 40% increase in strength for the no-cracking case has not been included ($\psi_4 = 1.0$). Notice that concrete pullout never controls for anchor rods with $F_y = 36$ ksi, and concrete with $f'_c = 4$ ksi. For higher strength anchor rods, washer plates may be necessary to obtain the full strength of the anchors. The size of the washers should be kept as small as possible to develop the needed concrete strength. Unnecessarily large washers can reduce the concrete resistance to pull out.

Hooked anchor rods can fail by straightening and pulling out of the concrete. This failure is precipitated by a localized bearing failure of the concrete above the hook. A hook is generally not capable of developing the required tensile strength. Therefore, hooks should only be used when tension in the anchor rod is small.

Appendix D of ACI 318-08 provides a pullout strength for a hooked anchor of $\phi\psi_4(0.9\, f_c'\, e_h\, d_o)$, which is based on an anchor with diameter d_o bearing against the hook extension of e_h. ϕ is taken as 0.70. The hook extension, e_h, is limited to a maximum of $4.5d_o$. ψ_4 equals 1 if the anchor is located where the concrete is cracked at service load levels, and ψ_4 equals 1.4 if it is not cracked at service load levels.

Concrete Capacity Design (CCD)

In the CCD method the concrete cone is considered to be formed at an angle of approximately 34° (1 to 1.5 slope). For simplification, the cone is considered to be square rather than round in plan. See Figure 3.2.1.

The concrete breakout stress (f_t in Figure 3.2.1) in the CCD method decreases with an increase in size of the breakout surface. Consequently, the increase in strength of the breakout in the CCD method is proportional to the embedment depth to the power of 1.5 (or to the power of 5/3 for deeper embedments).

According to Appendix D, Section D.4.2.2 of ACI 318-08 (ACI, 2008), the CCD method is valid for anchors with diameters not exceeding 2 in., and tensile embedment length not exceeding 25 in. in depth.

Anchor-rod design for structures subject to seismic loads and designed using a response modification factor, R, greater than 3, should be in accordance with Section 8.5 of the AISC *Seismic Provisions*.

Per ACI 318-08, Appendix D, the concrete breakout strength for a group of cast-in anchors in normal weight concrete is:

$$\phi N_{cbg} = \phi\psi_3 24\sqrt{f_c'}\; h_{ef}^{\,1.5}\,\frac{A_N}{A_{No}} \text{ for } h_{ef} < 11 \text{ in.}$$

$$\phi N_{cbg} = \phi\psi_3 16\sqrt{f_c'}\; h_{ef}^{\,5/3}\,\frac{A_N}{A_{No}} \text{ for 25 in.} \geq h_{ef} \geq 11 \text{ in.}$$

where

ϕ = 0.70 (condition B, i.e., where supplementary reinforcement is not present)

ψ_3 = 1.25 considering the concrete to be uncracked at service loads, otherwise = 1.0

h_{ef} = depth of embedment, in.

A_N = concrete breakout cone area for group, in.^{2}

A_{No} = concrete breakout cone area for single anchor, in.^{2}

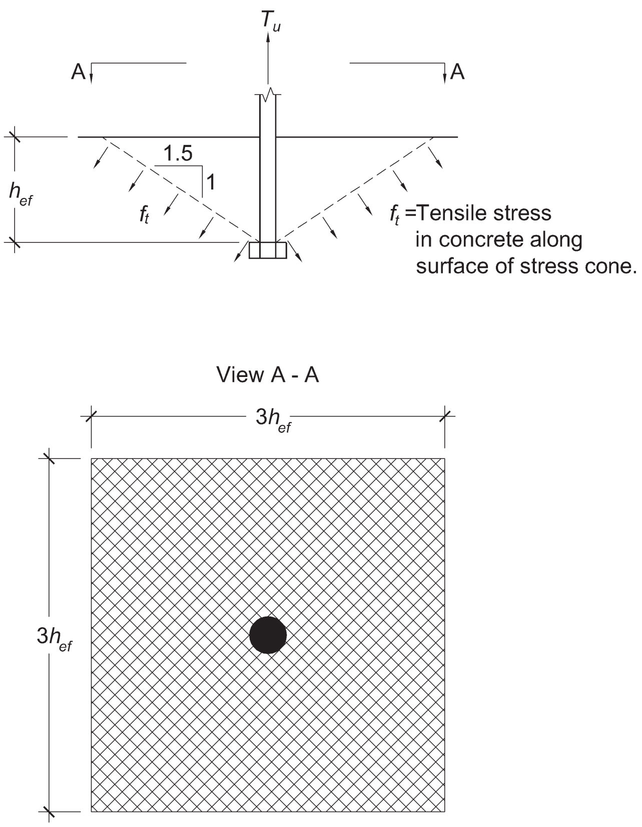


Fig. 3.2.1. Full breakout cone in tension per ACI 318-08.

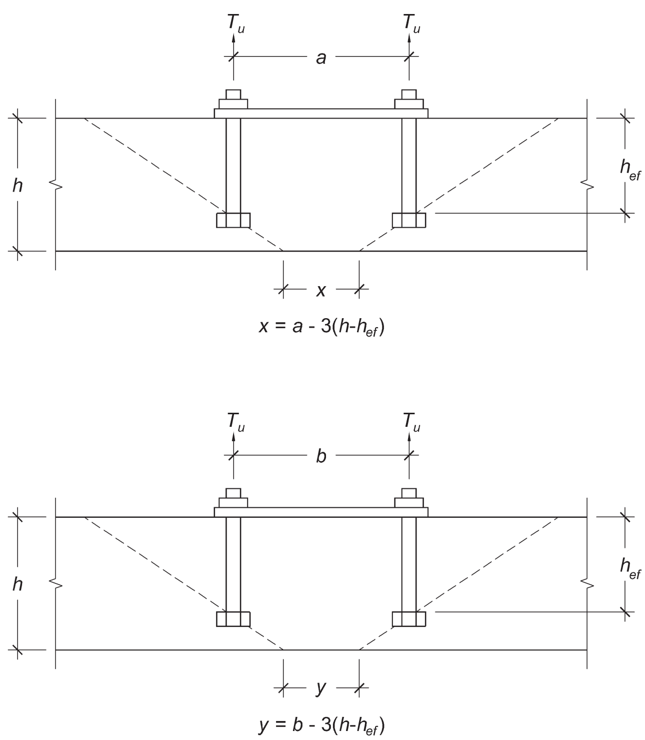


Fig. 3.2.2. Breakout cone for group anchors in thin slab.

Appendix D, Section D.5.4 of ACI 318-08 lists criteria for anchor rods to prevent concrete side-face blowout. These lateral bursting forces are associated with tension in the anchor rods. The failure plane or surface in this case is assumed to be cone shaped and radiating from the anchor head to the adjacent free edge or side of the concrete element. This is illustrated in Figure 3.2.4. It is recommended to use a minimum side cover, c_1, of 6 anchor diameters for anchor rods conforming to ASTM F1554 Grade 36 to avoid problems with side face breakout. As with the pullout stress cones, overlapping of the stress cones associated with these lateral bursting forces is considered in Appendix D of ACI 318-08. Use of washer plates can be beneficial by increasing the bearing area, which increases the side-face blowout strength.

The concrete breakout strengths presented here assume that the concrete is uncracked. The designer should refer to ACI 318-08 to determine if the concrete should be taken as cracked or uncracked. If the concrete is considered cracked, ψ_3 equals 1.0 resulting in 80% of the concrete capacity values for uncracked concrete.

Development by Lapping with Concrete Reinforcement

The extent of the stress cone is a function of the embedment depth, the thickness of the concrete, the spacing between adjacent anchors, and the location of adjacent free edges in the concrete. The shapes of these stress cones for a variety of situations are illustrated in Figures 3.2.1, 3.2.2 and 3.2.3.

The stress cone checks rely upon the strength of plain concrete for developing the anchor rods and typically apply when columns are supported directly on spread footings, concrete mats or pile caps. However, in some instances the projected area of the stress cones or overlapping stress cones is extremely limited due to edge constraints. Consequently, the tensile strength of the anchor rods cannot be fully developed with plain concrete. In general, when piers are used, concrete breakout capacity alone cannot transfer the significant level of tensile forces from the steel column to the concrete base. In these instances, steel reinforcement in the concrete is used to carry the force from the anchor rods. This reinforcement often doubles as the reinforcement required to accommodate the tension and/or bending forces in the pier. The reinforcement must be sized and developed for the required tensile strength of the anchor rods on both sides of the potential failure plane described in Figure 3.2.5.

If an anchor is designed to lap with reinforcement, the anchor strength can be taken as $\phi F_y A_{se}$ as the lap splice length will ensure that ductile behavior will occur. A_{se} is the effective cross-sectional area, which is the tensile stress area for threaded rods. ϕ equals 0.90, as prescribed in Chapter 9 of ACI 318-08.

The anchor rod embedment lengths are determined from the required development length of the spliced reinforcement. Hooks or bends can be added to the reinforcing steel to minimize development length in the breakout cone. See ACI 318-08, Appendix Section D.5.2.9 and commentary for additional discussion on reinforcement (see Figure 3.2.5).

3.3 Design of Column Base Plates with Small Moments

Drake and Elkin (1999) introduced a design approach using factored loads directly in a method consistent with the equations of static equilibrium and the LRFD method. The

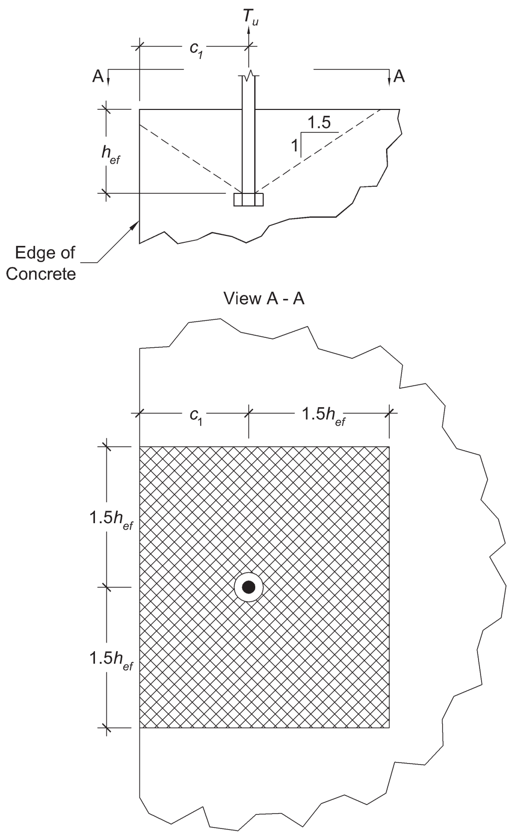


Fig. 3.2.3. Breakout cone in tension near an edge.

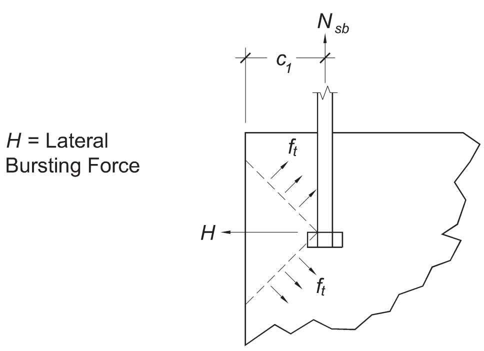


Fig. 3.2.4. Lateral bursting forces for anchor rods in tension near an edge.

procedure was modified by Fisher and Doyle (2005). Drake and Elkin proposed that a uniform distribution of the resultant compressive bearing stress is more appropriate when utilizing LRFD. The design is related to the equivalent eccentricity, e, equal to the moment, M_u, divided by the column axial force, P_u.

For small eccentricities, the axial force is resisted by bearing only with no uplift. For large eccentricities, it is necessary to use anchor rods to resist the uplift. The definition of small and large eccentricities, based on the assumption of uniform bearing stress, is discussed in the following. The variables T_u, P_u and M_u have been changed from the original work by Drake and Elkin to T, P_r and M_r, so that the method is applicable to both LRFD and ASD. A triangular bearing stress approach can also be used, as discussed in Appendix B.

Consider the force diagram shown in Figure 3.3.1. The resultant bearing force is defined by the product qY, in which:

$$q = f_p \times B \tag{3.3.1}$$

where

f_p = bearing stress between the plate and concrete, ksi
B = the base plate width [see Figure 3.1.1(b)], in.

The force acts at the midpoint of bearing area, or $Y/2$ to the left of point A. The distance of the resultant to the right of the centerline of the plate, ε, is therefore:

$$\varepsilon = \frac{N}{2} - \frac{Y}{2} \tag{3.3.2}$$

It is clear that as the dimension Y decreases, ε increases. Y will reach its smallest value when q reaches its maximum:

$$Y_{min} = \frac{P_r}{q_{max}} \tag{3.3.3}$$

where

$$q_{max} = f_{p(max)} \times B \tag{3.3.4}$$

The expression for the location of the resultant bearing force given in Equation 3.3.2 shows that ε reaches its maximum value when Y is minimum. Therefore:

$$\begin{aligned} \varepsilon_{max} &= \frac{N}{2} - \frac{Y_{min}}{2} \\ &= \frac{N}{2} - \frac{P_r}{2q_{max}} \end{aligned} \tag{3.3.5}$$

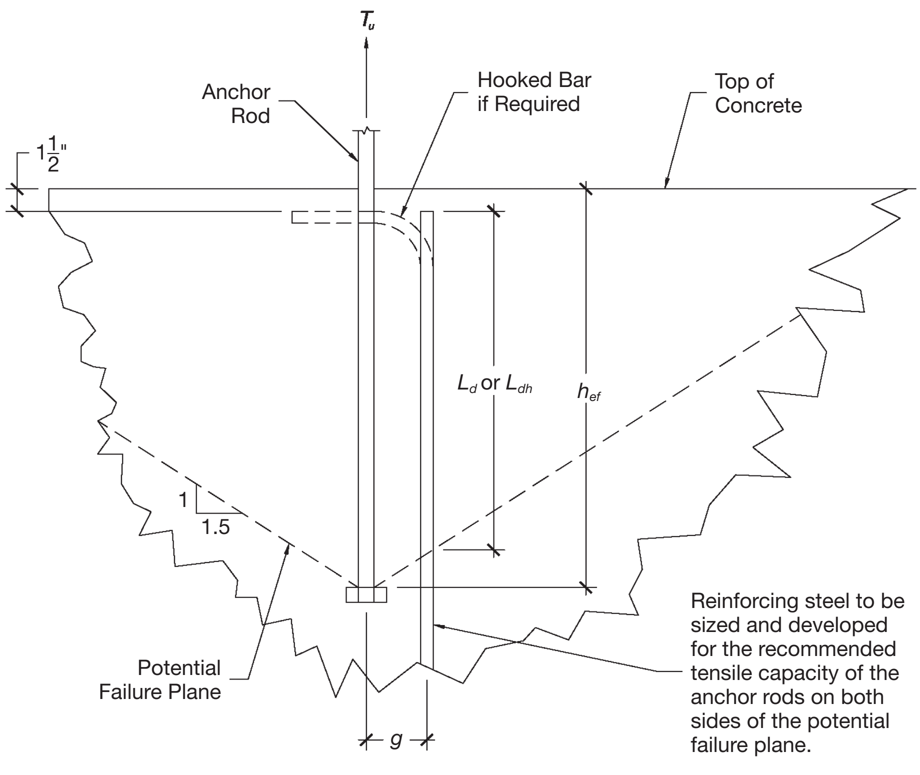


Fig. 3.2.5. The use of steel reinforcement for developing anchor rods.

For moment equilibrium, the line of action of the applied load, P_r, and that of the bearing force, qY, must coincide; that is, $e = \varepsilon$.

If the eccentricity

$$e = \frac{M_r}{P_r} \tag{3.3.6}$$

exceeds the maximum value that ε can attain, the applied loads cannot be resisted by bearing alone and anchor rods will be in tension.

In summary, for values of e less than ε_{max}, Y is greater than Y_{min} and q is less than q_{max}, and obviously, f_p is less than $f_{p(max)}$. For values of e greater than ε_{max}, $q = q_{max}$. Thus, a critical value of eccentricity of the applied load combination is:

$$e_{crit} = \varepsilon_{max} = \frac{N}{2} - \frac{P_r}{2q_{max}} \tag{3.3.7}$$

When analyzing various load and plate configurations, in the case where $e \le e_{crit}$ there will be no tendency to overturn, anchor rods are not required for moment equilibrium, and the force combination will be considered to have a small moment. On the other hand, if $e > e_{crit}$ moment equilibrium cannot be maintained by bearing alone and anchor rods are required. Such combinations of axial load and moment are referred to as large moment cases. The design of plates with large moments is outlined in Section 3.4.

3.3.1 Concrete Bearing Stress

The concrete bearing stress is assumed to be uniformly distributed over the area $Y \times B$. Equation 3.3.2, for the case of $e = \varepsilon$ provides an expression for the length of bearing area, Y:

$$\frac{N}{2} - \frac{Y}{2} = e$$

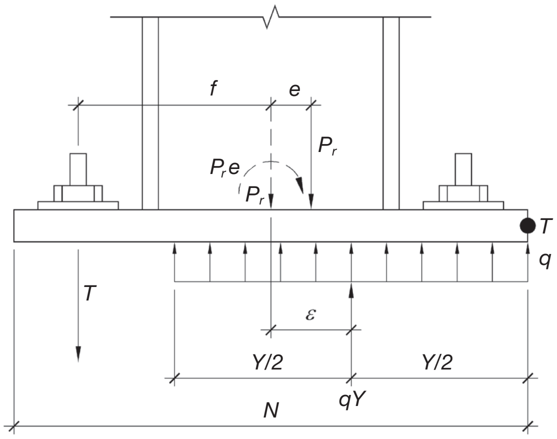


Fig. 3.3.1. Base plate with small moment.

therefore:

$$Y = N - 2e \tag{3.3.8}$$

The bearing stress can then be determined as:

$$q = \frac{P_r}{Y} \text{ ; from which } f_p = \frac{P_r}{BY}$$

For the small moment case, $e \le e_{crit}$. Therefore, as noted above, $q \le q_{max}$. From Equations 3.3.1 and 3.3.4, it follows that $f_p \le f_{p(max)}$.

For the condition $e = e_{crit}$, the bearing length, Y, obtained by use of Equations 3.3.7 and 3.3.8 is:

$$Y = N - 2\left(\frac{N}{2} - \frac{P_r}{2q_{max}}\right) = \frac{P_r}{q_{max}} \tag{3.3.9}$$

3.3.2 Base Plate Flexural Yielding Limit at Bearing Interface

The bearing pressure between the concrete and the base plate will cause bending in the base plate for the cantilever length, m, in the case of strong axis bending and cantilever length, n, in the case of weak axis bending [see Figure 3.1.1(b)]. For the strong axis bending, the bearing stress, f_p (ksi), is calculated as:

$$f_p = \frac{P_r}{BY} = \frac{P_r}{B(N - 2e)} \tag{3.3.10}$$

The required strength per inch of the base plate can then be determined as:

For $Y \ge m$:

$$M_{pl} = f_p\left(\frac{m^2}{2}\right) \tag{3.3.11}$$

For $Y < m$:

$$M_{pl} = f_p Y\left(m - \frac{Y}{2}\right) \tag{3.3.12}$$

where
M_{pl} = plate bending moment per unit width, kip-in.

The nominal bending resistance per unit width of the plate is given by:

$$R_n = \frac{F_y t_p^2}{4}$$

where
F_y = specified yield stress of the plate material, ksi
t_p = plate thickness, in.

The available flexural strength of the plate per unit width is:

$$\phi_b M_n = \phi_b F_y \frac{t_p^2}{4} \quad \text{(LRFD) (3.3.13a)}$$

$$\frac{M_n}{\Omega_b} = \frac{F_y}{\Omega_b}\frac{t_p^2}{4} \quad \text{(ASD) (3.3.13b)}$$

where

ϕ_b = resistance factor in bending = 0.90
Ω_b = safety factor in bending = 1.67

To determine the plate thickness, equate the right-hand sides of Equations 3.3.11 or 3.3.12, and 3.3.13 and solve for $t_{p(req)}$:

For $Y \geq m$:

$$t_{p(req)} = \sqrt{\frac{4\left[f_p\left(\frac{m^2}{2}\right)\right]}{0.90F_y}} = 1.49m\sqrt{\frac{f_p}{F_y}} \quad \text{(LRFD) (3.3.14a-1)}$$

$$t_{p(req)} = \sqrt{\frac{4\left[f_p\left(\frac{m^2}{2}\right)\right]}{F_y/1.67}} = 1.83m\sqrt{\frac{f_p}{F_y}} \quad \text{(ASD) (3.3.14b-1)}$$

For $Y < m$:

$$t_{p(req)} = 2.11\sqrt{\frac{f_p Y\left(m - \frac{Y}{2}\right)}{F_y}} \quad \text{(LRFD) (3.3.15a-1)}$$

$$t_{p(req)} = 2.58\sqrt{\frac{f_p Y\left(m - \frac{Y}{2}\right)}{F_y}} \quad \text{(ASD) (3.3.15b-1)}$$

where

$t_{p(req)}$ = minimum plate thickness, in.

Note: When n is larger than m, the thickness will be governed by n. To determine the required thickness, substitute n for m in Equations 3.3.14a-1, 3.3.14b-1, 3.3.15a-1 and 3.3.15b-1. While this approach offers a simple means of designing the base plate for bending, when the thickness of the plate is controlled by n the designer may choose to use other methods of designing the plate for flexure, such as yield-line analysis, or a triangular pressure distribution assumption, as discussed in Appendix B.

3.3.3 Base Plate Flexural Yielding at Tension Interface

With the moment such that $e \leq e_{crit}$, there will be no tension in the anchor rods and thus they will not cause bending in the base plate at the tension interface. Therefore, bearing at the interface will govern the design of the base plate thickness.

3.3.4 General Design Procedure

1. Determine the axial load and moment.
2. Pick a trial base plate size, $N \times B$.
3. Determine the equivalent eccentricity

$$e = M_r / P_r$$

and the critical eccentricity

$$e_{crit} = \frac{N}{2} - \frac{P_r}{2q_{max}}$$

If $e \leq e_{crit}$, go to next step (design of the base plate with small moment); otherwise, refer to design of the base plate with large moment (Section 3.4).

4. Determine the bearing length, Y.
5. Determine the required minimum base plate thickness, $t_{p(req)}$.
6. Determine the anchor rod size.

