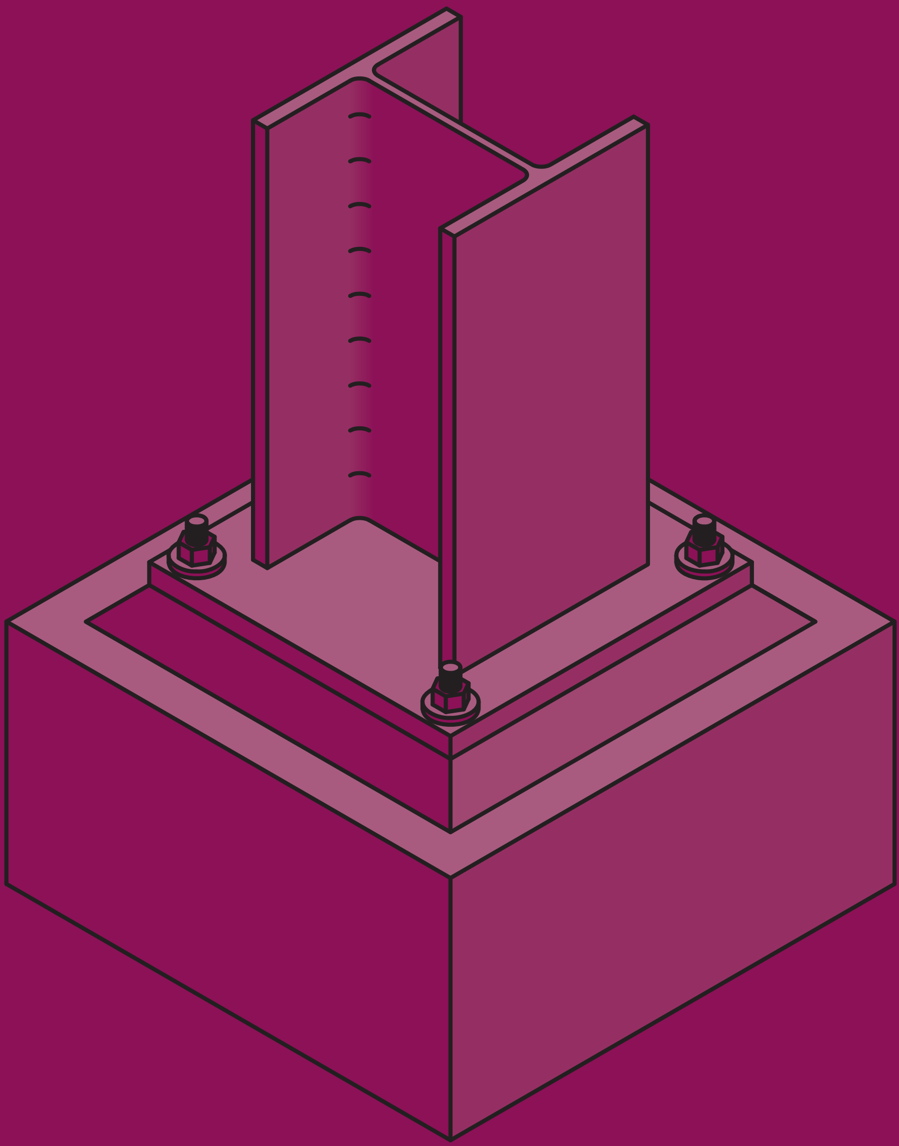


Design Guide 1

Base Connection Design for Steel Structures

Third Edition

Smarter.
Stronger.
Steel.

Design Guide 1

Base Connection Design for Steel Structures

Third Edition

Amit Kanvinde, PhD
Mahmoud Maamouri, SE, PE, PhD
Joshua Buckholt, SE, PE

American Institute of Steel Construction

Printed in the United States of America

Authors

Amit Kanvinde, Ph.D., is Professor of Civil and Environmental Engineering, College of Engineering, at the University of California–Davis, Davis, California.

Mahmoud Maamouri, Ph.D., P.E., S.E., is Executive Vice President, CSD Structural Engineers, Milwaukee, Wisconsin.

Joshua Buckholt, P.E., S.E., is Vice President, CSD Structural Engineers, Milwaukee, Wisconsin.

Acknowledgments

The authors thank the American Institute of Steel Construction for their support in the development of this document and for assistance in its preparation. The authors also wish to thank the following AISC reviewers whose contributions significantly improved the document.

Eric Bolin	Larry Kloiber	Chris Raebel
D.B. Davis	Conner R. Maines	Paul Richards
Rick Drake	Jim Malley	Tom Sabol
Jim Fisher	Margaret Matthew	Chia-Ming Uang
Steve Herlache	Scott Metzger	

The authors also wish to acknowledge the authors of the previous editions of this Design Guide whose work was foundational for this updated and expanded edition.

First Edition:	John DeWolf	David Ricker
Second Edition:	Jim Fisher	Larry Kloiber

Additionally, the authors would like to acknowledge the late Robert J. Dexter and Daeyong Lee for their authorship of Appendix A included in the second edition of the Design Guide. Their contributions were carried forward into this edition with minor updates. Special appreciation is also extended to George Evers for assistance with typesetting the manuscript and creating figures.

Preface

This is the third edition of AISC Design Guide 1. The first edition was published in 1990 as *Column Base Plates* by J.T. DeWolf and D.T. Ricker. The second edition was published in 2006 as *Base Plate and Anchor Rod Design* by J.M. Fisher and L.A. Kloiber. This third edition incorporates and updates the content of the previous editions while also providing significant expansions in coverage related to base connection design. Significant expansions to this Design Guide include the addition of Chapter 3 addressing the relationship between the structure and base connections; the addition of Chapter 5 pertaining to embedded base connection design; the addition of Chapter 6, which focuses on seismic design of base connections; and the addition of Appendices C and D, which provide guidance regarding the simulation and representation of base connections. This edition also significantly expands upon the number of design examples.

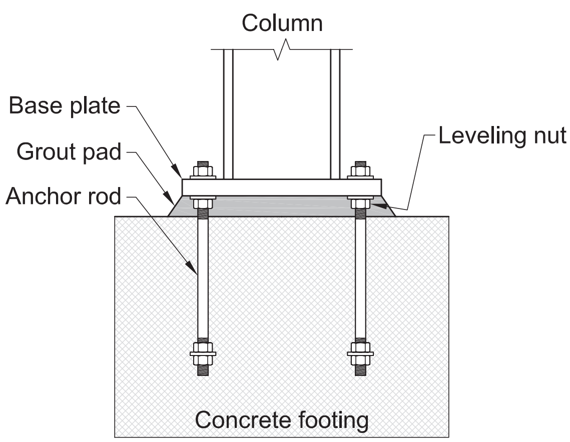
Table of Contents

Chapter 1
Introduction

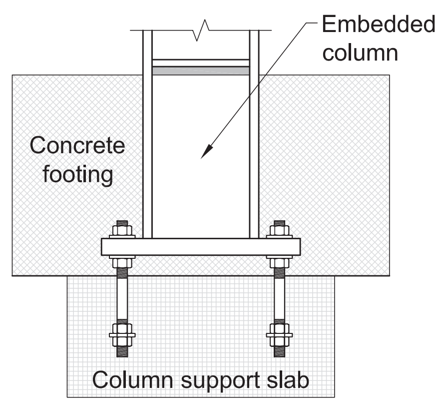
1.1 GENERAL

Column base connections are the critical interface between the steel structure and the foundation. These connections are used in buildings to support gravity loads and may function as part of lateral force-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. They also serve a critical function in seismically designed structures, wherein they may be subjected to large cyclic loading and deformations. In many cases, the base connections also interact with the structure they are a part of, influencing its response. Column base connections are used in nearly all types of steel structures, notably buildings (which encompass moment frames and braced frames), bridges, as well as special structures. Moreover, they are designed to resist axial compression and tension, moments, and shear (and combinations thereof). As a result, these connections take diverse forms and are subject to multiple design considerations.

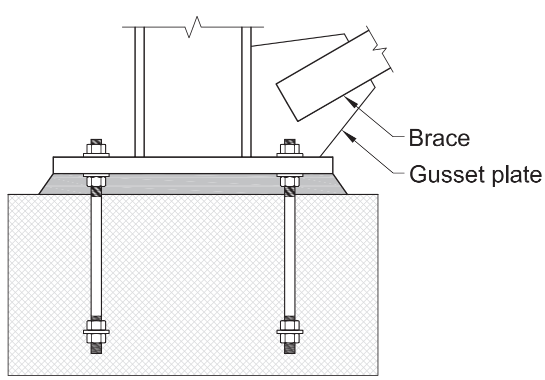
Figures 1-1(a), (b), and (c) illustrate some common base connection details. Figures 1-1(a) and (b) show details that are used in moment frames or when only a column needs to be attached to the footing, whereas Figure 1-1(c) shows a braced frame base connection where both a column and a diagonal brace are present. Each figure shows the various components of the corresponding connection, including the base plate, anchor rods, and footing (footing reinforcement is not shown for clarity). Figure 1-1(a) shows a base



(a) Exposed base plate connection with column only



(b) Embedded base connection



(c) Exposed base plate connection with gusset plate and brace

Fig. 1-1. Types of base connections.

connection that resists applied axial forces and moments through the development of tension in the anchor rods along with bearing stresses in the footing; shear may be resisted through friction, anchor rod bearing, or a shear-lug (also not shown). Figure 1-1(b) shows an embedded type connection where bearing between the column flange and footing is used to resist large base moments, typically used in seismic design. The braced connection shown in Figure 1-1(c) uses anchor rods similar to those shown in Figure 1-1(a), but also includes the gusset plate and diagonal brace. Note that the configurations shown in Figures 1-1(a) through (c) are generic, and multiple variations of these are used in practice. For example, shim stacks may be used instead of leveling nuts for setting the base plate. Similarly, in both embedded and exposed column bases (generically referred to as base connections), reinforcement is often used to supplement the strength of the concrete. Other variations include the anchor rod patterns, how they are anchored (e.g., headed or hooked ends), and weld details between the column and base plate (e.g., fillet, partial-joint-penetration, or complete-joint-penetration).

Base connections are often the last structural steel items to be designed but are the first items required on the jobsite. In recent years, with the acceleration of many fast-track projects or delegated designs, engineers are more frequently asked to complete and release the anchorage ahead of releasing the drawings for the whole structure. The schedule demands along with the problems that can occur at the interface of structural steel and concrete make it essential that the design details take into account not only structural requirements, but also 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 ensuring the accurate vertical alignment of the structure. Against this practical setting, material in this Design Guide is intended to provide guidelines for engineers, fabricators, and erectors to design, detail, and specify column base plate and anchor rod connections in a manner that (1) results in economic design, (2) provides safe and acceptable performance of both the connection and the structure under a range of conditions, and (3) avoids or addresses common fabrication and erection problems.

It is important to acknowledge the relationship of this Design Guide to related codes, standards, and design manuals including the 2022 AISC *Specification for Structural Steel Buildings* (AISC, 2022c), the 2022 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2022b), the AISC *Steel Construction Manual* (AISC, 2023), as well the AISC *Seismic Design Manual* (AISC, 2018), hereafter referred to as the AISC *Specification*, AISC *Seismic Provisions*, AISC *Manual*, and AISC *Seismic Design Manual*, respectively. The AISC *Specification* provides generally applicable requirements for the design and construction of structural steel buildings and other structures. The AISC *Seismic Provisions* pertain to the design, erection, and fabrication of structural steel and composite steel and concrete seismic force-resisting systems and are used in conjunction with the AISC *Specification*. These documents refer to this Design Guide for topical and detailed guidance for the design of base connections. The AISC *Seismic Design Manual* integrates the general guidance provided by the standards and the topical guidance provided by this Design Guide to develop design examples for both the structural system as well as connections, including the base connections. AISC Design Guide 7, *Industrial Building Design* (Fisher, 2019), contains additional examples and discussion relative to the design of anchor rods. Figure 1-2 illustrates the relationships among related codes, standards, and design manuals relevant to column base design. Design procedures in this Design Guide are based on *ACI Building Code Requirements for Structural Concrete and Commentary*, ACI 318-19(22) (ACI, 2022), hereafter referred to as ACI 318 in the remainder of this Design Guide.

This Design Guide includes guidance for designs made in accordance with load and resistance factor design (LRFD) or allowable strength design (ASD). Section 1.2 summarizes the significant research and improved design guidelines that have been issued subsequent to the publication of the second edition, and which are included in this edition. This Design Guide supersedes the second edition of AISC Design Guide 1 (Fisher and Kloiber, 2006).

1.2 HISTORY AND ADVANCEMENTS

1.2.1 Previous Editions of Design Guide 1 and Research Synthesis

The first edition of Design Guide 1 (DeWolf and Ricker, 1990) was published in 1990 and reflected research and design methods for column base plate connections current at that time. The first edition contains a compilation of information on the design of base plates and anchorages for steel columns with the intent of providing research background and a basic understanding of the connection behavior for design. An important aspect of the first edition is the introduction of the triangular stress block approach for the design of base plate connections subjected to axial compression and flexure. The edition also mentions the lack of research work (particularly experimental validation) of the design approach, recognizing the method reflects elastic, but possibly not ultimate response.

The second edition of Design Guide 1 (Fisher and Kloiber, 2006, which supersedes the first edition) was published in 2006, partly in response to new research and a new Occupational Safety and Health Administration (OSHA) provision requiring four anchor rods for most base plate connections (OSHA, 2001). In addition to the OSHA regulations, the second edition incorporates significant research and improved design guidelines issued subsequent to the publication of the first edition. These include the ACI *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08 (2008), with improved provisions for the pullout and breakout strength of anchor rods and other embedded anchors. The second edition also introduces the Drake and Elkin (1999) design approach for base plates with axial compression and moment, including the rectangular stress block approach, which was expected to be more consistent with the ultimate response of base plate connections.

A comprehensive review study sponsored by AISC (Grauvilardell et al., 2005) synthesized results from several experimental and analytical studies conducted worldwide regarding the behavior of a range of configurations of column base connections and provides a detailed description of the status of knowledge on behavior and design. A key contribution of the synthesis is identification of knowledge gaps and research priorities. Some of the main issues identified by the study include (1) the lack of research and design procedures for embedded base connections; (2) for various configurations (in unbraced and braced frames), the lack of applicability of the design methods to cyclic loading representative of seismic conditions; (3) lack of understanding of desired failure modes and hierarchies in base connections of various configurations; and (4) lack of understanding and methods to characterize load-deformation response, including flexibility of base connections. The publication of this review study (and similar knowledge gaps identified internationally) motivated significant research in the United States and elsewhere. Given that the timing of publication of this study was virtually coincident with the second edition of Design Guide 1, the findings of this research were not included in the second edition. A major objective of the third edition is to incorporate these findings into the Guide. Specifically, the third edition now incorporates research findings in several areas that are germane to the design, strength characterization, simulation, and construction of base connections. These are summarized in the next subsection.

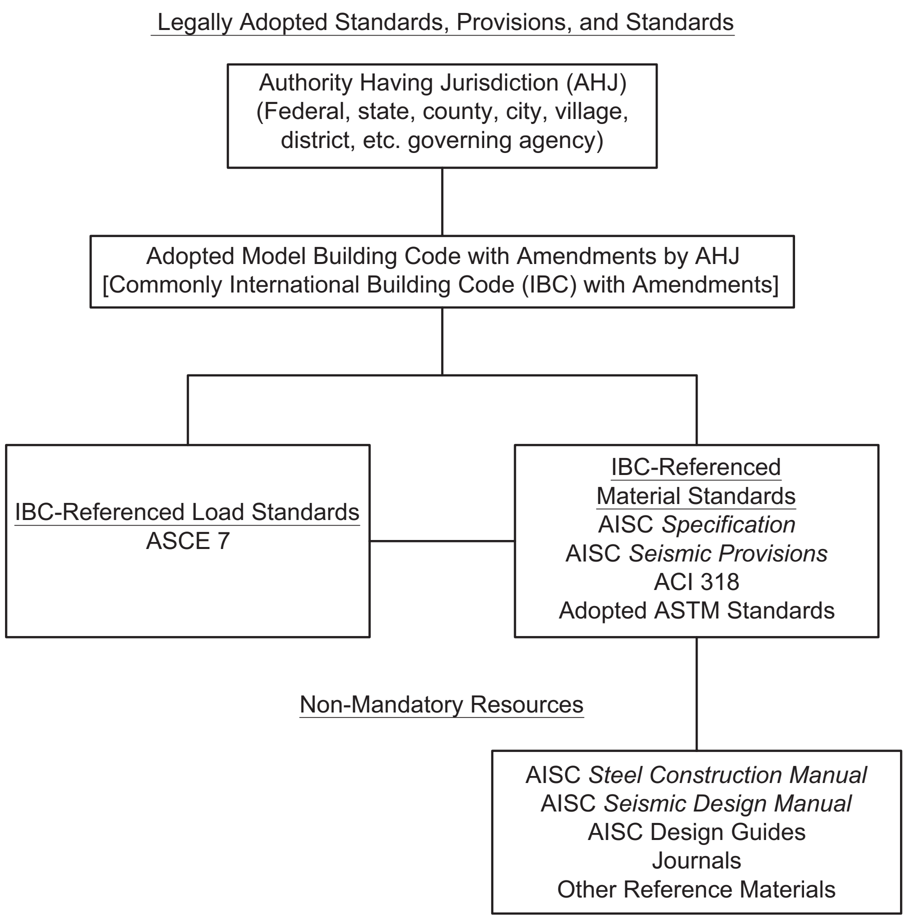


Fig. 1-2. Documents relevant to column base and anchorage design.

1.2.2 Relevant Developments since the Publication of Design Guide 1, 2nd Ed.

Research on column base connections published since the second edition of Design Guide 1 has addressed many of the issues identified by the Grauvilardell et al. (2005) research synthesis. This research has studied various types of base connections (exposed as well as embedded) in multiple contexts (strength, stiffness, seismic response, and interactions with the building frame) using various methodologies (experimental, analytical, and computational). Additionally, there have been changes to related codes and standards that necessitate updating the Guide for the purpose of consistency.

Research on Column Base Plate Connections

The vast majority of research in this area has focused on column base plate connections of the general type shown in Figure 1-1(a), which connects the column to a base plate, which is then attached to a footing using pre-installed anchors. The research has mainly focused on connections without a brace or gusset plate [i.e., Figure 1-1(c)]. Gomez et al. (2010), Kanvinde et al. (2014), Trautner et al. (2017b), and Hassan et al. (2022) conducted large-scale experiments on base plate connections consistent with U.S. construction practice, subjected to combinations of axial, flexure, and cyclic lateral loads. Gomez et al. (2011) experimentally investigated three different shear transfer mechanisms (anchor bearing, friction, and shear-lug bearing). These studies provided new insights about the strength characterization methods included in previous editions of the Design Guide, integrated new configurations in terms of anchor rod patterns and column sections, and resulted in new information about the cyclic response of these connections relevant to earthquake applications. Additionally, the experiments (along with previously conducted experiments summarized in Grauvilardell et al., 2005) provide benchmark data for validation of computational simulations as well as stiffness and load-deformation models. Tests on base plate connections in Europe (e.g., Wald et al., 2008a; Gresnigt et al., 2008; and Di Sarno et al., 2007) supplement the experimental data for model validation and development. Studies in Japan (Choi and Ohi, 2005), South Korea (Choi and Choi, 2013), and France (Seco et al., 2021) have examined column base plate connections subjected to biaxial bending and axial compression.

The experimental datasets have been complemented by finite element studies that generalize the findings to untested configurations and provide insights regarding internal force distributions. Examples of such studies include Kanvinde et al. (2013), Hassan et al. (2021), and Inamasu and Lignos (2022). Particularly notable in this regard is the Component-Based Finite Element Method (CBFEM) developed by Wald et al. (2008b) that has been used to characterize internal stress distributions in base plate connections.

In large part, the experimental data outlined in the preceding has suggested modest refinements to the strength models for base plate connections loaded in uniaxial bending and axial compression. However, new analytical models for base plate connections under biaxial bending and axial compression have been proposed by Hassan et al. (2021), Seco et al. (2021), and Fasaee et al. (2018). Models for the rotational stiffness of these as well as the full load-deformation response of these connections (under both monotonic and cyclic loading) have also been developed, by Kanvinde et al. (2012), Dumas et al. (2006), and Torres-Rodas et al. (2016). Reliability studies on base plate connections have been conducted in the United States by Song et al. (2020), Torres-Rodas et al. (2020), and Aviram et al. (2010) as well as in Nigeria (Idris and Umar, 2007) to examine the level of safety provided by various design approaches, including the approach outlined in the second edition of Design Guide 1.

Although research has not been conducted on braced frame base plate connections [such as the one shown in Figure 1-1(c)], it is relevant to mention here the publication of a SteelTIPS report (Astaneh-Asl, 2008) that provides guidance for the design of such details.

Research on Embedded Base Connections

Research on embedded base connections consistent with U.S. construction practice was nonexistent prior to the publication of the second edition of this Guide. Since then, significant research in the United States has resulted in experimental data on the strength and load-deformation characteristics of deeply embedded base connections (Grilli et al., 2017; Hassan and Kanvinde, 2023) as well as shallowly embedded base connections (Richards et al., 2018; Hassan and Kanvinde, 2023). This data, in addition to tests on embedded base connections conducted in Japan (Cui et al., 2009), has led to the development of strength (Grilli and Kanvinde, 2017; Barnwell, 2015), stiffness (Richards et al., 2018), and load-deformation/hysteretic models (Torres-Rodas et al., 2018a) for embedded base connections.

