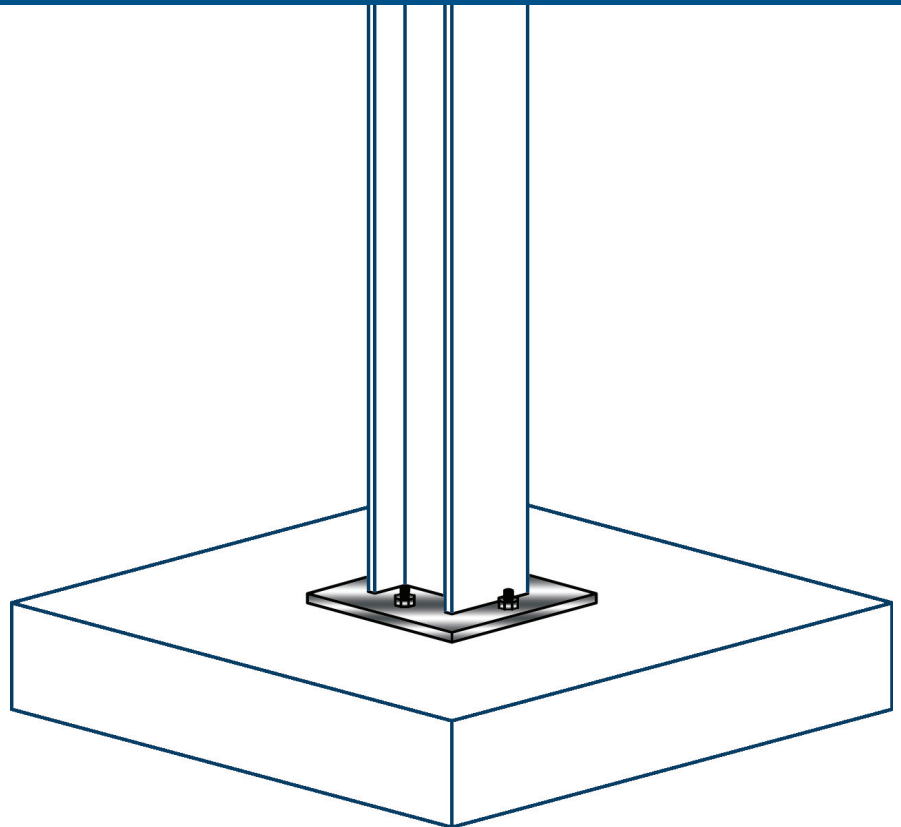




Steel Design Guide

*Base Plate and
Anchor Rod Design*

Second Edition





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Printed in the United States of America

First Printing: May 2006

Acknowledgements

The authors would like to thank Robert J. Dexter from the University of Minnesota, and Daeyong Lee from the Steel Structure Research Laboratory, Research Institute of Industrial Science & Technology (RIST), Kyeonggi-Do, South Korea, for their writing of Appendix A and the first draft of this Guide. The authors also recognize the contributions of the authors of the first edition of this guide, John DeWolf from the University of Connecticut and David Ricker (retired) from Berlin Steel Construction Company, and thank Christopher Hewitt and Kurt Gustafson of AISC for their careful reading, suggestions, and their writing of Appendix B. Special appreciation is also extended to Carol T. Williams of Computerized Structural Design for typing the manuscript.

AISC would also like to thank the following individuals who assisted in reviewing the drafts of this Design Guide for their insightful comments and suggestions.

Victoria Arbitrio	Donald Johnson
Reidar Bjorhovde	Geoffrey L. Kulak
Crystal Blanton	Bill R. Lindley II
Charles J. Carter	David McKenzie
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James Doyle	William T. Segui
Richard M. Drake	David F. Sharp
Samuel S. Eskildsen	Victor Shneur
Daniel M. Falconer	Bozidar Stojadinovic
Marshall T. Ferrell	Raymond Tide
Roger D. Hamilton	Gary C. Violette
John Harris	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 over-emphasized. This is the 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, 2005), and includes guidance for designs made in accordance with load and resistance factor design (LRFD) or allowable stress design (ASD).

This Guide follows the format of the 2005 AISC Specification, developing strength parameters for foundation system design in generic terms that facilitate either load and resistance factor design (LRFD) or allowable strength design (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 Appendix D, which is exclusively based on the strength approach (LRFD) for the design of such embedment. Other steel elements of the foundation system, including the column base plate and the sizing of anchor diameters are equally proficient to evaluation using LRFD or ASD load methods. 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 the *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 2005 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.

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 that 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.