3.4 Design of Column Base Plates with Large Moments

When the magnitude of the bending moment is large relative to the column axial load, anchor rods are required to connect the base plate to the concrete foundation so that the base does not tip, nor fail the concrete in bearing at the compressed edge. This is a common situation for rigid frames designed to resist lateral earthquake or wind loadings and is schematically presented in Figure 3.4.1.

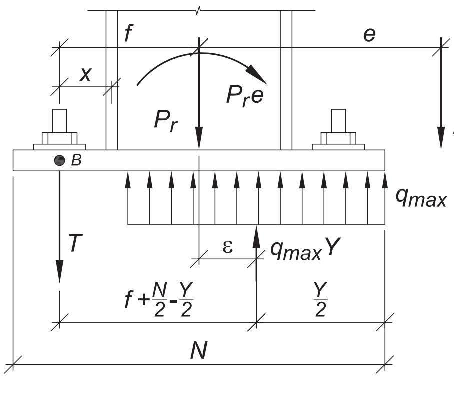


Fig. 3.4.1. Base plate with large moment.

As discussed in the previous section, large moment conditions exist when:

$$e > e_{crit} = \frac{N}{2} - \frac{P_r}{2q_{max}} \tag{3.4.1}$$

3.4.1 Concrete Bearing and Anchor Rod Forces

The bearing pressure, q, is equal to the maximum value, q_{max}, for eccentricities greater than e_{crit}. In order to calculate the total concrete bearing force and the anchor rod forces, consider the force diagram shown in Figure 3.4.1.

Vertical force equilibrium requires that:

$$\sum F_{vertical} = 0$$

$$T = q_{max}Y - P_r \tag{3.4.2}$$

where T equals the anchor rod required tensile strength.

Also, the summation of moments taken about Point B must equal zero. Hence:

$$q_{max}Y\left(f + \frac{N}{2} - \frac{Y}{2}\right) - P_r(e+f) = 0$$

After rearrangement, a quadratic equation for the bearing length, Y, is obtained:

$$Y^2 - 2\left(f + \frac{N}{2}\right)Y + \frac{2P_r(e+f)}{q_{max}} = 0$$

and the solution for Y is:

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e+f)}{q_{max}}} \tag{3.4.3}$$

The concrete bearing force is given by the product $q_{max}Y$. The anchor rod tensile force, T, is obtained by solving Equation 3.4.2.

For certain force, moment and geometry combinations, a real solution of Equation 3.4.3 is not possible. In that case, an increase in plate dimensions is required. In particular, only if the following holds:

$$\left(f + \frac{N}{2}\right)^2 \geq \frac{2P_r(e+f)}{q_{max}} \tag{3.4.4}$$

will the quantity under the radical in Equation 3.4.3 be positive or zero and provide a real solution. If the expression in Equation 3.4.4 is not satisfied, a larger plate is required.

3.4.2 Base Plate Yielding Limit at Bearing Interface

For the case of large moments, the bearing stress is at its limiting value, i.e., $f_p = f_{p(max)}$. The required plate thickness may be determined from either Equations 3.3.14a-2 and 3.3.14b-2 or 3.3.15a-2 and 3.3.15b-2.

If $Y \geq m$

$$t_{p(req)} = 1.49m\sqrt{\frac{f_{p(max)}}{F_y}} \quad \text{(LRFD) (from 3.3.14a-2)}$$

$$t_{p(req)} = 1.83m\sqrt{\frac{f_{p(max)}}{F_y}} \quad \text{(ASD) (from 3.3.14b-2)}$$

If $Y < m$

$$t_{p(req)} = 2.11\sqrt{\frac{f_{p(max)}Y\left(m - \frac{Y}{2}\right)}{F_y}} \quad \text{(LRFD) (from 3.3.15a-2)}$$

$$t_{p(req)} = 2.58\sqrt{\frac{f_{p(max)}Y\left(m - \frac{Y}{2}\right)}{F_y}} \quad \text{(ASD) (from 3.3.15b-2)}$$

Note: When n is larger than m, the thickness will be governed by n. To determine the required thickness, substitute n for m in Equations 3.3.14a-2, 3.3.14b-2, 3.3.15a-2 and 3.3.15b-2.

3.4.3 Base Plate Yielding Limit at Tension Interface

The tension force T_u (LRFD) and T_a (ASD) in the anchor rods will cause bending in the base plate. Cantilever action is conservatively assumed with the span length equal to the distance from the rod centerline to the center of the column flange, x. Alternatively, the bending lines could be assumed as shown in Figure 3.1.1. For a unit width of base plate, the required bending strength of the base plate can be determined as:

$$M_{pl} = \frac{T_u x}{B} \quad \text{(LRFD)} \tag{3.4.5a}$$

$$M_{pl} = \frac{T_a x}{B} \quad \text{(ASD)} \tag{3.4.5b}$$

where

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{3.4.6}$$

with:

d = depth of wide flange column section (See Figure 3.1.1), in.

t_f = column flange thickness, in.

The available flexural strength per unit length for the plate is given in Equation 3.3.13. Setting that strength equal to the applied moment given by Equation 3.4.5 provides an expression for the required plate thickness:

$$t_{p(req)} = 2.11\sqrt{\frac{T_u x}{BF_y}} \text{ (LRFD)} \tag{3.4.7a}$$

$$t_{p(req)} = 2.58\sqrt{\frac{T_a x}{BF_y}} \text{ (ASD)} \tag{3.4.7b}$$

3.4.4 General Design Procedure

1. Determine the axial load and moment.

2. Pick a trial base plate size, $N \times B$.

3. Determine the equivalent eccentricity

$$e = M_r / P_r$$

and the critical eccentricity.

$$e_{crit} = \frac{N}{2} - \frac{P_r}{2q_{max}}$$

If $e > e_{crit}$, go to next step (design of the base plate with large moment); otherwise, refer to design of the base plate with small moment described in Section 3.3.

Check the inequality of Equation 3.4.4. If it is not satisfied, choose larger plate dimensions.

4. Determine the equivalent bearing length, Y, and tensile force in the anchor rod, T_u (LRFD) and T_a (ASD).

5. Determine the required minimum base plate thickness, $t_{p(req)}$, at the bearing and tension interfaces. Choose the larger value.

6. Determine the anchor rod size.

3.5 Design for Shear

There are three principal ways of transferring shear from column base plates into concrete:

1. Friction between the base plate and the grout or concrete surface

2. Bearing of the column and base plate, and/or shear lug, against a concrete surface

3. Shear in the anchor rods

3.5.1 Friction

In typical base plate situations the compression force between the base plate and the concrete will usually develop shear resistance sufficient to resist the lateral forces. The contribution of the shear should be based on the most unfavorable arrangement of required compressive loads, P_u, that is consistent with the lateral force being evaluated, V_u. The shear strength can be calculated in accordance with the following, based on ACI 318-08 and ACI 349-06 Appendix D criteria,

$$\phi V_n = \phi \mu P_u \le (\phi 0.2 f_c' A_c \text{ or } \phi 800 A_c \text{, whichever is smaller})$$

For friction between steel base plates and concrete a μ value of 0.4 is given in ACI 349-06, Appendix D. As an upper limit on the design shear strength, ACI 349-06, Section 11.7.5 indicates that ϕV_n shall not exceed $\phi 0.2 f_c' A_c$ or $\phi 800 A_c$, whichever is smaller, where ϕ is taken as 0.75. Only LRFD requirements are addressed in the ACI documents. There are no comparable ASD provisions.

3.5.2 Bearing

Shear forces can be transferred in bearing by the use of shear lugs or by embedding the column in the foundation. These methods are illustrated in Figure 3.5.1.

When shear lugs are used, Appendix D of ACI 349-06 (ACI, 2006) permits use of confinement in combination with bearing for transferring shear from shear lugs into the concrete. The commentary to ACI 349-06 suggests this mechanism is developed as follows:

1. Shear is initially transferred through the anchor rods to the grout or concrete by bearing augmented by shear resistance from confinement effects associated with tension anchors and external concurrent axial load.

2. Shear then progresses into a shear-friction mode.

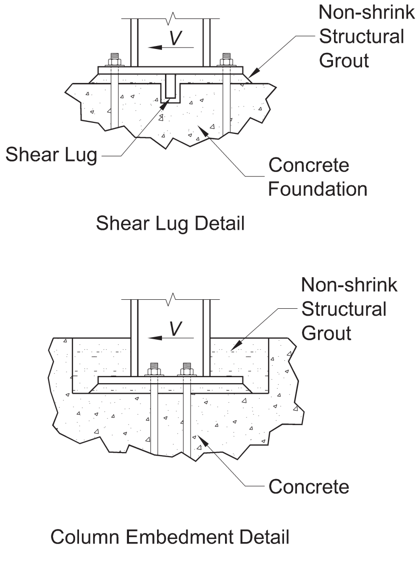


Fig. 3.5.1. Transfer of base shear through bearing.

The recommended bearing limit, ϕP_{ubrg}, per Appendix Section D.4.6.2 of ACI 349-06, is $\phi 1.3 f'_c A_l$. Using a ϕ consistent with ASCE 7 load factors ($\phi = 0.65$), $\phi P_{ubrg} \approx 0.85 f'_c A_l$, where A_l = embedded area of the shear lug (this does not include the portion of the lug in contact with the grout above the pier). Only LRFD requirements are addressed in the ACI documents. There are no comparable ASD provisions.

For bearing against an embedded base plate or column section where the bearing area is adjacent to the concrete surface, ACI 318-08 recommends that $\phi P_{ubrg} = 0.55 f'_c A_{brg}$. A_{brg} is the contact area between the base plate and/or column against the concrete.

According to the Commentary Section RD.11.1 of Appendix D of ACI 349-06, the anchorage shear strength due to confinement can be taken as $\phi K_c(N_y\text{-}P_a)$, with ϕ equal to 0.85, where N_y is the yield strength of the tension anchors equal to $nA_{se}F_y$, and P_a is the factored external axial load on the anchorage; P_a is positive for tension and negative for compression. This shear strength due to confinement considers the effect of the tension anchors and external loads acting across the initial shear fracture planes. When P_a is negative, one must verify that P_a will actually be present while the shear force is occurring. The value of K_c is 1.6 for inset base plates without shear lugs, or for anchorage with multiple shear lugs of height h and spacing s (clear distance face-to-face between shear lugs) less than or equal to $0.13h\sqrt{f'_c}$. For anchorage with a single shear lug located a distance h or greater from the front edge of the base plate, K_c is 1.8.

In summary, the lateral resistance can be expressed as

$\phi P_n = 0.85 f'_c A_l + 1.36(N_y\text{-}P_a)$ when multiple shear lugs are used and

$\phi P_n = 0.55 f'_c A_{brg} + 1.36(N_y\text{-}P_a)$ for bearing on a column or the side of a base plate

If the designer wishes to use shear-friction strength as well, the provisions of ACI 349-06 can be followed. Additional comments related to the use of shear lugs are provided below:

1. For shear lugs or column embedments bearing in the direction of a free edge of the concrete, Appendix D of ACI 349-06 states that in addition to considering bearing failure in the concrete, "the design shear strength for each lug or plate edge shall be determined based on a uniform tensile stress of $4\phi\sqrt{f'_c}$ acting on an effective stress area defined by projecting a 45° plane from bearing edges of the shear lug or base plate to the free surface." The bearing area of the shear lug (or column embedment) is to be excluded from the projected area. Use ϕ equal to 0.75. This criterion may control or limit the shear strength of the shear lug or column embedment details in concrete piers.

2. Consideration should be given to bending in the base plate resulting from forces in the shear lug. This can be of special concern when the base shears (most likely due to bracing forces) are large and bending from the force on the shear lug is about the weak axis of the column. As a rule of thumb, the authors recommend that the base plate should be of equal or greater thickness than the shear lug.

3. Multiple shear lugs may be used to resist large shear forces. Appendix D of ACI 349-06 provides criteria for the design and spacing of multiple shear lugs.

4. Grout pockets must be of sufficient size for ease of grout placement. Non-shrink grout of flowable consistency should be used.

The design of a shear lug is illustrated in Example 4.9.

3.5.3 Shear in Anchor Rods

It should be noted that the use of anchor rods to transfer shear forces must be carefully examined due to several assumptions that must be made. Particular attention must be paid to the manner in which the force is transferred from the base plate to the anchor rods.

Using the AISC-recommended hole sizes for anchor rods, which can be found in Table 2.3, considerable slip of the base plate may occur before the base plate bears against the anchor rods. The effects of this slip must be evaluated by the engineer. The reader is also cautioned that, due to placement tolerances, it is likely that not all of the anchor rods will receive the same force.

The authors recommend a cautious approach, such as using only two of the anchor rods to transfer the shear, unless special provisions are made to equalize the load to all anchor rods (Fisher, 1981). Lateral forces can be transferred equally to all anchor rods, or to selective anchor rods, by using a plate washer welded to the base plate between the anchor rod nut and the top of the base plate. The plate washers should have holes 1⁄16 in. larger than the anchor rod diameter. Alternatively, to transfer the shear equally to all anchor rods, a setting plate of proper thickness can be used and then field welded to the base plate after the column is erected. It cannot be emphasized enough that the use of shear in the anchor rods requires attention in the design process to the construction issues associated with column bases.

Once the shear is delivered to the anchor rods, the shear must be transferred into the concrete. If plate washers are used to transfer shear to the rods, some bending of the anchor rods can be expected within the thickness of the base plate. If only two anchor rods are used for shear transfer, as suggested previously, the shear is transferred within the base plate and bending of the rods can be neglected. Based on bending friction theory, no bending of the anchor rod within the grout need be considered. The moment in the anchor rods can be determined by assuming reverse curvature bending. The lever arm can be taken as the half distance between the center of bearing of the plate washer to the top of the grout surface. Where anchors are used with a built-up grout pad, ACI 318-08 requires that the anchor capacity be multiplied by 0.8. No explanation of the reduction is provided; however, it is the authors' understanding that the requirement is to adjust the strength to account for bending of the anchor rods within the grout pad. Limitations on grout pad thicknesses are not provided. It is the authors' opinion that the reduction is not required when AISC combined bending and shear checks are made on the anchor rods, and the resulting area of the anchor rod is 20% larger than the rod without bending.

Appendix D of ACI 318-08 employs the CCD method to evaluate the concrete breakout strength from shear forces re-sisted by anchor rods.

For the typical cast-in-place anchor group used in building construction, the shear strength determined by concrete breakout as illustrated in Figure 3.5.2 is evaluated as

$$\phi V_{cbg} = \phi \frac{A_v}{A_{vo}} \psi_5 \psi_6 \psi_7 V_b \text{, kips}$$

where

ϕ = 0.70

ψ_5 = 1 (all anchors at same load)

ψ_7 = 1.4 (uncracked or with adequate supplementary reinforcement)

$V_b = 7\left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f'_c}\, c_1^{1.5}$ for normal weight concrete

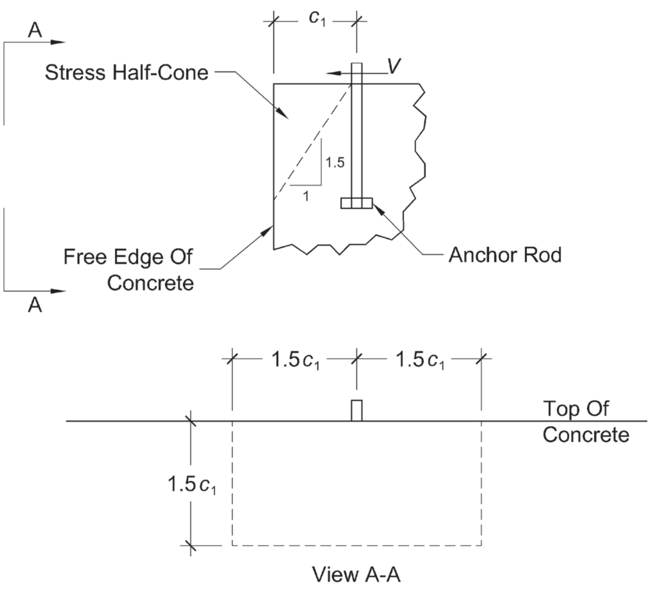


Fig. 3.5.2. Concrete breakout cone for shear.

c_1 = the edge distance (in.) in the direction of load as illustrated in Figure 3.5.2

f'_c = concrete compressive strength, psi

ℓ = embedment depth, in.

d_o = rod diameter, in.

Typically $\frac{\ell}{d_o}$ becomes 8 since the load bearing length is limited to $8d_o$.

Substituting,

$$\phi V_{cbg} = 10.4 \frac{A_v}{A_{vo}} \psi_6 \sqrt{d_o}\sqrt{f'_c}\, c_1^{1.5}$$

where

ψ_6 = a modifier to reflect the capacity reduction when side cover limits the size of the breakout cone

A_{vo} = $4.5c_1^2$ (the area of the full shear cone for a single anchor as shown in View A-A of Figure 3.5.2), in.^{2}

A_v = total breakout shear area for a single anchor, or a group of anchors, in.^{2}

It is recommended that the rod diameter, d_o, used in the square root term of the V_b expression, be limited to a maximum of 1.25 inches, based on research results conducted at the University of Stuttgart. If the edge distance c_1 is large enough, then the anchor rod shear strength will govern. The nominal shear strength of a single anchor rod equals $0.4F_uA_b$, if the threads are not excluded from the shear plane, and $0.5F_uA_b$, if the threads are excluded; $\phi = 0.75$ and $\Omega = 2.00$.

In evaluating the concrete breakout strength, the breakout either from the most deeply embedded anchors or breakout on the anchors closer to the edge should be checked. When breakout is being determined on the outer two anchors (those closest to the concrete edge) the inner two anchors (those farthest from the concrete edge) should be considered to carry the same load as the outer two anchors. When the concrete breakout is considered from the inner two anchors, all of the shear is to be taken by the inner anchors. Shown in Figure 3.5.3 are the two potential breakout surfaces and an indication of which will control, based on anchor location relative to the edge distance.

In many cases it is necessary to use reinforcement to anchor the breakout cone in order to achieve the shear strength as well as the ductility desired. Ties placed atop piers as required in Section 7.10.5.6 of ACI 318-08 can also be used structurally to transfer the shear from the anchors to the piers.

In addition to the concrete breakout strength, ACI 318-08 also contains provisions for a limit state called pryout strength. The authors have checked several common situations and have not found pryout strength to control for typical anchor rod designs. ACI 318-08 defines the pryout strength of a single anchor in shear as:

$$\phi V_{cp} = \phi k_{cp} N_{cb}$$

where

ϕ = 0.70

N_{cb} = nominal concrete breakout strength in tension of a single anchor, kips

h_{ef} = effective anchor embedment length, in.

k_{cp} = 1.0 for $h_{ef} < 2.5$ in.

= 2.0 for $h_{ef} \geq 2.5$ in.

3.5.4 Interaction of Tension and Shear in the Concrete

When the concrete is subjected to a combination of pullout and shear, ACI 318-08, Appendix D uses an interaction equation solution. The reader is referred to ACI for further explanation.

3.5.5 Hairpins and Tie Rods

To complete the discussion on anchorage design, transfer of shear forces to reinforcement using hairpins or tie rods will be addressed. Hairpins are typically used to transfer load to the floor slab. The friction between the floor slab and the subgrade is used in resisting the column base shear when individual footings are not capable of resisting horizontal forces. The column base shears are transferred from the anchor rods to the hairpin (as shown in Figure 3.5.4) through bearing. Problems have occurred with the eccentricity between the base plate and the hairpin due to bending in the anchor rods after the friction capacity is exceeded. This problem can be avoided as shown in Figure 3.5.5 or by providing shear lugs. Since hairpins rely upon the frictional restraint provided by the floor slab, special consideration should be given to the location and type of control and construction joints used in the floor slab to assure no interruption in load transfer, yet still allowing the slab to move. In addition, a vapor barrier should not be used under the slab.

Tie rods (continuous rods that run through the slab to the opposite column line) are typically used to counteract large shear forces associated with gravity loads on rigid frame structures. When using tie rods with large clear span rigid frames, consideration should be given to elongation of the tie rods and to the impact of these elongations on the frame analysis and design. In addition, significant amounts of sagging or bowing should be removed before tie rods are encased or covered, since the tie rod will tend to straighten when tensioned.

Tie rods and hairpin bars should be placed as close to the top surface of the concrete slab as concrete cover requirements allow.

4.0 DESIGN EXAMPLES

4.1 Example: Base Plate for Concentric Axial Compressive Load (No Concrete Confinement)

A W12×96 column bears on a 24 in. × 24 in. concrete pedestal. The minimum concrete compressive strength is $f'_c = 3$ ksi, and the base plate yield stress is $F_y = 36$ ksi. Determine the base plate plan dimensions and thickness for the given required strength, using the assumption that $A_2 = A_1$ (Case I).

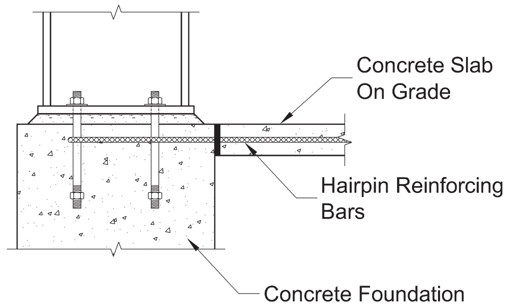


Fig. 3.5.4. Typical detail using hairpin bars.

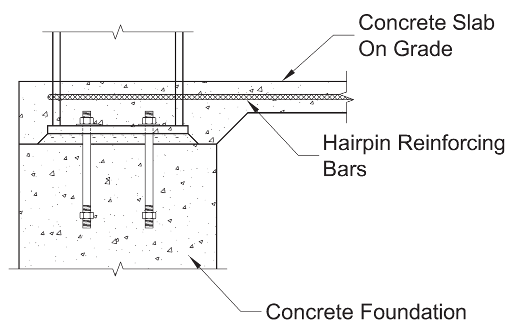
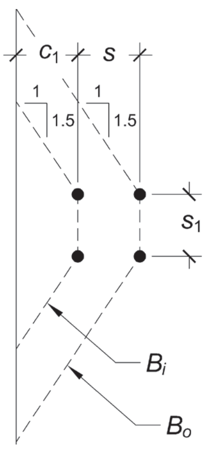


Fig. 3.5.5. Alternate hairpin detail.



s_1/s	c_1/s For B_o To Control
0.5	> 2.33
2/3	> 2.31
1.0	> 2.26
1.5	> 2.19
2.0	> 2.12

Fig. 3.5.3. Concrete breakout surfaces for group anchors.