Other Relevant Research

The research outlined in the preceding was conducted to assess overall response of base connections. In addition to this, since 2006, there have been important investigations of specific effects and mechanisms that are important to base connections. For example, Myers et al. (2009) studied the effect of weld details between the base plate and column, while Christensen (2010) and Wilsmann (2012) studied welds in HSS base plates with corner anchor rods. A study by Cozzens et al. (2021) has revealed new information about the response of plate washers for column anchor rod applications. Studies have been conducted on the topic of anchorages as well—these include anchorage details specific to steel column to concrete footing attachments (Grilli and Kanvinde, 2016; Worsfold et al., 2022; Worsfold and Moehle, 2023). Work by Haninger and Tong (2014) and Denavit (private communications, 2022) has resulted in new and topical insights about plate bending limit states when subjected to bearing stresses from the footing.

System Studies Investigating the Interactions between Base Connection and Frame

As discussed earlier, the flexibility and load-deformation response of the base connection has the potential to significantly influence global structural response and has implications for design and simulation of the base connection as well as the entire structure. Building on models for base connection load-deformation response, numerous studies have investigated interactions between the base connection and the frame. These include Nonlinear Response History Analysis (NLRHA) studies by Zareian and Kanvinde (2013) and studies on instrumented buildings (Falborski et. al., 2020a) that investigated the effect of base connection stiffness on moment frame performance under earthquakes. Falborski et al. (2020b) and Inamasu et al. (2019, 2022) examined the feasibility of developing inelastic rotation in the base connection itself, rather than in the attached column. Other NLRHA studies (Torres-Rodas et al., 2018b) and Inamasu et al. (2020) have sought to establish appropriate base connection design loads under seismic conditions.

Relevant Changes in Related Codes and Standards

Subsequent to the publication of Design Guide 1, Second Edition, Second Printing, in 2010, incremental changes to codes and standards relevant to base plate and anchorage design have been adopted. Following are some of the significant changes related to ACI 318, the *International Building Code*, and the AISC *Seismic Provisions*.

As compared to the 2008 Edition of the ACI *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08 (ACI, 2008), the 2022 Edition of the ACI *Building Code Requirements for Structural Concrete*, ACI 318-19(22) (ACI, 2022), now includes:

1. Revisions to anchorage seismic provisions.
2. New provisions encompassing the design of post-installed adhesive anchors.
3. Reorganization of anchorage provisions and their relocation from Appendix D to Chapter 17.
4. Added provisions addressing screw anchors post-installed into concrete.
5. Clarifying guidance on the design of anchor reinforcement.
6. New provisions outlining requirements for the design of shear lugs.

Additionally, the *International Building Code* (IBC) from the 2009 Edition (ICC, 2009) to the 2021 Edition (IBC, 2021) has trended toward additional deference to the ACI 318 anchorage provisions with modifications as incorporated in Chapter 19. Although this Design Guide is based on the 2021 Edition of the IBC, it is important to consult the authority having jurisdiction to confirm which edition of IBC and any potential amendments are applicable for each project.

Significant changes to the AISC *Seismic Provisions* from the 2005 Edition (AISC, 2005) to the 2022 Edition (AISC, 2022b) include:

1. Additional requirements for welding, weld tabs, and weld backing for columns participating and not participating in the seismic force-resisting system (SFRS).
2. A new stipulation that the flexural demand at column bases can be limited by the overstrength seismic load only if a ductile limit state in either the column base or the foundation controls the design.
3. Revisions to the required shear strength at column bases for columns participating and not participating in the SFRS.

1.3 SCOPE, UPDATES, AND PREVIEW

The third edition of the Design Guide retains all the topics previously included in the second edition—that is, those pertaining to the design, fabrication, erection, and repair of base plate connections subjected to a range of loadings. Based on the new developments outlined in the previous section, several new topics are introduced, and the structure of the Guide is reorganized to facilitate its use. The main changes include the following:

- The title of the Design Guide has been changed to "Base Connection Design for Steel Structures" to reflect the expanded scope of the new edition.
- A new chapter (Chapter 3) is included to establish the relationship between the structure and base connection to provide context for connection selection, design, and simulation.
- Chapter 4 in this edition (which addresses column base plate connection design) is now particularized to column base plate connections with some modifications; broadly, this was the topic of Chapter 3 of the second edition. A new section is introduced to address base plate connections subjected to biaxial bending and axial compression.
- A new chapter on embedded base connections is included (Chapter 5), reflecting findings from multiple research studies summarized in Section 1.2.

- A new chapter (Chapter 6) has been added focusing on seismic design.
- Two new appendices (Appendices C and D) have been added to provide methods for the representation of column base connections in frame analysis and design and guidance regarding their simulation through finite element analysis.
- Sections pertaining to fabrication and erection have been added to Chapters 4 and 5 for exposed and embedded base connections, respectively.

This Design Guide develops 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, Chapter 17, which is exclusively based on the strength approach (LRFD), for the design of such embedments. As such, this Guide only includes LRFD provisions for concrete limit states where ACI 318 is applicable. 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.

This Design Guide is not intended to be used for structures outside the scope of the AISC *Specification*, the AISC *Seismic Provisions*, or ACI 318. One such example is for structures in nuclear facilities that should reference the application-specific requirements developed by AISC and ACI.

Chapter 2
Materials—Specifications, Selection, and Other Considerations

Chapter 2 outlines specifications pertaining to base plate, anchor rod, weld, grout, and concrete materials. These specifications are provided in several sources such as AISC, ACI, ASTM, AWS, and other documents. Additionally, guidance on the selection of materials and other applicable considerations are included. This Design Guide does not address stainless steel applications. For stainless steel applications, the reader is referred to the AISC *Specification for Structural Stainless Steel Buildings* (AISC, 2021) and the relevant ASTM materials standards.

2.1 BASE PLATE AND ANCHOR ROD 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. Preferred material specifications noted in Table 2-1 are based on the recommendations of the AISC *Manual* that are "…based on consultations with fabricators to identify materials that are commonly used in steel construction, and reflects such factors as ready availability, ease of ordering and delivery, and pricing." The reader is referred to AISC *Manual* Table 2-5, AISC *Manual* Table 2-6, and AISC *Specification* Table J3.2 for additional information.

2.2 BASE PLATE MATERIAL SELECTION

Base plates should be designed using ASTM A572/A572M (2021d) Grade 50 material unless the availability of an alternative grade is confirmed prior to specification. Because ASTM A572/A572M Grade 50 plate is readily available, the plates can often be cut from stock material. 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 of a crane building, the use of an anchor rod chair 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, bases asymmetric due to bracing connections, and columns that are adjacent to walls or foundation edges.

Many structural engineers have established minimum thicknesses for base plates for typical gravity columns in buildings. 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 ANCHOR ROD SELECTION (MATERIAL, TYPE, AND WELDABILITY)

As shown in Table 2-2, the preferred specification for anchor rods is ASTM F1554 (2020a), with Grade 36 being the most common grade used. The availability of other grades should be confirmed prior to specification.

ASTM F1554 Grade 55 anchor rods are used when there are large tensile forces due to moment connections or uplift from overturning. ASTM F1554 Grade 105 material 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, oversized anchor rod holes, should be furnished as heavy hex nuts, preferably ASTM A563 (2021a) Grade A or DH for Grade 105 material. Additionally, recommended sizes for plate washers are provided in Table 4-3.

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

Table 2-1. Recommended Base Plate Materials	
Thickness, t_p	**Plate Availability**
$t_p \le 4$ in.	ASTM A36/A36M ASTM A572/A572M Gr. 42 or 50[a] ASTM A588/A588M Gr. 50
4 in. $< t_p \le 5$ in.	ASTM A36/A36M[a] ASTM A572/A572M Gr. 42 ASTM A588/A588M Gr. 46
5 in. $< t_p \le 6$ in.	ASTM A36/A36M[a] ASTM A572/A572M Gr. 42 ASTM A588/A588M Gr. 42
6 in. $< t_p \le 8$ in.	ASTM A36/A36M[a] ASTM A588/A588M Gr. 42
$t_p > 8$ in.	ASTM A36/A36M[a]

[a] Preferred material specification

Table 2-2. Recommended Anchor Rod Materials

ASTM Designations		Tensile Strength, F_u, ksi	Nominal Tensile Stress,[a] $F_{nt} = 0.75F_u$, ksi	Nominal Shear Stress (N-Type),[a], [c] $F_{nv} = 0.450F_u$, ksi	Nominal Shear Stress (X-Type),[a], [b] $F_{nv} = 0.563F_u$, ksi	Maximum Diameter, in.
F1554	Gr. 36[d]	58	43.5	26.1	32.7	4
F1554	Gr. 55[d]	75	56.3	33.8	42.2	4
F1554	Gr. 105	125	93.8	56.3	70.4	3
A449		120	90.0	54.0	67.6	1
A449		105	78.8	47.3	59.1	1½
A449		90	67.5	40.5	50.7	3
A36/A36M		58	43.5	26.1	32.7	15
A354 Gr. BD		150	113	67.5	84.5	4

[a] Nominal stress on unthreaded body area of threaded part (gross area)
[b] Threads excluded from shear plane
[c] Threads included in shear plane
[d] Preferred material specification

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

ASTM 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. For rods with diameters larger than approximately 1 in., 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 embedded plate washers or other similar devices may increase the pullout strength of the anchor rod; however, they 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 breakout and side-face blowout could occur (see Section 4.3.2 of this Guide). In these cases, calculations should be done 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, that places restrictions on chemical composition and carbon equivalent content to provide weldability when needed. Adding this supplement is helpful should welding become anticipated for fixes in the field. Grade 36 is typically weldable without supplement. ASTM F1554 permits the manufacturer to substitute Grade 55 with supplementary requirement S1 when Grade 36 is specified. This may have an impact on design for seismic loading when the anchor rod capacity is being developed.

There are also supplemental provisions, S4, available for Grades 55 and 105 regarding Charpy V-Notch (CVN) toughness. These provide for CVN testing of 15 ft-lb at 40°F for Grade 55 and either 40°F or −20°F for Grade 105. 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 (Paris and Erdogan, 1963). 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 is typically 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 F2329/F2329M; 2015) or the mechanical galvanizing process (ASTM B695; 2021c) 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 (2020b) and A36/A36M (2019a) 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.

Post-installed anchors are not covered in this Design Guide. If used, they must be installed and inspected in accordance with the manufacturer's installation procedures, the IBC, and any applicable code approval reports.

2.4 WELD MATERIALS

Welding is commonly used at the base plate-to-column interface. Welding is also used in applications such as the attachment of shear lugs, stiffeners, and bracing gussets to the base plate; in cases where welded washer plates are utilized to transfer shear forces from anchor rods to the base plate; and in cases where base plates are welded to setting plates. Shop welding is typically more economical than field welding and is therefore preferred. Consumables for welding (filler metals and fluxes) are specified in AISC *Specification* Section A3.5. All welding must be in conformance with AWS *Structural Welding Code—Steel*, AWS D1.1/D1.1M (2020) as modified by AISC *Specification* Section J2. Matching filler metals must be used when CJP groove welds are subject to tension normal to the effective area per AISC *Specification* Section J2.6. Additional requirements for a minimum CVN toughness of 20 ft-lb at 40°F or lower are required per AISC *Specification* Section J2.6 for "CJP groove welded T- and corner joints with steel backing left in place, subjected to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor, as applicable, for a PJP groove weld."

Seismic applications where the AISC *Seismic Provisions* are enforced are subject to additional criteria. All welds for base connections participating in the seismic force-resisting system (SFRS) require filler metals conforming to AWS D1.8/D1.8M, clauses 6.1, 6.2, and 6.3 (2021) per AISC *Seismic Provisions* Section A3.4a. Base connection welds, when classified as demand critical, must also satisfy the additional provisions in AWS D1.8/D1.8M, clauses 6.1, 6.2, and 6.3 for demand critical welds.

2.5 GROUT MATERIALS

Grout serves as the connection between the steel base plate and the concrete foundation to transfer compression and shear through friction. Grout also serves to assist with maintaining the levelness of column base plates and plumbness of columns during erection. 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 if concrete confinement is used in calculating the available concrete bearing stress. This will ensure that the grout is not the limiting factor when the maximum available bearing strength of the concrete foundation is desired. The design thickness of the grout space will depend on how fluid the grout is and how accurately 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, but on the top of a footing or pier, normally the space should be 1½ to 2 in. Large base plates, plates with shear lugs, base details with large anchor rods, or applications with leveling nuts 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 at least two grout holes be provided. Grout holes are typically 2 to 3 in. in diameter and are typically thermally cut in the base plate. ACI 318, Section 17.11.1.2, requires the addition of inspection and vent holes with at least a 1 in. diameter for horizontal base plates with shear lugs. 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 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 engineer or record preparing design documents. 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. In this case, the erector then must 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 position.

2.6 CONCRETE MATERIALS

Requirements pertaining to concrete properties and durability (including reinforcement cover) are contained in ACI 318, Chapter 19. The requirements contained in Chapter 19 are dependent upon the application, seismic design category, and exposure class. Generally, more stringent limits are relevant in applications with elevated seismicity, applications exposed to freezing and thawing, applications exposed to sulfate, applications in contact with water, or applications exposed to chlorides such as deicing chemicals.

Requirements relating to concrete reinforcement materials are contained in ACI 318, Chapter 20. The most common deformed bar (rebar) specification is ASTM A615/A615M (2022a) Grade 60. Special requirements are applicable when deformed reinforcement is used as anchor reinforcement. Anchor reinforcement used in structures in Seismic Design Categories C through F are required to conform to ASTM A706/A706M (2022b) Grades 60 or 80. Alternatively, ASTM A615/A615M Grade 60 may be used if the requirements of ACI 318, Section 20.2.2.5(b), are satisfied. These requirements include limitations on actual yield strength based on mill tests, ratio of actual tensile strength-to-actual yield strength, minimum fracture elongation, and minimum uniform elongation.

Specifications for structural concrete are also contained in ACI 301-20 (2020) and are commonly adopted by reference in many construction documents. This document contains general requirements and additional guidance for items including but not limited to specifications for formwork and formwork accessories; reinforcement and reinforcement supports; concrete mixtures; handling, placing, and construction of concrete; mass concrete; and industrial floor slabs.

Chapter 3
Base Selection, Design, and Simulation

3.1 OVERVIEW AND ORGANIZATION

As discussed in Chapter 1, column base connections have a diversity of configurations, depending on the type of structural system in which they are used (e.g., moment frame versus braced frame) and the types of loads and actions they are used to resist (e.g., gravity versus lateral—wind or seismic). The combination of the different configurations with design scenarios results in a multitude of loading cases and details, which may further be discussed in the context of design procedures or simulation methods. This Guide addresses these various situations. Against this backdrop, the main objective of this chapter is to provide context for the interpretation and use of material in the Guide that addresses specific details, in terms of their design as well as simulation. It is important to note that the design, detailing, and simulation guidance provided in this Guide is applicable once an overall connection configuration is selected. Consequently, this chapter also outlines considerations for selection of a particular configuration. The chapter is divided into two sections: (1) Section 3.2 discusses different base connection configurations, the factors that drive their selection, the loading conditions they may be subjected to, and where the design guidance for each of these situations may be found in the Guide, and (2) Section 3.3 discusses the structural interaction of the base connection with the frame and directs the user to guidance (provided in this Guide) for appropriate representation of base connections in structural models.

3.2 BASE CONNECTION CONFIGURATIONS

Base connections may be classified in various ways. A convenient way to classify them is based on the structural system they are used within, which affects their basic configuration. These connections may be categorized as those used when only a column must be attached to a concrete footing versus when a column and another member (typically a diagonal brace) must be attached to the base connection. The former is common in moment-resisting frames, gravity frames, or in cantilever columns (e.g., as used in mezzanines), whereas the latter is common in braced frame structures. Figures 1-1(a) through (c) in Chapter 1 illustrate these basic types of configurations. With reference to these figures, Section 3.2.1 addresses columns without braces, while Section 3.2.2 addresses columns with braces.

3.2.1 Base Connections for Columns without Braces

Base connections for columns without braces represent the most common condition in many structures. In fact, the previous editions of Design Guide 1 have been focused on this condition. Figures 1-1(a) and (b) introduced previously show some generic details for this condition. Column base connections without braces may be broadly classified into two categories—exposed base plate connections and embedded base connections. Exposed base plate connections [Figure 1-1(a)] are by far the most common when large bending moments and shears are not carried by the base connection. This is due to the economy and convenience of fabrication and erection because the concrete installation is completed almost entirely before the steel columns are erected. As a result, they are often used in gravity frames, cantilever columns, or in moment frames in which the base moments are low (e.g., in nonseismic regions or in seismic regions for low- to mid-rise moment frames). When large moments and forces need to be resisted by the base connection, it is not feasible to rely only on anchor rods to transfer these moments and forces because these result in other expenses such as thicker or stiffened base plates, larger or additional anchor rods, or deeper anchorage depths. In these situations, the column may be embedded in the foundation [see Figure 1-1(b)], and resistance is obtained by direct bearing of the column against the concrete or through the attachment of reinforcement to the column flanges. However, this involves additional expense and coordination between the steel erection and concrete installation because multiple concrete pours are necessary, both before and after the erection of the column. Both exposed and embedded base connections are now summarized with respect to their details and navigation of this Guide.

Exposed Base Plate Connections for Columns without Braces

Exposed base plate connections typically consist of a column welded to a base plate that is then anchored to a concrete footing using anchor rods. Usually, a grout layer is present between the base plate and the footing to ensure a uniform transfer of stress between the plate and the footing, as shown in Figure 1-1(a). Within this general design concept, variations in detail selection may arise from the following factors:

- Member type: Various cross sections may be used for columns; the common ones are W-sections, square or rectangular HSS, channels, and round HSS sections, as shown in Figure 3-1. The shape of the column cross section affects the formation of yield lines in the base plate and the type of welds that can be used. Typically, the selection of the column precedes the design of the base connection. In this Guide, the focus is on W-sections and rectangular or square HSS sections. Round cross sections and nonrectangular base plates are outside the scope of this Guide. For these, finite element simulations (Appendix D provides guidelines), other design guides (e.g., Horn, 2011) for monopole bases, or research findings (Horová et al., 2011) may be more appropriate.

- Anchor rod pattern: Anchor rods may be used in various patterns, some of which are shown in Figure 3-2; these patterns may be necessitated by the magnitude of loads to be resisted in conjunction with the base plate size and type of attached column section.
- Anchor rod type: The Guide focuses on pre-installed (cast-in-place) anchor rods; these may be headed or hooked at the bottom. At the top, various details may be used. Plate washers welded to the base plate may be specified if shear is intended to be carried through the anchor rods; this allows for simultaneous engagement of all anchor rods in shear.
- Shear lug: If large shear forces must be transferred into the footing, a shear lug (see Figure 3-3) is often provided on the underside of the base plate.
- Welds: Welds between the column section and the base plate may be fillet welds or partial-joint-penetration (PJP) or CJP groove welds, depending on the type of column to be attached, and the strength and detailing requirements.
- Stiffening: Base plates may be stiffened with haunches to increase the flexural capacity; these are outside the scope of this Guide. The design of stiffened bases could use an elastic approach with an established load path or a yield line approach similar to connections discussed in AISC Design Guide 39, *End-Plate Moment Connections* (Eatherton and Murray, 2023).
- Seismic details: If connection ductility is required in addition to strength (to meet seismic requirements), additional detailing may be specified. This may include the use of upset thread anchor rods, or the use of chairs on top of the base plate, to increase the stretch length and deformation capacity of the anchor rods. Such details are discussed at length in Chapter 6.

Embedded Base Connections

Embedded base connections consist of the bottom portion of column embedded within the concrete footing as shown previously in Figure 1-1(b) and in Figure 3-4. Usually, a column support slab is provided below the base of the column for erection purposes, after which the footing is poured around the column. Flexure is typically resisted through a combination of two mechanisms: (1) horizontal bearing of the concrete against the column flange in conjunction with development of a shear panel and (2) vertical bearing of the embedded base plate against the footing. Variations to this basic detail—discussed at length in Chapter 5—depend on the magnitudes of applied loads (see Figure 3-4):

- Attachment of reinforcement to the column flange, or running the reinforcement through the column flanges, to supplement the concrete bearing and overall flexural strength.

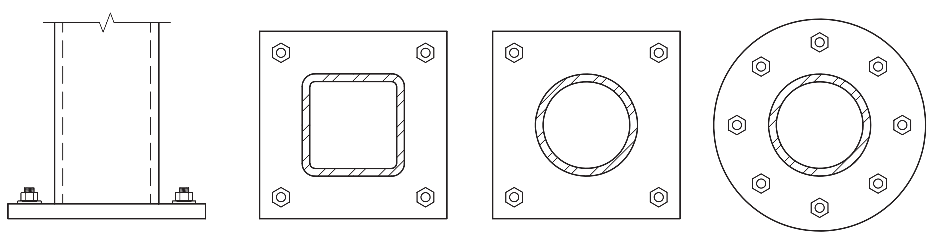


Fig. 3-1. Common base plate details based on type of column attached.