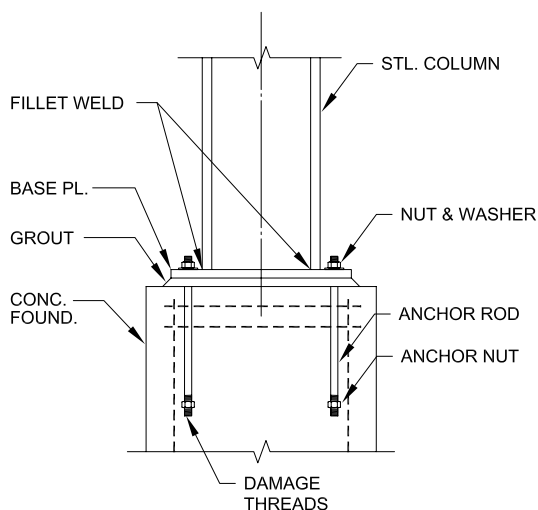


Figure 1.1. Column base connection components.

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

The vast majority of building columns are designed for axial compression only with little or no uplift. For such columns, the simple column-base-plate connection detail 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 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 lb (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, 2002) 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.

The AISC Design Guide 7, 2nd edition, *Industrial Buildings: Roofs to Column Anchorage* (Fisher, 2004), 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

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	57.5	42.0	1½
		90	67.5	45.0	36.0	3
A36	58	43.5	29.0	23.2	4	
A307	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

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 1/8-in. increments up to 1 1/4 in. thickness and in 1/4-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 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 typical gravity columns. For posts and light HSS columns, the minimum plate thickness is typically 1/2 in., and for other structural columns a plate thickness of 3/4 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. deep ... and sharp V-shaped notches. Gouges deeper than 3/16 in. ... 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 on steel plates are covered in Section M2.8 of the AISC Specification as follows:

“Steel bearing plates 2 in. ... or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. ... but not over 4 in. ... in thickness are permitted to be straightened by press-

ing or, if presses are not available, by milling for bearing surfaces ... to obtain a satisfactory contact bearing. Steel bearing plates over 4 in. ... 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 baseplate.

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

“Lack of contact bearing not exceeding a gap of $\frac{1}{16}$ in. ... regardless of the type of splice used ... is permitted. If the gap exceeds $\frac{1}{16}$ in. ... but is less than $\frac{1}{4}$ in. ... and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel shims. Shims need not be other than mild steel, regardless of the grade of 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 $\frac{1}{4}$ in. to $\frac{1}{2}$ 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 $\frac{5}{16}$ -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 provided here:

- 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 $\frac{5}{16}$ -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 $\frac{3}{4}$ 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 moment or axial tension and anchor rods 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 2005 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 building 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 col-

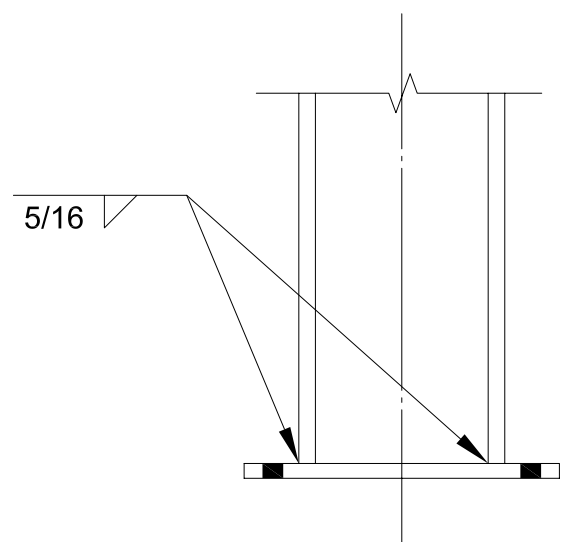


Figure 2.1. Typical gravity column base plate weld.

umn 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.

2.5 Anchor Rod Material

As shown 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.



Figure 2.2. Base plate with adjusting screws.

The ASTM specification allows F1554 anchor rods to be supplied either straight (threaded with nut for anchorage), bent or headed. Rods up to approximately 1 in. 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 with 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 3.22 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 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.

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.
3/4	1 5/16	2	1/4
7/8	1 9/16	2 1/2	5/16
1	1 13/16	3	3/8
1 1/4	2 1/16	3	1/2
1 1/2	2 5/16	3 1/2	1/2
1 3/4	2 3/4	4	5/8
2	3 1/4	5	3/4
2 1/2	3 1/4	5 1/2	7/8

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 below regarding the use of alternate 1 1/8-in. hole size for 3/4-in.-diameter anchor rods, with plates less than 1/4 in. thick.

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 153) 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 this 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.

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*.

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 Design Guide 7, that the washer thickness be approximately one-third 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 1/16 in. with 3/4-in.-diameter rods and base plates less than 1 1/4 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) washers 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 3/4-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 1/2 in. of material from the edge of the hole to the edge of the plate will normally suffice, although 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 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 tem-

plate 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 forgoing recommendations, fast-track projects and projects with complex layouts may require special 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 accurate 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 117-90, Section 2.3, *Placement of embedded items*, allows a tolerance on vertical, lateral, and level alignment of ± 1 in. AISC *Code of Standard Practice* (AISC, 2005), Section 7.5.1, lists the following tolerances:

“(a) The variation in dimension between the centers of any two Anchor Rods within an Anchor-Rod Group shall be equal to or less than $\frac{1}{8}$ in.”