1. The required strength due to axial loads.

LRFD	ASD
$P_u = 700$ kips	$P_a = 430$ kips

2. Calculate the required base plate area.

LRFD	ASD
$A_{1(req)} = \dfrac{P_u}{\phi_c 0.85 f_c'} = \dfrac{700\text{ kips}}{(0.65)(0.85)(3\text{ ksi})} = 422\text{ in.}^2$	$A_{1(req)} = \dfrac{\Omega_c P_u}{0.85 f_c'} = \dfrac{(2.50)(430\text{ kips})}{(0.85)(3\text{ ksi})}\sqrt{f_c'} = 422\text{ in.}^2$

Note: Throughout these examples a resistance factor for bearing on concrete of $\phi_c = 0.65$ has been applied, per ACI 318-08. This resistance factor is more liberal than the resistance factor of $\phi_c = 0.60$ presented in the 2005 AISC *Specification*. Although it was intended that the AISC provision would match the ACI provision, this deviation was overlooked. As both documents are consensus standards endorsed by the building code, and ACI 318-08 has been adopted by reference into the 2005 AISC *Specification for Structural Steel Buildings*, the authors consider a ϕ factor of 0.65 appropriate for use in design. However, ACI 318 is written using strength design only, and does not publish an equivalent Ω factor. Therefore, $\Omega_c = 2.50$ has been used in the ASD calculations presented here, to remain consistent with the value published in the AISC *Specification*.

3. Optimize the base plate dimensions, N and B.

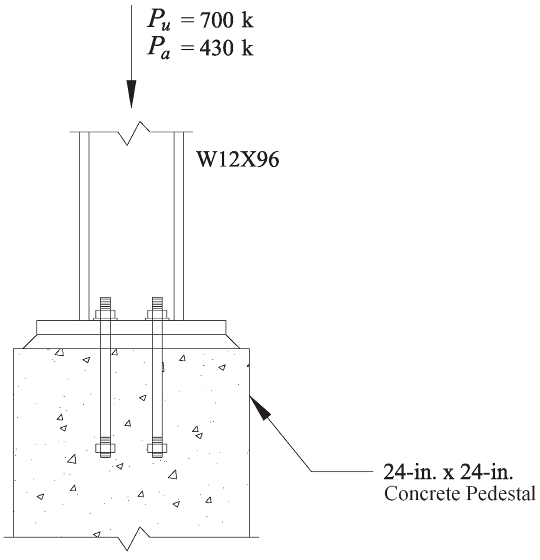


Fig. 4.1.1. Example 4.1.

$$\Delta = \frac{0.95d - 0.8b_f}{2}$$

$$= \frac{0.95(12.7\text{ in.}) - 0.8(12.2\text{ in.})}{2}$$

$$= 1.15\text{ in.}$$

$$N \approx \sqrt{A_{1(req)}} + \Delta$$

$$\approx \sqrt{422\text{ in.}^2} + 1.15\text{ in.}$$

$$\approx 21.7\text{ in.}$$

Try $N = 22$ in.
$B = 422\text{ in.}^2/22\text{ in.} = 19.2$ in.

Try $B = 20$ in.
$A_1 = (22\text{ in.})(20\text{ in.}) = 440\text{ in.}^2 > 422\text{ in.}^2$

4. Check axial compressive strength of the concrete.

LRFD	ASD
$P_u \le \phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\dfrac{A_2}{A_1}} = (0.65)(0.85)(3\text{ ksi}) \times (440\text{ in.}^2)\sqrt{\dfrac{440\text{ in.}^2}{440\text{ in.}^2}}$ = 729 kips > 700 kips **o.k.**	$P_a \le \dfrac{P_p}{\Omega_c} = \dfrac{0.85 f_c' A_1 \sqrt{\dfrac{A_2}{A_1}}}{\Omega_c} = \dfrac{(0.85)(3\text{ ksi})(440\text{ in.}^2)\sqrt{\dfrac{440\text{ in.}^2}{440\text{ in.}^2}}}{2.50}$ = 449 kips > 430 kips **o.k.**

5. Calculate required base plate thickness.

$$m = \frac{N - 0.95d}{2}$$

$$= \frac{22\text{ in.} - 0.95(12.7\text{ in.})}{2}$$

$$= 4.97\text{ in.}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$= \frac{20.0\text{ in.} - 0.8(12.2\text{ in.})}{2}$$

$$= 5.12\text{ in.}$$

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{P_u}{\phi P_p} = \left[\dfrac{4(12.7\text{ in.})(12.2\text{ in.})}{(12.7\text{ in.} + 12.2\text{ in.})^2}\right]\dfrac{700\text{ kips}}{729\text{ kips}} = 0.960$	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{\Omega P_a}{P_p} = \left[\dfrac{4(12.7\text{ in.})(12.2\text{ in.})}{(12.7\text{ in.} + 12.2\text{ in.})^2}\right]\dfrac{430\text{ kips}}{449\text{ kips}} = 0.960$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$$
$$= \frac{2\sqrt{0.960}}{1+\sqrt{1-0.960}}$$
$$= 1.63 \quad \Rightarrow \quad 1$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$
$$= (1)\frac{\sqrt{(12.7\text{ in.})(12.2\text{ in.})}}{4}$$
$$= 3.11\text{ in.}$$

$$l = \max(m, n, \lambda n')$$
$$= \max(4.97\text{ in.},\ 5.12\text{ in.},\ 3.11\text{ in.})$$
$$= 5.12\text{ in.}$$

LRFD	ASD
$t_{min} = l\sqrt{\frac{2P_u}{\phi_b F_y BN}}$ $= (5.12\text{ in.})\sqrt{\frac{(2)(700\text{ kips})}{(0.90)(36\text{ ksi})(20\text{ in.})(22\text{ in.})}}$ $= 1.60\text{ in.}$ Use $t_p = 1\frac{3}{4}$ in.	$t_{min} = l\sqrt{\frac{2\Omega_b P_a}{F_y BN}}$ $= (5.12\text{ in.})\sqrt{\frac{(2)(1.67)(430\text{ kips})}{(36\text{ ksi})(20\text{ in.})(22\text{ in.})}}$ $= 1.54\text{ in.}$ Use $t_p = 1\frac{3}{4}$ in.

6. Determine the anchor rod size and location.

   Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements and practical considerations.

   Use four ¾-in.-diameter rods, ASTM F1554, Grade 36.

   Rod length = 12 in.

4.2 Example: Base Plate for Concentric Axial Compressive Load (Using Concrete Confinement)

Determine the base plate plan dimensions from Example 4.1, using concrete confinement (Case III).

1. Calculate the required axial compressive strength.

LRFD	ASD
$P_u = 700$ kips	$P_a = 430$ kips

2. Calculate the required base plate area using the strength increase for concrete confinement.

LRFD	ASD
$A_{1(req)} = \frac{P_u}{2\phi_c 0.85 f_c'}$ $= \frac{700\text{ kips}}{(2)(0.65)(0.85)(3\text{ ksi})}$ $= 211\text{ in.}^2$	$A_{1(req)} = \frac{\Omega_c P_a}{2(0.85 f_c')}$ $= \frac{(2.50)(430\text{ kips})}{(2)(0.85)(3\text{ ksi})}$ $= 211\text{ in.}^2$

3. Optimize the base plate dimensions, N and B.

$$\Delta = \frac{0.95d - 0.8b_f}{2}$$
$$= \frac{0.95(12.7\text{ in.}) - 0.8(12.2\text{ in.})}{2}$$
$$= 1.15\text{ in.}$$

$$N \approx \sqrt{A_{1(req)}} + \Delta$$
$$\approx \sqrt{211\text{ in.}^2} + 1.15\text{ in.}$$
$$\approx 15.7\text{ in.}$$

Try $N = 16$ in.
$B = 211\text{ in.}^2/16\text{ in.} = 13.2$ in.
Try $B = 14$ in.
$A_1 = (16\text{ in.})(14\text{ in.})$
$= 224\text{ in.}^2 > 211\text{ in.}^2$ **o.k.**

4. Calculate A_2 geometrically similar to A_1.

   Based on the 24 in. pier,

$N_2 = 24$ in.

Ratio $B/N = 14\text{ in.}/16\text{ in.}$
$= 0.875$

$B_2 = (0.875)(24\text{ in.})$
$= 21.0$ in.

$A_2 = (24\text{ in.})(21.0\text{ in.})$
$= 504\text{ in.}^2$

$504\text{ in.}^2 \le 4A_1 = (4)(224\text{ in.}^2)$
$= 896\text{ in.}^2$

Case III applies.

5. Use trial and error solution.

Try $N = 20$ in., $B = 18$ in.
$A_1 = (20\text{ in.})(18\text{ in.})$
$= 360\text{ in.}^2$
$N_2 = 24$ in.

Ratio B/N = 18 in./20 in. = 0.900

$$B_2 = (0.900)(24 \text{ in.}) = 21.6 \text{ in.}$$

$$A_2 = (24 \text{ in.})(21.6 \text{ in.}) = 518 \text{ in.}^2$$

LRFD	ASD
6. Determine if $P_u \le \phi_c P_p$; if not, revise N and B, and retry until criteria is satisfied. $\phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$ $= (0.65)(0.85)(3 \text{ ksi})(360 \text{ in.}^2)\sqrt{\frac{518 \text{ in.}^2}{360 \text{ in.}^2}}$ $= 716$ kips; $P_u \le \phi_c P_p$; 700 kips ≤ 716 kips **o.k.** Use N = 20 in., B = 18 in.	6. Determine if $P_a \le \frac{P_p}{\Omega_c}$; if not, revise N and B, and retry until criteria is satisfied. $\frac{P_p}{\Omega_c} = \frac{0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}}{\Omega_c}$ $= \frac{(0.85)(3 \text{ ksi})(360 \text{ in.}^2)\sqrt{\frac{518 \text{ in.}^2}{360 \text{ in.}^2}}}{2.50}$ $= 440$ kips; $P_a \le \frac{P_p}{\Omega_c}$; 430 kips ≤ 440 kips **o.k.** Use N = 20 in., B = 18 in.

7. Calculate required base plate thickness.

$$m = \frac{N - 0.95d}{2} = \frac{20 \text{ in.} - 0.95(12.7 \text{ in.})}{2} = 3.97 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} = \frac{18.0 \text{ in.} - 0.8(12.2 \text{ in.})}{2} = 4.12 \text{ in.}$$

LRFD	ASD
$X = \left[\frac{4db_f}{(d+b_f)^2}\right]\frac{P_u}{\phi P_p}$ $= \left[\frac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\frac{700 \text{ kips}}{716 \text{ kips}}$ $= 0.977$	$X = \left[\frac{4db_f}{(d+b_f)^2}\right]\frac{\Omega P_a}{P_p}$ $= \left[\frac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\frac{430 \text{ kips}}{440 \text{ kips}}$ $= 0.977$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1 = \frac{2\sqrt{0.977}}{1+\sqrt{1-0.977}} = 1.72 \quad \Rightarrow \quad 1$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} = (1)\frac{\sqrt{(12.7 \text{ in.})(12.2 \text{ in.})}}{4} = 3.11 \text{ in.}$$

$$l = \max(m, n, \lambda n') = \max(3.97 \text{ in.}, 4.12 \text{ in.}, 3.11 \text{ in.}) = 4.12 \text{ in.}$$

LRFD	ASD
$t_{min} = l\sqrt{\frac{2P_u}{\phi_b F_y BN}}$ $= (4.12 \text{ in.})\sqrt{\frac{(2)(700 \text{ kips})}{(0.90)(36 \text{ ksi})(18 \text{ in.})(20 \text{ in.})}}$ $= 1.43$ in. Use t_p = 1½ in.	$t_{min} = l\sqrt{\frac{2P_a\Omega_b}{F_y BN}}$ $= (4.12 \text{ in.})\sqrt{\frac{(2)(430 \text{ kips})(1.67)}{(36 \text{ ksi})(18 \text{ in.})(20 \text{ in.})}}$ $= 1.37$ in. Use t_p = 1½ in.

4.3 Example: Available Tensile Strength of a ¾-in. Anchor Rod

Calculate the available tensile strength of a ¾-in.-diameter ASTM F1554 Grade 36 anchor rod.

$$R_n = (0.75) F_u A_b = (0.75)(58 \text{ ksi})(0.442 \text{ in.}^2) = 19.2 \text{ kips}$$

The available tensile strength is determined as:

LRFD	ASD
ϕR_n = (0.75)(19.2 kips) = 14.4 kips	R_n/Ω = 19.2 kips/2.00 = 9.60 kips

Alternatively, the rod strength could be obtained in accordance with ACI 318 Appendix D.

4.4 Example: Concrete Embedment Strength

Calculate the tensile design strength of the concrete for a single smooth ¾-in.-diameter headed anchor rod with an embedment length of 6 in. The ACI 318 concrete breakout design strength (using equation for h_{ef} < 11 in.) for uncracked 4,000 psi concrete is:

$$\phi N_{cbg} = \phi\psi_3 24\sqrt{f_c'}\, h_{ef}^{1.5}\frac{A_N}{A_{No}}$$

Assuming uncracked concrete, ψ_3 = 1.25. For a single anchor rod, $A_N = A_{No}$.

$$\phi N_{cbg} = 0.70(1.25)24\sqrt{4{,}000 \text{ psi}}(6 \text{ in.})^{1.5}(1)$$
$$= 19{,}500 \text{ lb or } 19.5 \text{ kips}$$

Note that the breakout strength is theoretically independent of the size of the anchor rod. This embedment, at only 6 in., is enough to make the design tensile strength of a Grade 36 anchor rod up to ¾-in. diameter govern the design.

As discussed in Section 3.2.2, the ACI 318-08 pullout strength equations do not typically control provided that the anchor rod yield strength equals 36 ksi and the concrete strength is 4 ksi. In this case, the pullout strength shown in Table 3.2 may be multiplied by 1.4 to obtain the pullout strength, as the concrete is uncracked. The resulting pullout strength is

$$\phi N_p = 1.4 \times 20.3 \text{ kips}$$
$$= 28.4 \text{ kips} > 19.5 \text{ kips}$$

No equivalent ASD solution to this check exists in ACI 318-08.

4.5 Example: Column Anchorage for Tensile Loads

Design a base plate and anchorage for a W10×45 column subjected to a net uplift, as a result of the nominal loads shown in Figure 4.5.1. The reinforcing steel has F_y = 60 ksi.

a. Assume the anchorage is as shown in Figure 4.5.1.
b. Re-evaluate the anchorage if the column is on a 20-in. by 20-in. pier.

Procedure:

1. Determine the required strength due to uplift on the column.
2. Select the type and number of anchor rods.
3. Determine the appropriate base plate thickness and welding to transfer the uplift forces from the column to the anchor rods.
4. Determine the method for developing the anchor rods in the concrete in the spread footing.

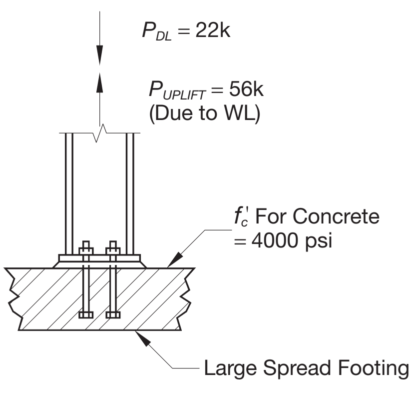


Fig. 4.5.1. Nominal loading diagram for Example 4.5.

Solution a.:

1. Determine the required strength due to uplift on the column.

LRFD	ASD
Uplift = − 0.9 P_{DL} + 1.6 P_{UPLIFT} = − 0.9(22 kips) + 1.6(56 kips) = 69.8 kips	Uplift = − 0.6(P_{DL}) + P_{UPLIFT} = − (0.6)(22 kips) + 56 kips = 42.8 kips

2. Select the type and number of anchor rods. Use four anchor rods (minimum per OSHA requirements).

LRFD	ASD
T_u/rod = 69.8 kips/4 = 17.5 kips	T_a/rod = 42.8 kips/4 = 10.7 kips

Using an ASTM F1554 Grade 36 material, select a ⅞-in.-diameter rod. The nominal tensile strength of each anchor rod is:

$$R_n = 0.75F_uA_b$$
$$= (0.75)(58 \text{ ksi})(0.601 \text{ in.}^2)$$
$$= 26.1 \text{ kips}$$

LRFD	ASD
The design strength of the rod = ϕR_n. ϕR_n = (0.75)(26.1 kips) = 19.6 kips/rod > 17.5 kips/rod **o.k.**	The allowable strength of the anchor rod = R_n/Ω. R_n/Ω = (26.1 kips)/2.00 = 13.1 kips/rod > 10.7 kips/rod **o.k.**

3. The rods are positioned inside the column profile with a 4-in. square pattern. Prying forces are negligible. To simplify the analysis, conservatively assume the tensile loads in the anchor rods generate one-way bending in the base plate about the web of the column. This assumption is illustrated by the assumed bending lines shown in Figure 4.5.2. If the column web strength controls the design, then consider distributing the forces to the flanges as well as the web. If the bolts are placed outside of the flanges, the 45° load distribution can be used to distribute the forces to the flanges.

The required flexural strength of the base plate per rod equals the rod force times the lever arm to the column web face.

LRFD	ASD
$M_u = 17.5 \text{ kips}\left(2 \text{ in.} - \dfrac{0.350 \text{ in.}}{2}\right)$ = 31.9 kip-in.	$M_a = 10.7 \text{ kips}\left(2 \text{ in.} - \dfrac{0.350 \text{ in.}}{2}\right)$ = 19.5 kip-in.

The effective width, b_{eff}, of base plate for resisting the required moment strength at the face of web is determined using a 45° distribution for the rod loads (width shown between the dashed lines in Figure 4.5.2),

$$b_{eff} = \left(2\text{ in.} - \frac{0.350\text{ in.}}{2}\right)(2)$$
$$= 3.65\text{ in.}$$

$$Z = \frac{b_{eff}t^2}{4}$$
$$F_y = 36\text{ ksi}$$

LRFD	ASD
$t_{req'd} = \sqrt{\dfrac{M_u(4)}{b_{eff}(\phi_b F_y)}}$ $= \sqrt{\dfrac{(31.9\text{ kip-in.})(4)}{(3.65\text{ in.})(0.90)(36\text{ ksi})}}$ $= 1.04$ in. Use a 1¼-in.-thick plate (F_y = 36 ksi).	$t_{req'd} = \sqrt{\dfrac{M_a(4)\Omega_b}{b_{eff}(F_y)}}$ $= \sqrt{\dfrac{(19.5\text{ kip-in.})(4)(1.67)}{(3.65\text{ in.})(36\text{ ksi})}}$ $= 0.996$ in. Use a 1-in.-thick plate (F_y = 36 ksi).

For welding of the column to the base plate:

$$\text{Maximum weld load} = \frac{T/\text{rod}}{b_{eff}}$$

LRFD	ASD
$\text{Maximum weld load} = \dfrac{17.5\text{ kips}}{3.65\text{ in.}}$ $= 4.79$ kips/in.	$\text{Maximum weld load} = \dfrac{10.7\text{ kips}}{3.65\text{ in.}}$ $= 2.93$ kips/in.

Minimum weld size for a 0.350-in. column web = $\frac{3}{16}$ in. (Table J2.4 of AISC *Specification*)

Nominal weld strength per inch for a $\frac{3}{16}$-in. fillet weld with E70 electrode (using the 50% directional increase according to AISC *Specification* Section J2.4):

$$R_n = F_w A_w$$

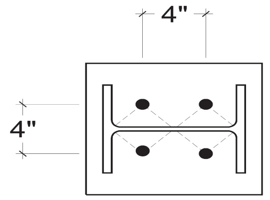


Fig. 4.5.2. Rod load distribution.

where

F_w = $0.60F_{EXX}(1.0 + 0.50\sin^{1.5}\theta)$
A_w = effective area of the weld

$$R_n = (0.60)(70\text{ ksi})(1.5)(0.707)(\tfrac{3}{16}\text{ in.})$$
$$= 8.35\text{ kips/in.}$$

LRFD	ASD
$\phi R_n = (0.75)(8.35\text{ kips/in.})$ $= 6.26$ kips/in. 4.79 kips/in. < 6.26 kips/in. $\frac{3}{16}$-in. fillet weld on each side of the column web is **o.k.**	$R_n/\Omega = (8.35\text{ kips/in.})/2.00$ $= 4.18$ kips/in. 2.93 kips/in. < 4.18 kips/in. $\frac{3}{16}$-in. fillet weld on each side of the column web is **o.k.**

Check web:
Web stress = Force per inch/Area of web per inch

LRFD	ASD
$\text{Web stress} = \dfrac{(2)(4.79\text{ kips/in.})}{0.350\text{ in.}}$ $= 27.4$ ksi in web $< 0.9F_y = 32.4$ ksi **o.k.**	$\text{Web stress} = \dfrac{(2)(2.93\text{ kips/in.})}{0.350\text{ in.}}$ $= 16.7$ ksi in web $< \dfrac{F_y}{1.67} = 21.6$ ksi **o.k.**

4. As noted earlier, this column is anchored in the middle of a large spread footing. Therefore, there are no edge constraints on the concrete tensile cones and there is no concern regarding edge distance to prevent lateral breakout of the concrete.

   Try using a 3.5-in. hook on the embedded end of the anchor rod to develop the rod. As mentioned earlier in this Guide the use of hooked anchor rods is generally not recommended. The use of hooked anchor rods here is to demonstrate the limited pullout strength of this type of rod.

   Note that no equivalent ASD solution exists for concrete bearing strength.

   Based on uniform bearing on the hook, the hook bearing strength per ACI 318-08, Appendix D is:

$$\text{Hook bearing strength} = \phi\psi_4\,(0.9f'_c e_h d_o)$$

where

ϕ = 0.70
ψ_4 = cracking factor (1.0 for cracked, 1.4 for uncracked concrete)
d_o = hook diameter
e_h = hook projection
f'_c = concrete compressive strength

Hook bearing strength $= 0.70(1.4)(0.9)(4{,}000 \text{ psi}) \times (3.5 \text{ in.} - \tfrac{7}{8} \text{ in.})(\tfrac{7}{8} \text{ in.})$

$= 8{,}100$ lb

$= 8.10$ kips < 17.5 kips **n.g.**

Thus a 3.5-in. hook is not capable of developing the required tensile force in the rod.

Therefore, use a heavy hex nut and a threaded rod end to develop the anchor rod.

The pullout strength of a ⅞-in.-diameter anchor rod from Table 3.2 is 27.3 kips, which is greater than the required strength per anchor rod.