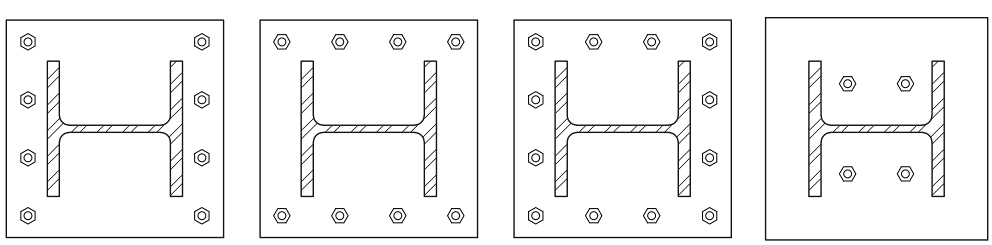


Fig. 3-2. Common anchor rod patterns.

- Hoops or stirrups to supplement shear strength of the footing, especially if vertical bearing of the embedded plate is an active mechanism.
- Installation of plates at the top of the footing (with grout) to transfer compressive loads through the footing. This may be similar to a stiffener plate between the column web and flanges, or a larger plate, if axial loads are high. The latter requires significantly more fabrication and welding.
- Typically, the base plate at the bottom (designed for erection forces) also resists uplift in the column; however, its size may be adjusted for this purpose.
- Other aspects of the embedded base connection detailing (e.g., reinforcement patterns) may depend on the type of foundation system (e.g., pile caps, mat foundations, or grade beams), and the load path from the column to the soil.

It is relevant to note here that while embedded base connections are often designed to obtain additional strength from the concrete, in some cases the embedment may be incidental. This is common, for example, in "blockout" connections wherein a slab-on-grade is cast on top of the base plate (see Figure 3-5). To achieve this, the column is first connected to the footing as in a conventional exposed base plate connection but through a diamond shaped blockout as shown in Figure 3-5. This blockout allows for the installation of the remainder of the slab-on-grade prior to the installation of any structural steel (minimizing/eliminating the overlap of concrete and steel workers on the job site). Subsequently, the blockout is filled with unreinforced concrete, grout, or felt strips, creating a cold joint between the blockout concrete and the remainder of the slab. The blockout, and the surrounding slab, create a base connection that has a shallow embedment, which provides supplemental flexural strength

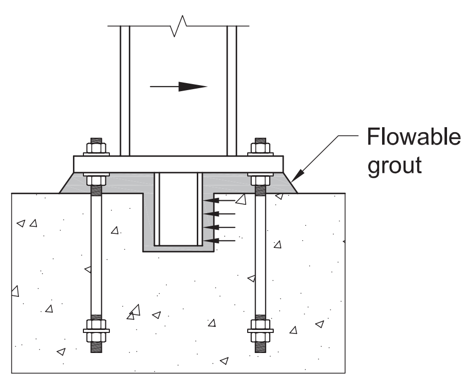


Fig. 3-3. Shear lug to resist large shear forces.

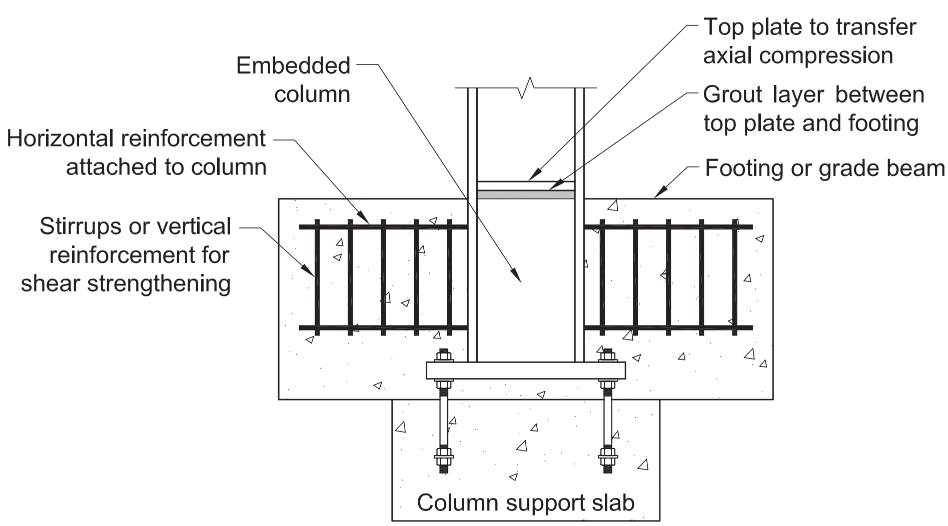


Fig. 3-4. Embedded base connection showing details.

and stiffness. This flexural resistance is usually discounted in design (except for shear transfer through base plate bearing) but becomes important in the context of performance assessment. As a result, the blockout connection is discussed in this Guide only in the context of its simulation within structural models. This guidance can be found in Appendix C.

Loading Conditions Considered and Navigation of the Guide

In general, base connections without a brace may be subjected to a combination of axial force, biaxial moments (with respect to the column cross-section axes), and biaxial shears. These may be applied in a static sense or in a seismic sense. The Guide contains comprehensive guidance for the design of these connections. Specifically, the design guidance is organized as follows:

- For exposed base plate connections without a brace:
    - Strength design guidance may be found in Chapter 4. This chapter provides guidance for design under 11 common loading scenarios, featuring various combinations of axial tension and compression, along with moments (in both directions) and shear.
    - Seismic design guidance may be found in Chapter 6. This chapter defers to Chapter 4 for strength design guidance but outlines detailing and additional considerations that are relevant in a seismic context.
- For embedded base connections without a brace:
    - Strength design guidance may be found in Chapter 5. This chapter exclusively focuses on embedded base connections without a brace. Given the relatively limited research on this topic, only in-plane loading cases are considered, with axial compression and tension combined with uniaxial flexure and shear. Supplementary information for seismic design may be found in Chapter 6.

In either case, torsion in the column is not in the scope of the Guide owing to the lack of research in this area and assuming that torsion in the column will be low relative to other forces. It is noted here that torsion in the column may produce shear in the anchors and also induce tension in the anchors if significant warping is present in the column along with the torsion.

3.2.2 Base Connections for Columns with Braces

Column bracing is commonly used in various types of structures, with the brace directly connected to the base connection and the column, typically through a gusset plate as shown in Figure 1-1(c). These connections are used in both nonseismic as well as seismic contexts. In a nonseismic context, they may be used for stability bracing or for bracing systems to resist lateral loads such as wind. In a seismic context, they form an integral part of lateral load-resisting systems, including ordinary and special concentrically braced frames as well as eccentrically braced frames and buckling restrained braced frames (AISC, 2022b). Similar to base connections without braces, these connections may be constructed as exposed base plate or embedded base plate connections; the latter is used when it is unfeasible to carry the large base forces through the anchor rods alone. However, as in the case of base connections without a brace, this involves additional expense and coordination between the steel erection and concrete installation. Within the generic configuration shown in Figure 1-1(c), numerous variations are possible in the detail. Some common variations along with factors that influence their selection are:

- Exposed versus embedded connections: Although exposed base plate connections with a brace [Figure 1-1(c)] are

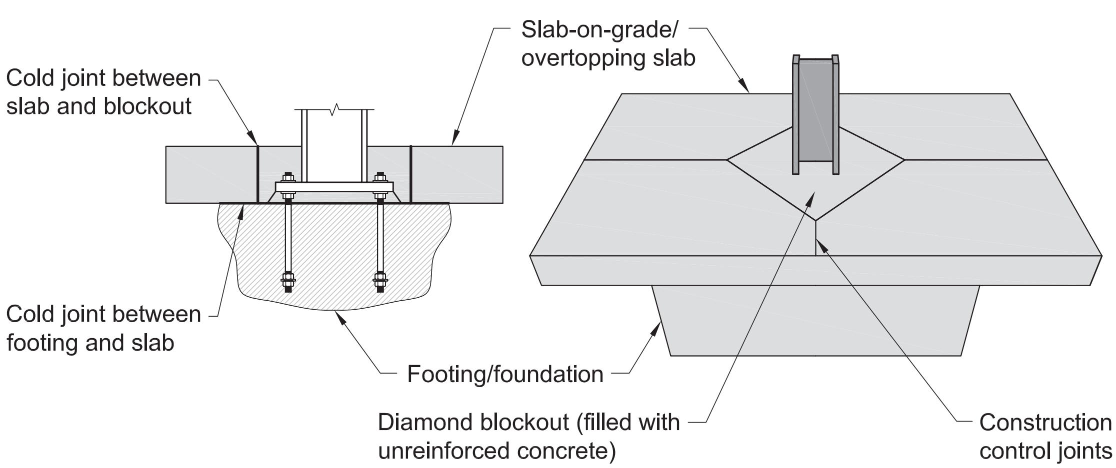


Fig. 3-5. Blockout connections resulting in shallow embedment of base plate connections.

common, embedded (or encased) connections for base connections with a brace may become necessary if the loads (especially the base shear) are large and cannot be resisted effectively through the anchor rods or a shear lug (see Figure 3-6).

- Shear lug: If large shear forces must be transferred into the footing, a shear lug (see Figure 3-3) is often provided on the underside of the base plate.
- Drag strut: Drag struts or grade beams are often used to carry horizontal forces from the brace into adjacent footings.
- Anchor rod patterns: The shape of the column, gusset plate, and the loads affect anchor rod patterns.
- Vertical stiffeners: These are often provided to increase the flexural strength of the base plate. Gusset plates incidentally provide vertical stiffening.

Loading Conditions Considered and Navigation of the Guide

In general, base connections with a brace may be subjected to similar types of loading as those without a brace—that is, a combination of axial force, biaxial moments (with respect to the column cross-section axes), and biaxial shears. These may be applied in a static sense or in a seismic sense. However, in base connections with a brace, the vertical and horizontal forces (i.e., axial forces and shear) are likely to be significantly higher relative to the moments, in contrast to connections without a brace. These moments in connections with a brace arise due to fixity of the base connections (which results in a deviation from the truss assumptions) or to an eccentricity between the working point of the connection and the centroid of the base plate. In either case, as far as the design of the base connection itself is concerned, the main difference is in the relative magnitudes of the forces, rather than the design procedure itself. The design guidance for base connections with a brace is included in Chapter 4 for exposed base plate connections and Chapter 5 for embedded base connections. In Chapter 4, specific design examples are provided for two loading cases (Section 4.3.4—Design for Combined Tension and Shear, and Section 4.3.5—Design for Combined Compression and Shear) that are considered to be commonly applicable to base connections with braces.

3.3 INTERACTION OF BASE CONNECTIONS WITH FRAMES

The flexibility and load-deformation response of base connections influence the internal force and moment distribution of the entire structure in addition to the structural deformations. As a result, it is important to appropriately represent the base connection in structural models. These interactions are particularly important in moment frames. These connections are often represented as either pinned or fixed in structural models, both of which introduce error into estimation of structural response. The discussion in this Guide is restricted to the moment-rotation response of base connections in moment frames (including the initial rotational stiffness and the subsequent yielding and hysteretic behavior). This section is divided into three subsections: Section 3.3.1 provides a general qualitative commentary of the load deformation response of base connections, with implications for the response of moment frames, and Sections 3.3.2 and 3.3.3 address simulation of base connections for seismic performance assessment for two different types of design—one in which the base connection remains elastic (a strong-base design) versus one in which it is expected to

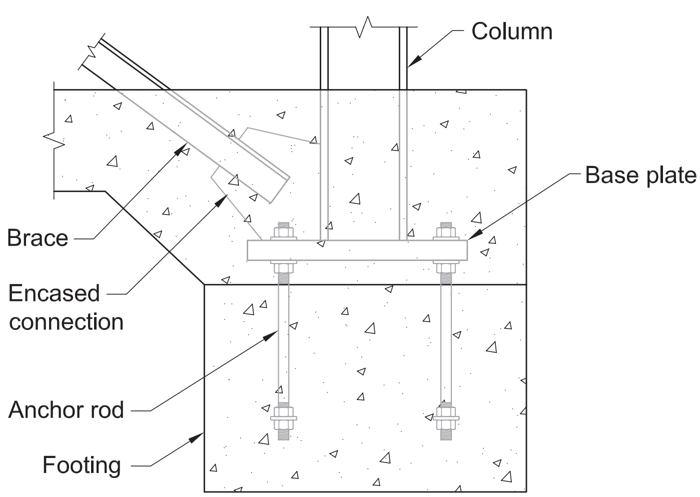


Fig. 3-6. Embedded plate brace connection.

yield (weak-base design). These sections conclude by directing the reader to modeling guidance for base connections.

3.3.1 General Observations about Base Connection Load-Deformation Response

Figure 3-7 shows the typical moment-rotation response (experiment by Gomez et al., 2010) of an exposed base plate type connection subjected to a cyclic loading increasing rotation in the presence of axial compression. Referring to the figure, it is evident that:

1. Even before the connection yields, there is significant rotation in the base connection. In fact, if it is assumed that design moments occur at ~70–80% of capacity (owing to safety factors) and sizing considerations, a rotation of 0.01–0.02 rad is obtained at yield. This type of response is observed across different types of base connections, both exposed and embedded. This suggests that the base connections possess partial fixity and cannot be assumed as fixed or free without further analysis or context.
2. Even during this initial "elastic" response of the connection, there is some nonlinearity in the load-deformation response. This occurs due to effects such as uplift of the base plate from the footing and nonlinearity in the concrete stress-strain response in bearing. The implication of this is that a secant stiffness is usually measured at the point of yielding of the base connection (see Figure 3-7) or at a fixed fraction of ultimate capacity is more appropriate for representation of the base plate stiffness.
3. The base connections possess significant ductility under both monotonic and cyclic conditions. As an example, the base connection response shown in Figure 3-7 showed anchor rod fracture at a rotation of over 0.08 rad. While the precise degree of deformation capacity is sensitive to detailing, a review of experimental data indicates that both exposed and embedded base connections (tested since 1984) in general, have rotation capacities well above 0.04 rad.

3.3.2 Modeling Base Connections for Strong-Base Design

In a vast majority of design scenarios (with exceptions noted in Section 3.3.3), base connections are expected to remain elastic. This includes almost all static/gravity and wind load situations, as well as most situations for seismic design (wherein the base connection is capacity designed to fully develop the plastic moment capacity of the attached column—see Chapter 6). As a result, the only attribute of the base connection that participates in interaction with the frame is its elastic (or initial) rotational stiffness—shown in Figure 3-7. In these cases, the base rotational flexibility (reciprocal of rotational stiffness) influences the structural response in three ways:

- Under lateral (e.g., seismic) loading, the rotational flexibility of the base connection lowers the point of inflection in the column with respect to the fixed base assumption. This increases the moment at the top of the column, increasing the susceptibility of the frame to weak-story collapse.
- Under lateral loading, the rotational flexibility increases the interstory drift in the first story with respect to the fixed base assumption.
- The base flexibility may influence the column end fixity, affecting its effective length, and consequently its compressive strength.

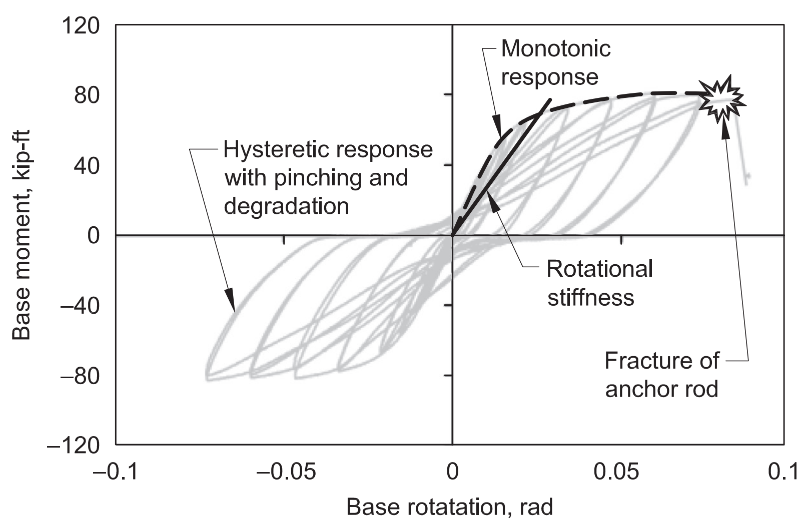


Fig. 3-7. Moment rotation response of an exposed base plate connection (from Gomez et al., 2010) showing rotational stiffness.

While it is known that base connections are not rigid, approaches to estimate their flexibility are not commonly used in the design and performance assessment process, and they are often represented as pinned or fixed in structural models. Representing them as pinned in structural models is a conservative assumption, which results in increased estimates of story drifts and column moments; this in turn leads to the selection of heavier or larger members and an increase in steel tonnage. On the other hand, sometimes these connections are considered to be fixed in structural models because they are designed to be stronger than the column. While this may appear to be a reasonable assumption, it indicates a conflation of strength and stiffness. In contrast, experimental data suggests that even base connections that are significantly stronger than the attached column (including embedded base connections) exhibit significant rotational flexibility, such that simulating them as fixed may be erroneous and may lead to unconservative characterization of performance. That said, the influence of the base rotation stiffness on overall structural response is highly dependent on the remainder of the structure—for example, a frame with highly stiff beams and columns will show low base rotations regardless of base flexibility. Thus, modeling the most accurate estimate of base connection flexibility is the best option to avoid unnecessary conservatism or unconservatism. To this end, stiffness estimation methods for column base connections of various configurations have been developed. These have been extensively validated against both experimental data as well as recorded seismic response data from instrumented buildings. Moreover, these methods are fairly straightforward to apply. Given this, it is desirable to represent the elastic stiffness of base connections (and the foundation system generally) in the same manner as the remainder of the structure is represented in simulation. Approaches to estimate the elastic stiffness of various types of base connections are provided in Appendix C.

3.3.3 Modeling Base Connections for Weak-Base Design

Designing the base connections to be stronger than the attached column (i.e., designing them to have a higher moment capacity as compared to the strain-hardened plastic moment capacity of the column) is costly. As a result, weak base design may become desirable in which the column base connection is designed to yield in an inelastic cyclic manner (in a manner similar to that illustrated in Figure 3-7) while the column remains elastic. This type of response is explicitly allowed by the AISC *Seismic Provisions* as long as ductile response can be achieved in the base connection. Design methods that leverage the ductility of base connections in this manner are currently under development along with details that provide such ductility (outlined in Chapter 6). However, performance assessment of such structures requires simulation of the hysteretic response of base connections. Referring to Figure 3-7, this response is somewhat complex, showing characteristics such as cyclic degradation, pinching, and loss of strength. Appendix C (Section C.3) provides guidelines for simulating this type of response in structural models.

Chapter 4
Design of Exposed Column Base Connections

4.1 OVERVIEW AND ORGANIZATION

This chapter provides the design requirements for exposed column bases, such as those shown in Figures 1-1(a) and (c). Several different design load cases and combinations in exposed column base connections are discussed in Section 4.3:

- Section 4.3.1 Design for Axial Compression
- Section 4.3.2 Design for Axial Tension
- Section 4.3.3 Design for Shear
- Section 4.3.4 Design for Combined Axial Tension and Shear
- Section 4.3.5 Design for Combined Axial Compression and Shear
- Section 4.3.6 Design for Bending
- Section 4.3.7 Design for Combined Axial Compression and Bending
- Section 4.3.8 Design for Combined Axial Tension and Bending
- Section 4.3.9 Design for Combined Axial Compression, Bending, and Shear
- Section 4.3.10 Design for Combined Axial Tension, Bending, and Shear
- Section 4.3.11 Design for Combined Axial Compression and Biaxial Bending

For loading cases or combinations where bending is considered, two conditions are discussed—low moment and high moment. In each section, the design methodology is outlined. Detailed design examples follow in the Section 4.7.

Section 4.4 provides methodologies available for the design of anchorage reinforcement. The anchor rods for base connections are designed for steel strength and concrete strength. In many situations, either due to the concrete thickness or the closeness of the anchor rods to the edge of the concrete, the concrete breakout strength is reduced, and the required anchor strength cannot be achieved. For such cases, anchor reinforcement is typically added to transfer the design load from the anchors into the structural concrete member.

In addition, the chapter provides information related to fabrication and installation in Section 4.5 and repair and field fixes in Section 4.6.

4.2 OVERALL DESIGN PROCESS AND FLOW

The general behavior and distribution of forces for a column base 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 crushes in bearing, the column to base plate weld fractures, the anchor rods yield in tension, or the concrete strength of the anchor rod group is reached. If the concrete strength of the anchor rod group is larger than the lowest of the other limit states, the behavior generally will be ductile, notwithstanding weld fracture, if it occurs. However, it is not always necessary or even possible to design a foundation that prevents concrete failure.

The overall base connection design process includes six steps: (1) base plate footprint selection; (2) determination of appropriate distribution of internal forces; (3) base plate thickness selection; (4) anchor rod, anchor group, and reinforcement design; (5) considerations for footing design; and (6) welding design and detailing.

The regulations of the Occupational Safety and Health Administration (OSHA) Safety Standards for Steel Erection (2020) 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 (¾ in. diameter) on a 4 in. by 4 in. pattern. If one considers only the moments from the eccentric loads (because 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.-diameter ASTM F1554 Grade 36 anchor rods (see Table 4-1 in Section 4.3.2) equals $(2)(14.5 \text{ kips})(4 \text{ in.}) = 116$ kip-in. For a 14-in.-deep column, the OSHA required moment strength is only $(1.6)(0.300 \text{ kips})(18 \text{ in.} + 7 \text{ in.}) = 12.0$ kip-in.

4.3 LOAD COMBINATIONS

4.3.1 Design for Axial Compression

Overview of Mechanics and Method

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).