“(b) The variation in dimension between the centers of adjacent Anchor-Rod Groups shall be equal to or less than $\frac{1}{4}$ in.”

“(c) The variation in elevation of the tops of Anchor Rods shall be equal to or less than plus or minus $\frac{1}{2}$ in.”

“(d) 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 $\frac{1}{4}$ in. per 100 ft, but not to exceed a total of 1 in.”

“(e) 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 $\frac{1}{4}$ in.”

Thus, ACI 117 is much more generous for embedded items than the AISC *Code of Standard Practice* (AISC, 2005) is 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 CSI Division 3, require that the anchor rods be set in accordance with the AISC *Code of Standard Practice* (AISC, 2005) 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

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 tighten 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 plumb, 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 $\frac{1}{4}$ 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 $\frac{1}{16}$ 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 earlier. 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 $1\frac{1}{2}$ in. 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.

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



Figure 2.3. Anchor rods run over by crane.

Diameter of Rod, in.	Width Across Flats, in.	Width Across Corners, in.	Height of Nut, in.
$\frac{3}{4}$	$1\frac{1}{8}$	$1\frac{5}{16}$	$2\frac{1}{4}$
$\frac{7}{8}$	$1\frac{5}{16}$	$1\frac{1}{2}$	$2\frac{5}{8}$
1	$1\frac{1}{2}$	$1\frac{3}{4}$	3
$1\frac{1}{4}$	$1\frac{7}{8}$	$2\frac{3}{16}$	$3\frac{3}{4}$
$1\frac{1}{2}$	$2\frac{1}{4}$	$3\frac{1}{8}$	$4\frac{1}{2}$
$1\frac{3}{4}$	$2\frac{3}{4}$	$3\frac{3}{16}$	$5\frac{1}{4}$
2	$3\frac{1}{8}$	$3\frac{5}{8}$	6
$2\frac{1}{2}$	$3\frac{7}{8}$	$4\frac{1}{2}$	$7\frac{1}{2}$

Dimensions based on IFI #128 of Industrial Fastener Institute. Material conforms to ASTM A563 Grade A.

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 typi-

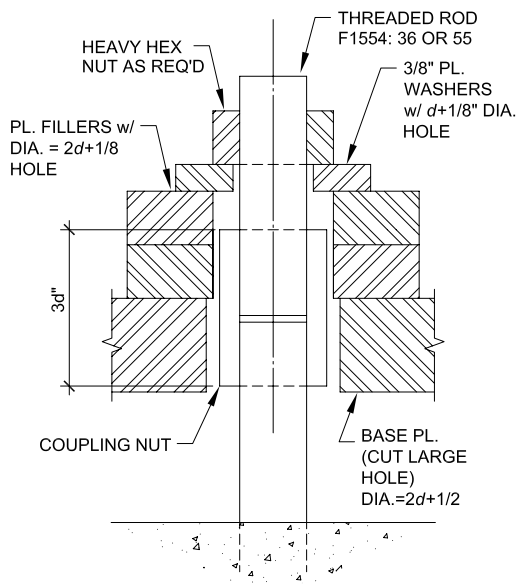
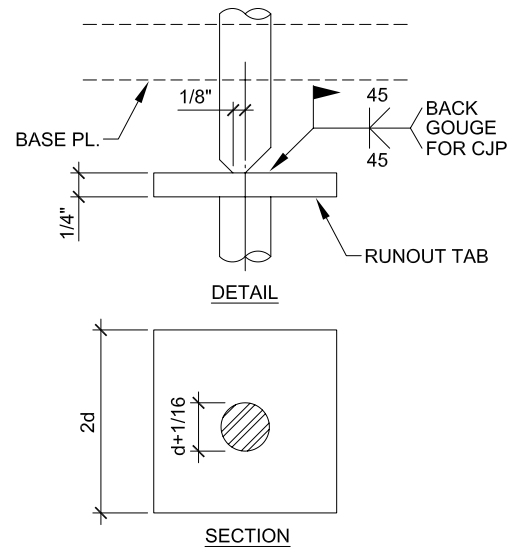


Figure 2.4. Coupling nut detail for extending anchor rod.

cal 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).



NOTE: CAN TRIM & GRIND RUN OFF TAB AFTER WELDING IF REQ'D MATERIAL: F1554: 36 OR 55 WITH SUPPLEMENT S1.

Figure 2.5a. Groove weld splice.

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 earlier, 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°

Nonsymmetrical 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.

2.12 Details for Seismic Design D

The 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005) 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 connection is given in ACI 318, 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 must either 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.