The required embedment depth to achieve a concrete breakout strength, ϕN_{cbg}, that exceeds the required uplift of 69.8 kips (LRFD) can be determined by trial and error. The final trial with an embedment length of 13 in. follows.

Per ACI 318-08, Appendix D, the concrete breakout strength is:

$$\phi N_{cbg} = \phi \psi_3 24 \sqrt{f_c'}\, h_{ef}^{1.5} \frac{A_N}{A_{No}} \text{ for } h_{ef} < 11 \text{ in.}$$

and

$$\phi N_{cbg} = \phi \psi_3 16 \sqrt{f_c'}\, h_{ef}^{5/3} \frac{A_N}{A_{No}} \text{ for } 11 \text{ in.} \le h_{ef} \le 25 \text{ in.}$$

where

ϕ = 0.70

ψ_3 = 1.25 considering the concrete to be uncracked

h_{ef} = 13 in.

A_N = concrete breakout cone area for group (see Figure 4.5.3)

= $[2(1.5)(13 \text{ in.}) + 4 \text{ in.}]^2$

= 1,850 in.^{2}

A_{No} = concrete breakout cone area for single anchor

= $(3h_{ef})^2$

= $[(3)(13 \text{ in.})]^2$

= 1,520 in.^{2}

$$\phi N_{cbg} = 0.70(1.25)(16)\sqrt{4{,}000 \text{ psi}}\,(13 \text{ in.})^{5/3} \left(\frac{1{,}850 \text{ in.}^2}{1{,}520 \text{ in.}^2} \right)$$

$= 77{,}500$ lb

$= 77.5$ kips > 69.8 kips **o.k.**

With the ⅞-in.-diameter anchors, a 13 in. embedment is adequate to achieve the anchor strength considering the full breakout strength.

Solution b.:

If the anchors were installed in a 20-in. square pier the concrete breakout strength would be limited by the pier cross section. With an 8-in. maximum edge distance, the effective h_{ef} need be only 8 in. /1.5 = 5.33 in. to have the breakout cone area equal the pier cross-sectional area. This leads to:

$$\phi N_{cbg} = 0.70(1.25)(24)\sqrt{4{,}000 \text{ psi}}\,(5.33 \text{ in.})^{1.5} \left[\frac{(20 \text{ in.})^2}{[(3)(5.33 \text{ in.})]^2} \right]$$

$= 25{,}600$ lb

$= 25.6$ kips < 69.8 kips **n.g.**

Thus, it is necessary to transfer the anchor load to the vertical reinforcing steel in the pier. The required area of steel is:

$$A_s = \frac{69.8 \text{ kips}}{0.90(60 \text{ ksi})} = 1.29 \text{ in.}^2$$

The minimum area of longitudinal reinforcement per ACI 318-08, Section 10.9.1 is:

$$A_s = 0.01A_g = 0.01(20 \text{ in.})(20 \text{ in.}) = 4.00 \text{ in.}^2$$

Use 4-#10 bars.

With the bars located in the corners of the piers, use a horizontal distance from the center of the anchor to the center of the reinforcing steel,

$$g = \left[\frac{(20 \text{ in.} - 4 \text{ in.})}{2} - 2.6 \text{ in.} \right] \sqrt{2} = 7.64 \text{ in.}$$

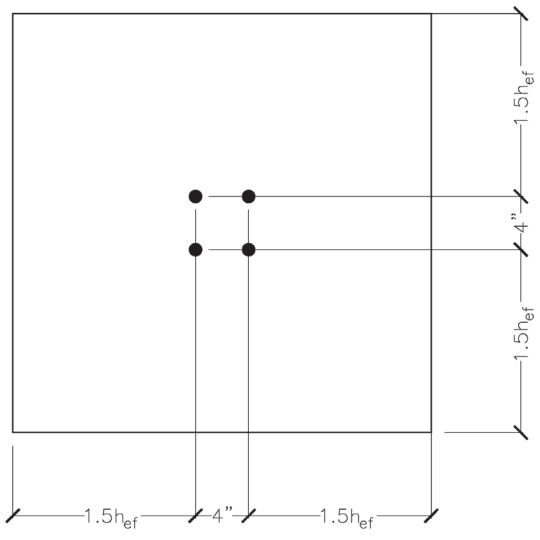


Fig. 4.5.3. Breakout cone for Example 4.5.

as shown in Figure 4.5.4. Using a Class B splice factor with a 1.3 value (ACI 318-08, Section 12.15) and with a development length from ACI 318-08, Section 12.2.3.

$$\ell_d = \left[\frac{3}{40} \frac{f_y}{\lambda\sqrt{f_c'}} \frac{\psi_t \psi_e \psi_s}{\left(\dfrac{c_b + K_{tr}}{d_b} \right)} \right] d_b$$

where

ψ_t = factor used to modify development length based on reinforcement location

ψ_e = factor used to modify development length based on reinforcement coating

ψ_s = factor used to modify development length based on reinforcement size

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete

K_{tr} = transverse reinforcement index

c_b = smaller of (a) the distance from the center of the bar to the nearest concrete surface, and (b) one-half the center-to-center spacing of bars being developed, in.

d_b = nominal diameter of bar, in.

f_c' = specified compressive strength of concrete, psi

f_y = specified yield strength of reinforcement, psi

For the #10 bars:

$$\ell_d = \left[\left(\frac{3}{40}\right) \left(\frac{60{,}000 \text{ psi}}{1.0\sqrt{4{,}000 \text{ psi}}} \right) \frac{(1.0)(1.0)(1.0)}{\left(\dfrac{2.6 \text{ in.} + 0.00}{1.27 \text{ in.}} \right)} \right] (1.27 \text{ in.})$$

$$= 44.1 \text{ in.}$$

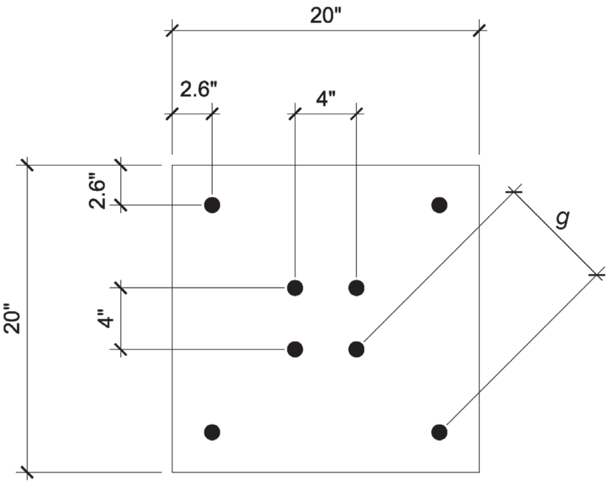


Fig. 4.5.4. Plan for calculation of dimension g.

compute ℓ_e from the ratio

$$\frac{\ell_e}{69.8} = \frac{1.3\ell_d}{nA_s\phi F_y}$$

$$= \frac{1.3(44.1 \text{ in.})}{4(1.27 \text{ in.}^2)(0.9)(60 \text{ ksi})}$$

which leads to

$\ell_e = 14.6$ in.

where

ℓ_e is the effective steel reinforcement lap required to develop the load in the reinforcing steel

Therefore minimum required h_{ef} = 14.6 in. + 1.5 in. (concrete cover) + 7.64 in. /1.5 = 21.2 in. as illustrated on Figure 3.2.5. Select 22-in. embedment for the anchors.

4.6 Example: Small Moment Base Plate Design

Design a base plate for axial dead and live loads equal to 100 kips and 160 kips, respectively, and moments from the dead and live loads equal to 250 kip-in. and 400 kip-in., respectively. Bending is about the strong axis for the wide flange column W12×96 with d = 12.7 in. and b_f = 12.2 in. The ratio of the concrete to base plate area is unity. F_y of the base plate is 36 ksi and f_c' of the concrete is 4 ksi.

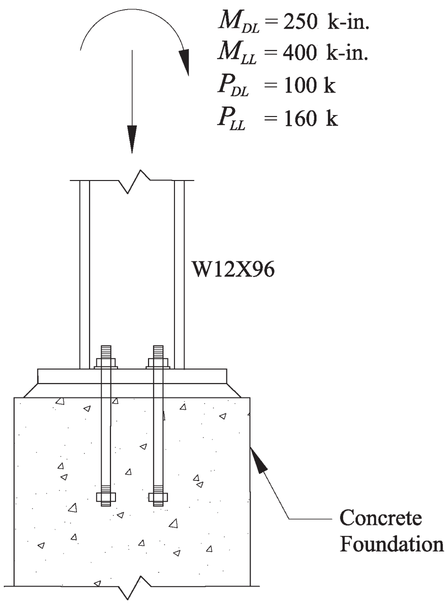


Fig. 4.6.1. Example 4.6.

Solution:

1. Compute the required strength.

LRFD	ASD
$P_u = 1.2(100) + 1.6(160)$ $= 376$ kips $M_u = 1.2(250) + 1.6(400)$ $= 940$ kip-in.	$P_a = 100 + 160$ $= 260$ kips $M_a = 250 + 400$ $= 650$ kip-in.

2. Choose trial base plate size.

   The base plate dimension $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA (3 in. is the minimum concrete cover).

   $N > d + (2)(3.0 \text{ in.}) = 18.7$ in.
   $B > b_f + (2)(3.0 \text{ in.}) = 18.2$ in.

   Try $N = 19$ in. and $B = 19$ in.

3. Determine e and e_{crit}.

LRFD	ASD
$e = \dfrac{M_u}{P_u}$ $= \dfrac{940 \text{ kip-in.}}{376 \text{ kips}}$ $= 2.50$ in. $f_{p(max)} = \phi_c(0.85f'_c)\sqrt{\dfrac{A_2}{A_1}}$ $= (0.65)(0.85)(4 \text{ ksi})(1)$ $= 2.21$ ksi $q_{max} = f_{p(max)} \times B$ $= (2.21 \text{ ksi})(19 \text{ in.})$ $= 42.0$ kips/in. $e_{crit} = \dfrac{N}{2} - \dfrac{P_u}{2q_{max}}$ $= \dfrac{19 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{2(42.0 \text{ kips/in.})}$ $= 5.02$ in.	$e = \dfrac{M_a}{P_a}$ $= \dfrac{650 \text{ kip-in.}}{260 \text{ kips}}$ $= 2.50$ in. $f_{p(max)} = \dfrac{(0.85f'_c)}{\Omega_c}\sqrt{\dfrac{A_2}{A_1}}$ $= \dfrac{(0.85)(4 \text{ ksi})(1)}{2.50}$ $= 1.36$ ksi $q_{max} = f_{p(max)} \times B$ $= (1.36 \text{ ksi})(19 \text{ in.})$ $= 25.8$ kips/in. $e_{crit} = \dfrac{N}{2} - \dfrac{P_a}{2q_{max}}$ $= \dfrac{19 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{2(25.8 \text{ kips/in.})}$ $= 4.46$ in.

   Therefore, $e \le e_{crit}$, and the design meets the criteria for the case of a base plate with small moment.

4. Determine bearing length, Y:

   $Y = N - 2e$
   $= 19 \text{ in.} - (2)(2.50 \text{ in.})$
   $= 14.0$ in.

   Verify bearing pressure:

LRFD	ASD
$q = \dfrac{P_u}{Y}$ $= \dfrac{376 \text{ kips}}{14.0 \text{ in.}}$ $= 26.9$ kips/in. $< 42.0 \text{ kips/in.} = q_{max}$ **o.k.**	$q = \dfrac{P_a}{Y}$ $= \dfrac{260 \text{ kips}}{14.0 \text{ in.}}$ $= 18.6$ kips/in. $< 25.8 \text{ kips/in.} = q_{max}$ **o.k.**

5. Determine minimum plate thickness.

   At the bearing interface:

$$m = \frac{N - 0.95d}{2} = \frac{19 \text{ in.} - 0.95(12.7 \text{ in.})}{2} = 3.47 \text{ in.}$$

LRFD	ASD
$f_p = \dfrac{P_u}{BY}$ $= \dfrac{376 \text{ kips}}{(19 \text{ in.})(14.0 \text{ in.})}$ $= 1.41$ ksi	$f_p = \dfrac{P_a}{BY}$ $= \dfrac{260 \text{ kips}}{(19 \text{ in.})(14.0 \text{ in.})}$ $= 0.977$ ksi

   The minimum thickness may be calculated from Equation 3.3.14 since $Y \ge m$:

LRFD	ASD
$t_{p(req)} = 1.49m\sqrt{\dfrac{f_p}{F_y}}$ $= (1.49)(3.47 \text{ in.})\sqrt{\dfrac{1.41 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.02$ in.	$t_{p(req)} = 1.83m\sqrt{\dfrac{f_p}{F_y}}$ $= (1.83)(3.47 \text{ in.})\sqrt{\dfrac{0.977 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.05$ in.

   Check the thickness using the value of n.

$$n = \frac{B - 0.8b_f}{2} = \frac{19 \text{ in.} - (0.8)(12.2 \text{ in.})}{2} = 4.62 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = (1.49)(4.62 \text{ in.})\sqrt{\dfrac{1.41 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.36$ in. Controls Use a base plate 1½ in. × 19 in. × 1 ft -7 in.	$t_{p(req)} = (1.83)(4.62 \text{ in.})\sqrt{\dfrac{0.977 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.39$ in. Controls Use a base plate 1½ in. × 19 in. × 1 ft-7 in.

6. Determine the anchor rod size.

   Since no anchor rod forces exist, the anchor rod size can be determined based on the OSHA requirements, and practical considerations.

   Use:
   (4) ¾-in.-diameter rods, ASTM F1554, Grade 36
   Rod length = 12 in.

4.7 Example: Large Moment Base Plate Design

Design a base plate for axial dead and live loads equal to 100 kips and 160 kips, respectively, and moments from the dead and live loads equal to 1,000 kip-in. and 1,500 kip-in., respectively. Bending is about the strong axis for a W12×96 wide flange column with $d = 12.7$ in. and $b_f = 12.2$ in. Conservatively consider the ratio of the concrete to base plate area is unity. F_y of the base plate is 36 ksi and f_c' of concrete is 4 ksi.

1. Compute the required strength.

LRFD	ASD
$P_u = 1.2(100 \text{ kips}) + 1.6(160 \text{ kips}) = 376 \text{ kips}$ $M_u = 1.2(1{,}000 \text{ kip-in.}) + 1.6(1{,}500 \text{ kip-in.}) = 3{,}600 \text{ kip-in.}$	$P_a = 100 \text{ kips} + 160 \text{ kips} = 260 \text{ kips}$ $M_a = 1{,}000 \text{ kip-in.} + 1{,}500 \text{ kip-in.} = 2{,}500 \text{ kip-in.}$

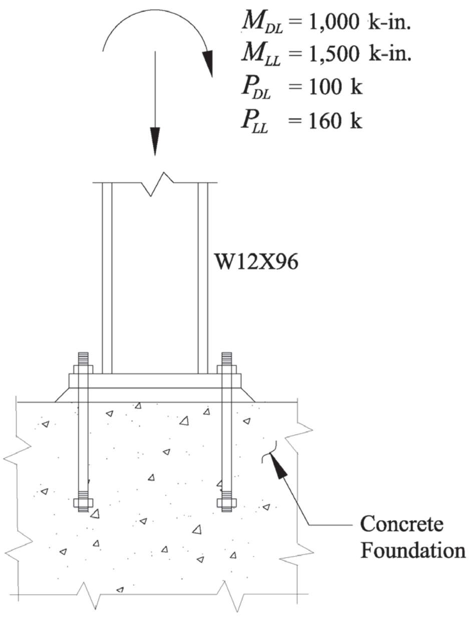


Fig. 4.7.1. Example 4.7.

2. Choose trial base plate size.

   The base plate dimension $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA (3 in. is the minimum concrete cover).

$$N > d + (2)(3.0 \text{ in.}) = 18.7 \text{ in.}$$

$$B > b_f + (2)(3.0 \text{ in.}) = 18.2 \text{ in.}$$

   Try $N = 19$ in. and $B = 19$ in.

3. Determine e and e_{crit}; check inequality in Equation 3.4.4 to determine if a solution exists:

LRFD	ASD
$q_{max} = 42.0$ kips/in. (see Example 4.6)	$q_{max} = 25.8$ kips/in. (see Example 4.6)
$e = \dfrac{M_u}{P_u} = \dfrac{3{,}600 \text{ kip-in.}}{376 \text{ kips}} = 9.57 \text{ in.}$	$e = \dfrac{M_a}{P_a} = \dfrac{2{,}500 \text{ kip-in.}}{260 \text{ kips}} = 9.62 \text{ in.}$
$e_{crit} = \dfrac{N}{2} - \dfrac{P_u}{2q_{max}} = \dfrac{19 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{(2)(42.0 \text{ kips/in.})} = 5.02 \text{ in.}$	$e_{crit} = \dfrac{N}{2} - \dfrac{P_a}{2q_{max}} = \dfrac{19 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{(2)(25.8 \text{ kips/in.})} = 4.46 \text{ in.}$

   Because $e > e_{crit}$, this is the case of a base plate with a large moment.

   Check the inequality of Equation 3.4.4:

   Assume that the anchor rod edge distance is 1.5 in. Therefore, from the geometry in Figure 3.4.1:

$$f = \frac{N}{2} - 1.5 \text{ in.} = \frac{19 \text{ in.}}{2} - 1.5 \text{ in.} = 8 \text{ in.}$$

$$\left(f + \frac{N}{2}\right)^2 = \left(8 \text{ in.} + \frac{19 \text{ in.}}{2}\right)^2 = 306 \text{ in.}^2$$

LRFD	ASD
$\dfrac{2P_u(e+f)}{q_{max}} = \dfrac{(2)(376 \text{ kips})(9.57 \text{ in.} + 8 \text{ in.})}{42.0 \text{ kips/in.}}$ $= 315 \text{ in.}^2$ Since 315 in.^{2} > 306 in.^{2}, the inequality is not satisfied. Hence, a larger plate dimension is required.	$\dfrac{2P_a(e+f)}{q_{max}} = \dfrac{(2)(260 \text{ kips})(9.62 \text{ in.} + 8 \text{ in.})}{25.8 \text{ kips/in.}}$ $= 355 \text{ in.}^2$ Since 355 in.^{2} > 306 in.^{2}, the inequality is not satisfied. Hence, a larger plate dimension is required.

As the second iteration, try a 22 in. × 22 in. plate.

The increased dimensions cause a modification in the maximum bearing pressure, q_{max}, f and e_{crit}. The new values become:

LRFD	ASD
$q_{max} = (2.21 \text{ ksi})(B)$ $= (2.21 \text{ ksi})(22 \text{ in.})$ $= 48.6 \text{ kips/in.}$ $f = \dfrac{22 \text{ in.}}{2} - 1.5 \text{ in.}$ $= 9.5 \text{ in.}$ $e_{crit} = \dfrac{22 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{(2)(48.6 \text{ kips/in.})}$ $= 7.13 \text{ in.}$ The eccentricity, e, still exceeds e_{crit}, therefore, the load combination is for large moments. Also: $\left(f + \dfrac{N}{2}\right)^2 = \left(9.5 \text{ in.} + \dfrac{22 \text{ in.}}{2}\right)^2$ $= 420 \text{ in.}^2$ $\dfrac{2P_u(e+f)}{q_{max}} = \dfrac{(2)(376 \text{ kips})(9.57 \text{ in.} + 9.5 \text{ in.})}{48.6 \text{ kips/in.}}$ $= 295 \text{ in.}^2$ 295 in.^{2} < 420 in.^{2}, therefore the inequality in Equation 3.4.4 is satisfied and a real solution for Y exists.	$q_{max} = (1.36 \text{ ksi})(B)$ $= (1.36 \text{ ksi})(22 \text{ in.})$ $= 29.9 \text{ kips/in.}$ $f = \dfrac{22 \text{ in.}}{2} - 1.5 \text{ in.}$ $= 9.5 \text{ in.}$ $e_{crit} = \dfrac{22 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{(2)(29.9 \text{ kips/in.})}$ $= 6.65 \text{ in.}$ The eccentricity, e, still exceeds e_{crit}, therefore, the load combination is for large moments. Also: $\left(f + \dfrac{N}{2}\right)^2 = \left(9.5 \text{ in.} + \dfrac{22 \text{ in.}}{2}\right)^2$ $= 420 \text{ in.}^2$ $\dfrac{2P_a(e+f)}{q_{max}} = \dfrac{(2)(260 \text{ kips})(9.62 \text{ in.} + 9.5 \text{ in.})}{29.9 \text{ kips/in.}}$ $= 333 \text{ in.}^2$ 333 in.^{2} < 420 in.^{2}, therefore the inequality in Equation 3.4.4 is satisfied and a real solution for Y exists.