Concrete Bearing Limit

The nominal bearing strength of column bases bearing on concrete is defined in ACI 318, Section 22.8.3.2, as $B_n = (0.85 f_c' A_1)$ 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 $\sqrt{A_2/A_1} \le 2$. The relationship between A_2 and A_1 is illustrated in ACI 318, Figure R22.8.3.2,

where

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

A_2 = area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having its upper base equal to the loaded area, in.^{2}

f_c' = specified compressive strength of concrete, 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, $\sqrt{A_2/A_1} = 2$.

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

On the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \qquad \text{(}\textit{Spec.}\text{ Eq. J8-1)}$$

On less than the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \qquad \text{(}\textit{Spec.}\text{ Eq. J8-2)}$$

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 for bearing on concrete as follows:

$$\phi_c = 0.65 \text{ (LRFD)}$$

$$\Omega_c = 2.31 \text{ (ASD)}$$

ACI 318, Section 21.2.1, also stipulates a resistance factor of $\phi = 0.65$ for bearing on concrete.

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

On the full area of a concrete support:

$$f_{p(max)} = 0.85 f_c' \qquad (4\text{-}1)$$

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

$$f_{p(max)} = 0.85 f_c' \sqrt{A_2/A_1} \le 1.7 f_c' \tag{4-2}$$

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

LRFD	ASD
$f_{pu(max)} = \phi_c f_{p(max)}$ (4-3a)	$f_{pa(max)} = \dfrac{f_{p(max)}}{\Omega_c}$ (4-3b)

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

LRFD	ASD
$\dfrac{P_u}{A_1} \le f_{pu(max)}$ (4-4a)	$\dfrac{P_a}{A_1} \le f_{pa(max)}$ (4-4b)

Thus,

LRFD	ASD
$A_{1(req)} = \dfrac{P_u}{f_{pu(max)}}$ (4-5a)	$A_{1(req)} = \dfrac{P_a}{f_{pa(max)}}$ (4-5b)

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

LRFD	ASD
$A_{1(req)} = \dfrac{P_u}{\phi_c 0.85 f_c'}$ (4-6a)	$A_{1(req)} = \dfrac{\Omega_c P_a}{0.85 f_c'}$ (4-6b)

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

LRFD	ASD
$A_{1(req)} = \dfrac{1}{2}\left(\dfrac{P_u}{\phi_c 0.85 f_c'}\right)$ (4-7a)	$A_{1(req)} = \dfrac{1}{2}\left(\dfrac{\Omega_c P_a}{0.85 f_c'}\right)$ (4-7b)

Many column base plates bear directly on a layer of grout. The grout compressive strength should always be higher than the concrete compressive strength. Because the grout compressive strength is always specified higher than the concrete strength, the concrete compressive strength, f_c', must be used in the preceding equations. The previous edition of this Design Guide recommended that the grout strength be specified as two times the concrete strength. Lower grout strengths may be justified if the bearing strength of the grout is checked against the required strength. The important dimensions of the column-base connection are shown in Figure 4-1.

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:

LRFD	ASD
$$f_{pu} = \frac{P_u}{BN} \quad (4\text{-}8a)$$	$$f_{pa} = \frac{P_a}{BN} \quad (4\text{-}8b)$$

This bearing pressure causes bending in the base plate at the assumed critical sections shown in Figure 4-1(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:

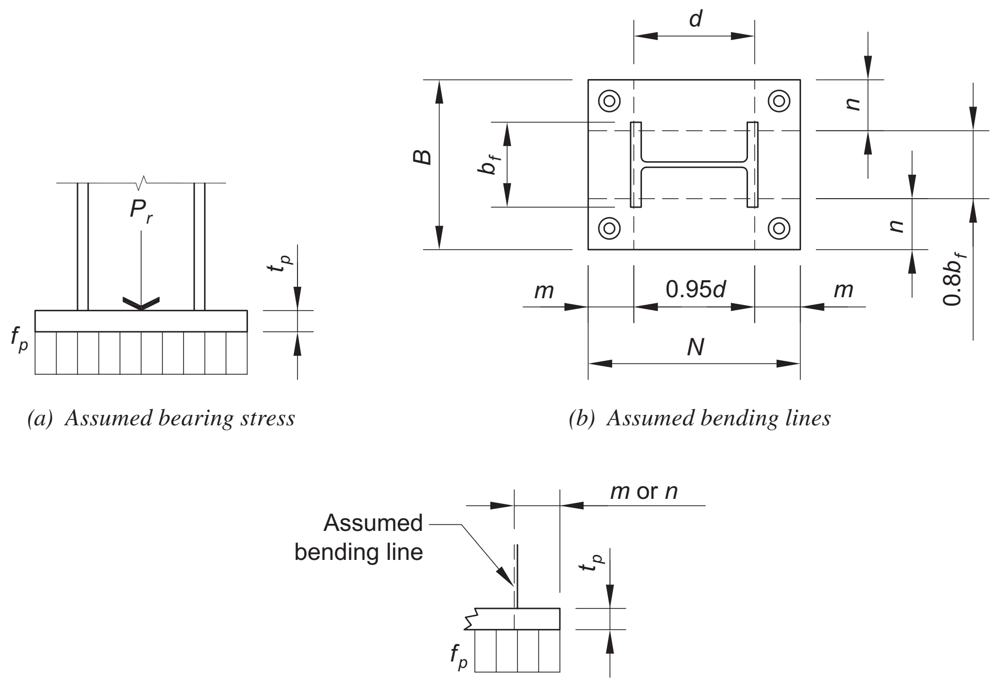
LRFD	ASD
$$M_{pl} = f_{pu}\left(\frac{l^2}{2}\right) \quad (4\text{-}9a)$$	$$M_{pl} = f_{pa}\left(\frac{l^2}{2}\right) \quad (4\text{-}9b)$$

where the critical base plate cantilever dimension, l, is the largest of m, n, and $\lambda n'$. The following equations are also found in AISC *Manual* Part 14.

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} \tag{4-12}$$



(a) Assumed bearing stress

(b) Assumed bending lines

(c) Base plate design moment determination

Fig. 4-1. Design of base plate with axial compressive load.

where

B = base plate width, in.

N = base plate length, 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.

$= \dfrac{\sqrt{db_f}}{4}$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \leq 1 \tag{4-13}$$

X is determined as:

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{P_u}{\phi_c P_p}$ (4-14a)	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{\Omega_c P_a}{P_p}$ (4-14b)

where

P_a = required axial compressive strength (ASD), kips

P_p = nominal strength of concrete under the base plate, kips

$= 0.85 f_c' A_1 \sqrt{A_2/A_1} \leq 1.7 f_c' A_1$ (*Spec*. Eq. J8-2)

P_u = required axial compressive strength (LRFD), kips

ϕ_c = 0.65

Ω_c = 2.31

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, 2023):

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{\phi_b F_y BN}}$ (4-15a) $\phi_b = 0.90$	$t_{min} = l\sqrt{\dfrac{2\Omega_b P_a}{F_y BN}}$ (4-15b) $\Omega_b = 1.67$

where F_y is the specified minimum yield stress of the base plate in ksi.

Because 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 typically accomplished by proportioning the base plate dimensions so that m and n are approximately equal.

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.

General Design Procedure

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

Case I: $A_2 = A_1$ (no consideration of concrete confinement)
Case II: $A_2 \geq 4A_1$ (consideration of concrete confinement)
Case III: $A_1 < A_2 < 4A_1$ (consideration of concrete confinement)

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—that is, $A_2 \geq 4A_1$ (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 and must be determined using an iterative process. 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 4-1(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 Cases I–III are detailed in the following. Anchor rod design is covered in Section 4.3.2.

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 using Equations 4-6.

LRFD	ASD
$A_{1(req)} = \dfrac{P_u}{\phi_c 0.85 f_c'}$ (4-6a)	$A_{1(req)} = \dfrac{\Omega_c P_a}{0.85 f_c'}$ (4-6b)

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

$$N \approx \sqrt{A_{1(req)}} + \Delta \tag{4-16}$$

where

$$\Delta = \frac{0.95d - 0.8b_f}{2} \tag{4-17}$$

then

$$B = \frac{A_{1(req)}}{N} \tag{4-18}$$

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 viewpoint select N equal to B.

4. Calculate the required base plate thickness.

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} \tag{4-12}$$

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1 \tag{4-13}$$

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{P_u}{\phi_c P_p}$ (4-14a)	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{\Omega_c P_a}{P_p}$ (4-14b)

where the available bearing strength is determined using AISC *Specification* Equation J8-1:

LRFD	ASD
$\phi_c P_p = \phi_c 0.85 f_c' A_1$	$\dfrac{P_p}{\Omega_c} = \dfrac{0.85 f_c' A_1}{\Omega_c}$

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

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{\phi_b F_y BN}}$ (4-15a)	$t_{min} = l\sqrt{\dfrac{2\Omega_b P_a}{F_y BN}}$ (4-15b)

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, except to provide lateral support to the bottom of the column, and need only to be sized for OSHA requirements, erection considerations such as wind during construction, and practical considerations.
6. Determine the welding required as necessary.

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.

LRFD	ASD
$A_{1(req)} = \dfrac{1}{2}\left(\dfrac{P_u}{\phi_c 0.85 f_c'}\right)$ (4-7a)	$A_{1(req)} = \dfrac{1}{2}\left(\dfrac{\Omega_c P_a}{0.85 f_c'}\right)$ (4-7b)

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 ($A_2 \ge 4A_1$).

   Calculate A_2 based on A_1 and using ACI 318, Section 22.8.3.2. If $A_2 \ge 4A_1$, calculate the required thickness using the procedure shown in Step 4 of Case I using Equations 4-19 to calculate the available bearing strength:

LRFD	ASD
$\phi_c P_p = 2\phi_c 0.85 f_c' A_1$ (4-19a)	$\dfrac{P_p}{\Omega_c} = \dfrac{2(0.85 f_c')A_1}{\Omega_c}$ (4-19b)

5. Determine the anchor rod size and location.
6. Determine the welding required as necessary.

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 II.

LRFD	ASD
$$A_{1(req)} = \frac{1}{2}\left(\frac{P_u}{\phi_c 0.85 f_c'}\right) \quad \text{(4-7a)}$$	$$A_{1(req)} = \frac{1}{2}\left(\frac{\Omega_c P_a}{0.85 f_c'}\right) \quad \text{(4-7b)}$$

3. Optimize the base plate dimensions, N and B. Use the same procedure from Case I, Step 3.
4. Calculate A_2, based on A_1 and using ACI 318, Section 22.8.3.2.
5. Determine if the required axial compressive strength is less than the available bearing strength using Equations 4-20:

LRFD	ASD
$$P_u \le \phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \quad \text{(4-20a)}$$	$$P_a \le \frac{P_p}{\Omega_c} = \left(\frac{0.85 f_c' A_1}{\Omega_c}\right)\sqrt{\frac{A_2}{A_1}} \quad \text{(4-20b)}$$

   If the condition is not satisfied, revise N and B, and retry until criterion is satisfied. This is an iterative process.

6. Determine the base plate thickness using the procedure from Case I, Step 4.
7. Determine the anchor rod size and location.
8. Determine the welding required as necessary.

4.3.2 Design for Axial Tension

Overview of Mechanics and Method

The design of base connections for tension consists of five 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 loads of the supported elements, the supporting columns are subjected to net uplift forces. In addition, columns in moment frames or braced frames may be subjected to net uplift forces due to overturning.

2. Design the welding required between the column and the base plate.

   Consideration should be given to the load path from the column to each anchor rod. If the base plate is sufficiently stiff such that it can be considered rigid, it may be reasonable to consider the entirety of the weld as fully effective to resist the forces flowing from the column, into the base plate, and toward the anchor rods. In most cases, however, only portions of the weld group may be effective in transferring forces flowing from the column. One such example is when anchor rods are located at the corners of HSS columns or flange tips of wide-flange columns. A method to address such stress concentrations in the weld group has been provided in publications such as AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Buildings* (West and Fisher, 2020), and the AISC *Hollow Structural Sections Connections Manual* (AISC, 1997). Subsequent testing by Christensen (2010) and Wilsmann (2012) has evaluated this approach and found it to be generally conservative for the tested cases. A second case is illustrated in Example 4.7-3, wherein only welding adjacent to the anchor rods is considered effective to resist the tension loading. A consistent model of load path from the column, through the effective portions of the welding, through the effective portions of the base plate in bending, through the anchor rods, and into the concrete should be used.

3. 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 2.3. The number of anchor rods required is a function of the maximum net uplift on the column, the distribution of the uplift reaction to the various anchors, and the strength per rod for the anchor rod material chosen. The force distribution among anchor rods will likely be affected by anchor rod locations, plate stiffness, and any base plate stiffening elements present. Variations in force distribution will occur where differences in relative stiffnesses among the anchor rods in a group exist and the force distribution is statically indeterminate. In these situations, tensile loads in the anchor rods should be proportioned considering the stiffness of the load path to that anchor rod. Alternatively, the base plate should be made sufficiently stiff, or stiffening added, to account for the differences in relative stiffnesses. Finite element analyses such as outlined in Appendix D may be used to evaluate the relative anchor rod tensions and plate stresses in cases where the plate is not sufficiently stiff to ensure rigid plate behavior and the distribution is statically indeterminate.

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 4 following), and because the length of the rods results in larger deflections than for steel-to-steel connections. The procedure to determine the required size of the anchor rods is discussed in Section 4.3.2.1.

4. Determine the appropriate base plate size and thickness 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 4-1. 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.

5. Determine the concrete tensile 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 discussed in Section 4.3.2.2.

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 of anchors is calculated in accordance with ACI 318. Section 4.3.2.1 provides the methodology to determine the steel tensile strength and Section 4.3.2.2 provides the approach used to determine the concrete tensile strength.

4.3.2.1 Anchor Rod Steel Tensile Strength

The steel tensile strength of an anchor 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). ASME B1.1 (2020) defines this threaded area as:

$$A_{se,N} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n}\right)^2 \tag{4-21}$$

where

d_a = major diameter, in.

n = number of threads per in.

Table 4-1 lists the net tensile stress area for diameters between ⅝ in. and 4 in.

Two methods of determining the required tensile stress area are commonly used. One is based directly on the ASME-stipulated tensile stress area as described previously. 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 the default method stipulated in the AISC

Table 4-1. ASTM F1554 Anchor Rod (Rod Only) Available Tensile Strength										
Rod Diameter, in.	Threads per inch (UNC)	Nominal Rod Area, A_b, in.^{2}	Tensile Stress Area, $A_{se,N}$, in.^{2}	Available Tensile Strength, kips						
				ϕR_n ($\phi = 0.75$)			R_n/Ω ($\Omega = 0.75$)			
				LRFD			ASD			
				Grade 36	Grade 55	Grade 105	Grade 36	Grade 55	Grade 105
⅝	11	0.307	0.226	9.83	12.7	21.2	6.55	8.48	14.1
¾	10	0.442	0.334	14.5	18.8	31.3	9.69	12.5	20.9
⅞	9	0.601	0.462	20.1	26.0	43.3	13.4	17.3	28.9
1	8	0.785	0.606	26.4	34.1	56.8	17.6	22.7	37.9
1⅛	7	0.994	0.763	33.2	42.9	71.5	22.1	28.6	47.7
1¼	7	1.23	0.969	42.2	54.5	90.8	28.1	36.3	60.6
1½	6	1.77	1.41	61.3	79.3	132	40.9	52.9	88.1
1¾	5	2.41	1.90	82.7	107	178	55.1	71.3	119
2	4½	3.14	2.50	109	141	234	72.5	93.8	156
2¼	4½	3.98	3.25	141	183	305	94.3	122	203
2½	4	4.91	4.00	174	225	375	116	150	250
2¾	4	5.94	4.93	214	277	462	143	185	308
3	4	7.07	5.97	260	336	560	173	224	373
3¼	4	8.30	7.10	309	399	—	206	266	—
3½	4	9.62	8.33	362	469	—	242	312	—
3¾	4	11.0	9.66	420	543	—	280	362	—
4	4	12.6	11.1	483	624	—	322	416	—
— Grade not available in the given diameter.									

Specification; however, footnote b of *Specification* Table J3.2 permits the tensile load to be calculated by multiplying the tensile stress area of the threaded rod by the specified minimum tensile stress.

The strength of structural fasteners in AISC documents has historically been based on the modifying factor and the nominal bolt diameter, while the direct tensile stress area approach is stipulated in ACI 318, Chapter 17. The designer should be aware of the differences in these design approaches and stay consistent within one system when determining the required anchor area.

Strength tables for commonly used anchor rod materials and sizes are easily developed by the procedures that follow, for either design method. Table 4-1 included herein has been developed for ASTM F1554 rods based on the tensile stress area approach for consistency with the ACI approach. (Note: ASTM F1554 is the suggested standard and preferred anchor rod material.)

AISC *Specification* Table J3.2, footnote b and ACI 318, Equation 17.6.1.2 stipulate the nominal tensile strength of an anchor rod as:

$$R_n = F_u A_{se,N} \tag{4-22}$$

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

$$\phi R_n = (0.75) F_u A_{se,N} \tag{4-23}$$

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

$$\frac{R_n}{\Omega} = \frac{F_u A_{se,N}}{2.00} \tag{4-24}$$

ACI 318, Section 17.6.1.2, requires the specified minimum tensile strength of the threaded rod, F_u, used in calculating the nominal tensile capacity not be taken larger than $1.9F_y$ and 125,000 psi. For ASTM F1554 threaded rods, F_u does not exceed these limits and may therefore be used directly in calculating the tensile strength of the threaded rod.

ACI 318, Table 17.5.3(a), requires a reduced resistance factor, ϕ, be used when the anchor rod material does not qualify as a ductile steel element as defined in ACI 318, Section 2.3. Threaded rods conforming to ASTM F1554 satisfy the ductile steel element requirements contained within ACI 318 and do not require the reduced resistance factor associated with brittle steel elements.

Shown in Table 4-1 are the design and allowable strengths for various anchor rods based on the AISC *Specification* and ACI 318.

4.3.2.2 Concrete Tensile Strength

It is presumed that ASCE/SEI 7 (ASCE, 2022) load factors are employed in this Guide. The ϕ factors used herein correspond to those in ACI 318, Section 17.5.3 and Chapter 21.

ACI 318, Chapter 17, addresses the anchoring to concrete of cast-in or post-installed expansion anchors, undercut anchors, adhesive anchors, and screw anchors. The provisions include limit states for concrete pullout, side-face blowout, and breakout strength following the concrete capacity design (CCD) method. Bond strength of adhesive anchors is also covered in Chapter 17.

Concrete Pullout Strength

ACI concrete pullout strength is based on ACI 318, Section 17.6.3.

For cast-in headed anchor rods

$$\phi N_{pn} = \phi\psi_{c,P}\left(8A_{brg}f'_c\right) \tag{4-25}$$

where

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

f'_c = specified compressive strength of concrete, psi

ϕ = 0.70 per ACI 318, Table 17.5.3(c)

$\psi_{c,P}$ = 1.4 if the anchor is located in a region of a concrete member where analysis indicates no cracking at service levels, otherwise $\psi_{c,P} = 1.0$.

Shown in Table 4-2 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 ($\psi_{c,P} = 1.0$). Notice that concrete pullout does not control over the steel strength for ASTM F1554 Grade 36 anchor rods with $f'_c = 5$ ksi for all listed diameters, with $f'_c = 4$ ksi for diameters less than or equal to 2¾ in. or $f'_c = 3$ ksi for diameters less than or equal to ¾ in. For higher strength anchor rods or concrete with a lower compressive strength, washer plates may be necessary to obtain the full strength of the anchors. The size of the washers should be minimized while developing the required strength.

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, as recommended in AISC *Manual* Part 14, hooks, if used, should be limited to "…axially loaded members subject to compression only to locate and prevent displacement or overturning of columns due to erection loads or accidental collisions during erection."

ACI 318, Chapter 17, provides a pullout strength for a hooked anchor of $\phi\psi_{c,P}\left(0.9f'_c e_h d_a\right)$, which is based on an anchor with diameter d_a 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 $4.5d_a$ but not less than $3d_a$. $\psi_{c,P}$ equals 1 if the anchor is located where the concrete is cracked at service load levels and equals 1.4 if it is not cracked at service load levels.

Concrete Breakout Strength

The concrete breakout strength is determined based on the CCD method. In the CCD method, the concrete cone is considered to be formed at an angle of approximately 35° (1 to 1.5 slope). For simplification, the cone is considered to be square rather than round in plan. The concrete breakout stress (f_t in Figure 4-2) 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). When the concrete breakout cone is influenced by an

Table 4-2. Anchor Rod Concrete Pullout Strength (LRFD Only)					
Rod Diameter, in.	Rod Area, A_b, in.^{2}	Heavy Hex Nut Bearing Area, A_{brg}, in.^{2}	Design Concrete Pullout Strength, ϕN_{pn}, kips		
			f'_c = 3,000 psi	f'_c = 4,000 psi	f'_c = 5,000 psi
⅝	0.307	0.671	**11.3**	**15.0**	**18.8**
¾	0.442	0.911	**15.3**	**20.4**	**25.5**
⅞	0.601	1.19	20.0	**26.7**	**33.3**
1	0.785	1.50	25.2	**33.6**	**42.0**
1⅛	0.994	1.85	31.1	**41.4**	**51.8**
1¼	1.23	2.24	37.6	**50.2**	**62.7**
1½	1.77	3.12	52.4	**69.9**	**87.4**
1¾	2.41	4.14	69.6	**92.7**	**116**
2	3.14	5.32	89.4	**119**	**149**
2¼	3.98	6.63	111	**149**	**186**
2½	4.91	8.10	136	**181**	**227**
2¾	5.94	9.70	163	**217**	**272**
3	7.07	11.5	193	258	**322**
3¼	8.30	13.4	225	300	**375**
3½	9.62	15.4	259	345	**431**
3¾	11.0	17.6	296	394	**493**
4	12.6	19.9	334	446	**557**

Note: Bold values above the heavy line indicate the available pullout capacity exceeds the available steel strength of the anchor rod in tension for ASTM F1554 Gr. 36.

edge (see Figure 4-3), the breakout area is reduced. According to ACI 318, Section 17.3, the CCD method is valid for anchors with diameters not exceeding 4 in. and specified concrete strength used for design not exceeding 10,000 psi. Anchors must also satisfy the edge distances, spacings, and thickness indicated in Section 17.9 unless supplementary reinforcement is provided to control splitting failure.