The base plate connection can be designed using concepts similar to beam-to-column connections. However, the Com-

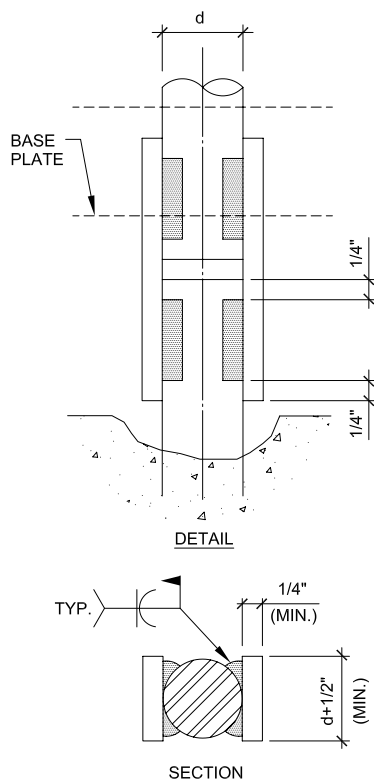


Figure 2.5b. Lap plate splice.

mentary to the AISC Seismic Provisions notes some significant differences:

1. Long anchor rods embedded in concrete will strain much more than high-strength bolts or welds in beam-to-column connections.
2. Column base plates are bearing on grout and concrete, which is more compressible than the column flanges of the beam-to-column connections.
3. 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.
4. 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.
5. AISC standard hole diameters for column base anchor rods are different than AISC standard holes for high-strength bolts.
6. 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.

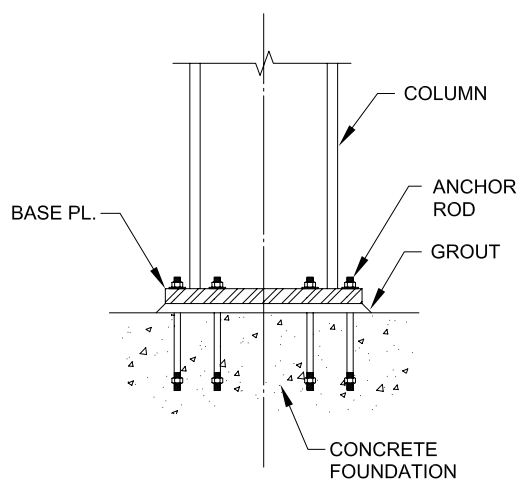


Figure 2.6. Typical moment base detail.

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 Base Plates with Small Moments
- Section 3.4 Base Plates 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.

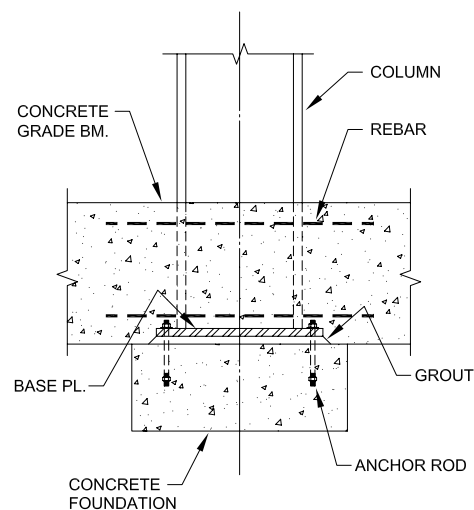


Figure 2.7. Embedded moment base detail.

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 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 lb. Columns, base plates, and their foundations must have sufficient moment strength to resist a minimum eccentric gravity load of 300 lb 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. × 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 design moment strength for ¾-in. anchor rods equals (2)(19.1 kips)(4 in.) = 306 kip-in. For a 14-in.-deep column, the OSHA required moment strength is only (1.6)(0.300)(18 + 7) = 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 318-02, Section 10.17, as $\phi(0.85f'_cA_1)$ 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 $\sqrt{A_2/A_1} \leq 2$.

The 2005 AISC Specification, Section J8, provides the nominal bearing strength, P_p , as follows:

Equation J8-1:

$P_p = 0.85f'_cA_1$ on the full area of a concrete support.

Equation J8-2:

$$P_p = (0.85f'_cA_1) \left(\sqrt{\frac{A_2}{A_1}} \right) \leq 1.7f'_cA_1$$

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

$$\phi = 0.60 \text{ (LRFD)} \quad \Omega = 2.50 \text{ (ASD)}$$

Alternatively, ACI 318-02 stipulates a ϕ 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 ϕ factor in designing column base plates.