4. Determine bearing length, Y, and anchor rod tension, T_u or T_a:

LRFD	ASD
$Y = \left(f + \dfrac{N}{2}\right) \pm \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2P_u(e+f)}{q_{max}}}$ $= \left(9.5 \text{ in.} + \dfrac{22 \text{ in.}}{2}\right)$ $\pm \sqrt{420 \text{ in.}^2 - 295 \text{ in.}^2}$ $= 20.5 \text{ in.} \pm 11.2 \text{ in.}$ $= 9.30 \text{ in.}$ $T_u = q_{max}Y - P_u$ $= (48.6 \text{ kips/in.})(9.30 \text{ in.}) - 376 \text{ kips}$ $= 76.0 \text{ kips}$	$Y = \left(f + \dfrac{N}{2}\right) \pm \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2P_a(e+f)}{q_{max}}}$ $= \left(9.5 \text{ in.} + \dfrac{22 \text{ in.}}{2}\right)$ $\pm \sqrt{420 \text{ in.}^2 - 333 \text{ in.}^2}$ $= 20.5 \text{ in.} \pm 9.33 \text{ in.}$ $= 11.2 \text{ in.}$ $T_a = q_{max}Y - P_a$ $= (29.9 \text{ kips/in.})(11.2 \text{ in.}) - 260 \text{ kips}$ $= 74.9 \text{ kips}$

5. Determine minimum plate thickness.

At bearing interface:

$$m = \frac{N - 0.95d}{2}$$
$$= \frac{22 \text{ in.} - 0.95(12.7 \text{ in.})}{2}$$
$$= 4.97 \text{ in.}$$

LRFD	ASD
From Example 4.6, $f_p = f_{p(max)}$ $= 2.21 \text{ ksi}$	From Example 4.6, $f_p = f_{p(max)}$ $= 1.36 \text{ ksi}$

Because $Y \geq m$:

LRFD	ASD
$t_{p(req)} = 1.49m\sqrt{\dfrac{f_{p(max)}}{F_y}}$ $= (1.49)(4.97 \text{ in.})\sqrt{\dfrac{2.21 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.83 \text{ in.}$	$t_{p(req)} = 1.83m\sqrt{\dfrac{f_{p(max)}}{F_y}}$ $= (1.83)(4.97 \text{ in.})\sqrt{\dfrac{1.36 \text{ ksi}}{36 \text{ ksi}}}$ $= 1.77 \text{ in.}$

At tension interface:

$$x = \frac{N}{2} - \frac{d}{2} + \frac{t_f}{2} - 1.5$$
$$= \frac{22 \text{ in.}}{2} - \frac{12.7 \text{ in.}}{2} + \frac{0.900 \text{ in.}}{2} - 1.5 \text{ in.}$$
$$= 3.60 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = 2.11\sqrt{\dfrac{T_u x}{BF_y}}$ $= 2.11\sqrt{\dfrac{(76.0 \text{ kips})(3.60 \text{ in.})}{(22 \text{ in.})(36 \text{ ksi})}}$ $= 1.24 \text{ in.}$	$t_{p(req)} = 2.58\sqrt{\dfrac{T_a x}{BF_y}}$ $= 2.58\sqrt{\dfrac{(74.9 \text{ kips})(3.60 \text{ in.})}{(22 \text{ in.})(36 \text{ ksi})}}$ $= 1.51 \text{ in.}$

Check the thickness using the value of n.

$$n = \frac{B - 0.8b_f}{2}$$
$$= \frac{22 \text{ in.} - 0.8(12.2 \text{ in.})}{2}$$
$$= 6.12 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = (1.49)(6.12 \text{ in.})\sqrt{\dfrac{2.21 \text{ ksi}}{36 \text{ ksi}}}$ $= 2.26 \text{ in.}$ Controls	$t_{p(req)} = (1.83)(6.12 \text{ in.})\sqrt{\dfrac{1.36 \text{ ksi}}{36 \text{ ksi}}}$ $= 2.18 \text{ in.}$ Controls

The bearing interface governs the design of the base plate thickness. Use 2½-in. plate.

6. Determine the anchor rod size and embedment (LRFD only).

From previous calculations, $T_u = 76.0$ kips. If two anchor rods are used on each face of the column, the force per rod equals 38.0 kips. From Table 3.1, the design strength of 1½-in.-diameter ASTM F1554 Grade 36 anchor rods is 57.7 kips. The recommended hole size for the 1½-in. rod is 2⁵⁄₁₆ in. per Table 2-3. Using an edge distance to the center of the hole of 2¼ in., the initial assumption of 1½ in. must be adjusted. Using the adjusted edge distance the 1½-in. rods are still adequate.

The pullout strength of each anchor rod with a heavy hex nut is selected from Table 3.2 as 70.1 kips, which is greater than the required strength per rod of 38.0 kips.

For completeness, determine the embedment length for the anchor rods.

Try 18 in. of embedment.

The design concrete breakout strength is:

$$\phi N_{cbg} = \phi\psi_3 16\sqrt{f_c'}\, h_{ef}^{\,5/3}\frac{A_N}{A_{No}} \quad \text{for 11 in.} \le h_{ef} \le \text{25 in.}$$

In the CCD method, the concrete cone is considered to form at a slope of 1.5 to 1 as discussed in Section 3.2.2 of this Design Guide. If the rods are placed 12 in. apart, the plan area of the failure cone is $2(1.5h_{ef}) = (3)(18 \text{ in.}) = 54$ in. in width and (3)(18 in.) + 12 in. = 66 in. in length as shown in Figure 4.7.2, thus the total area, $A_N = 3{,}564 \text{ in.}^2$ The plan area of the failure cone for a single anchor rod embedded to 18 in. is $(3h_{ef})^2 = [(3)(18)]^2 = 2{,}916 \text{ in.}^2$ The ratio of these areas is 1.22, so for uncracked 4,000 psi concrete, the design concrete breakout strength is:

$$\begin{aligned}\phi N_{cbg} &= 0.70(1.25)16\sqrt{4{,}000\text{ psi}}\,(18)^{5/3}(1.22)\\ &= 134{,}000 \text{ lbs or}\\ &= 134 \text{ kips} > 76.0 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

For moderate or high seismic risk, ACI 318-08 indicates that the strength of anchors is to be multiplied by 0.75. In this case, the steel strength would be 0.75(57.7) kips = 43.3 kips per rod. The anchor rods are still o.k.

4.8 Example: Shear Transfer Using Bearing

Calculate the minimum embedment depth of a shallowly embedded W12×50 in 6,000 psi grout for an LRFD shear load of 100 kips. The base plate is 15 in. by 15 in. and is 1.5 in. thick. The projected area of the plate, A_{brg}, equals (1.5 in.)(15 in.) = 22.5 in.² The design shear strength in bearing on the base plate edge per ACI 318-08 equals

$$\begin{aligned}\phi P_{ubrg} &= 0.55 f_c' A_{brg}\\ &= 0.55(6 \text{ ksi})(22.5 \text{ in.}^2)\\ &= 74.3 \text{ kips}\end{aligned}$$

The remaining 25.7 kips must be taken by bearing of the flange of the W12×50 against the concrete. The width of the flange is 8.08 in. The required bearing area equals

$$\begin{aligned}A_{brg} &= \frac{25.7 \text{ kips}}{(0.55)(6 \text{ ksi})}\\ &= 7.79 \text{ in.}^2\end{aligned}$$

Thus, the required flange embedment depth equals:

$$\left(\frac{7.79 \text{ in.}^2}{8.08 \text{ in.}}\right) = 0.964 \text{ in.}$$

The total bearing thickness must be at least 1.5 in. + 0.964 in. = 2.46 in. To allow for variations in the actual slab thickness, use a total embedment of 4 in. for the flange and base plate.

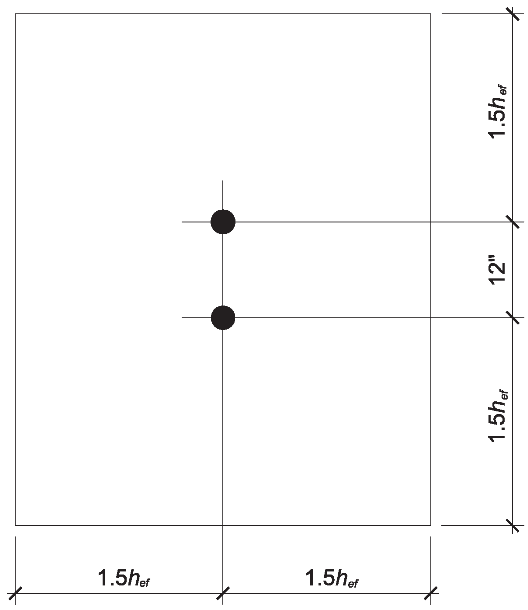


Fig. 4.7.2. Breakout cone for Example 4.7.

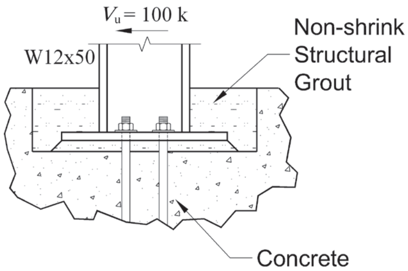


Fig. 4.8.1. Example 4.8.

4.9 Example: Shear Lug Design

Design a shear lug detail for the W10×45 column considered in Example 4.5, but with an additional shear of 23 kips (nominal load) due to wind. See Figure 4.9.1. The anchor rods in this example are designed only to transfer the net uplift from the column to the pier. The shear lug will be designed to transfer the entire shear load to the pier with the confinement component being ignored.

Procedure:

1. Determine the required embedment for the lug into the concrete pier.
2. Determine the appropriate thickness for the lug.
3. Size the welds between the lug and the base plate.

Solution:

1. Two criteria are used to determine the appropriate embedment for the lug. These criteria are the bearing strength of the concrete and the shear strength of the concrete in front of the lug. The shear strength of the concrete in front of the lug is evaluated (in strength terms) as a uniform tensile stress of $4\phi\sqrt{f'_c}$ with $\phi = 0.75$ acting on an effective stress area defined by projecting a 45° plane from the bearing edge of the shear lug to the free surface (the face of the pier). The bearing area of the lug is to be excluded from the projected area. Since this criterion is expressed in strength terms, the bearing strength of the concrete is also evaluated with a strength approach. The bearing strength of the concrete in contact with the lug is evaluated as $\phi 1.3 f'_c A_l$, where $\phi = 0.65$.

   Since the anchor rods are sized for only the required uplift tension, the $\phi K_c(N_y - P_a) = 0.85(1.8)(N_y - P_a) = 1.53(N_y - P_a)$ term addressed in Section 3.5.2 (assuming the shear lug is located a distance h from the front edge of the base plate) will be small and thus is ignored in this example.

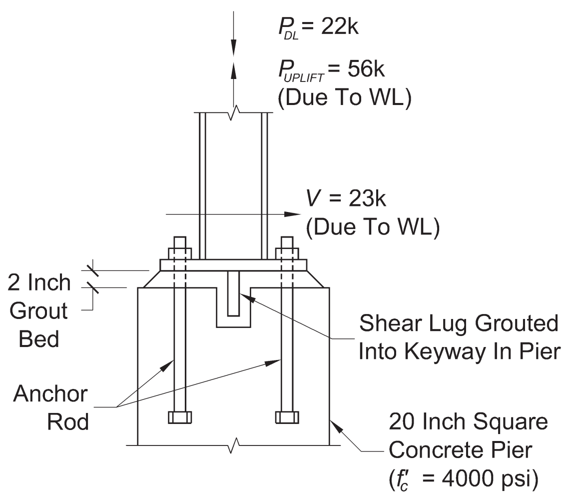


Fig. 4.9.1. Shear lug design.

The factored shear load = (1.6)(23 kips)
= 36.8 kips

Equating this load to the bearing strength of the concrete, the following relationship is obtained:

$$0.65(1.3)(4{,}000\text{ psi})(A_l)_{req'd} = 36{,}800\text{ lb}$$
$$(A_l)_{req'd} = 10.8\text{ in.}^2$$

Assuming the base plate and shear lug width is 9 in., the required embedded depth, d, of the lug (in the concrete) is calculated as:

$$d = 10.8\text{in.}/9\text{ in.} = 1.2\text{ in.}$$

Use 2 in.

See Figure 4.9.2.

Using this embedment, the shear strength of the concrete in front of the lug is checked. The projected area of the failure plane at the face of the pier is shown in Figure 4.9.3.

Assuming the lug is positioned in the middle of the pier and the lug is 1 in. thick:

a = 5.5 in. in the 20-in.-wide pier

b = 2 in. + 9.5 in.
= 11.5 in.

The projected area of this plane, A_v, excluding the area of the lug, is then calculated as:

$$A_v = (20\text{ in.})(11.5\text{ in.}) - (2\text{ in.})(9\text{ in.}) = 212\text{ in.}^2$$

Using this area, the shear strength of the concrete in front of the lug, V_u, is calculated as:

$$V_u = 4\phi\sqrt{f'_c}A_v$$
$$= \frac{4(0.75)\sqrt{4{,}000\text{ psi}}(212\text{ in.}^2)}{1{,}000}$$
$$= 40.2\text{ kips} > 36.8\text{ kips } \textbf{o.k.}$$

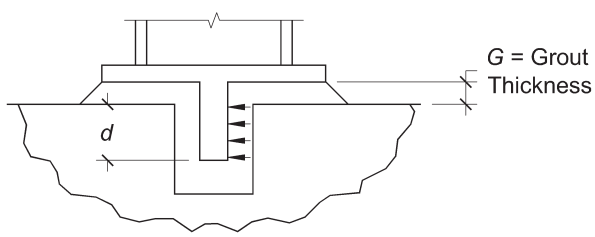


Fig. 4.9.2. Shear lug depth.

2. Using a cantilever model for the lug, determine the required lug thickness.

$$M_{ul} = V_u(G + d/2)$$
$$= (36.8 \text{ kips})(2 \text{ in.} + 2 \text{ in.}/2)$$
$$= 110 \text{ kip-in.}$$

Note: G = 2 in. = thickness of grout bed.

$$Z = \frac{bt_{req'd}{}^2}{4}$$

$$\phi_b M_n = \phi_b F_y Z$$
$$= \frac{\phi_b F_y b t^2_{req'd}}{4}$$
$$= \frac{(0.90)(36 \text{ ksi})(9 \text{ in.})t^2_{req'd}}{4}$$
$$= 72.9t^2_{req'd}$$

$$110 \text{ kip-in.} = 72.9t_{req'd}{}^2$$

$$t_{req'd} = 1.23 \text{ in.}$$

Use a 1¼-in.-thick lug (F_y = 36 ksi)

Based on the discussion in item 2 near the end of Section 3.5.2, it is recommended to use a base plate of 1¼ in. minimum thickness with this shear lug.

3. Most steel fabricators would prefer to use fillet welds up to ¾ in. size rather than partial- or complete-joint-penetration groove welds to attach the lug to the base plate. The forces on the welds are as shown in Figure 4.9.4.

Consider 5⁄16-in. fillet welds:

$$s = 1.25 \text{ in.} + (5/16 \text{ in.})(1/3)(2)$$
$$= 1.46 \text{ in.}$$

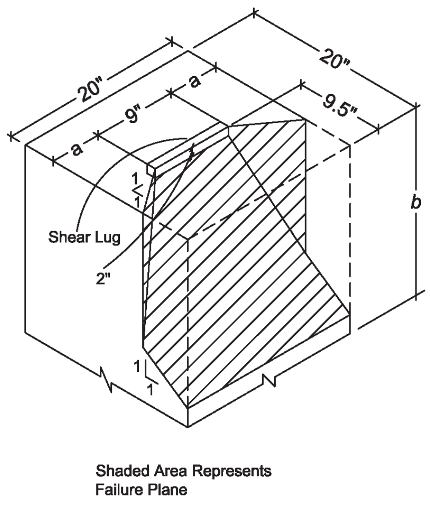


Fig. 4.9.3. Lug failure plane.

$$f_c = \frac{110 \text{ kip-in.}}{(1.46 \text{ in.})(9 \text{ in.})}$$
$$= 8.37 \text{ kips/in.}$$

$$f_v = \frac{1.6(23 \text{ kips})}{(9 \text{ in.})(2)}$$
$$= 2.04 \text{ kips/in.}$$

The resultant weld load, f_r, is calculated as:

$$f_r = \sqrt{(8.37 \text{ kips/in.})^2 + (2.04 \text{ kips/in.})^2}$$
$$= 8.62 \text{ kips/in.}$$

For a 5⁄16-in. fillet weld using E70 electrode:

$$\phi R_n = \phi F_w A_w$$
$$\phi F_w = \phi(0.60)F_{EXX}(1 + 0.5\sin^{1.5}\theta)$$
$$= (0.75)(0.60)(70 \text{ ksi})(1.5)$$
$$= 47.3 \text{ ksi}$$

Note: The 1.5 factor represents the increase for transverse load in a fillet weld ($\theta = 90°$).

$$\phi R_n = (47.3 \text{ ksi})(5/16 \text{ in.})(0.707)$$
$$= 10.5 \text{ kips/in.}$$

10.5 kips/in. > 8.62 kips/in. **o.k.**

Use 5⁄16-in. fillet welds.

4.10 Example: Edge Distance for Shear

Determine the required concrete edge distance to develop the shear strength of four ¾-in.-diameter anchor rods. A 4 in. by 4 in. pattern is used for the rods. The concrete strength is 4,000 psi.

Rod Shear Strength (per the AISC *Specification* Section J3.6):

$$\phi R_n = \phi\,(0.4)F_u A_b \text{ (threads included)}$$
$$= (0.75)(0.4)(58 \text{ ksi})(0.442 \text{ in.}^2)$$
$$= 7.69 \text{ kips}$$

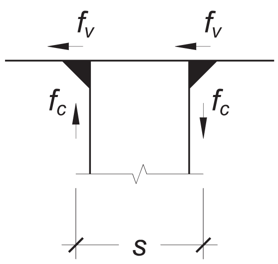


Fig. 4.9.4. Forces on shear lug welds.

For all four rods, $\phi R_n = 30.8$ kips. From the simplified equation given in Section 3.5.3:

$$\phi V_{cbg} = 10.4 \frac{A_v}{A_{vo}} \psi_6 \sqrt{d_o} \sqrt{f_c'}\, c_1^{1.5}$$

where

Trial c_1 = 14 in. (distance to the edge of concrete)
s = 4 in. (rod spacing)
c_1/s = 14 in./4 in.
= 3.5 > 2.26 (as given in Figure 3.5.3 for s_1/s = 1.0); therefore, the total group controls
ψ_6 = 1 (not limited by side encroachment)
A_{vo} = $4.5c_1^2$
= $4.5\,(14 \text{ in.})^2$
= 882 in.^{2} (the area of the full shear cone for a single anchor as shown in View A-A of Figure 3.5.2)
A_v = $4.5c_1^2 + s\,(1.5c_1)$
= 882 in.^{2} + (4 in.)(1.5)(14 in.)
= 966 in.^{2} (the total breakout shear area for a group of anchors)

$$\phi V_{cbg} = 10.4\left(\frac{966 \text{ in.}^2}{882 \text{ in.}^2}\right)(1)\sqrt{0.75 \text{ in.}}\sqrt{4{,}000 \text{ psi}}\left(14 \text{ in.}\right)^{1.5}$$

= 32,700 lb
= 32.7 kips

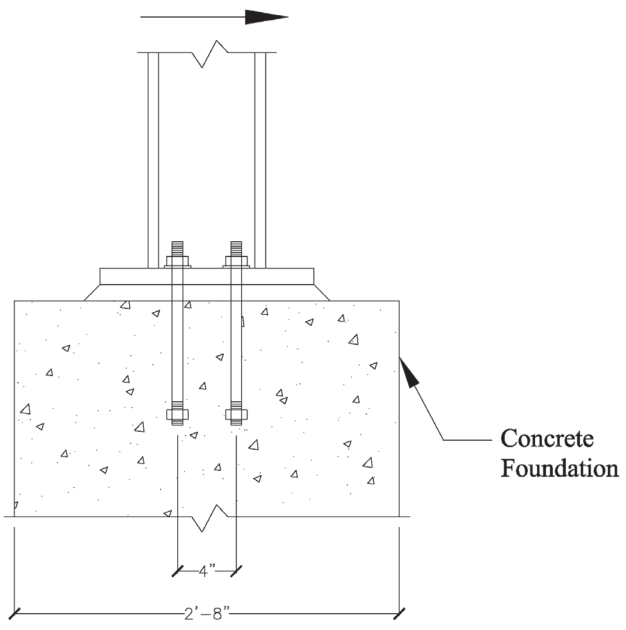


Fig. 4.10.1. Example 4.10.

Approximately 14-in. clearance in plain concrete is required.

The reader is referred to AISC *Design Guide 7* (Fisher, 2004) for a discussion on the reinforcement of concrete piers to resist lateral thrusts.

4.11 Example: Anchor Rod Resisting Combined Tension and Shear

Determine the required size of four anchor rods for the W10×45 column examined in Example 4.9, using the anchor rods to resist the wind shear.

The nominal wind shear force is 23 kips. Therefore, the required shear strength is

LRFD	ASD
$V_u = 1.6 \times 23$ kips = 36.8 kips	$V_u = 23.0$ kips

Solution:

1. As determined in Example 4.5, the required strength due to uplift on the column is:

LRFD	ASD
Uplift = 69.8 kips	Uplift = 42.8 kips

2. A total of four anchor rods is used. Use plate washers with standard holes welded to the top of the base plate to transfer the shear to all four anchor rods. Try four 1⅛-in.-diameter ASTM F1554 Grade 36 anchors. For combined shear and tension the anchor rods must meet the following AISC *Specification* provision (Section J3.7).

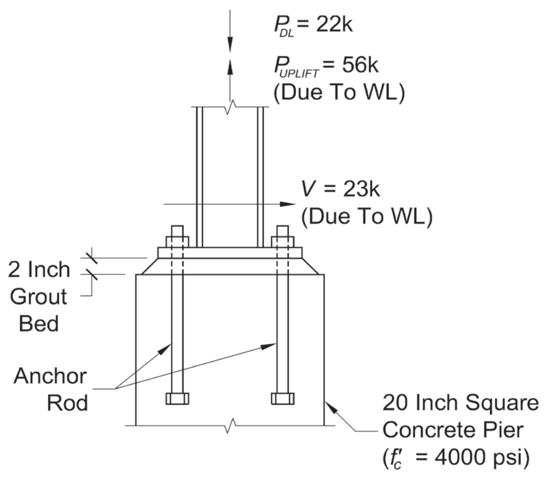


Fig. 4.11.1. Example 4.11.

LRFD	ASD
$f_t \le \phi F'_{nt} = \phi\left(1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v\right) \le \phi F_{nt}$ where $\phi = 0.75$	$f_t \le \dfrac{F'_{nt}}{\Omega} = \dfrac{\left(1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v\right)}{\Omega} \le \dfrac{F_{nt}}{\Omega}$ where $\Omega = 2.00$

Stresses in rods:

LRFD	ASD
Shear stress: $f_v = \dfrac{36.8 \text{ kips}}{4(0.994 \text{ in.}^2)} = 9.26 \text{ ksi}$	Shear stress: $f_v = \dfrac{23.0 \text{ kips}}{4(0.994 \text{ in.}^2)} = 5.78 \text{ ksi}$

Tensile stress: The tensile stress in the rods comes from two sources:

1. tension from bending, and
2. axial tension

The bending moment in each rod equals the shear per rod times the half distance from the center of the plate washer to the top of the grout.

Determine the plate washer thickness:

The bearing force per rod is:

LRFD	ASD
$P_{brg} = \dfrac{36.8 \text{ kips}}{4} = 9.20 \text{ kips}$	$P_{brg} = \dfrac{23.0 \text{ kips}}{4} = 5.75 \text{ kips}$

The deformation at the hole at service load is not a design consideration; therefore, the nominal bearing strength is:

$$R_n = 1.5L_c tF_u \le 3.0dtF_u$$

By inspection, ½-in. plate washers will suffice even with minimal edge distance. Thus, the lever arm can be taken as half the distance from the center of the plate washer to the top of the grout. The plate washer is ½-in. thick, and the base plate is 1.25-in. thick (LRFD) and 1.00-in. thick (ASD), as determined in Example 4.5.