ACI 318, Section 17.6.2, specifies that the nominal concrete breakout strength for a group of cast-in anchors is:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

where

A_{Nc} = projected concrete failure area of a group of anchors, in.^{2}

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

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

$\psi_{c,N}$ = breakout cracking factor based on the influence of cracks in concrete

$\psi_{cp,N}$ = breakout splitting factor to account for splitting tensile stresses

$\psi_{ec,N}$ = breakout factor to account for eccentric tension loading

$\psi_{ed,N}$ = breakout edge effect factor based on proximity to edges of concrete

The basic concrete breakout strength of a single anchor in cracked concrete, N_b, is given in ACI 318, Section 17.6.2.2, as:

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \qquad \text{(ACI 318, Eq. 17.6.2.2.1)}$$

where

f_c' = specified compressive strength of concrete, psi

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

k_c = 24 for cast-in anchors

λ_a = 1.0 for normal-weight concrete

When the anchor is a cast-in headed stud or cast-in headed bolt and the effective embedment of the anchor is between 11 in. and 25 in., inclusive, the value of N_b may be increased up to 14% by utilizing ACI 318, Section 17.6.2.2.3, as:

$$N_b = 16\lambda_a \sqrt{f_c'} h_{ef}^{5/3} \quad \text{(ACI 318, Eq. 17.6.2.2.3)}$$

Side-Face Blowout Strength

ACI 318, Section 17.6.4, provides the side-face blowout strength of headed anchors in tension with deep embedment close to an edge. 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 4-4. As with the concrete breakout stress cones, overlapping of the stress cones associated with these lateral bursting forces is considered in ACI 318, Chapter 17. Use of washer plates can be beneficial by increasing the bearing area, which increases the side-face blowout strength.

ACI 318, Section 17.6.4, stipulates the nominal side-face blowout strength, N_{sb}, of a single headed anchor rod with deep embedment close to an edge $(h_{ef} > 2.5c_{a1})$ as:

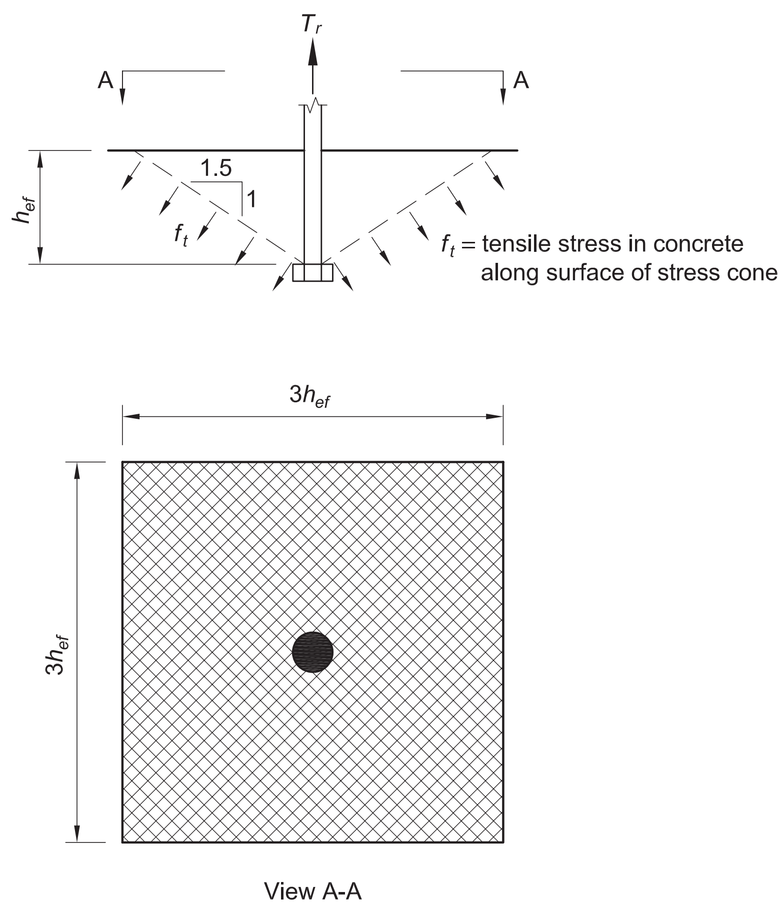


Fig. 4-2. Full breakout cone in tension per ACI 318.

$$N_{sb} = 160 c_{a1} \sqrt{A_{brg}} \lambda_a \sqrt{f'_c} \qquad \text{(ACI 318, Eq. 17.6.4.1)}$$

where

A_{brg} = net bearing area of the head of stud, anchor rod, or headed deformed bar, in.^{2}

c_{a1} = minimum distance from the center of an anchor shaft to the edge of concrete in one direction, in.

f'_c = specified compressive strength of concrete, psi

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

For multiple headed anchor rods with deep embedment close to an edge $(h_{ef} > 2.5c_{a1})$ and anchor spacing less than $6c_{a1}$, the nominal side-face blowout strength, N_{sbg}, of those anchors susceptible to a side-face blowout is:

$$N_{sbg} = \left(1 + \frac{s}{6c_{a1}}\right) N_{sb} \qquad \text{(ACI 318, Eq. 17.6.4.2)}$$

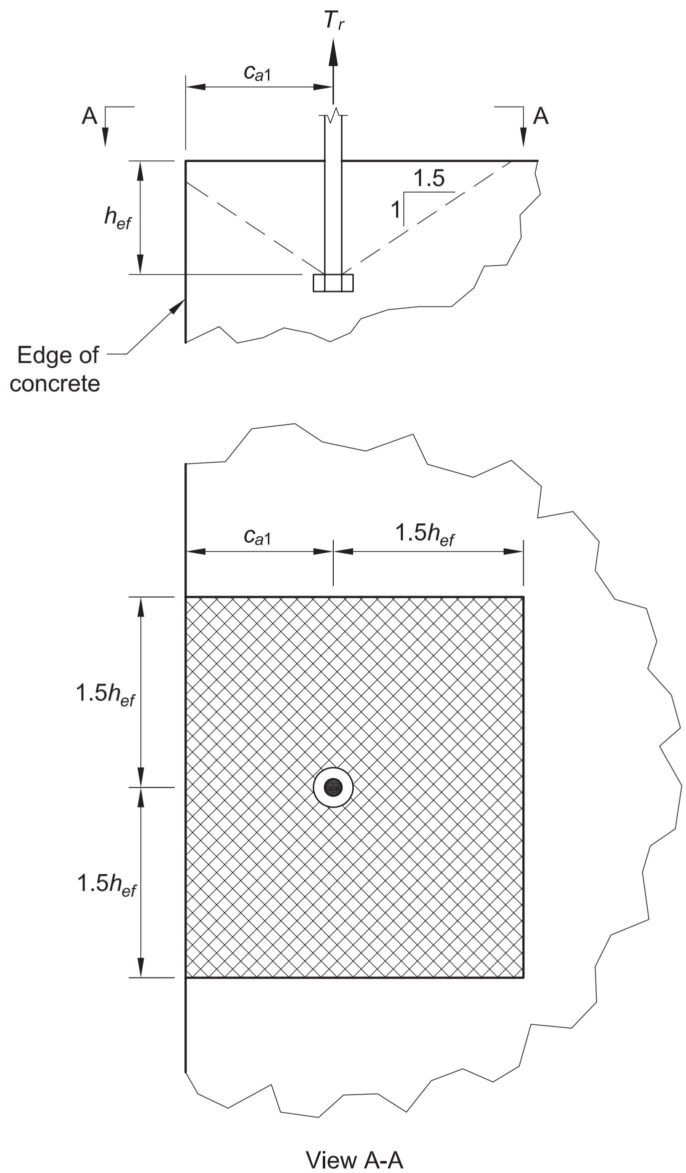


Fig. 4-3. Breakout cone in tension near an edge.

where

s = distance between the outer anchors along the edge, in.

4.3.3 Design for Shear

Overview of Mechanics and Method

For exposed column bases similar to those shown in Figures 1-1(a) and (c), there are three principal ways of transferring shear from the column and/or the gusset plate into the concrete: (1) through shear in the anchor rods, (2) using shear lugs, or (3) through friction when compression is present, as shown in Figure 4-5. The design for shear using the first two approaches is covered within this section. The design for shear using friction is covered in Section 4.3.5. The design for shear for embedded columns is covered in Chapter 5.

Shear in the 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. The design for shear requires a check of the steel strength of the anchor rods and the concrete strength in shear. The concrete limit states are the breakout strength in shear and pryout strength in shear as indicated in ACI 318, Table 17.5.2.

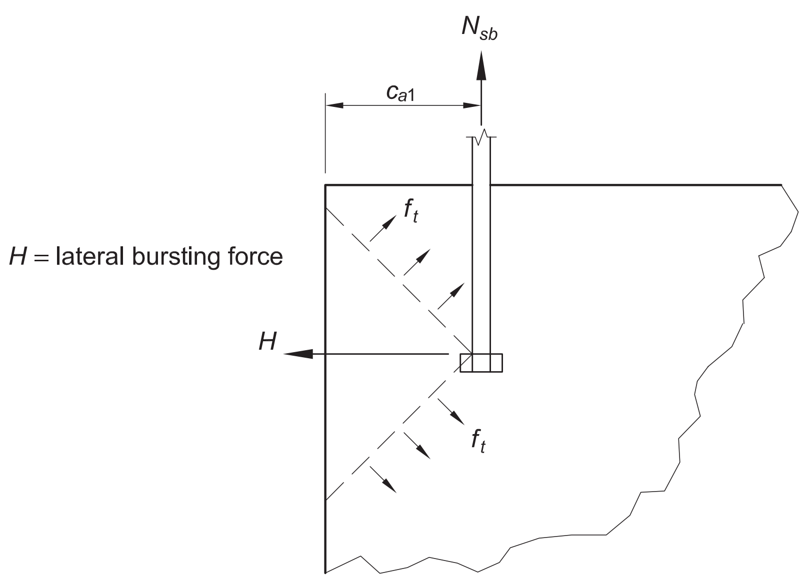


Fig. 4-4. Lateral bursting forces for anchor rods in tension near an edge.

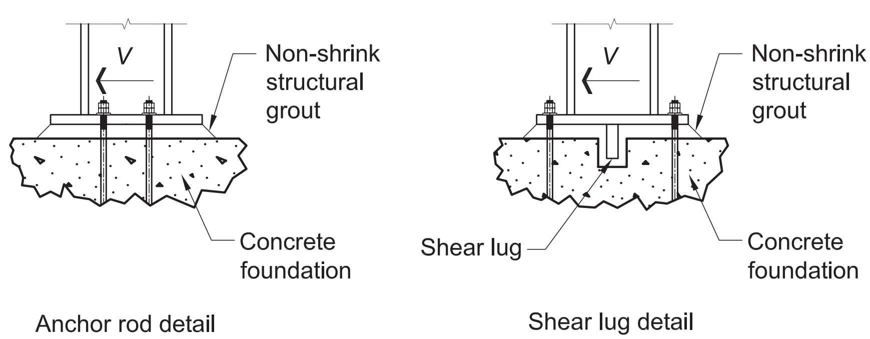


Fig. 4-5. Transfer of base shear.

When using the AISC-recommended hole sizes for anchor rods, which can be found in Table 4-3 (shown later in Section 4.5.3), or alternate oversized holes, 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 the anchor rods will receive the same force. The authors recommend a cautious approach, such as using only the anchor rods closest to the edge in the direction of the force to transfer the shear, unless special provisions are made to equalize the load to all anchor rods (Fisher, 1981).

Shear forces can be transferred equally to all anchor rods or to selective anchor rods. The engineer should consider the load distribution as indicated in ACI 318, Commentary Section R17.7.2.1, and also consider the effect of any oversized holes and the presence of plate washers. The plate washers should be detailed with standard, nonoversized, holes. 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 due 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. 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.

Anchor Rods Steel Strength in Shear

The design shear strength of an anchor rod given by AISC *Specification* Section J3.7 as:

$$\phi R_{nv} = \phi F_{nv} A_b \qquad \text{(from } \textit{Spec.} \text{ Eq. J3-1)}$$

where

A_b = nominal unthreaded body area of bolt or threaded part, in.^{2}

F_{nv} = nominal shear stress in bearing-type connections, ksi

= $0.450F_u$ if the threads are not excluded from the shear plane (from AISC *Specification* Table J3.2)

= $0.563F_u$ if the threads are excluded from the shear plane (from AISC *Specification* Table J3.2)

ϕ = 0.75

The ACI 318 steel design strength of anchor rods in shear is given by:

$$\phi V_{sa} = \phi(0.6) A_{se,v} f_{uta} \qquad \text{(from ACI 318, Eq. 17.7.1.2b)}$$

where

$A_{se,v}$ = effective cross-sectional area of an anchor in shear, in.^{2}

f_{uta} = specified tensile strength of anchor steel, ksi

ϕ = 0.65

Where anchors are used with a built-up grout pad, ACI 318, Section 17.7.1.2.1, requires that the anchor capacity be multiplied by 0.80. 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 the AISC *Specification* combined bending and shear checks are made on the anchor rods.

Anchor Rods Steel Strength in Bending

The bending strength of anchor rods can be determined as follows:

$$\phi M_n = \phi F_{nt} Z \qquad (4\text{-}26)$$

where

F_{nt} = nominal tensile strength of anchor steel according to AISC *Specification* Table J3.2, ksi

Z = plastic section modulus based on nominal diameter of anchor, in.^{3}

ϕ = 0.75

Interaction of shear, tension, and bending in the anchor rod is typically considered. The tension in the anchor rods may arise due to direct tension in the column, due to bending, or a combination of both. In such cases, the following interaction equation (Equation C-J3-4a from the AISC *Specification* Commentary) may be used to evaluate the combined stress limit state:

$$\left(\frac{f_t}{\phi F_{nt}}\right)^2 + \left(\frac{f_v}{\phi F_{nv}}\right)^2 \le 1 \qquad \text{(\textit{Spec}. Eq. C-J3-4a)}$$

In Equation C-J3-4a, F_{nt} and F_{nv} are the ultimate tensile strength and the ultimate shear strength of the anchor rod, whereas f_t and f_v are the applied tensile and shear stresses. These applied stresses may be determined from the loads as follows:

$$f_v = \frac{V_{rod}}{A} \qquad (4\text{-}27)$$

$$f_t = \frac{P_{rod}}{A} + \frac{M_{rod}}{Z} \qquad (4\text{-}28)$$

The terms P_{rod}, V_{rod}, and M_{rod} represent the factored ultimate axial force, shear, and moment, respectively, in the anchor rod being evaluated. As noted previously, the moment may be calculated by assuming reverse curvature bending over the distance between the center of the bearing plate washer and the top of the grout.

Anchor Rods Concrete Breakout Strength in Shear

ACI 318, Section 17.7.2, employs the CCD method to evaluate the concrete breakout strength from shear forces resisted 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 4-6 is evaluated as:

$$\phi V_{cbg} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b, \text{ lbf} \qquad \text{(ACI 318, Eq. 17.7.2.1b)}$$

where

$$V_b = \left[7\left(\frac{\ell_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\sqrt{f'_c}\,(c_{a1})^{1.5} \le 9\sqrt{f'_c}\,(c_{a1})^{1.5} \qquad \text{(ACI 318, Eq. 17.7.2.2.1a and b)}$$

for normal weight concrete, lbf

c_{a1} = the edge distance in the direction of load as illustrated in Figure 4-6, in.

d_a = rod diameter, in.

f'_c = concrete compressive strength, psi

ℓ_e = load-bearing length of the anchor in shear, which is equal to the embedment depth for anchors with a constant stiffness over the full length of embedded section, in.; ℓ_e shall not be taken larger than $8d_a$

ϕ = 0.70 (considering supplementary reinforcement not present)

$\psi_{c,V}$ = 1.4 [when an analysis confirms no cracking at service load levels or when adequate supplementary reinforcement is provided per ACI 318, Table 17.7.2.5.1]

$\psi_{ec,V}$ = 1.0 (for shear applied concentrically with the anchor group)

Typically, $\dfrac{\ell_e}{d_a}$ is equal to 8 because the load bearing length is limited to $8d_a$. In this case, when $d_a > 0.720$ in., then V_b will be governed by the $9\sqrt{f_c'}(c_{a1})^{1.5}$ term.

Substituting,

$$\phi V_{cbg} = 8.82\frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{h,V}\sqrt{f_c'}(c_{a1})^{1.5} \tag{4-29}$$

where

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

A_{Vco} = $4.5{c_{a1}}^2$ (the area of the full shear cone for a single anchor as shown in View A-A of Figure 4-6), in.^{2}

$\psi_{ed,V}$ = a modifier to reflect the capacity reduction when side cover limits the size of the breakout cone calculated per ACI 318, Section 17.7.2.4.

$\psi_{h,V}$ = a modifier to reflect the capacity increase when the concrete member thickness is less than $1.5c_{a1}$ calculated per ACI 318, Section 17.7.2.6.

If the edge distance c_{a1} is large enough, then the anchor rod steel shear strength will govern. 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.

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. Anchor reinforcement as permitted in ACI 318, Section 17.5.2.1(b), can be used structurally to transfer the shear from the anchors to the foundation. See Section 4.4 of this Guide for further discussion on anchor reinforcement.

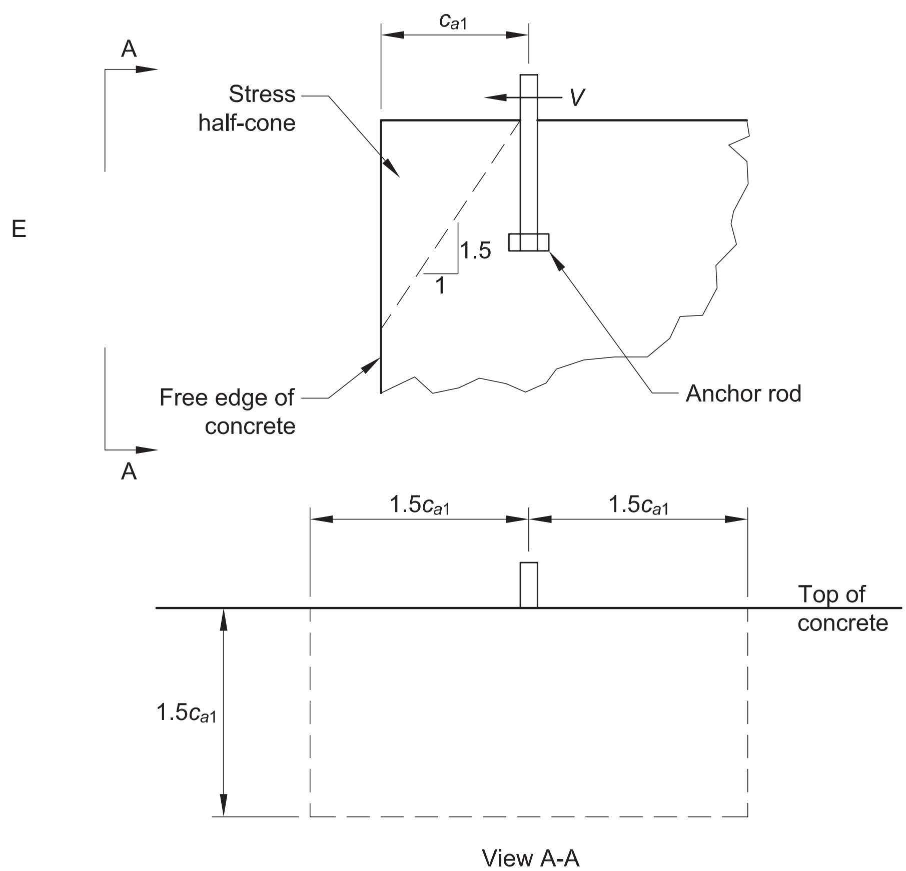


Fig. 4-6. Concrete breakout cone for shear.

Anchor Rods Concrete Pryout Strength in Shear

In addition to the concrete breakout strength, ACI 318, Section 17.7.3, also contains provisions for a limit state called pryout strength. ACI 318 defines the pryout strength of a single anchor in shear as:

$$\phi V_{cp} = \phi k_{cp} N_{cp} \qquad \text{(ACI 318, Equation 17.7.3.1a)}$$

where

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

h_{ef} = effective embedment length, in.

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

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

ϕ = 0.70

When the concrete is subjected to a combination of tension and shear, ACI 318, Chapter 17, uses an interaction equation solution. This will be covered in Section 4.3.4 of this Design Guide.