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

On the full area of a concrete support:

$$f_{p(max)} = 0.85f'_c$$

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

$$f_{p(max)} = (0.85f'_c) \left(\sqrt{\frac{A_2}{A_1}} \right) \leq 1.7f'_c$$

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

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

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

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 10.17 of ACI 318 (ACI, 2002), as follows:

$$f_{p(max)} = \phi \left(0.85f'_c \right) \sqrt{\frac{A_2}{A_1}}$$

$$\sqrt{\frac{A_2}{A_1}} \leq 2$$

where

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

ϕ = strength reduction factor for bearing, 0.65 per Section 9.3, ACI 318-02

f'_c = specified compressive strength of concrete, ksi

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

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 $f_{p(max)}$:

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

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

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

$$A_{1(req)} = \frac{P_u}{\phi 0.85 f'_c} \quad (\text{LRFD})$$

$$A_{1(req)} = \frac{\Omega P_a}{0.85 f'_c} \quad (\text{ASD})$$

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

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

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

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 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

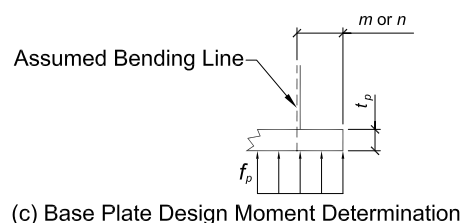
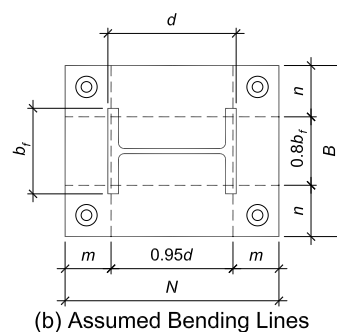
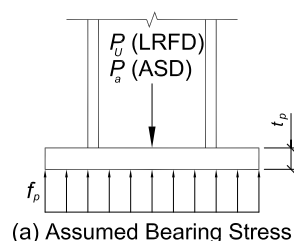


Figure 3.1.1. Design of base plate with axial compressive load.

bearing pressure also causes bending in the base plate in the area between the column flanges (Thornton, 1990; Drake and Elkin, 1999). The following procedure allows a single procedure to determine the base plate thickness for both situations.

The required strength of the base plate can be determined as

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

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

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

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

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

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

N = base plate length, in.

B = base plate width, in.

b_f = column flange width, in.

d = overall column depth, in.

n' = yield-line theory cantilever distance from column web or column flange, in.

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

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

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

where

P_u = the required axial compressive load (LRFD), kips

P_a = the required axial compressive load (ASD), kips

$$P_p = 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}$$

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, 2005):

$$t_{min} = l \sqrt{\frac{2P_u}{\phi F_y B N}} \text{ (LRFD)}$$

$$t_{min} = l \sqrt{\frac{2\Omega P_a}{F_y B N}} \text{ (ASD)}$$

where

ϕ = resistance factor for flexure, 0.90

Ω = factor of safety for ASD, 1.67

F_y = specified minimum yield stress of 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 λ term 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 bolts on the anchor rods.

Steps for obtaining base plates sizes for these 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.

$$A_{1(req)} = \frac{P_u}{\phi 0.85 f'_c} \text{ (LRFD)}$$

$$A_{1(req)} = \frac{\Omega P_a}{0.85 f'_c} \text{ (ASD)}$$

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

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

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

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 set N equal to B .

4. Calculate the required base plate thickness.

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

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

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

N = base plate length, in.

B = base plate width, in.

b_f = column flange width, in.

d = overall column depth, in.

n' = yield-line theory cantilever distance from column web or column flange, in.

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

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

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

where

$$\phi P_p = \phi 0.85 f'_c A_1 \text{ (LRFD)}$$

$$\frac{P_p}{\Omega} = \frac{0.85 f'_c A_1}{\Omega} \text{ (ASD)}$$

Find $l_{max}(m, n, \lambda n')$

$$t_{min} = l \sqrt{\frac{2P_u}{\phi F_y B N}} \text{ (LRFD)}$$

$$t_{min} = l \sqrt{\frac{2P_a \Omega}{F_y B N}} \text{ (ASD)}$$

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: $A_2 \geq 4A_1$

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

1. Calculate the factored axial compressive load, P_u (LRFD) or P_a (ASD).
2. Calculate the required base plate area.

$$A_{1(req)} = \frac{P_u}{2\phi 0.85 f'_c} \text{ (LRFD)}$$

$$A_{1(req)} = \frac{\Omega P_a}{2(0.85 f'_c)} \text{ (ASD)}$$

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 ($A_2 \geq 4A_1$).

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 $(N_2)(B_2)$. If $A_2 \geq 4A_1$, calculate the required thickness using the procedure shown in Step 4 of Case I, except that

$$\phi P_p = \phi f'_c 2A_1 \text{ (LRFD)}$$

$$\frac{P_p}{\Omega} = \frac{f'_c 2A_1}{\Omega} \text{ (ASD)}$$

5. Determine the anchor rod size and location.

Case III: $A_1 < A_2 < 4A_1$

1. Calculate the factored axial compressive load, P_u (LRFD) or P_a (ASD).
2. Calculate the approximate base plate area based on the assumption of Case III.

$$A_{1(req)} = \frac{P_u}{2\phi 0.85 f'_c} \text{ (LRFD)}$$

$$A_{1(req)} = \frac{\Omega P_a}{2(0.85 f'_c)} \text{ (ASD)}$$

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

$$P_u \leq \phi P_p = \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \text{ (LRFD)}$$

$$P_a \leq \frac{P_p}{\Omega} = \left(\frac{0.85 f'_c A_1}{\Omega} \right) \sqrt{\frac{A_2}{A_1}} \text{ (ASD)}$$

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 their locations.