LRFD	ASD
Lever arm $= \dfrac{1.25 \text{ in.} + (0.500 \text{ in.}/2)}{2} = 0.750 \text{ in.}$ Thus $M_l = \dfrac{36.8 \text{ kips}(0.750 \text{ in.})}{4} = 6.90 \text{ kip-in.}$	Lever arm $= \dfrac{1.00 \text{ in.} + (0.500 \text{ in.}/2)}{2} = 0.625 \text{ in.}$ Thus $M_l = \dfrac{23.0 \text{ kips}(0.625 \text{ in.})}{4} = 3.59 \text{ kip-in.}$

The stress in the rod due to bending equals

$$f_{tb} = \frac{M_l}{Z}$$

where

$$Z = \frac{d^3}{6} = \frac{(1.125 \text{ in.})^3}{6} = 0.237 \text{ in.}^3$$

LRFD	ASD
$f_{tb} = \dfrac{6.90 \text{ kip-in.}}{0.237 \text{ in.}^3} = 29.1 \text{ ksi}$	$f_{tb} = \dfrac{3.59 \text{ kip-in.}}{0.237 \text{ in.}^3} = 15.1 \text{ ksi}$

LRFD	ASD
The axial stress in the rods is: $f_{ta} = \dfrac{P_u}{A} = \dfrac{69.8 \text{ kips}}{4(0.994 \text{ in.}^2)} = 17.6 \text{ ksi}$	The axial stress in the rods is: $f_{ta} = \dfrac{P_a}{A} = \dfrac{42.8 \text{ kips}}{4(0.994 \text{ in.}^2)} = 10.8 \text{ ksi}$
The tensile stress is: $f_t = 29.1 \text{ ksi} + 17.6 \text{ ksi} = 46.7 \text{ ksi}$	The tensile stress is: $f_t = 15.1 \text{ ksi} + 10.8 \text{ ksi} = 25.9 \text{ ksi}$
Combined shear and tension strength: $F_{nt} = 0.75F_u = (0.75)(58 \text{ ksi}) = 43.5 \text{ ksi}$	Combined shear and tension strength: $F_{nt} = 0.75F_u = (0.75)(58 \text{ ksi}) = 43.5 \text{ ksi}$
$F_{nv} = 0.4F_u = (0.4)(58 \text{ ksi}) = 23.2 \text{ ksi (threads included)}$	$F_{nv} = 0.4F_u = (0.4)(58 \text{ ksi}) = 23.2 \text{ ksi (threads included)}$
$\phi F'_{nt} = \phi\left(1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v\right) \le \phi F_{nt}$ $= 0.75\left[(1.3)(43.5 \text{ ksi}) - \dfrac{43.5 \text{ ksi}(9.26 \text{ ksi})}{(0.75)(23.2 \text{ ksi})}\right]$ $= 25.1 \text{ ksi}$ $\le (0.75)(43.5 \text{ ksi}) = 32.6 \text{ ksi}$	$\dfrac{F'_{nt}}{\Omega} = \dfrac{\left(1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v\right)}{\Omega} \le \dfrac{F_{nt}}{\Omega}$ $= \dfrac{\left[(1.3)(43.5 \text{ ksi}) - \dfrac{2.00(43.5 \text{ ksi})(5.78 \text{ ksi})}{(23.2 \text{ ksi})}\right]}{2.00}$ $= 17.4 \text{ ksi}$ $\le \dfrac{(43.5 \text{ ksi})}{2.00} = 21.8 \text{ ksi}$
46.7 ksi > 25.1 ksi **n.g.**	25.9 ksi > 17.4 ksi **n.g.**

Try four 1½-in.-diameter rods.

LRFD	ASD
Shear stress: $f_v = \dfrac{36.8\text{ kips}}{4\left(1.77\text{ in.}^2\right)}$ $= 5.20\text{ ksi}$ $Z = \dfrac{d^3}{6}$ $= \dfrac{\left(1.50\text{ in.}\right)^3}{6}$ $= 0.563\text{ in.}^3$ $f_{tb} = \dfrac{6.90\text{ kip-in.}}{0.563\text{ in.}^3}$ $= 12.3\text{ ksi}$	Shear stress: $f_v = \dfrac{23.0\text{ kips}}{4\left(1.77\text{ in.}^2\right)}$ $= 3.25\text{ ksi}$ $Z = \dfrac{d^3}{6}$ $= \dfrac{\left(1.50\text{ in.}\right)^3}{6}$ $= 0.563\text{ in.}^3$ $f_{tb} = \dfrac{3.59\text{ kip-in.}}{0.563\text{ in.}^3}$ $= 6.38\text{ ksi}$

LRFD	ASD
The axial stress equals: $f_{ta} = \dfrac{P_u}{A}$ $= \dfrac{69.8\text{ kips}}{4\left(1.77\text{ in.}^2\right)}$ $= 9.86\text{ ksi}$ The tensile stress is $f_t = 12.3\text{ ksi} + 9.86\text{ ksi}$ $= 22.2\text{ ksi}$ $\phi F'_{nt} = 0.75\left[\left(1.3\right)\left(43.5\text{ ksi}\right) - \dfrac{43.5\text{ ksi}\left(5.20\text{ ksi}\right)}{\left(0.75\right)\left(23.2\text{ ksi}\right)}\right]$ $= 32.7\text{ ksi}$ $\le \left(0.75\right)\left(43.5\text{ ksi}\right) = 32.6\text{ ksi}$ $\phi F'_{nt} = 32.6\text{ ksi}$ 22.2 ksi < 32.6 ksi **o.k.** Use four 1½-in.-diameter rods.	The axial stress equals: $f_{ta} = \dfrac{P_a}{A}$ $= \dfrac{42.8\text{ kips}}{4\left(1.77\text{ in.}^2\right)}$ $= 6.05\text{ ksi}$ The tensile stress is $f_t = 6.38\text{ ksi} + 6.05\text{ ksi}$ $= 12.4\text{ ksi}$ $\dfrac{F'_{nt}}{\Omega} = \dfrac{\left[\left(1.3\right)\left(43.5\text{ ksi}\right) - \dfrac{2.00\left(43.5\text{ ksi}\right)\left(3.25\text{ ksi}\right)}{\left(23.2\text{ ksi}\right)}\right]}{2.00}$ $= 22.2\text{ ksi}$ $\le \dfrac{\left(43.5\text{ ksi}\right)}{2.00} = 21.8\text{ ksi}$ $\dfrac{F'_{nt}}{\Omega} = 21.8\text{ ksi}$ 12.4 ksi < 21.8 ksi **o.k.** Use four 1½-in.-diameter rods.

Due to the size of the rods, they will have to be positioned beyond the column flanges.

As a matter of interest, assume that welded washers are not provided. It should be noted that a slip of ¾ in. could occur before the anchor rods go into bearing. Check the 1½-in. anchor rods using the authors' suggestion that only two anchor rods should be considered to carry the shear in this case. Bending can be neglected in the rods, but the 0.8 reduction in shear strength per ACI 318 is included. Rather than using the 0.8 reduction, use a 1.25 magnifier on the shear load.

LRFD	ASD
$f_v = \dfrac{\left(1.25\right)\left(36.8\text{ kips}\right)}{2(1.77\text{ in.}^2)}$ $= 13.0\text{ ksi}$ $f_{ta} = \dfrac{69.8\text{ kips}}{4(1.77\text{ in.}^2)}$ $= 9.86\text{ ksi}$ $\phi F'_{nt} = \phi\left(1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v\right) \le \phi F_{nt}$ $= 0.75\left[\left(1.3\right)\left(43.5\text{ ksi}\right) - \dfrac{43.5\text{ ksi}\left(13.0\text{ ksi}\right)}{\left(0.75\right)\left(23.2\text{ ksi}\right)}\right]$ $= 18.0\text{ ksi}$ $\le 0.75(43.5\text{ ksi}) = 32.6\text{ ksi}$ 9.86 ksi < 18.0 ksi **o.k.**	$f_v = \dfrac{\left(1.25\right)\left(23\text{ kips}\right)}{2(1.77\text{ in.}^2)}$ $= 8.12\text{ ksi}$ $f_{ta} = \dfrac{42.8\text{ kips}}{4(1.77\text{ in.}^2)}$ $= 6.05\text{ ksi}$ $\dfrac{F'_{nt}}{\Omega} = \dfrac{\left(1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v\right)}{\Omega} \le \dfrac{F_{nt}}{\Omega}$ $= \dfrac{\left[\left(1.3\right)\left(43.5\text{ ksi}\right) - \dfrac{2.00(43.5\text{ ksi})\left(8.12\text{ ksi}\right)}{\left(23.2\text{ ksi}\right)}\right]}{2.00}$ $= 13.1\text{ ksi}$ $\le \dfrac{43.5\text{ ksi}}{2.00} = 21.8\text{ ksi}$ 6.05 ksi < 13.1 ksi **o.k.**

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Appendix A

SPECIAL CONSIDERATIONS FOR DOUBLE-NUT JOINTS, PRETENSIONED JOINTS AND SPECIAL STRUCTURES

A1. Design Requirements

Anchor rods are sometimes used in special applications that require special design details, such as anchor rods designed without a grout base (double-nut anchor rods), anchor rods in sleeves, pretensioned applications and special moment bases or stools.

Double-nut anchor rods are different from building column anchor rods that may use a setting nut, but are not designed for compression in the completed structure. Double-nut joints are very stiff and reliable for transmitting moment to the foundation. Because tall pole-type structures are non-redundant and are subject to fatigue due to wind flutter, special inspection and tightening procedures should be used. Studies have shown that pretension in the rod between the two nuts improves fatigue strength and assures good load distribution among the anchor rods (Frank, 1980; Kaczinski, Dexter and Van Dien, 1996). The base plates of light and sign standards are not grouted after erection and the rod carries all of the structural load. The anchor rods must be designed for tension, compression and shear and the foundation must be designed to receive these loads from the anchor rods.

Machinery bases and certain columns may require very close alignment of the anchor rods. Oversized sleeves can be used when setting the rods to provide substantial flexibility in the rod so that it can be adjusted to fit the machinery base. The anchorage at the bottom of the rod must be designed to span the sleeve and develop the required bearing on the concrete.

Often machinery, process equipment, and certain building columns may be subject to vibration or cyclical loads, which may in turn subject the anchor rod to fatigue. Pretensioning the rod can improve its fatigue life, but anchor rods can effectively be pretensioned only against steel. Even when tensioning a 55-ksi rod, 24 in. long, it only takes concrete creep/shrinkage of 0.05 in. to relieve all of the pretension. Thus, it is recommended, when it is necessary to pretension an anchor rod, that a steel sleeve be used that is adequate to transfer the anchor rod pretension from the anchor plate to the base plate. See Figure A1.1.

Large mill building columns that have to be set accurately and have large moments at the base can be designed using a stool-type detail as shown in Figure A1.2. The advantage of this type of detail is that the base plate can be set in advance using large oversized holes. The use of the fillet welded stool avoids having to complete-joint-penetration groove weld the column base to the heavy base plate. If the column and base plate are over 2 in. thick, using a complete-joint-penetration weld detail would require special material toughness. The use of the stool has the added advantage that the extended anchor rod length will allow easier adjustment to meet the holes in the stool cap plate.

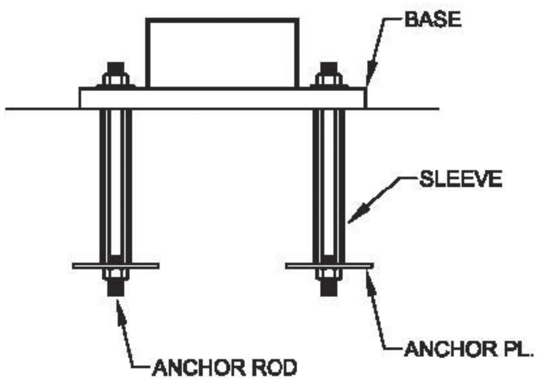


Fig. A1.1. Anchor rods with sleeves.

Fig. A1.2. Column moment base using stool.

A1.1 Compression Limit State for Anchor Rods

With the usual short length involved, the nominal steel compressive strength for anchor rods in double-nut moment joints is the product of its yield stress and the gross area. Yielding could initiate at lower load levels on the reduced area of the threads, but it is assumed that the consequences of this yielding would be relatively minor. The available strength, $\phi_c R_c$ or R_c/Ω_c, is determined with:

$$R_c = F_y A_g$$

$$\phi_c = 0.90 \qquad \Omega_c = 1.67$$

where

R_c = nominal steel compressive strength of an anchor rod, kips

F_y = specified minimum yield stress, ksi

A_g = gross area based on the nominal diameter of the anchor rod for cut threads or the pitch diameter for rolled threads, in.^{2}

Typically, the clear distance under the base plate should not exceed 2.5 in. In general, if the clear distance between the bottom of the bottom leveling nut and the top of concrete is greater than four rod diameters, buckling of the anchor rod should be considered using the column design criteria of the AISC *Specification*.

Headed anchor rods transfer the compressive force to the concrete by bearing of the head, and deformed bars transfer the compressive force to the concrete along their length. The compressive strength of the anchor rod due to concrete failure should be calculated using the American Concrete Institute (ACI 318-08) criteria.

A1.2 Tensile Fatigue Limit State for Anchor Rods

Column base connections subject to more than 20,000 repeated applications of axial tension and/or flexure must be designed for fatigue. When the maximum fatigue stress range is less than the threshold fatigue stress range, 7 ksi, anchor rods need not be further checked for fatigue.

Four-anchor-rod joints are of low cost and suitable for small sign, signal and light supports, and other miscellaneous structures. In other cases, although only four anchor rods may be required for strength, there should ideally be at least six and preferably eight anchor rods in a joint in a non-redundant structure subject to fatigue.

There is a trend toward using fewer very large anchor rods in high-demand dynamically loaded structures. When there are eight anchor rods in a joint, and the first one fails from fatigue, the stress range on the neighboring rods increases only about 25%. These rods would then be expected to last an additional 35 to 50% of the time it took to fail the first rod, assuming the loading remains approximately constant. This gives the column-base-plate connection some measure of redundancy, even if the structure is non-redundant. Fatigue of anchor rod joints with only four rods will fail completely only a short time after the first rod failure.

For circular patterns of six or more double-nut anchor rods, testing has shown that the thickness of the base plate must at least equal or exceed the diameter of the anchor rods, and also that the bending in the anchor rod is negligible when the distance between the bottom of the leveling nut and the top of the concrete is less than the anchor rod diameter (Kaczinski et al., 1996). However, tests on four-anchor-rod patterns show that neither of these simple rules is sufficient when determining the proper base plate thickness and the bending in the anchor rods.

In column-base-plate connections subject to fatigue, the anchor rod will fail before the concrete fatigue strength is reached. Therefore, it is not necessary to consider the fatigue strength of the concrete.

Corrosion protection is particularly important for fatigue-critical anchor rods, since corrosion pitting can degrade the fatigue resistance. It is generally accepted that galvanizing does not decrease the fatigue strength significantly.

Stresses in anchor rods for fatigue analysis should be based on elastic distribution of service loads. The tensile stress area should be used in the computation of stresses in threaded anchors. The stress range should be calculated including the external load range due to repeated live loads and any prying action due to those loads. The bending stress range should be added to the axial stress range to determine the total stress range to check for fatigue.

The S-N curve for galvanized non-pretensioned anchor rods corresponds to detail Category E'; however, the fatigue threshold of 7 ksi is much greater than for other Category E' details. For other cases, 7 ksi is the threshold associated with Category D. If the anchor rod in double-nut moment and pretensioned joints is properly pretensioned, the S-N curve for infinite life increases to Category E; however, the fatigue threshold is not significantly increased. When tests were conducted with an eccentricity of 1:40, the appropriate Category for both pretensioned and non-pretensioned anchor rods was Category E'. Therefore, for design, it is recommended that Category E' be used with a fatigue threshold of 7 ksi, regardless of the pretension. This design would be tolerant of limited misalignment up to 1:40.

Since the fatigue resistance of various grades of anchor rod is the same, it is not advantageous to use strengths higher than 55 ksi in fatigue applications. The fracture toughness of higher strength anchor rods is generally somewhat less.

Base plates, nuts and other components need not be checked for fatigue, unless required by the invoking specification. Axial forces in the anchor rods from tension, compression and flexure must be considered. For all types of joints, the entire force range is assumed to be applied to the anchor rods, even if they are pretensioned. Bending of the anchor rods need not be considered, with the exception of double-nut joints when there are only four anchor rods, or when the clear distance between the bottom of the leveling nut and the concrete exceeds the diameter of the anchor rods. In cases where the bending stress range must be calculated, the minimum bending moment is the shear force in the anchor rod times the distance between the bottom of the base plate and the top of concrete. Shear forces may be ignored for purposes of calculating the fatigue effect, even if they act in combination with the axial forces.

Stress range is defined as the magnitude of the change in service stress due to the application or removal of the service live load. The entire range in stress must be included, even if during part of the cycle the stress is in compression. In the case of a load reversal, the stress range in an individual anchor rod is computed as the algebraic difference between the peak stress due to the live load applied in one direction and the peak stress due to the live load applied in the other direction. If the base plate thickness is less than the diameter of the anchor rods, the applied stress ranges should include any additional tension resulting from prying action produced by the unfactored live load.

The applied stress range is computed by dividing the axial force ranges by the tensile stress area. If bending of the anchor rods is included in the analysis, the bending stress range must be added to the stress range from the axial forces from a consistent load case. The stress range need not be amplified by stress concentration factors.

No further evaluation of fatigue resistance is required if the stress in the anchor rod remains in compression during the entire cycle (including the minimum dead load), or if the stress range is less than the threshold stress range, F_{TH}. The maximum applied stress range must not exceed the allowable stress range computed as follows:

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.333} \geq F_{TH}$$

where

F_{SR} = allowable stress range, ksi
C_f = constant equal to 3.9×10^8
N = number of stress range cycles during the life of the structure
F_{TH} = threshold stress range equal to 7 ksi

For posts and poles, the base plate thickness can influence the fatigue resistance of thin posts. As shown in the following, 3 in. is the optimum thickness, but as long as the thickness is greater than 2 in., the fatigue resistance is generally adequate.

Finite-element analyses illustrate the effect of base plate thickness. In the model generated by the authors, the base plate thickness was varied from 1 to 6 in. Obviously, a 6-in.-thick base plate is unreasonable for most common applications, but was used to show the effect over a large range of thickness. The results of the study indicate that increasing the thickness of the base plate can significantly decrease the stresses immediately adjacent to the pole-to-base plate weld. The reduction in stress is due to the decrease in base plate flexibility that occurs as the base plate becomes thicker (i.e., greater than 1.5 in.). As the base plate gets thicker, it can more efficiently distribute the stresses from the tower to the anchor rods without bending. In thinner base plates, the local base plate bending results in significant bending moments in the tube wall at the connection. For the 1-in.-thick base plate, there are stress concentrations at the bend lines, which means that the membrane stresses are not well distributed around the perimeter, but rather concentrated at the bends in the tube. This observation is consistent with crack initiation locations observed in cracked towers. However, with increasing thickness, the base plate becomes less flexible and the influence of the stress concentrations is less pronounced.

This finding is consistent with fatigue test data from the University of Texas (Koenigs, Botros, Freytag and Frank, 2003). In these tests, a socket joint detail with a 2-in.-thick base plate performed much better in fatigue than one with a 1.5-in.-thick base plate.

To assess the relative effect of base plate thickness, longitudinal stresses on the outer surface from the model are compared in Figure A1.3 at 1.5 in. above the top of the base plate. The stresses were normalized to the stresses extracted from the model of the actual "as-built" 1.25-in.-thick base plate. The results of interest are labeled "outer stress @ 1.5 in." the results for the case with "12 in. hole" may be ignored. It can be seen that, for a base plate 2.25 in. thick, the outer stress at this location decreases to about 65% of what it would be for a 1.25-in.-thick base plate. For a 3-in.-thick base plate, the stress decreases further but not much, down to about 60% of what it would be for a 1.25-in.-thick base plate.

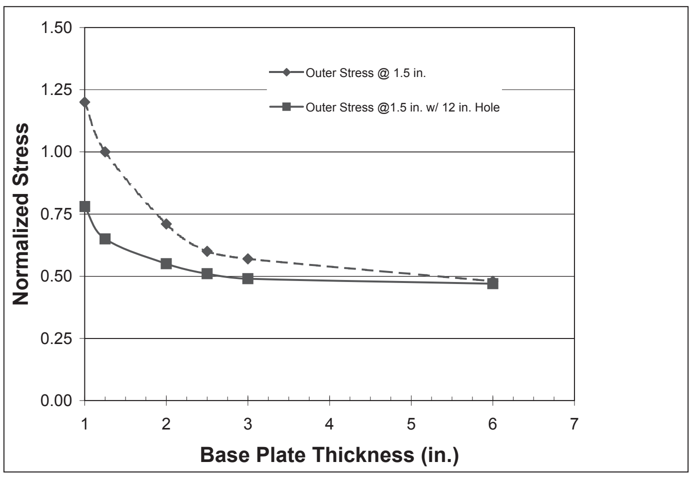


Fig. A1.3. Stresses in base plate.

A2. Installation Requirements for Pretensioned Joints

Proper installation is usually the responsibility of the Contractor. However, the Engineer of Record, or their representative, may witness the inspection and testing.

In any anchor-rod installation, there will be some amount of misalignment. It is assumed that the tolerances will be stated in the invoking specification and that the tolerances correspond with the tolerances specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges*. For anchor rods subjected to fatigue loading, it is also recommended that a tolerance for vertical misalignment of anchor rods be specified as less than 1:40. Provisions should be made to minimize misalignments and to meet required tolerances. The best way to maintain alignment is the use of a template. Templates comprising rings with nuts on both sides at two locations along the length of the anchor rods are recommended.

Vibratory machine joints and double-nut joints designed for Seismic Design Category D or greater, according to SEI/ASCE 7 (ASCE, 2005), or designed for fatigue as described herein, require pretensioning. Failure to follow the nut tightening procedure can lead to inadequately pretensioned anchor rods and associated uneven distribution of loads among the contributing anchor rods. Inadequately tightened bolts can also lead to fatigue failures and further loosening of the nuts under cyclic loading. A less likely outcome of failure to follow the tightening procedure is tightening to the point of damage—plastic deformation and stripping of the threads—which may require removal and replacement.

The starting point for tightening procedures is between 20 and 30% of the final tension. For anchor rods, this is defined as a function of torque, as:

$$T_v = 0.12\ d_b T_m$$

where

T_v = verification torque, in.-kips

d_b = nominal body diameter of the anchor rod, in.