4.3.4 Design for Combined Axial Tension and Shear

Overview of Mechanics and Method

Exposed column bases subjected to combined tension and shear must be designed in accordance with the approaches outlined in Section 4.3.2 (Design for Axial Tension) and Section 4.3.3 (Design for Shear). In addition to all the limit states considered in those sections, the combination of tension and shear affects the design methodology as follows:

1. The column-to-base plate welds need to be designed for the combined normal tensile and shear forces.
2. The anchor rods above the concrete and grout pad must be checked for combined shear and tensile loads following the requirement of AISC *Specification* Section J3.8. Anchor rod bending is to be considered as discussed in Section 4.3.3 of this Guide.
3. The anchor rod anchorage into the concrete must be designed using the tensile and shear interaction equation requirements as outlined in ACI 318, Section 17.8. As discussed previously, bending of the anchor rods within the grout pad does not need to be considered when using the ACI approach, but rather the nominal steel shear strength of the anchor is multiplied by a 0.80 factor per ACI 318, Section 17.7.1.2.1.

Because the AISC *Specification* and ACI 318 handle the interaction between tension and shear differently, the authors do not recommend combining approaches but, rather, checking the anchor rods using both documents separately as outlined in items 2 and 3. AISC considers the strength of the steel anchor rods and ACI considers both the steel and concrete limit states.

Two examples are provided in Section 4.7. Example 4.7-6 illustrates the design of an exposed base connection subjected to combined tension and shear. Only the AISC limit states are considered in this example. Example 4.7-7 illustrates a base connection with a tension-only brace that produces a case of combined tension and shear at the base. This example focuses on the ACI approach for shear and tension interaction.

4.3.5 Design for Combined Axial Compression and Shear

Overview of Mechanics and Method

Exposed column bases subjected to combined compression and shear must be designed in accordance with the approaches outlined in Section 4.3.1 (Design for Axial Compression) and Section 4.3.3 (Design for Shear). In addition to all the limit states considered in those sections, the combination of compression and shear affects the design approaches as follows:

1. The column to base plate welds are to be designed for both compression (when a smooth and notch-free contact bearing surface is not sufficient per AISC *Specification* Section M4.4) and shear loads.
2. The base plate thickness must be designed for the moment due to the compression stress on the concrete/grout from the compression load and the moment due to the eccentric shear in the case of shear lugs discussed in Section 4.3.3.

3. The compression load generates friction between the base plate and the grout or concrete surface that can be used to transfer shear into the concrete. This compression is considered a clamping force that generates a shear resistance in the perpendicular direction. The friction force can be used to resist the entire shear load or contribute to resist a portion of the shear load, while the balance of the load can be resisted by the anchor rods in shear or shear lugs.

In typical base connection 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 due to friction can be calculated in accordance with the following, based on ACI 318, Section 22.9, and ACI 349-13, Appendix D, Section D.6.1.4, criteria (ACI, 2013),

$$\phi V_n = \phi_{friction}(\mu P_u) \le \min\left[\phi 0.2 f_c' A_c,\ \phi(800\text{ psi})A_c\right] \tag{4-30}$$

For friction between steel base plates and concrete, a μ value of 0.4 is given in ACI 349-13, Appendix D. ACI 349-13, Section D.6.1.4, permits the nominal shear strength due to friction to be added to the nominal steel shear strength of the anchor in shear. As such, the resistance factor for friction is taken as the resistance factor for shear in an anchor, $\phi_{friction} = 0.65$. As an upper limit on the design shear strength, ACI 318, Section 22.9.4.4, 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 and A_c is given in ACI 318 as the area of concrete section resisting shear transfer. Only LRFD requirements are addressed in the ACI documents.

It is noteworthy that in accordance with ASCE/SEI 7, Chapters 13 and 15, for some seismic applications, friction cannot be used to transfer shear loading. Also, many specifications in delegated design applications do not allow the use of friction to transfer shear loads. The use of friction to transfer shear loads can only be used for some seismic applications and when the project specifications allow it. And as indicated previously, even when relying on friction to transfer shear loading, columns must be anchored to the foundations with a minimum of four anchor rods per OSHA requirements.

4.3.6 Design for Bending

Overview of Mechanics and Method

It is unlikely in practice to have a loading case of pure bending at the base of a column. However, this section is provided herein to illustrate the methodology that can be used for situations of combined bending with shear. The design approach for bending in exposed base connections follows the methodology used by Drake and Elkin (1999), which is discussed in detail in Section 4.3.7 (Design of Combined Axial Compression and Bending). The reader is therefore referred to Section 4.3.7 for further discussion of the method and Figure 4-8 for the definition of variables.

For the case of pure bending, the small moment case in the Drake and Elkin approach is not applicable; only the large moment derivation is valid. For base connections with large moments, the Drake and Elkin approach is thus modified as described by Doyle and Fisher (2005) by removing the applied axial load and setting the applied moments as $P_r e = M_r$ and $2P_r f = 0$. The basic equations then become:

$$T = C = q_{max} Y \tag{4-31}$$

where

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2M_r}{q_{max}}} \tag{4-32}$$

Similar to cases of combined compression and bending, a real nonzero solution will only exist if:

$$\left(f + \frac{N}{2}\right)^2 > \frac{2M_r}{q_{max}} \tag{4-33}$$

Once T and C are determined, the large moment procedures in Section 4.3.7 may be utilized to calculate the required plate thickness and confirm the anchorage capacity.

Elements of the design procedure are as follows:

1. Design the column-to-base plate welds.
2. Pick a trial base plate size, $N \times B$.
3. Check the inequality in Equation 4-33. If it is not satisfied, choose larger plate dimensions.
4. Determine the equivalent bearing length, Y, and total tensile force in the anchor rods, 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 largest value.
6. Determine the anchor rod size.
7. Design the anchorage of the anchor rods into the concrete.

Example 4.7-9 illustrates a base connection design subjected to only a concentrated moment.

4.3.7 Design for Combined Axial Compression and Bending

Design of Column Base Plates with Low 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 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 (low moment), the axial force is resisted by bearing only with no uplift. For large eccentricities (large moment), 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 this section. The variables T_u, P_u, and M_u have been changed from the original work by Drake and Elkin to T_r, 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 4-7. The resultant bearing force is defined by the product qY, in which:

$$q = f_p B \tag{4-34}$$

where

B = the base plate width [see Figure 4-1(b)], in.

f_p = bearing stress between the plate and concrete or grout, ksi

The force acts at the midpoint of the 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:

$$\varepsilon = \frac{N}{2} - \frac{Y}{2} \tag{4-35}$$

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

$$Y_{min} = \frac{P_r}{q_{max}} \tag{4-36}$$

where

$$q_{max} = f_{p(max)} B \tag{4-37}$$

The expression for the location of the resultant bearing force given in Equation 4-35 shows that ε reaches its maximum value when Y is minimum. Therefore:

$$\varepsilon_{max} = \frac{N}{2} - \frac{Y_{min}}{2} \qquad (4\text{-}38)$$
$$= \frac{N}{2} - \frac{P_r}{2q_{max}}$$

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

$$e = \frac{M_r}{P_r} \qquad (4\text{-}39)$$

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:

$$e_{crit} = \varepsilon_{max} \qquad (4\text{-}40)$$
$$= \frac{N}{2} - \frac{P_r}{2q_{max}}$$

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 moments is outlined in this section.

Concrete Bearing Stress (for low moment case)

The concrete bearing stress is assumed to be uniformly distributed over the area $Y \times B$. Equation 4-35, for the case of $e = \varepsilon$ provides an expression for the length of bearing area, Y:

$$\frac{N}{2} - \frac{Y}{2} = e \qquad (4\text{-}41)$$

Therefore:

$$Y = N - 2e \qquad (4\text{-}42)$$

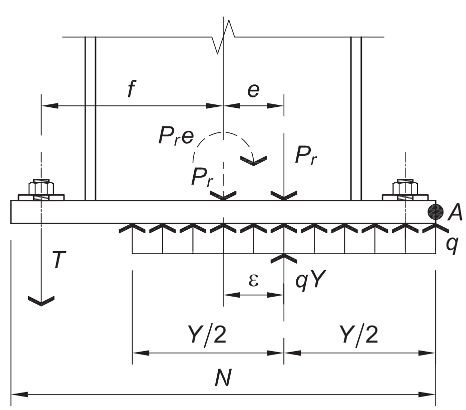


Fig. 4-7. Base plate with low moment.

The bearing stress can be determined as:

$$q = \frac{P_r}{Y} \tag{4-43}$$

From which:

$$f_p = \frac{P_r}{BY} \tag{4-44}$$

for the low moment case, $e \le e_{crit}$. Therefore, as noted previously, $q \le q_{max}$. From Equations 4-34 and 4-37, it follows that $f_p \le f_{p(max)}$.

For the condition $e = e_{crit}$, the bearing length, Y, obtained by use of Equations 4-40 and 4-42 is:

$$\begin{aligned} Y &= N - 2\left(\frac{N}{2} - \frac{P_r}{2q_{max}}\right) \\ &= \frac{P_r}{q_{max}} \end{aligned} \tag{4-45}$$

Base Plate Flexural Yielding Limit at Bearing Interface (for low moment case)

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 4-1(b)]. For the strong-axis bending, the bearing stress, f_p (ksi), is calculated as:

$$\begin{aligned} f_p &= \frac{P_r}{BY} \\ &= \frac{P_r}{B(N - 2e)} \end{aligned} \tag{4-46}$$

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

For $Y \ge m$:

$$M_{pl} = f_p\left(\frac{m^2}{2}\right) \tag{4-47}$$

For $Y < m$:

$$M_{pl} = f_p Y\left(m - \frac{Y}{2}\right) \tag{4-48}$$

where

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

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

$$R_n = \frac{F_y t_p^2}{4} \tag{4-49}$$

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:

LRFD	ASD
$$\phi_b M_n = \phi_b F_y \frac{t_p^2}{4} \quad \text{(4-50a)}$$ where ϕ_b = resistance factor in bending = 0.90	$$\frac{M_n}{\Omega_b} = \frac{F_y}{\Omega_b}\frac{t_p^2}{4} \quad \text{(4-50b)}$$ where Ω_b = safety factor in bending = 1.67

To determine the plate thickness, equate the right-hand sides of Equation 4-47 or 4-48 and Equation 4-50 and solve for $t_{p(req)}$.

For $Y \geq m$:

LRFD	ASD
$$t_{p(req)} = \sqrt{\frac{4\left[f_p\left(\frac{m^2}{2}\right)\right]}{0.90F_y}} = 1.49m\sqrt{\frac{f_p}{F_y}} \quad \text{(4-51a)}$$	$$t_{p(req)} = \sqrt{\frac{4\left[f_p\left(\frac{m^2}{2}\right)\right]}{F_y/1.67}} = 1.83m\sqrt{\frac{f_p}{F_y}} \quad \text{(4-51b)}$$

For $Y < m$:

LRFD	ASD
$$t_{p(req)} = \sqrt{\frac{4\left[f_pY\left(m-\frac{Y}{2}\right)\right]}{0.90F_y}} = 2.11\sqrt{\frac{f_pY\left(m-\frac{Y}{2}\right)}{F_y}} \quad \text{(4-52a)}$$	$$t_{p(req)} = \sqrt{\frac{4\left[f_pY\left(m-\frac{Y}{2}\right)\right]}{F_y/1.67}} = 2.58\sqrt{\frac{f_pY\left(m-\frac{Y}{2}\right)}{F_y}} \quad \text{(4-52b)}$$

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 4-51a and 4-51b for both $Y \geq m$ and $Y < m$. 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.

Base Plate Flexural Yielding at Tension Interface (for low moment case)

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.

General Design Procedure (for low moment case)

1. Determine the axial load and moment.
2. Design base plate-to-column welds.
3. Pick a trial base plate size, $N \times B$.
4. Determine the equivalent eccentricity:

$$e = \frac{M_r}{P_r} \tag{4-39}$$

and the critical eccentricity:

$$e_{crit} = \frac{N}{2} - \frac{P_r}{2q_{max}} \tag{4-40}$$

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

5. Determine the bearing length, Y.
6. Determine the required minimum base plate thickness, $t_{p(req)}$.
7. Determine the anchor rod size.
8. Design anchorage into concrete.

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 loads and is schematically presented in Figure 4-8.

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

$$\begin{aligned} e &> e_{crit} \\ &> \frac{N}{2} - \frac{P_r}{2q_{max}} \end{aligned} \tag{4-53}$$

Concrete Bearing and Anchor Rod Forces (for large moment case)

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 4-8.

Vertical force equilibrium requires that:

$$\sum F_{vertical} = 0 \tag{4-54}$$

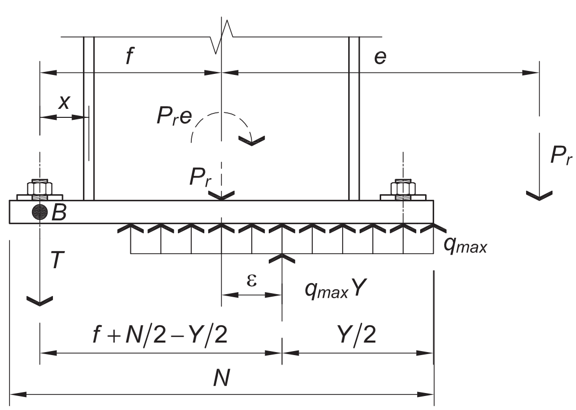


Fig. 4-8. Base plate with large moment.

and

$$T = q_{max} Y - P_r \tag{4-55}$$

where T equals the summation of the required strengths of all anchor rods.

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

$$q_{max} Y \left(f + \frac{N}{2} - \frac{Y}{2} \right) - P_r (e + f) = 0 \tag{4-56}$$

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

$$Y^2 - 2\left(f + \frac{N}{2} \right) Y + \frac{2P_r (e+f)}{q_{max}} = 0 \tag{4-57}$$

and the solution for Y is:

$$Y = \left(f + \frac{N}{2} \right) \pm \sqrt{\left(f + \frac{N}{2} \right)^2 - \frac{2P_r (e+f)}{q_{max}}} \tag{4-58}$$

The concrete bearing force is given by the product $q_{max}Y$. The anchor rod tensile force, T, is obtained by solving Equation 4-55.

For certain force, moment, and geometry combinations, a real solution of Equation 4-58 is not possible. In that case, an increase in plate dimensions is required. In particular, only if the following holds:

$$\left(f + \frac{N}{2} \right)^2 \geq \frac{2P_r (e+f)}{q_{max}} \tag{4-59}$$

will the quantity under the radical in Equation 4-58 be positive or zero and provide a real solution. If the expression in Equation 4-59 is not satisfied, a larger plate is required.

Base Plate Yielding Limit at Bearing Interface (for large moment case)

For the case of large moments, the bearing stress is at its limiting value—that is, $f_p = f_{p(max)}$. The required plate thickness may be determined from either Equations 4-51a and 4-51b or 4-52a and 4-52b.

If $Y \geq m$:

LRFD	ASD
$t_{p(req)} = 1.49m\sqrt{\frac{f_{p(max)}}{F_y}}$ (from 4-51a)	$t_{p(req)} = 1.83m\sqrt{\frac{f_{p(max)}}{F_y}}$ (from 4-51b)

If $Y < m$:

LRFD	ASD
$t_{p(req)} = 2.11\sqrt{\frac{f_{p(max)}Y\left(m - \frac{Y}{2}\right)}{F_y}}$ (from 4-52a)	$t_{p(req)} = 2.58\sqrt{\frac{f_{p(max)}Y\left(m - \frac{Y}{2}\right)}{F_y}}$ (from 4-52b)

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 4-51a and 4-51b, for both $Y \geq m$ and $Y < m$.

Base Plate Yielding Limit at Tension Interface (for large moment case)

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 4-1. For a unit width of base plate, the required bending strength of the base plate can be determined as:

LRFD	ASD
$M_{pl} = \dfrac{T_u x}{B}$ (4-60a)	$M_{pl} = \dfrac{T_a x}{B}$ (4-60b)

where

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{4-61}$$

d = depth of wide-flange column section (see Figure 4-1), in.

t_f = column flange thickness, in.

The available flexural strength per unit length for the plate is given in Equation 4-50. Setting that strength equal to the applied moment given by Equations 4-60 provides an expression for the required plate thickness:

LRFD	ASD
$t_{p(req)} = \sqrt{\dfrac{4T_u x}{B(0.90F_y)}}$ $= 2.11\sqrt{\dfrac{T_u x}{BF_y}}$ (4-62a)	$t_{p(req)} = \sqrt{\dfrac{4T_a x}{B(F_y/1.67)}}$ $= 2.58\sqrt{\dfrac{T_a x}{BF_y}}$ (4-62b)

General Design Procedure (for large moment case)

1. Determine the axial load and moment.
2. Design base plate-to-column weld.
3. Pick a trial base plate size, $N \times B$.
4. Determine the equivalent eccentricity:

$$e = \frac{M_r}{P_r} \tag{4-39}$$

and the critical eccentricity:

$$e_{crit} = \frac{N}{2} - \frac{P_r}{2q_{max}} \tag{4-40}$$

If $e > e_{crit}$, go to the next step (design of the base plate with large moment); otherwise, refer to the design of the base plate with small moment described in this section. Check the inequality of Equation 4-59. If it is not satisfied, choose larger plate dimensions.

5. Determine the equivalent bearing length, Y, and tensile force in the anchor rod, T_u (LRFD) and T_a (ASD).
6. Determine the required minimum base plate thickness, $t_{p(req)}$, at the bearing and tension interfaces. Choose the larger value.
7. Determine the anchor rod size.
8. Design anchorage to concrete.

4.3.8 Design for Combined Axial Tension and Bending

Overview of Mechanics and Method

Base connections subject to combined axial tension and bending may be designed using derived equations that satisfy static equilibrium. In the case of a large moment where the eccentricity of the applied tension falls outside of the bounds of the anchor group $(e = M_r/P_r > f)$, compression is necessary for equilibrium. In this case, an approach analogous to the combined axial compression and bending methodology outlined in Section 4.3.7 may be derived using the model shown in Figure 4-9 and the following equations of equilibrium:

$$P_r - T_r + q_{max}Y = 0 \tag{4-63}$$

Summation of moments about point B yields the following quadratic equation in Y:

$$Y^2 - 2\left(f + \frac{N}{2}\right)Y + \frac{2P_r(e-f)}{q_{max}} = 0 \tag{4-64}$$

where

P_r = required axial tension of base connection, kips

T_r = total required axial tension resisted by anchor rods, kips

Y = length of bearing compression force between base plate and concrete, in.

e = eccentricity between center of column and resultant required axial tension of base connection, in.

f = distance from center of column to anchors resisting tension, in.

q_{max} = maximum uniform bearing compression at concrete, kips/in.

Solving Equation 4-64 for Y, the length of bearing compression force between the base plate and concrete is given by:

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e-f)}{q_{max}}} \tag{4-65}$$

The concrete bearing force is given by the product $q_{max}Y$. The anchor rod tensile force, T_r, is obtained by solving Equation 4-63.

For certain force, moment, and geometry combinations, a real solution for Equation 4-65 is not possible. In that case, an increase in plate dimensions is required. In particular, only if the following holds:

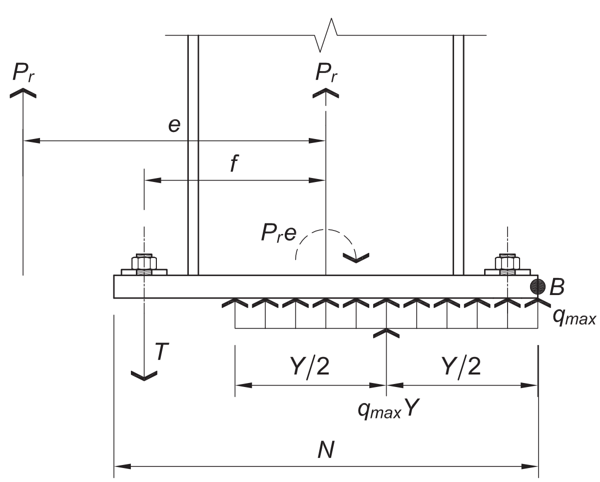


Fig. 4-9. Tension force falling outside of anchor rod group extents.

$$\left(f+\frac{N}{2}\right)^2 \geq \frac{2P_r(e-f)}{q_{max}} \tag{4-66}$$

will the quantity under the radical in Equation 4-65 be positive or zero and provide a real solution. If the expression in Equation 4-66 is not satisfied, a larger plate is required. The case where a real solution is not available occurs when Y exceeds the distance available between the compression edge of the plate and the anchor rod location $(f+N/2)$.

Once the anchor rod tension and concrete bearing force are determined, base plate yielding at the anchor rods and at the concrete compression bearing interface may be checked using the procedures for high moment baseplates in Section 4.3.7. In addition, column-to-base plate welding and anchorage into concrete design can be accomplished following previous sections in this chapter.

In the case of a low moment where the eccentricity of the applied tension falls within of the bounds of the anchor group ($e \leq f$), compression is not necessary for equilibrium and the tension and moment may be resolved in the anchor group, thus producing varying levels of tension among the anchor group anchors. The tension in each anchor may be determined by:

$$r_{r,i} = \frac{P_r}{n} + \frac{(P_r e)\,y_i}{I_x} \tag{4-67}$$

where

I_x = moment of inertia of the bolt group about its centroid, in.^{4}/in.^{2}

$= \sum_{i=1}^{n}(y_i)^2$

P_r = required axial tension of base connection, kips

e = distance perpendicular to the axis of bending between center of applied tension and centroid of the anchor group, in.

n = number of anchors resisting tension

$r_{r,I}$ = required tension for anchor i, kips

y_i = distance perpendicular to the axis of bending between the centroid of the anchor group and anchor i, in.