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.
2. Select the anchor rod material and the number and size of anchor rods required to resist uplift.
3. Determine the appropriate base plate size, thickness, and welding to transfer the uplift forces.
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).

Step 1—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.

Step 2—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 below), 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.

Step 3—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 com-

patibility 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.

Step 4—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 or the development length of deformed bars is calculated in accordance with current American Concrete Institute (ACI, 2002) criteria.

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

$$\text{Tensile stress area} = \left(D - \frac{0.7854}{n} \right)^2$$

where

D = major diameter

n = number of threads per in.

Table 7–18 in the AISC *Steel Construction Manual* list the 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 2005 AISC Specification.

The strength of structural fasteners has historically been based on the nominal bolt diameter, and the direct tensile stress area approach is stipulated in ACI 318 Appendix D. The designer should be aware of the differences in 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 consistent 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 2005 AISC Specification stipulates the nominal tensile strength of a fastener as

$$R_n = 0.7F_uA_b$$

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

$$\text{Design tensile strength} = (0.75)(0.75)F_uA_b = 0.5625F_uA_b$$

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

$$\text{Allowable tensile strength} = \frac{0.75}{2.00}F_uA_b = 0.375F_uA_b$$

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

$$\text{Design tensile strength} = \phi F_uA_{ts} = 0.75F_uA_{ts}$$

where $\phi = 0.75$

Shown in Table 3.1 are the design and allowable strengths for various anchor rods.

3.2.2 Concrete Anchorage for Tensile Forces

It is presumed that ASCE 7 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 318-02.

Appendix D of ACI 318-02 (ACI, 2002) 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 [concrete capacity design (CCD) method].

Concrete Pullout Strength

ACI concrete pullout strength is based on the ACI Appendix D provisions (Section D5.3):

$$\phi N_p = \phi \Psi_4 A_{brg} 8f'_c$$

where

N_p = the nominal pullout strength

$\phi = 0.7$

Rod Diameter, in.	Rod Area, A_r , in ²	LRFD $\phi R_n, \phi = 0.75$			ASD $R_n / \Omega, \Omega = 2.00$		
		Grade 36, kips	Grade 55, kips	Grade 105, kips	Grade 36, kips	Grade 55, kips	Grade 105, kips
5/8	0.307	10.0	12.9	21.6	6.7	8.6	14.4
3/4	0.442	14.4	18.6	31.1	9.6	12.4	20.7
7/8	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
1 1/8	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1 1/4	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1 1/2	1.77	57.7	74.6	124	38.4	49.7	82.8
1 3/4	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2 1/4	3.98	130	168	280	86.5	112	186
2 1/2	4.91	160	207	345	107	138	230
2 3/4	5.94	194	251	418	129	167	278
3	7.07	231	298	497	154	199	331
3 1/4	8.30	271	350	583	180	233	389
3 1/2	9.62	314	406	677	209	271	451
3 3/4	11.0	360	466	777	240	311	518
4	12.6	410	530	884	273	353	589

$\psi_4 = 1.4$ if the anchor is located in a region of a concrete member where analysis indicates no cracking ($f_i - f_r$) at service levels, otherwise $\psi_4 = 1.0$

$A_{brg} =$ the bearing area of the anchor rod head or nut, and f'_c is the concrete strength

Shown in Table 3.2 are design pullout strengths for anchor rods with heavy hex head nuts. The 40% increase in strength has not been included. 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 318-02 provides a pullout strength for a hooked anchor of $\phi\psi_4(0.9f'_c e_h d_o)$, 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 is limited to a maximum of $4.5d_o$; $\psi_4 = 1$ if the anchor is located where the concrete is cracked at service load, and $\psi_4 = 1.4$ if it is not cracked at service loads.

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_i in Figure 3.2.1) in the CCD method is considered to decrease 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).

The CCD method is valid for anchors with diameters not exceeding 2 in. and tensile embedment length not exceeding 25 in. in depth.