T_m = minimum installation pretension, kips, given in Table A1

Till and Lefke (1994) has shown that a multiplier of 0.12 in this relationship is adequate for common sizes and coatings of anchor rods. Other researchers have suggested a value of 0.20 for less-well-lubricated rods.

If an anchor rod has a nut-head or the head is fastened with nuts, the nut should be prevented from rotation while the anchor rod is tightened. This can be achieved with a jam nut or another type of locking device. The jam nut will affect the ultimate or fatigue strength of the rod.

Very large torques may be required to properly tighten anchor rods greater than 1 in. in diameter. A slugging wrench or a hydraulic torque wrench is required. For the leveling nuts, an open-end slugging wrench may be used.

A2.1 Double-Nut Joints

Prior to installation of anchor rods in a double-nut moment joint, an anchor-rod rotation capacity test should be performed with at least one anchor rod from each lot. This test attempts to recreate the conditions to which the anchor rod will be subjected during installation.

After the test and before placing the concrete, anchor rods should be secured to a template or other device to avoid movement during placing and curing of the concrete that may lead to misalignments larger than what may be tolerated. The hole pattern in the template should be verified by comparing the top template to the base plate to be erected if it is on site.

Beveled washers should be used:

1. Under the leveling nut if the slope of the bottom face of the base plate has a slope greater than 1:20.
2. Under the leveling nut if the leveling nut could not be brought into firm contact with the base plate.
3. Under the top nut if the slope of the top face of the base plate has a slope greater than 1:20.
4. Under the top nut if the top nut could not be brought into firm contact with the base plate.

If a beveled washer is required, the contractor should disassemble the joint, replace nuts adding the beveled washer(s) and retightening in a star pattern to the initial condition. Beveled washers can typically accommodate a slope up to 1:6.

Top nuts should be pretensioned. The procedure for pretensioning is a turn-of-nut procedure, although they are inspected using torque. Pretensioning the nuts should be accomplished in two full tightening cycles following a star pattern.

Experience indicates that even properly tightened galvanized anchor rods can subsequently become loose, especially in the first few days after installation, presumably because of creep in the galvanizing. Therefore, a final installation check should be made after at least 48 hours using a calibrated wrench and 110% of the torque calculated using the torque equation. It is expected that properly tightened joints will not move even if 110% of the minimum installation torque is applied. If a rod assembly cannot achieve the required torque, it is very likely that the threads have stripped.

When it is required that the nuts be prevented from loosening, a jam nut or other suitable device can be used. Any other method for preventing nut loosening should be approved by the Engineer of Record. Tack welding the top side of the top nut has been used, although this is not consistent with the AWS Structural Welding Code. While tack welding to the unstressed top of the anchor rod is relatively harmless, under no circumstance should any nut be tack welded to the washer or the base plate.

Table A1. Minimum Anchor Rod Pretension for Double-Nut-Moment Joints				
Anchor Rod Diameter, in.	Minimum Anchor Rod Pretension T_m, kips			
	ASTM F1554 Rod Grade 36[a]	ASTM F1554 Rod Grade 55[b]	ASTM F1554 Rod Grade 105[b]	ASTM A615 and A706 Bars Grade 60[b]
½	4	6	11	7
⅝	7	10	17	11
¾	10	15	25	16
⅞	13	21	35	22
1	18	27	45	29
1⅛	22	34	57	37
1¼	28	44	73	47
1½	41	63	105	67
1¾	55	86	143	91
2	73	113	188	–
2¼	94	146	244	156
2½	116	180	300	–
2¾	143	222	370	–
3	173	269	448	–
3¼	206	320	533	–
3½	242	375	625	–
3¾	280	435	725	–
4	321	499	831	–

[a] Equal to 50% of the specified minimum tensile strength of rods, rounded to the nearest kip.
[b] Equal to 60% of the specified minimum tensile strength of rods, rounded to the nearest kip.

Installation sequence:

1. The torque wrench used for tightening the nuts or final torque verification should have a torque indicator that is calibrated annually. A certification of such calibration should be available to the Engineer of Record. A torque multiplier may be used.

2. The verification torque is computed using

$$T_v = 0.12 d_b T_m$$

3. Prior to placing the anchor rods in the concrete, an anchor rod rotation capacity test should be conducted with at least one anchor rod from every lot. This test should be conducted using the base plate or a plate of equivalent grade, thickness and finish. The plate must be restrained against movement from the torque that will be applied. The test consists of Steps 11 through 19 that follow, with the exception of Step 13 (since there is only one anchor rod). The nut should be rotated to at least the required rotation given in Table A2. After the test, the nuts should be removed and inspected for damage to their threads. Then, the anchor rod is removed from the test plate and restrained while the nuts should be turned onto the bolts at least one rod diameter past the location of the leveling nut and top nut in the test, then backed off by one worker using an ordinary wrench (without a cheater bar). The threads are considered damaged if an unusual effort is required to turn the nut. If there is no damage to the anchor rod or nut during this test, they may be used in the joint. If there is damage to the threads or an inability to attain at least the verification torque, the lot of anchor rods should be rejected.

4. Anchor rods should be secured against relative movement and misalignment.

Table A2. Nut Rotation for Turn-of-Nut Pretensioning of UNC Threads		
Anchor Rod Diameter, in.	Nut Rotation[a,b,c]	
	F1554 Grade 36	F1554 Grades 55 and 105 A615 Grade 60 and 75 and A706 Grade 60
≤ 1½	⅙ turn	⅓ turn
> 1½	1/12 turn	⅙ turn

[a] Nut rotation is relative to anchor rod. The tolerance is plus 20 degrees.
[b] Applicable only to UNC threads.
[c] Beveled washer should be used if: a) the nut is not into firm contact with the base plate or b) the outer face of the base plate is sloped more than 1:40.

5. A template is required for leveling the leveling nuts. The hole pattern in the template should be verified. Any deviation between the hole positions outside of the tolerances must be reported to the Engineer of Record. The template set (or other device) with anchor rods should be secured in its correct position in accordance with the contract documents.

6. The concrete should be placed and cured.

7. If a top template is above the concrete surface, it may be removed 24 hours after placing the concrete.

8. The exposed part of the anchor rods should be cleaned with a wire brush or equivalent and lubricated if galvanized.

9. The anchor rods should be inspected visually to verify that there is no visible damage to the threads and that their position, elevation and projected length from the concrete are within the tolerances specified in the contract documents. In the absence of required tolerances, the position, elevation and projected length from the concrete should be according to the AISC *Code of Standard Practice for Steel Buildings and Bridges*. If the joint is required to be designed for fatigue, the misalignment from vertical should be no more than 1:40. Nuts should be turned onto the bolts well past the elevation of the bottom of the leveling nut and backed off by a worker using an ordinary wrench without a cheater bar. Thread damage requiring unusually large effort should be reported to the Engineer of Record.

10. If threads of galvanized anchor rods were lubricated more than 24 hours before placing the leveling nut, or have been wet since they were lubricated, the exposed threads of the anchor rod should be relubricated. Leveling nuts should be cleaned, threads and bearing surfaces lubricated (if galvanized), and placed on the anchor rods.

11. Leveling nut washers should be placed on the anchor rods. Beveled washers should be used if the nut cannot be brought into firm contact with the base plate.

12. The template should be placed on top of the leveling nuts to check the level of the nuts. In some cases, if indicated in the contract documents, it is permitted to set the base plate at some other angle other than level. If this angle exceeds 1:40, beveled washers should be used. Verify that the distance between the bottom of the bottom leveling nut and the top of concrete is not more than one anchor rod diameter (unless specified otherwise in the contract documents).

13. The base plate and structural element to which it is attached should be placed.

14. Top nut washers should be placed. Beveled washers should be used if the nut can not be brought into firm contact with the base plate.

15. Threads and bearing surfaces of the top nuts should be lubricated, placed and tightened to between 20 and 30% of the verification torque following a star pattern.

16. Leveling nuts should be tightened to between 20 and 30% of the verification torque following a star pattern.

17. Before further turning the nuts, the reference position of the top nut in the initial condition should be marked on an intersection between flats with a corresponding reference mark on the base plate at each bolt. Top nuts should be turned in increments following a star pattern (using at least two full tightening cycles) to the nut rotation specified in Table A2 if UNC threads are used. If 8UN threads are used, the appropriate nut rotation should be shown in the contract documents or specified by the Engineer of Record. After tightening, the nut rotation should be verified.

18. A torque wrench should be used to verify that a torque at least equal to the verification torque is required to additionally tighten the leveling nuts and the top nuts. An inability to achieve this torque means it is likely that the threads have stripped and this must be reported to the Engineer of Record.

19. After at least 48 hours, the torque wrench should again be used to verify that a torque at least equal to 110% of the verification torque is required to additionally tighten the leveling nuts and the top nuts. For cantilever or other nonredundant structures, this verification should be made at least 48 hours after erection of the remainder of the structure and any heavy attachments to the structure.

20. If the joint was designed for Seismic Design Category D or greater according to SEI/ASCE 7, or designed for fatigue, the nut should be prevented from loosening unless a maintenance plan is in place to verify at least every four years that a torque equal to at least 110% of the verification torque is required to additionally tighten the leveling nuts and the top nuts.

A2.2 Pretensioned Joints

The installation procedures for pretensioned joints are very similar to the first steps for double-nut-moment joints, except for the inclusion of the sleeve. The sleeve must be cleaned and sealed off to prevent inclusion of debris.

Anchor rods are typically tensioned using a center-hole ram with access to the nut for retightening. The nut is tightened down while the tension is maintained on the anchor rod, and the anchor rod tension is released. It is recognized that part of the tension will be lost to relaxation after the tension is released. Since there are many variations of pretensioned joints, the Engineer of Record should provide the specific procedures for tightening these joints.

Installation sequence:

1. The assembly of sleeve and anchor rod should be secured in its correct position in accordance with the contract documents.

2. If a template is used, the hole pattern should be verified by comparing the top template to the base plate to be erected and any deviation between the hole positions outside of the tolerances must be reported to the Engineer of Record.

3. The concrete should be placed and cured.

4. If a top template is above the concrete surface, it may be removed no sooner than 24 hours after placing the concrete.

5. The exposed part of the anchor rods should be cleaned with a wire brush or equivalent and lubricated.

6. The opening of the sleeve should be cleaned of debris and sealed off.

7. After removal of the template, if any, the anchor rods should be inspected visually to verify that there is no visible damage to the threads, and that their position, elevation and projected length from the concrete are within the tolerances specified in the contract documents. In the absence of required tolerances, the position, elevation and projected length from the concrete should be within the tolerances specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges*. The nuts should be turned onto the bolts at least one rod diameter past the elevation of the bottom of the base plate and backed off by a worker using an ordinary wrench without a cheater bar. Any damage resulting in unusual effort to turn the nut should be reported to the Engineer of Record.

8. The base plate and attached structural element, or piece of equipment or machinery, should be placed.

9. Washers should be placed.

10. If threads of anchor rods were lubricated more than 24 hours before placing the nut or have been wet since they were lubricated, the exposed threads of the anchor rod should be relubricated. Nuts should be cleaned and the threads and bearing surfaces lubricated.

11. The pretension and pretensioning method should be as specified in the contract documents, along with the procedures and requirements for an installation verification test, if necessary.

A3. Inspection and Maintenance after Installation

Regular inspection and maintenance should be conducted for joints that are designed for the fatigue. All joints designed for Seismic Design Category D or greater, according to SEI/ASCE 7, should also be inspected and maintained as follows after a significant seismic event.

1. **Anchor rod appearance**—Draw a diagram of the anchor rod pattern and number in a clockwise pattern. Check each anchor rod for corrosion, gouges or cracks. Suspected cracks may be more closely examined using the dye-penetrant technique. If there is heavy corrosion near the interface with the concrete, there may be more severe corrosion hidden below the concrete where the pocket around the anchor rod stays wet. Verify that all the anchor rods have top nuts with washers. Lock washers

should not be used. Galvanized nuts or washers should not be used with unpainted weathering steel. Check for inadequately sized washers for oversize holes. If there is no grout pad, verify that all the anchor rods have leveling nuts with washers. Check for loose nuts, gouges, thread damage or corrosion. Note any anchor rods that are significantly misaligned or bent to fit in the base plate hole. Note any anchor rods that are not flush with or projecting past the nut. If the anchor rod is not projecting past the nut, measure the distance from the top of the nut to the top of the anchor rod.

2. **Sounding the anchor rods**—Anchor rods may be struck by a hammer (a large ball peen hammer is suggested) to detect broken rods. Strike the side of the top nut and the top of the rod. Good tight anchor rods will all have a similar ring. Broken or loose anchor rods will have a distinctly different and duller sound.

3. **Tightness of anchor rod nuts**—It should be verified that the top nuts still have a sound tack weld (at the top of the top nut only) or a jam nut. Tack welds to the washer or the base plate are undesirable and should be reported. If one of these is not used to prevent loosening of the nut, the tightness should be verified by applying a torque equal to 110% of the torque computed using the torque equation, in accordance with Step 20 of the installation procedure for double-nut joints.

   If one nut in a joint is loose (the tack weld is fractured or the nut does not reach the required torque), it should be unscrewed, cleaned, inspected for possible thread stripping, lubricated, placed and brought to the initial condition and retightened to the pretension specified in Table A1 using the turn-of-nut method.

   If more than one nut in a joint is loose, the entire joint should be disassembled, all the anchor rods visually inspected and the joint reassembled with new nuts. If more than one nut is loose, the joint may have been poorly installed or fatigue problems may exist. A close following of the performance of the joint should be made.

4. **Ultrasonic test of anchor rods**—An ultrasonic test of anchor rods need be performed only if:
   - Welded repairs have been made.
   - Similar structures subject to similar loading have had fatigue problems.
   - Anchor rods were not adequately designed for fatigue in accordance with this Specification.

   The inspection should include at least:

   a. Verification that the joint is kept free of debris, water and vegetation.
   b. Verification that there is not severe corrosion, gouges or cracks.
   c. Verification that the grout and concrete in the vicinity of the anchor rods is in good condition.
   d. A hammer sound test of anchor rods.
   e. Verification of the tightness of nuts. It should be verified that the nuts still have a jam nut or other locking device or the tightness should be verified by applying 110% of the verification torque.
   f. Retightening of anchor rods, if needed.

If similar structures subject to similar loading have had anchor rod fatigue cracking problems, an ultrasonic test of anchor rods should be performed. The top of the rod or extension should be ground flush and the ultrasonic test and its interpretation should be in accordance with a procedure approved by a qualified engineer.

Appendix B

TRIANGULAR PRESSURE DISTRIBUTION

B.1 Introduction

When a column is subjected to either an eccentricity of axial load or a moment due to base rigidity, a simplifying assumption must be made to determine a design pressure on the base plate. Throughout this Design Guide, design procedures and examples have been presented using an assumption of a uniform pressure distribution on the base plate, which is consistent with procedures adopted by ACI. Alternatively, it is permissible to assume a triangular pressure distribution on the base plate.

This alternative does not in and of itself represent an elastic design or an ASD approach to design. Rather, both triangular and uniform distributions represent simplifying approximations that are equally applicable for LRFD and ASD applications. The use of a triangular pressure distribution, as shown in Figure B.1, will often require slightly thicker base plates and slightly smaller anchor rods than the uniform pressure approach, since the centroid of the pressure distribution is closer to the cantilevered edge of the plate.

B.2 Determining Required Base Plate Thickness from Required Strength

At times the base plate designer may wish to determine the base pressure separately from determining the required thickness. To facilitate this approach, a general format for sizing the base plate thickness based on the flexural moment caused by the pressure on the plate surface can be derived by setting the required flexural moment strength over the width of the base plate equal to the available flexural strength and solving for t:

$$t_{req} = \sqrt{\frac{4M_{u\,pl}}{\phi_b BF_y}} \quad \text{(LRFD)}$$

$$t_{req} = \sqrt{\frac{4M_{a\,pl}\Omega_b}{BF_y}} \quad \text{(ASD)}$$

where

ϕ_b = 0.90 and $\Omega_b = 1.67$

The designer may wish to solve directly for the plate thickness based on the applied loads and the geometry of the base conditions. However, an assumption of pressure distribution must be made to determine the moment used in the preceding equations. This process is illustrated in the following sections.

B.3 Determination of Required Stress and Effects of Eccentricity

The axial and flexural components of the applied loads are treated separately to determine the resulting stresses between the base plate and foundation, and then combined by superposition to calculate the pressure distribution across the plate.

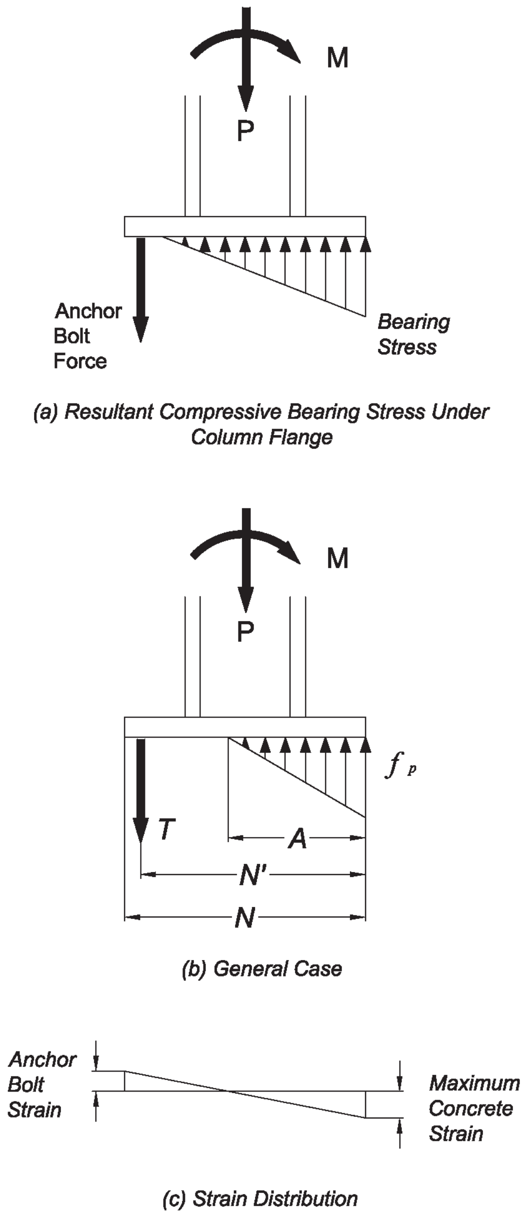


Fig. B.1. Elastic analysis for axial load plus moment, triangular distribution.

Assuming that the supported column and base plate have coincident centroids, if:

$$f_{pa} = P_r / A_1$$

$$f_{pb} = M_r / S_{pl}$$

where

P_r = applied axial compressive load, kips
M_r = applied bending moment, kip-in.
A_1 = area of base plate plan dimensions ($B \times N$), in.^{2}
S_{pl} = section modulus of base plate area with respect to direction of applied moment, in.^{3} For bending of a rectangular plate, $S_{pl} = \dfrac{BN^2}{6}$.

Equating $f_{pa} = f_{pb}$ will result in a triangular pressure distribution across the length of the base plate in the direction of the applied moment, with the maximum pressure on the compressive side of the moment and zero pressure on the tensile side of the moment. This is the theoretical condition where no tension exists on the interface between the base plate and foundation, and any applied additional moment at the same axial compressive load will result in tension.

The applied bending moment can be expressed as an axial compressive force applied at a distance from the centroid of the column/base plate. This distance, designated as the eccentricity, e, can be determined as

$$e = M_r / P_r$$

The balance point where the base plate pressure changes from zero tension to positive tension can be defined by a relationship between the eccentricity and the base plate length or width, as applicable. It was previously indicated that this transition point occurs when $f_{pa} = f_{pb}$. Therefore, assuming the applied moment is parallel to N:

$$\frac{P}{A_1} = \frac{M}{S_{pl}}$$

$$\frac{P}{BN} = \frac{Pe}{\left(\dfrac{BN^2}{6}\right)}$$

$$e = N/6$$

This point, where $e = N/6$, is commonly called the *kern* of the base plate.

B.4.1 Design Procedure for a Small Moment Base

1. Choose trial base plate sizes (B and N) based on the geometry of the column and the minimum of four anchor rods requirement.

$$N > d + (2)(3 \text{ in.})$$

$$B > b_f + (2)(3 \text{ in.})$$

2. Determine plate cantilever dimension, m or n, in the direction of the applied moment.

$$m = (N - 0.95d)/2$$

$$n = (B - 0.80b_f)/2$$

3. Determine applied loads, P and M (P_u and M_u for LRFD, P_a and M_a for ASD) based on SEI/ASCE 7 load combinations.

4. Determine eccentricity e and e_{kern}.

$$e = M/P$$

$$e_{kern} = N/6$$

If $e \le e_{kern}$, this is a small moment base, no tension exists between the base plate and the foundation. See Figure B.2a.

If $e > e_{kern}$, this is a large moment base, and must be designed for tension anchorage. See Section B4.2.

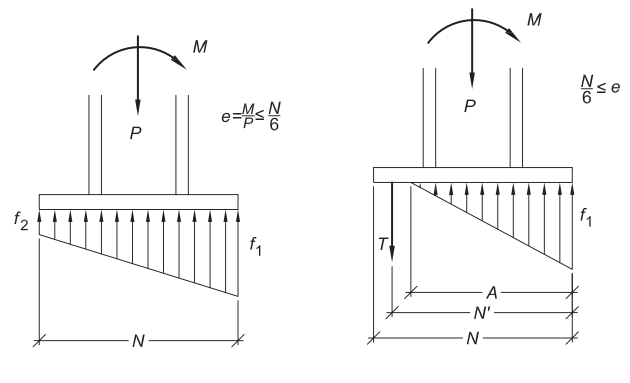
5. Determine base pressures.

Due to axial compression:

$$f_{p(ax)} = \frac{P}{A_1}$$

$$= \frac{P}{BN}$$

where
$P = P_u$ for LRFD, P_a for ASD



(a) Small Eccentricity - Bearing on Full Plate

(b) Large Eccentricity - Bearing on Partial Plate

Fig. B.2. Effect of eccentricity on bearing.