Once the anchor rod tension is determined, base plate yielding at the anchor rods may be checked using the procedures for large moment base plates in Section 4.3.7. In addition, column-to-base plate welding and anchorage into concrete design can be accomplished similar to previous sections in this chapter.

General Design Procedure

1. Determine the axial load and moment.
2. Design base plate-to-column weld.
3. Pick a trial base plate size, $N \times B$.
4. Determine the equivalent eccentricity, $e = M_r/P_r$.
5. If $e > f$, compression will be necessary for equilibrium. In this case, use Equations 4-63 through 4-66 to determine the tension and compression forces as discussed previously.
6. If $e \leq f$, compression will not be necessary for equilibrium. In this case, use Equation 4-67 to determine the tensile forces in the anchor rods as discussed previously.
7. Determine the required minimum base plate thickness, $t_{p(req)}$, at the bearing and tension interfaces, as applicable. Choose the larger value.
8. Determine the anchor rod size.
9. Design anchorage to concrete.

4.3.9 Design for Combined Axial Compression, Bending, and Shear

The design of base connections for combined axial compression, bending, and shear follows from the previous sections and are not repeated here except to note cases of interactions among the combined load effects.

Anchor rods in these cases may be subject to combined tension and shear. Shear in the anchor rods may also contribute to bending over a height of the anchor rod such as when welded washer plates are used with oversized holes. When shear lugs are utilized, the eccentric location of the concrete bearing against the shear lug will also increase the amount of tension in the anchor rods. In cases where the full base plate is not in compression bearing against the concrete (large moment), a reduced area, A_c, will be available when it is desired to utilize friction to resist shear. Combined tension and shear in concrete anchorage are interacted according to ACI 318-19(22), Section 17.8.

4.3.10 Design for Combined Axial Tension, Bending, and Shear

The design of base connections for combined axial tension, bending, and shear follows from the previous sections and is not repeated here except to note cases of interactions among the combined load effects.

Anchor rods in these cases may be subject to combined tension and shear. Shear in the anchor rods may also contribute to bending over a height of the anchor rod such as when welded washer plates are used with oversized holes. When shear lugs are utilized, the eccentric location of the concrete bearing against the shear lug will also increase the amount of tension in the anchor rods. In cases where the full base plate is not in compression bearing against the concrete, a reduced area, A_c, will be available when it is desired to utilize friction to resist shear. Combined tension and shear in concrete anchorages are interacted according to ACI 318, Section 17.8.

4.3.11 Design for Combined Axial Compression and Biaxial Bending

When exposed column base plates are subjected to axial compression and biaxial bending, the approaches provided in previous sections (for axial compression and uniaxial bending) are inapplicable directly because they utilize the equilibrium equations for vertical force and moment to determine the two unknowns—that is, the anchor forces and the bearing width (for the high-moment condition). Under biaxial bending, the base plate is rotated in a manner that multiple anchor rods may be engaged, with different forces (see Figure 4-10). In such cases, two issues arise: (1) the number of unknowns, corresponding to the different anchor rod forces and the bearing width, may exceed the number of equations—that is, three (moment in each direction and vertical force)—that are available, and (2) estimating the orientation of the axis of rotation is not trivial and adds another unknown to the

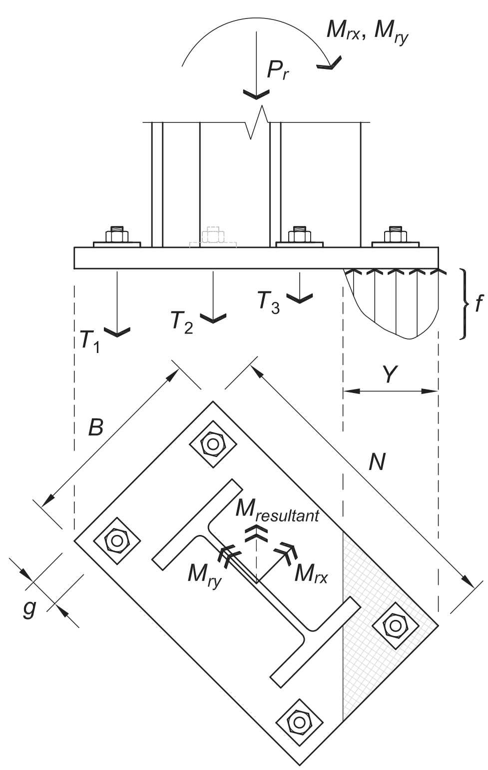


Fig. 4-10. Base plate subjected to biaxial bending resulting in static indeterminacy.

problem. Resolving this requires introduction of additional compatibility equations (Hassan et al., 2021) and a solution process that requires an iterative computer solution and is not amenable to hand calculation. An alternative way to estimate the resistance of exposed base plate connections under axial compression and biaxial bending involves the estimation of moment strength in each direction (i.e., strong- and weak-axis bending) under a given axial compressive force and then using an empirical interaction equation based on these moments to determine whether the connection is able to resist the applied loading. Variants of this approach have been proposed by Fasaee et al. (2018) and Da Silva Seco (2019). Experimental data by these researchers along with data by Choi and Ohi (2005) indicate that such an approach is acceptable.

Specifically, if the applied axial compression is P_r and the applied strong- and weak-axis moments are M_{rx} and M_{ry}, then an interaction equation may be defined as follows:

$$\left(\frac{M_{rx}}{M_{cx,P_r}}\right)^2 + \left(\frac{M_{ry}}{M_{cy,P_r}}\right)^2 = 1 \tag{4-68}$$

In Equation 4-68, the terms $M_{cx,Pr}$ and $M_{cy,Pr}$ represent the moment strengths (including the appropriate ϕ factors for LRFD) in each direction, considering all modes of failure, given the applied axial compression P_r. The terms $M_{cx,Pr}$ and $M_{cy,Pr}$ may be determined using Sections 4.3.6 and 4.3.7. An acceptable design is obtained when:

$$\left(\frac{M_{rx}}{M_{cx,P_r}}\right)^2 + \left(\frac{M_{ry}}{M_{cy,P_r}}\right)^2 \le 1 \tag{4-69}$$

The design process differs from that of uniaxial bending because individual components (e.g., the anchor or the base plate) are not directly sized for induced tensile forces of bending moments, but the entire connection is checked using the interaction Equation 4-69. The individual terms in the interaction equation are in turn based on estimates of internal anchor forces or base plate moments. As a result, the connection must be designed using a trial and error approach that accounts for this interaction; this is illustrated in the Example 4.7-14. It is noted that the design check using Equation 4-69 is acceptable when (1) no tension is present in the connection and (2) shear is transferred independently through a shear lug or friction, and not the anchors.

4.4 ANCHORAGE DESIGN FOR CONCRETE LIMIT STATES

4.4.1 Approaches for Using Reinforcement to Strengthen Concrete Limit States

The concrete breakout strength of anchors is a function of the embedment depth, the thickness of the concrete, the spacing between adjacent anchors, and the location of adjacent free edges of the concrete member, among other variables. In many situations, increasing the anchor embedment does not result in a significant increase in the breakout strength due to geometric limitations of the breakout cone. The concrete breakout strength equations provided in ACI 318, Chapter 17, were developed based on the concrete capacity design (CCD) method considering unreinforced concrete.

For situations where it is not possible to increase the concrete breakout strength by increasing the anchor embedment to achieve the required design strength or develop the anchor full strength, anchor reinforcement can be used instead of concrete breakout strength for both tension and shear loading per ACI 318, Section 17.5.2.1. For tension, the anchor reinforcement must be developed on both sides of the concrete breakout surface; see Figure 4-11. For shear, the anchor reinforcement must be developed on both sides of the concrete breakout surface or specified such that it encloses and contacts the anchor and is developed beyond the breakout surface; see Figure 4-12. In cases where anchor reinforcement is provided that exceeds the amount required to resist the required strength, ACI 318, Section 25.4.10, permits a reduction in the required development length in limited situations. The reduction in required development length is not permitted for hooked, headed, and mechanically anchored deformed reinforcement nor in seismic force-resisting systems in Seismic Design Categories C–F. Additionally, ACI 318, Chapter 25, sets minimum development length limits that apply even when excess reinforcement is provided. Recommended detailing practices of anchor reinforcement are provided in the ACI 318, Commentary Section R17.5.2.1.

The strength reduction factor for anchor reinforcement design is $\phi = 0.75$ per ACI 318, Sections 17.5.2.1.1 and 17.5.3. The anchor reinforcement development length is determined based on ACI 318, Chapter 25.

In general, when piers are used, concrete breakout capacity alone cannot transfer the significant level of tensile force from the steel column to the concrete base. Therefore, steel anchor reinforcement in the concrete can be used to transfer the force from the anchor rods into the concrete. The anchor reinforcement is in addition to the reinforcement required to accommodate the bending forces in the pier.

It is important to make the distinction between anchor reinforcement and supplementary reinforcement. As discussed, anchor reinforcement is an alternate approach to using the concrete breakout strength equations in ACI 318 and is designed to resist the required strength of the base connection. However, supplementary reinforcement is provided to restrain the breakout cones and not specifically designed to resist any loads. When supplementary reinforcement is provided, the strength reduction ϕ factor for breakout and side-face blowout strength are increased from 0.70 to 0.75 per ACI 318, Table 17.5.3(b).

The use of anchor reinforcement in practice has extended beyond its intended use as an alternate to concrete breakout strength. When side-face blowout strength, as determined by ACI 318 equations, is lower than the required strength, anchor reinforcement can also be used to resist the bursting forces of the breakout cone at the base of the anchor; see Figure 4-13.

Hairpins are sometimes used to transfer loads 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. 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 by properly designing the anchor rods for bending, by encasing the column in the concrete slab as shown in Figure 4-14, or by providing shear lugs. Because 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 ensure no interruption in load transfer, yet still allowing the slab to move. In addition, a vapor barrier should not be used under the slab when friction is relied upon to transfer shear to the soil.

In pre-engineered metal buildings, 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 because 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.

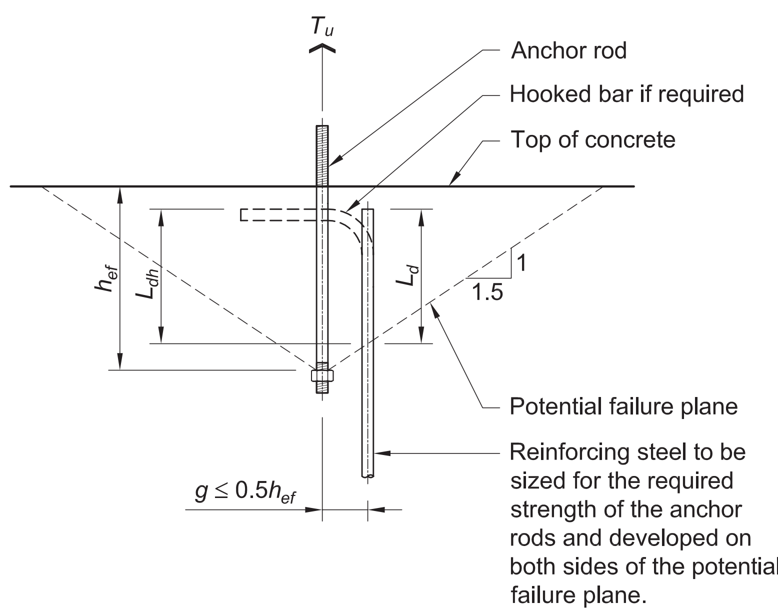


Fig. 4-11. The use of steel reinforcement for restraining tension concrete breakout.

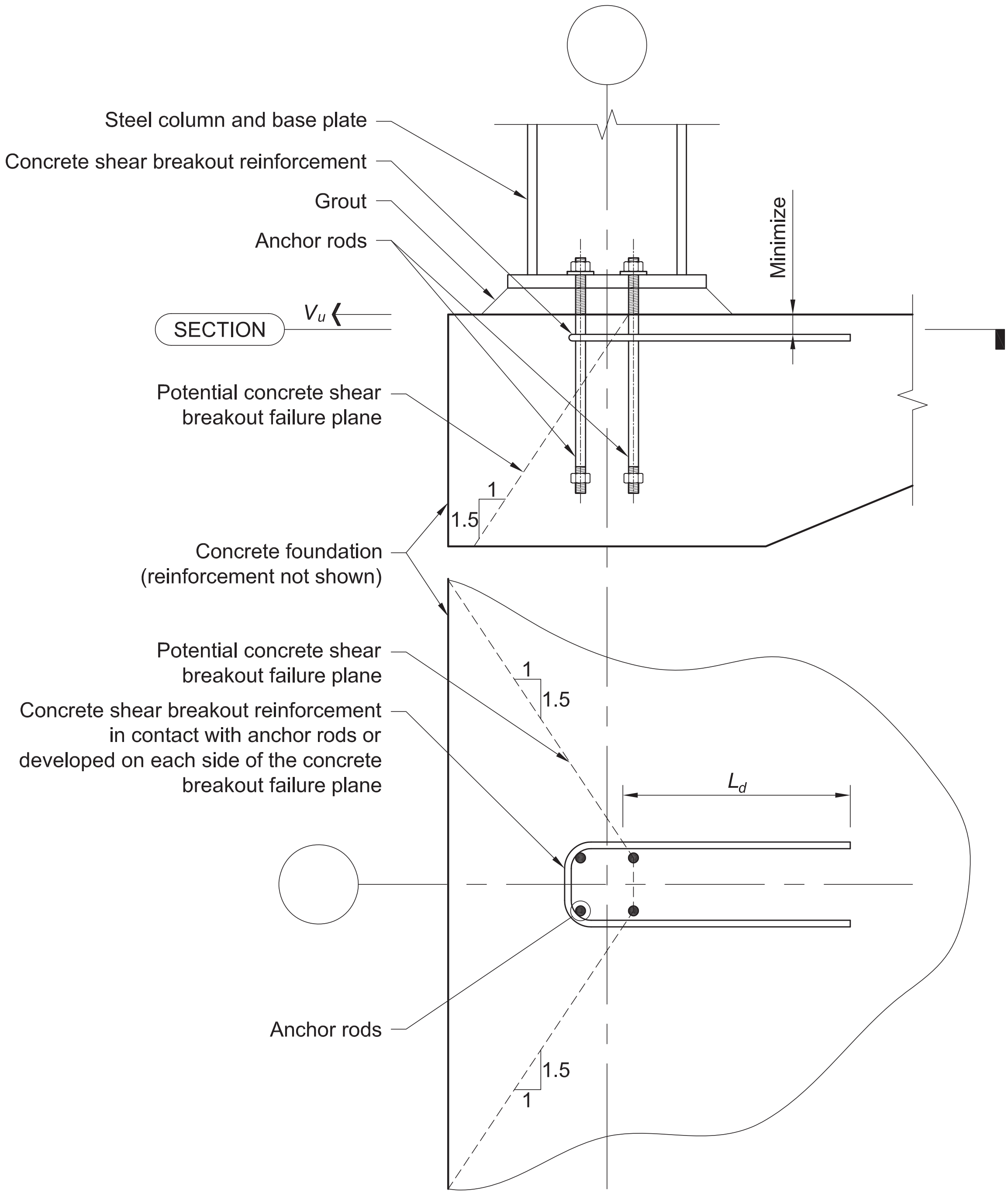


Fig. 4-12. The use of steel reinforcement for restraining shear concrete breakout.

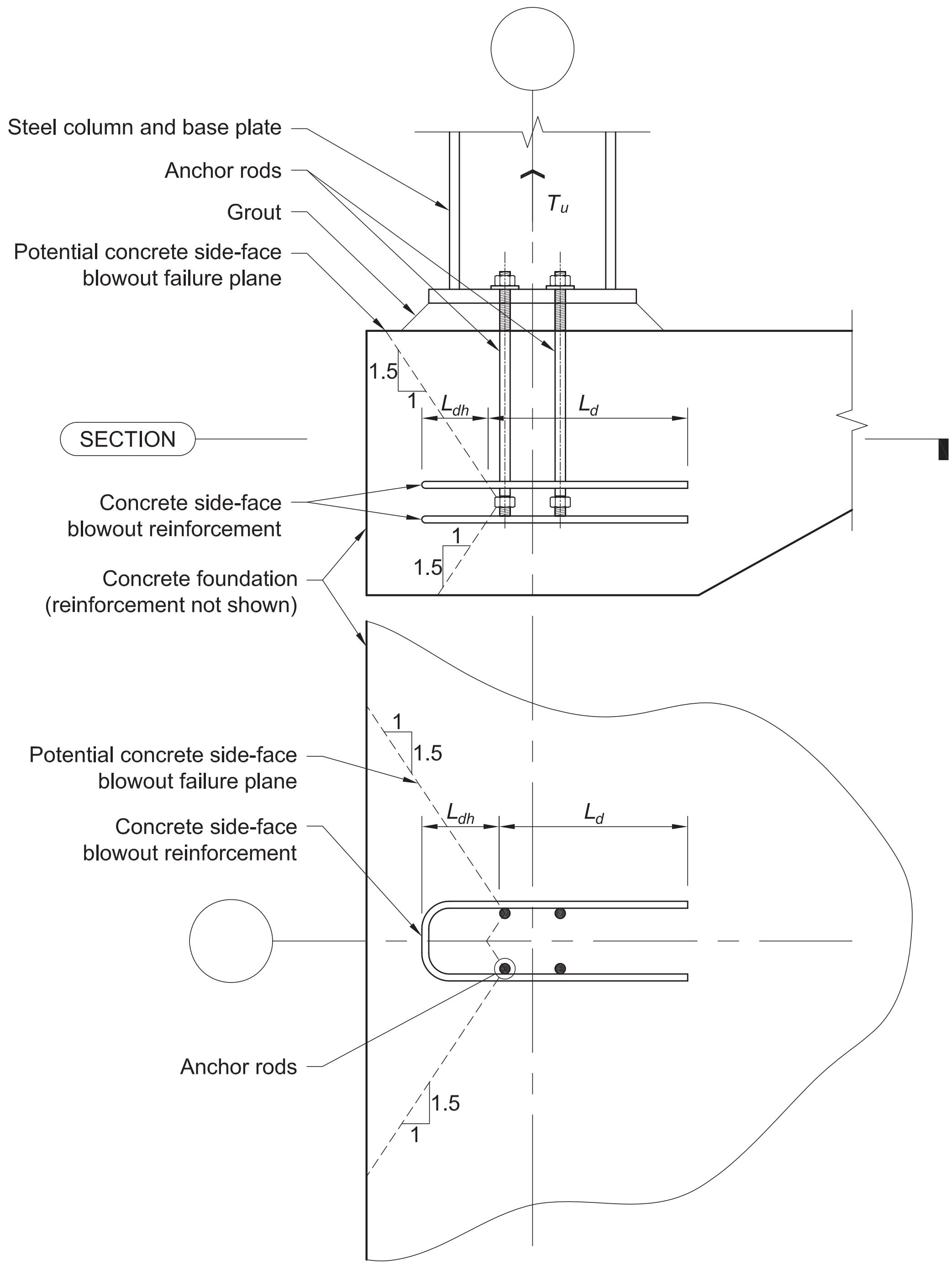


Fig. 4-13. The use of steel reinforcement for restraining concrete side-face blowout.

4.4.2 Use of Strut-and-Tie Methodologies in Anchorage Design

Strut-and-tie is an analysis method that can be used to design concrete members, or regions of members, where discontinuities cause nonlinear distribution of strains within a cross section. Discontinuities include changes in the geometry of a structural element or points of concentrated load or reactions. The points where the anchor rod forces are transferred into the concrete are considered discontinuity points, and thus a strut-and-tie approach can be used for the anchorage design of anchor rods.

In the strut-and-tie method, the region of discontinuity is modeled as an idealized truss. The compression elements of the truss represent the concrete struts and the tension elements of the truss represent the steel reinforcement ties. Generally, the strut-and-tie method is simply another approach to design steel reinforcement that will facilitate the transfer of the anchor rod forces to the concrete supporting member. ACI 318, Chapter 23, provides the design provisions for the design of the struts, ties, and nodal zones.

A report produced by the ASCE Petrochemical Energy Committee, titled *Anchorage Design for Petrochemical Facilities*, provides potential strut-and-tie models that can be used to resist tension and/or shear forces for breakout and side-face blowout limit states (ASCE, 2013).

4.5 EXPOSED BASE PLATE CONNECTIONS—FABRICATION AND INSTALLATION

4.5.1 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. of AISC *Specification* Section M2.2 lists requirements for thermal cutting as follows:

> Thermally cut edges shall meet the requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M) clauses 7.14.5.2, 7.14.8.3, and 7.14.8.4, hereafter referred to as AWS D1.1/D1.1M, with the exception that thermally cut free edges that will not be subject to fatigue 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 or repaired by welding.

Anchor rod hole sizes and grouting are covered in Sections 4.5.3 and 4.5.6 of this Guide.

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

> Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling provided a smooth and notch-free contact bearing surface 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 smooth and notch-free contact bearing surface. 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.

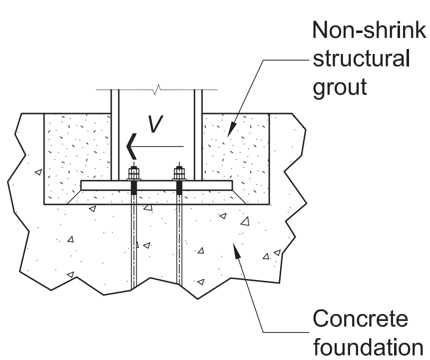


Fig. 4-14. Transfer of base shear through bearing by encasing the column.