Table 3.2. Anchor Rod Concrete Pullout Strength, kips					
Rod Diameter, in.	Rod Area, A_r , in ²	Bearing Area, in ²	Concrete Pullout Strength, ϕN_p		
			Grade 36, kips	Grade 55, kips	Grade 105, kips
5/8	0.307	0.689	11.6	15.4	19.3
3/4	0.442	0.906	15.2	20.3	25.4
7/8	0.601	1.22	20.5	27.3	34.1
1	0.785	1.50	25.2	33.6	42.0
1 1/8	0.994	1.81	30.4	40.5	50.7
1 1/4	1.23	2.24	37.7	50.2	62.8
1 1/2	1.77	3.13	52.6	70.1	87.7
1 3/4	2.41	4.17	70.0	93.4	117
2	3.14	5.35	90.0	120	150
2 1/4	3.98	6.69	112	150	187
2 1/2	4.91	8.17	137	183	229
2 3/4	5.94	9.80	165	220	274
3	7.07	11.4	191	254	318
3 1/4	8.30	13.3	223	297	372
3 1/2	9.62	15.3	257	343	429
3 3/4	11.0	17.5	294	393	491
4	12.6	19.9	334	445	557

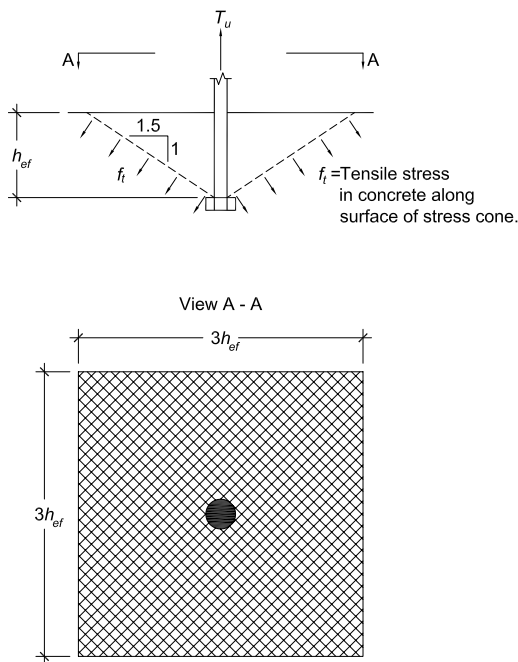


Figure 3.2.1. Full breakout cone in tension per ACI 318-02.

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 2005 AISC *Seismic Provisions for Structural Steel Buildings*.

Per ACI 318-02, Appendix D, the concrete breakout strength for a group of anchors 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

$$\phi N_{cbg} = \phi \Psi_3 16 \sqrt{f'_c} h_{ef}^{5/3} \frac{A_N}{A_{No}} \text{ for } h_{ef} \geq 11 \text{ in.}$$

where

$$\phi = 0.70$$

$$\Psi_3 = 1.25 \text{ considering the concrete to be uncracked at service loads, otherwise } = 1.0$$

$$h_{ef} = \text{depth of embedment, in.}$$

$$A_N = \text{concrete breakout cone area for group}$$

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

Appendix D of ACI 318-02 also lists criteria for anchor rods to prevent “failure due to lateral bursting forces at the anchor head.” 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 six 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 318-02. Use of washer plates can be beneficial by increasing the bearing area, which increases the side-face blowout strength.

The concrete breakout capacities assume that the concrete is uncracked. The designer should refer to ACI 318-02 to determine if the concrete should be taken as cracked or uncracked. If the concrete is considered cracked, ($\psi_3 = 1.0$) and 80% of the concrete capacity values should be used.

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.

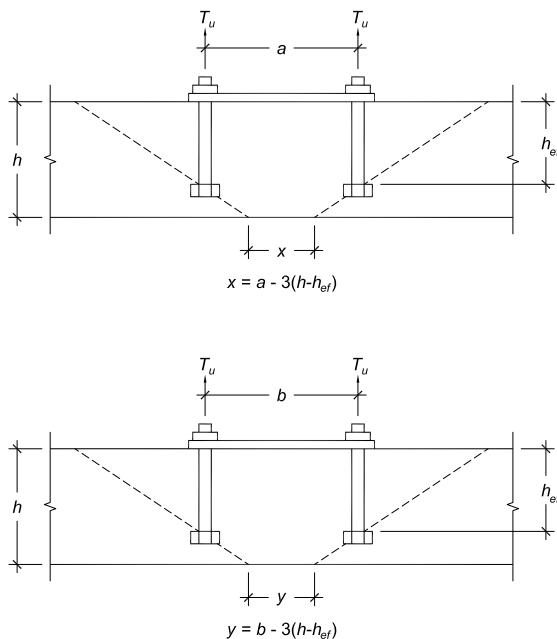


Figure 3.2.2. Breakout cone for group anchors in thin slab.

The stress cone checks rely on 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 $\phi A_{sc} F_y$ as the lap splice length will ensure that ductile behavior will occur. A_{sc} is the effective cross-sectional area, which is the tensile stress area for threaded rods. $\phi = 0.90$, as prescribed in Chapter 9 of ACI 318-02.