Due to applied moment:

$$f_{p(b)} = \frac{M}{S_{pl}}$$

$$= \frac{6Pe}{BN^2}$$

where
$P = P_u$ for LRFD, P_a for ASD
$M = M_u$ for LRFD, M_a for ASD

Combined pressure:

$$f_{p(max)} = f_{p(ax)} + f_{p(b)}$$

$$= \frac{P}{BN}\left(1+\frac{6e}{N}\right) \le f_{p\ avail}$$

where

$f_{p\ avail} = \phi_c 0.85 f'_c$ (LRFD)

$f_{p\ avail} = \dfrac{0.85 f'_c}{\Omega_c}$ (ASD)

If $f_{p(max)} \ge f_{p\ avail}$, adjust the base plate dimensions.

$$f_{p(min)} = f_{p(ax)} - f_{p(b)}$$

$$= \frac{P}{BN}\left(1-\frac{6e}{N}\right)$$

where
$P = P_u$ for LRFD, P_a for ASD

6. Determine pressure at m distance from $f_{p(max)}$.

$$f_{p(m)} = f_{p(max)} - 2f_{p(b)}(m/N)$$

7. Determine M_{pl} for bending about critical planes at m and n:

Bending of a 1-in.-wide strip of plate about a plane at m, in the direction of the applied moment:

$$M_{pl} = \left(f_{p(m)}\right)\left(\frac{m^2}{2}\right) + \left(f_{p(max)} - f_{p(m)}\right)\left(\frac{m^2}{3}\right)$$

For bending about a plane at n, perpendicular to the applied moment:

$$M_{pl} = f_{p(ax)}\left(\frac{n^2}{2}\right)$$

The critical moment is the larger of M_{pl} about the m and n critical planes.

8. Determine required plate thickness.

$$t_{req} = \sqrt{\frac{4M_{u\ pl}}{\phi_b B F_y}} \text{ (LRFD)}$$

$$t_{req} = \sqrt{\frac{4M_{a\ pl}\ \Omega_b}{BF_y}} \text{ (ASD)}$$

where
ϕ_b = 0.90 and $\Omega_b = 1.67$

B.4.2 Design Procedure for a Large Moment Base

When the effective eccentricity is large (greater than e_{kern}), there is a tensile force in the anchor rods due to the moment. See Figure B.2b. To calculate this force, the anchor rod force, T, and the length of bearing, A, must be determined, as shown in Figure B.3.

By static equilibrium, the following equations can be derived:

$$T + P = \frac{f_p AB}{2}$$

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

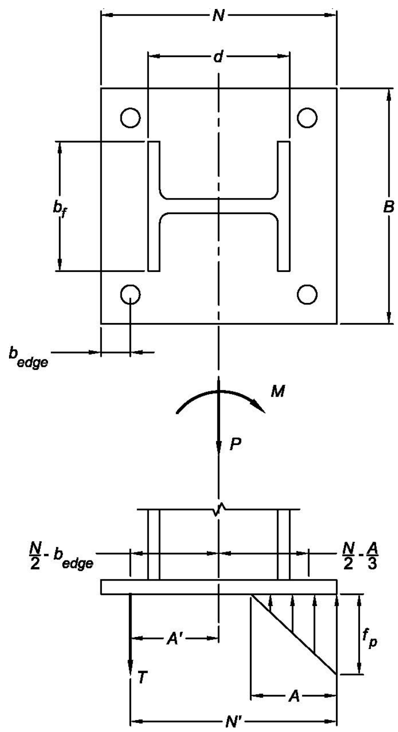


Fig. B.3. General definition of variables.

where

A' = the distance between the anchor rod and the column center, in.
T = T_u for LRFD, T_a for ASD
P = P_u for LRFD, P_a for ASD
M = M_u for LRFD, M_a for ASD

By summing the moments about the resulting rod force and solving as a quadratic function, the following expression can be determined for calculating the bearing distance, A:

$$A = \frac{3N' \pm \sqrt{\left(3N'\right)^2 - \dfrac{24\left(PA' + M\right)}{f_p B}}}{2}$$

where

P = P_u for LRFD, P_a for ASD
M = M_u for LRFD, M_a for ASD

The resulting tensile force in the anchor rods is then:

$$T = \frac{f_p AB}{2} - P$$

where

T = T_u for LRFD, T_a for ASD
P = P_u for LRFD, P_a for ASD

The design procedure is as follows:

1. Determine the available bearing strength, $\phi_c P_p$ or P_p/Ω_c, with

$$P_p = 0.85 f'_c A_1 \sqrt{A_2/A_1} \le 1.7 f'_c A_1$$

$$\phi_c = 0.65 \qquad \Omega_c = 2.50$$

2. Choose trial base plate sizes (B and N) based on geometry of the column and the minimum of four anchor rods requirement.

3. Determine the length of bearing, A, equal to the smallest positive value from the previous equation. If the value is reasonable, go on to the next step. If it is close to the value of N', the solution is not practical since this implies that bearing exists in the vicinity of the anchor rod. If this were so, the anchor rod could not develop its full tensile strength. It is then necessary to return to Step 2 and choose another, larger plate size.

4. Determine the resultant anchor rod force, T, from the equation above. If it is reasonable, go to the next step. Otherwise return to Step 2.

5. Determine the required flexural strength per inch of plate as the greater of the moment due to the pressure and the moment due to tension in the anchor rods. Each is to be determined at the appropriate critical section.

6. Determine the plate thickness based on the required flexural strength per inch of plate:

LRFD	ASD
$t_p = \sqrt{\dfrac{4M_{upl}}{\phi_b F_y}}$	$t_p = \sqrt{\dfrac{4M_{apl}\Omega_b}{F_y}}$

B.5.1 Example: Small Moment Base Plate Design, Triangular Pressure Distribution Approach

Design a base plate for axial dead and live loads equal to 100 kips and 160 kips, respectively, and moments from the dead and live loads equal to 250 kip-in. and 400 kip-in., respectively. Bending is about the strong axis for the W12×96 column with d = 12.7 in. and b_f = 12.2 in. The ratio of the concrete to base plate area is unity. F_y of the base plate is 36 ksi and f_c' of the concrete is 4 ksi.

1. Choose trial base plate sizes (B and N) based on geometry of the column and the four anchor rod requirement.

$$N > d + (2 \times 3.0 \text{ in.}) = 12.7 \text{ in.} + 6 \text{ in.} = 18.7 \text{ in.}$$

$$B > b_f + (2 \times 3.0 \text{ in.}) = 12.2 \text{ in.} + 6 \text{ in.} = 18.2 \text{ in.}$$

Try N = 19 in., B = 19 in.

2. Determine plate cantilever dimension, m or n.

$$\begin{aligned} m &= \frac{(N - 0.95d)}{2} \\ &= \frac{19 \text{ in.} - 0.95(12.7 \text{ in.})}{2} \\ &= 3.47 \text{ in.} \end{aligned}$$

$$\begin{aligned} n &= \frac{B - 0.80b_f}{2} \\ &= \frac{19 \text{ in.} - 0.80(12.2 \text{ in.})}{2} \\ &= 4.62 \text{ in.} \end{aligned}$$

3. Determine applied loads, P and M, based on ASD or LRFD load combinations.

LRFD
$P_u = 1.2(100 \text{ kips}) + 1.6(160 \text{ kips})$ $= 376 \text{ kips}$ $M_u = 1.2(250 \text{ kip-in.}) + 1.6(400 \text{ kip-in.})$ $= 940 \text{ kip-in.}$
ASD
$P_a = 100 \text{ kips} + 160 \text{ kips}$ $= 260 \text{ kips}$ $M_a = 250 \text{ kip-in.} + 400 \text{ kip-in.}$ $= 650 \text{ kip-in.}$

4. Determine eccentricity e and e_{kern}.

LRFD	ASD
$e_u = \dfrac{M_u}{P_u} = \dfrac{940 \text{ kip-in.}}{376 \text{ kips}} = 2.50 \text{ in.}$	$e_a = \dfrac{M_a}{P_a} = \dfrac{650 \text{ kip-in.}}{260 \text{ kips}} = 2.50 \text{ in.}$

$$e_{kern} = \frac{N}{6} = \frac{19 \text{ in.}}{6} = 3.17 \text{ in.}$$

$e = 2.5$ in. < $e_{kern} = 3.17$ in. Thus, this is a small moment base and no tension exists between the base plate and foundation.

5. Determine base pressures for a 1-in. strip of plate.

Due to axial compression:

$$f_{p(ax)} = \frac{P}{A_1} = \frac{P}{BN}$$

LRFD	ASD
$f_{pu(ax)} = \dfrac{P_u}{BN} = \dfrac{376 \text{ kips}}{(19 \text{ in.})(19 \text{ in.})} = 1.04 \text{ ksi}$	$f_{pa(ax)} = \dfrac{P_a}{BN} = \dfrac{260 \text{ kips}}{(19 \text{ in.})(19 \text{ in.})} = 0.720 \text{ ksi}$

Due to applied moment:

$$f_{p(b)} = \frac{M}{S_{pl}} = \frac{6Pe}{BN^2}$$

LRFD	ASD
$f_{pu(b)} = \dfrac{6P_u e}{BN^2} = \dfrac{6(376 \text{ kips})(2.50 \text{ in.})}{(19 \text{ in.})(19 \text{ in.})^2} = 0.822 \text{ ksi}$	$f_{pa(b)} = \dfrac{6P_a e}{BN^2} = \dfrac{6(260 \text{ kips})(2.50 \text{ in.})}{(19 \text{ in.})(19 \text{ in.})^2} = 0.569 \text{ ksi}$

Combined pressure:

LRFD	ASD
$f_{pu(max)} = f_{pu(ax)} + f_{pu(b)}$ $= 1.04 \text{ ksi} + 0.822 \text{ ksi}$ $= 1.86 \text{ ksi}$ $f_{pu(min)} = f_{pu(ax)} - f_{pu(b)}$ $= 1.04 \text{ ksi} - 0.822 \text{ ksi}$ $= 0.218 \text{ ksi}$	$f_{pa(max)} = f_{pa(ax)} + f_{pa(b)}$ $= 0.720 \text{ ksi} + 0.569 \text{ ksi}$ $= 1.29 \text{ ksi}$ $f_{pa(min)} = f_{pa(ax)} - f_{pa(b)}$ $= 0.720 \text{ ksi} - 0.569 \text{ ksi}$ $= 0.151 \text{ ksi}$

6. Determine pressure at critical bending plane (m distance from $f_{p(max)}$)

LRFD	ASD
$f_{pu(m)} = f_{pu(max)} - 2f_{pu(b)}\left(\dfrac{m}{N}\right)$ $= 1.86 \text{ ksi} - 2(0.822 \text{ ksi})\left(\dfrac{3.47 \text{ in.}}{19 \text{ in.}}\right)$ $= 1.56 \text{ ksi}$	$f_{pa(m)} = f_{pa(max)} - 2f_{pa(b)}\left(\dfrac{m}{N}\right)$ $= 1.29 \text{ ksi} - 2(0.569 \text{ ksi})\left(\dfrac{3.47 \text{ in.}}{19 \text{ in.}}\right)$ $= 1.08 \text{ ksi}$

7. Determine M_{pl} for bending about critical planes at m and n:

Bending of a 1-in.-wide strip of plate about a plane at m, in the direction of the applied moment:

LRFD	ASD
$M_{u\,pl} = \left(f_{pu(m)}\right)\left(\dfrac{m^2}{2}\right) + \left(f_{pu(max)} - f_{pu(m)}\right)\left(\dfrac{m^2}{3}\right)$ $= \left(1.56 \text{ ksi}\right)\dfrac{(3.47 \text{ in.})^2}{2}$ $+ \left(1.86 \text{ ksi} - 1.56 \text{ ksi}\right)\dfrac{(3.47 \text{ in.})^2}{3}$ $= 10.6 \text{ kip-in./in.}$	$M_{a\,pl} = \left(f_{pa(m)}\right)\left(\dfrac{m^2}{2}\right) + \left(f_{pa(max)} - f_{pa(m)}\right)\left(\dfrac{m^2}{3}\right)$ $= \left(1.08 \text{ ksi}\right)\dfrac{(3.47 \text{ in.})^2}{2}$ $+ \left(1.29 \text{ ksi} - 1.08 \text{ ksi}\right)\dfrac{(3.47 \text{ in.})^2}{3}$ $= 7.34 \text{ kip-in./in.}$

For bending about a plane at n, perpendicular to the applied moment: For the case of axial loads plus small moments the following procedure can be used (using the axial load only). For axial loads plus large moments a more refined analysis is required.

LRFD	ASD
$M_{u\,pl} = f_{pu(ax)}\left(\dfrac{n^2}{2}\right)$ $= 1.04 \text{ ksi}\dfrac{(4.62 \text{ in.})^2}{2}$ $=11.1$ kip-in./in.	$M_{a\,pl} = f_{pa(ax)}\left(\dfrac{n^2}{2}\right)$ $= 0.720 \text{ ksi}\dfrac{(4.62 \text{ in.})^2}{2}$ $= 7.68$ kip-in./in.

The critical moment is the larger of M_{pl} about m and n critical planes.

LRFD	ASD
$M_{u\,crit} = 11.1$ kip-in./in.	$M_{a\,crit} = 7.68$ kip-in./in.

8. Determine required plate thickness:

   Note: Since the M_{pl} is expressed in units of kip-in./in., the plate thickness expressions can be formatted without the plate width (B) as such:

LRFD	ASD
$t_{u\,req} = \sqrt{\dfrac{4M_{u\,crit}}{\phi_b F_y}}$ $= \sqrt{\dfrac{(4)(11.1 \text{ kip-in.})}{(0.90)36 \text{ ksi}}}$ $= 1.17$ in.	$t_{a\,req} = \sqrt{\dfrac{4M_{a\,crit}\Omega_b}{F_y}}$ $= \sqrt{\dfrac{(4)(7.68 \text{ kip-in.})(1.67)}{36 \text{ ksi}}}$ $= 1.19$ in.

9. Use plate size:

$N = 19$ in.
$B = 19$ in.
$t = 1¼$ in.

B.5.2 Example: Large Moment Base Plate Design, Triangular Pressure Distribution Approach

Design the base plate shown in Figure B.4 for an ASD and LRFD required strength of 60 and 90 kips, respectively, and ASD and LRFD moments of 480 kip-in. and 720 kip-in., respectively. The ratio of the concrete to base plate area (A_2/A_1) is 4.0. Bending is about the strong axis for the W8×31 column with $d = b_f = 8.00$ in. F_y of the base plate and anchor rods is 36 ksi and f'_c of the concrete is 3 ksi.

1. Determine the available bearing strength.

LRFD	ASD
$P_u = 90$ kips $M_u = 720$ kip-in. $\dfrac{\phi_c P_p}{A_1} = \phi_c 0.85 f'_c\sqrt{A_2/A_1} \le \phi_c 1.7 f'_c$ $= 0.65(0.85)(3.0 \text{ ksi})(2)$ $= 3.32$ ksi $\le 0.65(1.7)(3.0\text{ksi}) = 3.32$ ksi $f_{pu} = 3.32$ ksi	$P_a = 60$ kips $M_a = 480$ kip-in. $\dfrac{P_p}{\Omega_c A_1} = \dfrac{0.85 f'_c\sqrt{A_2/A_1}}{\Omega_c} \le \dfrac{1.7 f'_c}{\Omega_c}$ $= \dfrac{(0.85)(3.0 \text{ ksi})(2)}{2.50}$ $= 2.04$ ksi $\le \dfrac{(1.7)(3.0 \text{ ksi})}{2.50} = 2.04$ ksi $f_{pa} = 2.04$ ksi

2. Assume a 14 in. × 14 in. base plate. The effective eccentricity is:

LRFD	ASD
$e = 720$ kip-in./90 kips $= 8.00$ in.	$e = 480$ kip-in./60 kips $= 8.00$ in.

$e > e_{kern} = N/6 = 2.33$ in., therefore anchor rods are required to resist the tensile force. The anchor rods are assumed to be 1.5 in. from the plate edge.

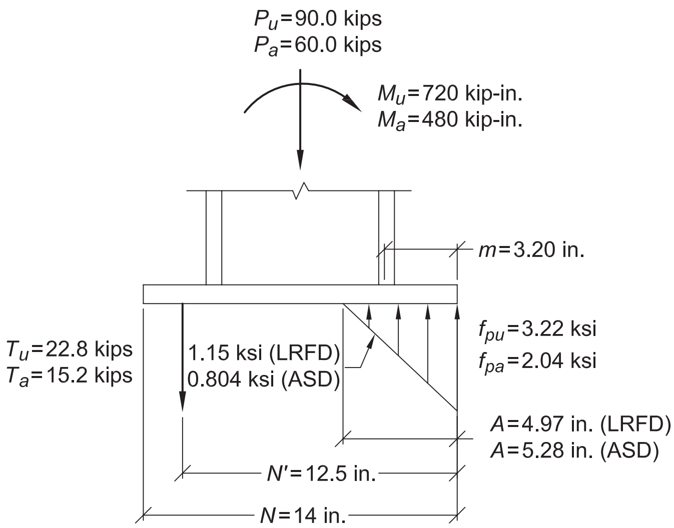


Fig. B.4. Design example with large eccentricity.

3. Determine the length of bearing:

LRFD
$A = \dfrac{3N' \pm \sqrt{\left(3N'\right)^2 - \dfrac{24\left(P_u A' + M_u\right)}{f_{pu}B}}}{2}$ $= \dfrac{3\left(12.5\text{ in.}\right) \pm \sqrt{\left[\left(3\right)\left(12.5\text{ in.}\right)\right]^2 - \dfrac{24\left[\left(90\text{ kips}\right)\left(5.5\text{ in.}\right) + 720\text{ kip-in.}\right]}{\left(3.22\text{ ksi}\right)\left(14\text{ in.}\right)}}}{2}$ $= 4.97$ in.
ASD
$A = \dfrac{3N' \pm \sqrt{\left(3N'\right)^2 - \dfrac{24\left(P_a A' + M_a\right)}{f_{pa}B}}}{2}$ $= \dfrac{3\left(12.5\text{ in.}\right) \pm \sqrt{\left[\left(3\right)\left(12.5\text{ in.}\right)\right]^2 - \dfrac{24\left[\left(60\text{ kips}\right)\left(5.5\text{ in.}\right) + 480\text{ kip-in.}\right]}{\left(2.04\text{ ksi}\right)\left(14\text{ in.}\right)}}}{2}$ $= 5.28$ in.

In the calculation of A, the minus sign before the radical controls the solution.

4. Determine the required tensile strength of the anchor rod:

LRFD	ASD
$T_u = \dfrac{f_{pu}AB}{2} - P_u$ $= \dfrac{(3.22\text{ ksi})(4.97\text{ in.})(14\text{ in.})}{2} - 90\text{ kips}$ $= 22.0$ kips $T_{rod} = T_u/2$ $= 22.0$ kips/2 $= 11.0$ kips	$T_a = \dfrac{f_{pa}AB}{2} - P_a$ $= \dfrac{(2.04\text{ ksi})(5.28\text{ in.})(14\text{ in.})}{2} - 60\text{ kips}$ $= 15.4$ kips $T_{rod} = T_a/2$ $= 15.4$ kips/2 $= 7.70$ kips

5. Determine the required plate thickness

The moment for this determination is to be taken at the critical plate width. This is determined by assuming that the load spreads at 45° to a location 0.95d of the column. The width is then taken as twice the distance from the rod to the critical section for each rod, provided that the critical section does not intersect the edge of the plate.

The critical section, as shown in Figure B.5, is at

$$\frac{14\text{ in.} - (0.95)(8\text{ in.})}{2} = 3.20\text{ in.}$$

The required moment strength, $M_{u\,pl}$ *or* $M_{a\,pl}$, for a 1-in. strip of plate, determined from the bearing stress distribution in Figure B.4, is:

LRFD	ASD
$f_{pu(m)} = \left(3.22\text{ ksi}\right)\left(\dfrac{4.97\text{ in.} - 3.20\text{ in.}}{4.97\text{ in.}}\right)$ $= 1.15$ ksi $M_{u\,pl} = \dfrac{(1.15\text{ ksi})\left(3.20\text{ in.}\right)^2}{2} + \dfrac{(3.22\text{ ksi} - 1.15\text{ ksi})(3.20\text{ in.})^2}{3}$ $= 13.0$ kip-in./in.	$f_{pa(m)} = 2.04\text{ ksi}\left(\dfrac{5.28\text{ in.} - 3.20\text{ in.}}{5.28\text{ in.}}\right)$ $= 0.804$ ksi $M_{a\,pl} = \dfrac{(0.804\text{ ksi})\left(3.20\text{ in.}\right)^2}{2} + \dfrac{(2.04\text{ ksi} - 0.804\text{ ksi})(3.20\text{ in.})^2}{3}$ $= 8.34$ kip-in./in.

Anchor rods are placed at a 1½ in. edge distance. The required moment strength, $M_{u\,pl}$ or $M_{a\,pl}$, for a 1-in. strip of plate due to the tension in the anchor rods is:

LRFD	ASD
$M_{u\,pl} = \dfrac{\left(11.0\text{ kips}\right)(3.20\text{ in.} - 1.5\text{ in.})}{2(3.20\text{ in.} - 1.5\text{ in.})}$ $= 5.50$ kip-in./in.	$M_{a\,pl} = \dfrac{7.70\text{ kips}(3.20\text{ in.} - 1.5\text{ in.})}{2(3.20\text{ in.} - 1.5\text{ in.})}$ $= 3.85$ kip-in./in.

The required moment strength due to the bearing stress distribution is critical.

The required plate thickness is:

LRFD	ASD
$t_p = \sqrt{\dfrac{4M_{u\,pl}}{\phi_b F_y}}$ $= \sqrt{\dfrac{4(13.0\text{ kip-in.})}{(0.90)(36\text{ ksi})}}$ $= 1.27$ in.	$t_p = \sqrt{\dfrac{4M_{a\,pl}\,\Omega_b}{F_y}}$ $= \sqrt{\dfrac{4(8.34\text{ kip-in.})(1.67)}{36\text{ ksi}}}$ $= 1.24$

Use a 14×14×1½ in. base plate.

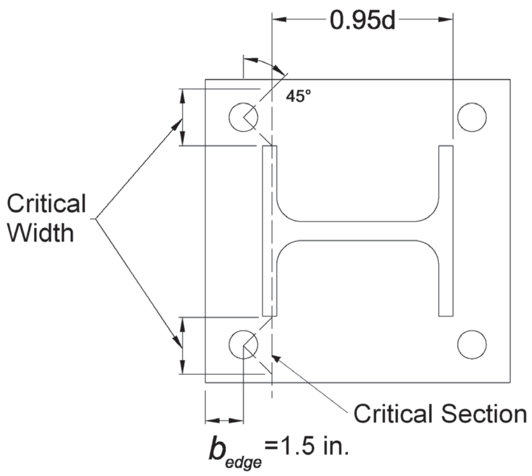


Fig. B.5. Critical plate width for anchor rod (tension side).