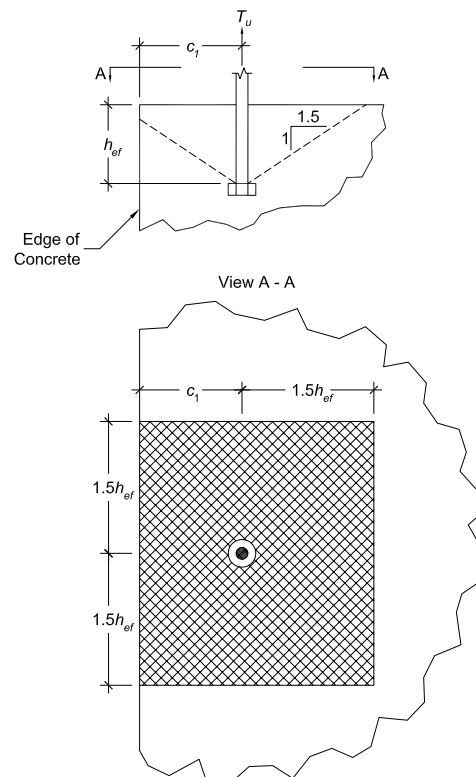


Figure 3.2.3. Breakout cone in tension near an edge.

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. From ACI 318, anchor rod embedment length equals the top cover to reinforcing plus L_d or L_{dh} (if hooked) plus 0.75 times g (see Figure 3.2.5). The minimum length is 17 times the rod diameter.

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 procedure was modified by Doyle and Fisher (2005). Drake and Elkin proposed that a uniform distribution of the resultant compressive bearing stress is more appropriate when

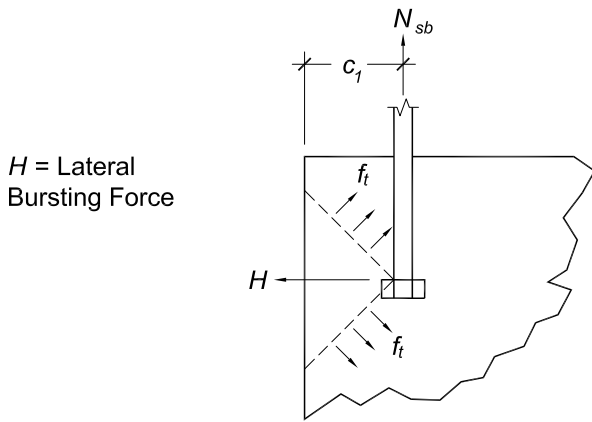


Figure 3.2.4. Lateral bursting forces for anchor rods in tension near an edge.

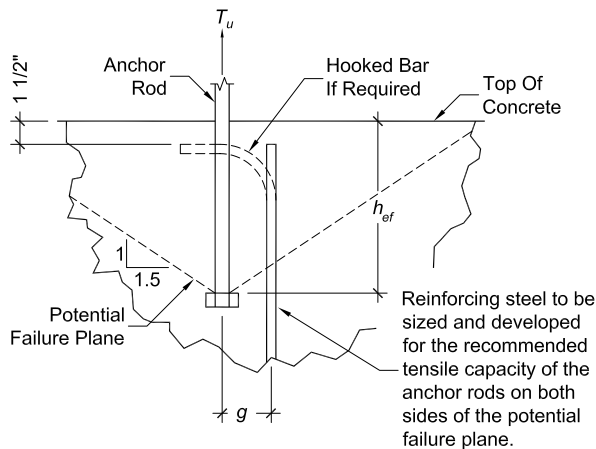


Figure 3.2.5. The use of steel reinforcement for developing anchor rods.

utilizing LRFD. The design is related to the equivalent eccentricity e , equal to the moment M_w divided by the column axial force P_u .

For small eccentricities, the axial force is resisted by bearing only. For large eccentricities, it is necessary to use anchor rods. 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_w 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

$$q = f_p \times B \quad (3.3.1)$$

where

f_p = bearing stress between the plate and concrete

B = the base plate width

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

$$\epsilon = \frac{N}{2} - \frac{Y}{2} \quad (3.3.2)$$

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

$$Y_{min} = \frac{P_r}{q_{max}} \quad (3.3.3)$$

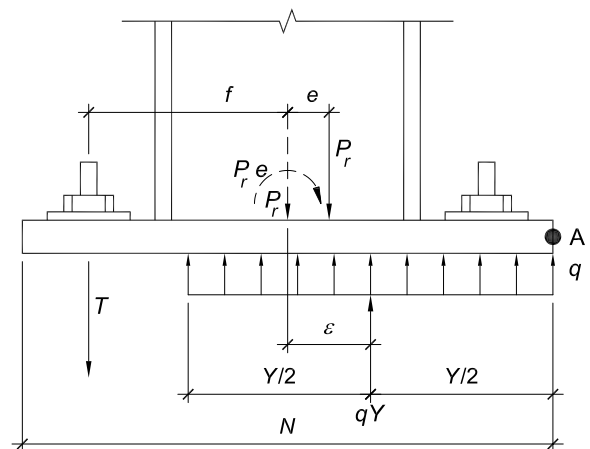


Figure 3.3.1. Base plate with small moment.