AISC *Specification* Section M4.4 defines a smooth and notch-free 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 equal to or 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 nontapered 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 smooth and notch-free 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.

4.5.2 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 roughly 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:

1. Fillet welds are preferable to groove welds where the fillet weld size is such that it is economical for fabricators to perform.
2. The use of the weld-all-around symbol should be avoided, especially on wide-flange shapes, because 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. The authors recommend that weld symbols specify welding of flats only.
3. For most wide-flange columns subject to axial compression only, welding on one side of each flange (see Figure 4-15) with the minimum AWS fillet weld size 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. This maximum size should be coordinated with the fabricator based on economy.
4. 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 because the critical yield line will form in the plate at the corners of the HSS.
5. AISC *Specification* Section J2 requires that the minimum fillet weld size is based on the thinner of the materials joined.

Most column base plates are shop welded to the column shaft. In the past it was common to detail heavy base plates for multistory buildings as loose pieces to be set and grouted before erecting the column shaft. The base plate was detailed with three

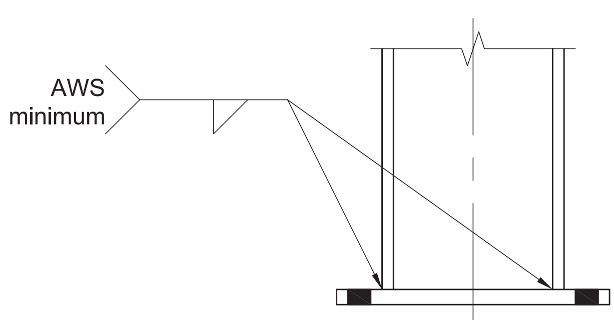


Fig. 4-15. Typical gravity column base plate weld.

adjusting screws, as shown in Figure 4-16, 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.

4.5.3 Anchor Rod Holes and Washers

A very 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 4-3.

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, and are currently, included in Part 14 of the AISC *Manual*.

The washer diameters shown in Table 4-3 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 if the thickness is adequate to prevent pulling through the hole.

The designer may consider using a smaller hole diameter as allowed in Footnote 4 in Table 4-3. This will allow the use of ASTM F844 (2019d) washers in lieu of the custom washers of dimensions shown in the table. This potential fabrication savings is not recommended because of potential problems with the placement of anchor rods being out of tolerance.

For anchor rods designed to resist moment or axial tension, the hole and washer sizes recommended in Table 4-3 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 4.3.3).
- ASTM F436/F436M (2019b) washers are not used on anchor rods because they generally are of insufficient size.
- Washers for anchor rods are not hardened and do not need to be.

Fig. 4-16. Base plate with adjusting screws.

Table 4-3. Recommended Sizes for Washers and Anchor Rod Holes in Base Plates							
Anchor Rod Diameter, in.	Base Plate Hole Diameter, in.	Minimum Washer Width, in.	Minimum Washer Thickness, in.	Anchor Rod Diameter, in.	Base Plate Hole Diameter, in.	Minimum Washer Width, in.	Minimum Washer Thickness, in.
ASTM F1554, Grade 36							
¾	1⁵⁄₁₆	2	¼	1½	2⅜	4	⅜
⅞	1⁹⁄₁₆	2½	⅜	1¾	2⅞	4½	¾
1	1⅞	3	⅜	2	3¼	5	¾
1¼	2⅛	3½	⅜	2½	3¾	5½	¾
ASTM F1554, Grade 55							
¾	1⁵⁄₁₆	2	¼	1½	2⅜	4	½
⅞	1⁹⁄₁₆	2½	⅜	1¾	2⅞	4½	¾
1	1⅞	3	⅜	2	3¼	5	¾
1¼	2⅛	3½	½	2½	3¾	5½	¾
ASTM F1554, Grade 105							
¾	1⁵⁄₁₆	2	⅜	1½	2⅜	4	⅝
⅞	1⁹⁄₁₆	2½	½	1¾	2⅞	4½	¾
1	1⅞	3	½	2	3¼	5	¾
1¼	2⅛	3½	⅝	2½	3¾	5½	⅞

Notes:
1. Hole sizes provided are based on anchor rod size and correlate with ACI 117 (2010).
2. Circular or square washers meeting the washer width are acceptable. Washer plate material: ASTM A572/A572M, Grade 50.
3. Clearance must be considered when choosing an appropriate anchor rod hole location, noting effects such as the position of the rod in the hole with respect to the column, weld size, and other interferences.
4. ASTM F844 washers may be used instead of plate washers when hole diameter is limited to rod diameter plus ⁵⁄₁₆ in. for rod diameters up to 1 in., rod diameter plus ½ in. for diameters over 1 in. up to 2 in., and rod diameters plus 1 in. for rod diameters over 2 in. This exception should not be used unless the general contractor has agreed to meet tighter tolerances for anchor rod placement than those specified in ACI 117.

- 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 thread length. Whenever possible, thread lengths should be specified at least 3 in., preferably 6 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 2 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 and post-tensioning tendons 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.

4.5.4 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. Additionally, a preconstruction meeting is recommended with the general contractor and their foundation construction team to review the anchor setting plans.

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 special considerations. In a fast-track project, the steel design and detailing may lag behind the initial foundation work, and the structural layout may change 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 117-10 [2010, reapproved 2015, and adopted by IBC 2021 (ICC, 2021)], 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.-diameter rods .... ±¼ in.
for 1-in., 1¼-in. and 1½-in.-diameter rods .... ±⅜-in.
for 1¾- in., 2-in., and 2½ in.-diameter rods .... ±½ in.

AISC *Code of Standard Practice* (2022a) Section 7.5.1 lists the following tolerances:

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.-diameter rods .... ±¼ in.
for 1-in., 1¼-in. and 1½-in.-diameter rods .... ±⅜-in.
for 1¾- in., 2 in., and 2½ in.-diameter rods .... ±½ in.

Thus, ACI 117-10 provisions are similar to the AISC *Code of Standard Practice* for anchor rod tolerances. Furthermore, because 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, 2020) Division 3, require that the anchor rods be set in accordance with the ACI 117-10 specification for tolerances for concrete construction and materials 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.

4.5.5 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 customs generally determine which of these methods is used. It is important in all methods that the erector tightens all 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.

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. Use of plate washers in lieu of standard washers will be needed at the bottom of the base plate because of the size of the large base plate holes. Typically, the design of the anchor rods and plate washers for loads during erection would be the responsibility of the erection engineer and should be designed to span across the hole.

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 typically ¼ in. thick and slightly larger than the base plate. Plates of this thickness tend to warp when fabricated; consequently, 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 or to transfer shear, the holes are made to follow AISC *Specification* Table J3.3 for standard holes. Otherwise, standard oversize 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 previously. 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. It is recommended that means to address this possibility should be established in advance of erecting the columns on the leveling plates. It should be noted that AISC *Specification* Section M2.8(b) waives the requirement for milling the bottom of base plates that are grouted. Not milling the bottom of thick base plates that bear on leveling plates may also result in the Section M4.4 tolerance being exceeded.

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.

Shim Stack Method

Column erection on shim stacks (steel or other materials) 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 steel shim stacks are typically large enough to carry substantial dead load prior to grouting of the base plate.

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 4-16 in Section 4.5.2. 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 4-3 in Section 4.5.3. 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.

4.5.6 Grouting Requirements

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

It is recommended that grout 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. However, grout with less strength can be used if its compressive strength is confirmed by calculation. 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 with large anchor rods and plates with shear lugs may require more space, especially if the setting nut and washer method is used to erect the column.

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. Additional requirements for grouting horizontally installed base plates with shear lugs are found in ACI 318, Section 17.11.1.2. 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 sides.

It is important to follow the manufacturer's recommendations for mixing and curing times. When placing grout in cold weather, it is especially important to ensure that 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 must 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.

4.6 EXPOSED COLUMN BASE CONNECTIONS—REPAIR AND FIELD FIXES

Anchor rods may require repair or modification during installation or later 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 if 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.

4.6.1 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 post-installed anchor rods. When using such rods, carefully follow the manufacturer's recommendations for installation and ACI 318, Chapter 17, for the anchorage design 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 must be made.

4.6.2 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. Additionally, products that protect anchor rods in the field and make them more visible are available. The anchor rods shown in Figure 4-17 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.

4.6.3 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. Additional information is available in Labelle (2016). Welding the nut to the anchor rod is not a prequalified welded joint and is not recommended. Additionally, ASTM F1554 only considers Grade 36 and Grade 55 (if in compliance with Supplement S1) anchor rod material to be weldable, and in these cases, it may be feasible to weld the anchor rod to the plate washer.

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 post-installed 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 4-18 details 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 4-4 lists the dimensions of typical coupling nuts that can be used to detail the required hole size and plate fillers. Alternatively, proprietary coupling nut extenders are available and could be considered.

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 to make a proper groove weld. Figure 4-19 shows a recommended detail for butt welding. The run-out tab can be trimmed off after welding, if necessary, and the rod can 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, 2017).

It is also possible to extend an anchor by using splice bars to connect a threaded rod extension. Details similar to Figure 4-20 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., and preferably 6 in., of thread beyond what the detail dimension requires to compensate for some variation in anchor rod projection.

Fig. 4-17. Anchor rods run over by a crane.

Table 4-4. Hex Coupling Nut Dimensions			
Diameter of Rod, in.	Width across Flats, in.	Approximate Width across Corners, in.	Height of Nut, in.
¾	1	1⅛	2¼
⅞	1¼	1 7/16	2½
1	1⅜	1 9/16	2¾
1¼	1⅝	1⅞	3
1½	2	2 5/16	3½
1¾	2¾	3 3/16	5¼
2	3⅛	3⅝	6
2½	3⅞	4½	7½
Dimensions based on ASME B18.2.2-2022 (2022). Material conforms to ASTM A563/A563M (2021a) Grade A.			

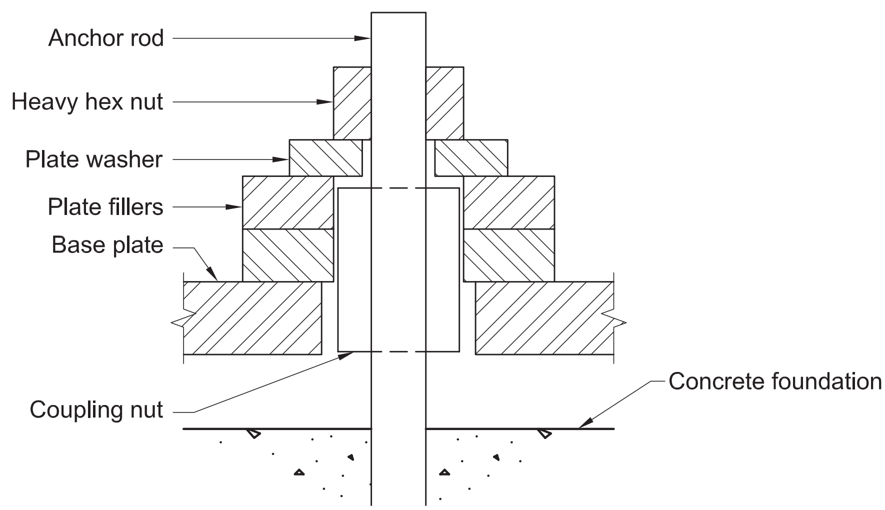


Fig. 4-18. Coupling nut detail for extending anchor rods.

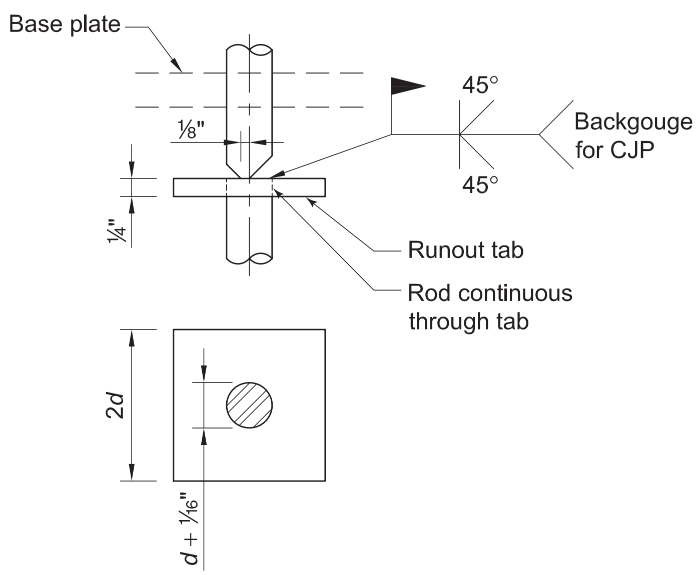


Fig. 4-19. Groove weld splice.

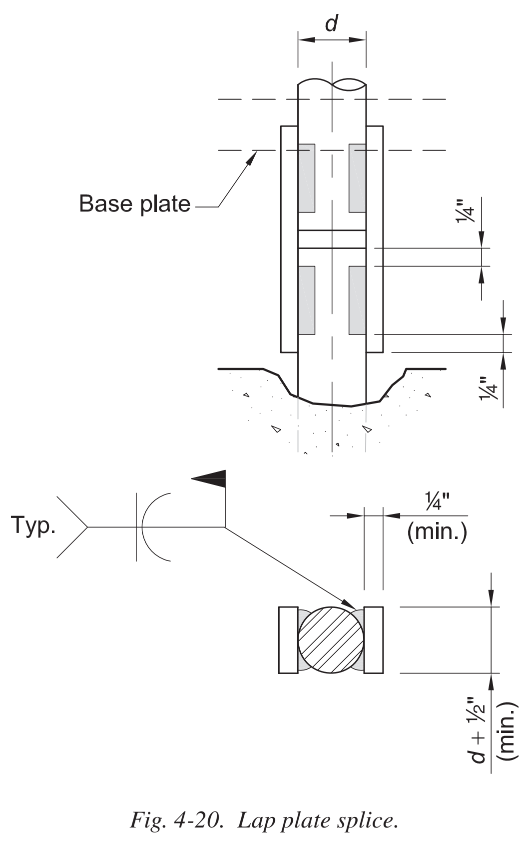


Fig. 4-20. Lap plate splice.

4.6.4 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.

4.7 DESIGN EXAMPLES

EXAMPLE 4.7-1—Base Connection for Concentric Axial Compression Load (No Concrete Confinement)

A base connection for a wide-flange column is designed for a concentric compression load. The dimensions of the base plate (width, length, and thickness) are determined considering the concrete bearing strength and flexural yielding strength of the plate. An increase in concrete bearing strength resulting from concrete confinement is not considered. The anchor rod quantity and configuration are determined.

Given:

A W12×96 column bears on a 24 in. × 24 in. concrete pedestal as shown in Figure 4-21. The minimum concrete compressive strength is $f_c' = 3$ ksi. The base plate is ASTM A572/A572M Grade 50 material.

The required strength due to axial loads is:

LRFD	ASD
$P_u = 700$ kips	$P_a = 466$ kips

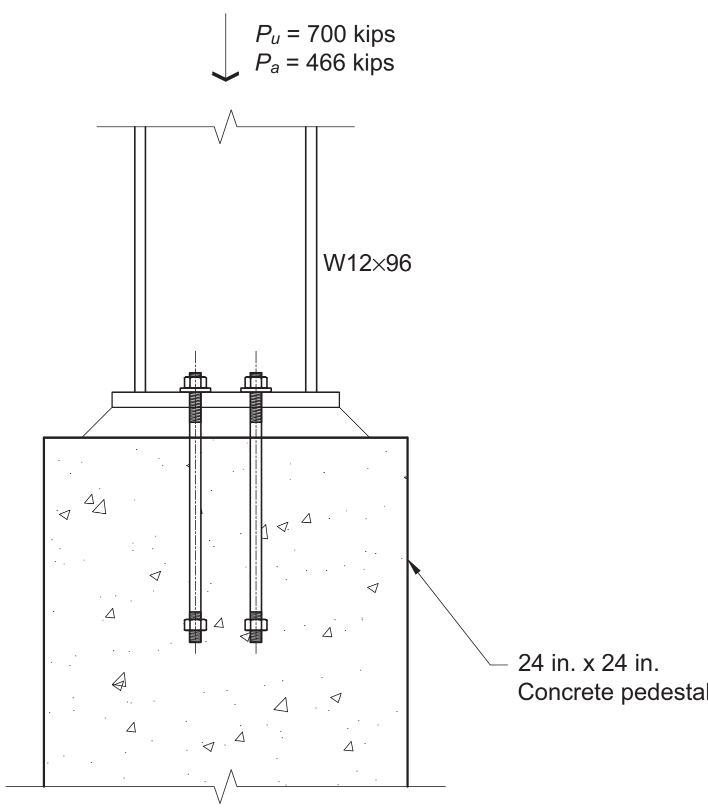


Fig. 4-21. Example 4.7-1 base detail.

Solution:

From AISC *Manual* Table 2-5

Base plate
ASTM A572/A572M Gr. 50
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the W-shape geometric properties are as follows:

W12×96
$d = 12.7$ in.
$b_f = 12.2$ in.

Determine the base plate plan dimensions and thickness for the given required strength, using the assumption that $A_2 = A_1$ (Case I).

Calculate the required base plate area

LRFD	ASD
$$A_{1(req)} = \frac{P_u}{\phi_c 0.85 f_c'} \qquad (4\text{-}6a)$$ $$= \frac{700 \text{ kips}}{(0.65)(0.85)(3 \text{ ksi})}$$ $$= 422 \text{ in.}^2$$	$$A_{1(req)} = \frac{\Omega_c P_a}{0.85 f_c'} \qquad (4\text{-}6b)$$ $$= \frac{(2.31)(466 \text{ kips})}{(0.85)(3 \text{ ksi})}$$ $$= 422 \text{ in.}^2$$

Optimize the base plate dimensions, N *and* B

Setting $m = n$ and $\Delta = B - N$ will yield:

$$\Delta = \frac{0.95d - 0.8b_f}{2} \tag{4-17}$$

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

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

$$N \approx \sqrt{A_{1(req)}} + \Delta \tag{4-16}$$

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

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

Round N up to its nearest whole number, $N = 22.0$ in.

$$B = \frac{A_{1(req)}}{N} \tag{4-18}$$

$$= \frac{422 \text{ in.}^2}{22.0 \text{ in.}}$$

$$= 19.2 \text{ in.}$$

Round B up to its nearest whole number, $B = 20.0$ in.

$$A_1 = BN$$

$$= (22.0 \text{ in.})(20.0 \text{ in.})$$

$$= 440 \text{ in.}^2 > 422 \text{ in.}^2$$

Check bearing strength of the concrete without considering confinement of the concrete ($A_2 = A_1$) using AISC *Specification* Equation J8-1:

LRFD	ASD
$\phi_c P_p = \phi_c 0.85 f'_c A_1$ $= (0.65)(0.85)(3 \text{ ksi})(440 \text{ in.}^2)$ $= 729 \text{ kips} > 700 \text{ kips}$ **o.k.**	$\dfrac{P_p}{\Omega_c} = \left(\dfrac{0.85 f'_c A_1}{\Omega_c}\right)$ $= \left[\dfrac{(0.85)(3 \text{ ksi})(440 \text{ in.}^2)}{2.31}\right]$ $= 486 \text{ kips} > 466 \text{ kips}$ **o.k.**

Calculate required base plate thickness

$$m = \frac{N - 0.85d}{2} \tag{4-10}$$

$$= \frac{22.0 \text{ in.} - 0.85(12.7 \text{ in.})}{2}$$

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

$$N = \frac{B - 0.8b_f}{2} \tag{4-11}$$

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

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

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{P_u}{\phi_c P_p}$ (4-14a) $= \left[\dfrac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\left(\dfrac{700 \text{ kips}}{729 \text{ kips}}\right)$ $= 0.960$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ (4-13) $= \dfrac{2\sqrt{0.960}}{1+\sqrt{1-0.960}}$ $= 1.63 > 1$ $= 1$	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{\Omega_c P_a}{P_p}$ (4-14b) $= \left[\dfrac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\left(\dfrac{466 \text{ kips}}{486 \text{ kips}}\right)$ $= 0.958$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ (4-13) $= \dfrac{2\sqrt{0.958}}{1+\sqrt{1-0.958}}$ $= 1.62 > 1$ $= 1$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} \tag{4-12}$$

$$= (1)\frac{\sqrt{(12.7 \text{ in.})(12.2 \text{ in.})}}{4}$$

$$= 3.11 \text{ in.}$$

$$l = \max\left(m, n, \lambda_n'\right)$$

$$= \max\left(4.97 \text{ in.}, 5.12 \text{ in.}, 3.11 \text{ in.}\right)$$

$$= 5.12 \text{ in}$$

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{\phi_b F_y BN}}$ (4-15a) $= (5.12\text{ in.})\sqrt{\dfrac{(2)(700\text{ kips})}{(0.90)(50\text{ ksi})(20.0\text{ in.})(22.0\text{ in.})}}$ $= 1.36\text{ in.}$	$t_{min} = l\sqrt{\dfrac{2\Omega_b P_a}{F_y BN}}$ (4-15b) $= (5.12\text{ in.})\sqrt{\dfrac{(2)(1.67)(466\text{ kips})}{(50\text{ ksi})(20.0\text{ in.})(22.0\text{ in.})}}$ $= 1.36\text{ in.}$

Use a 1½-in.-thick base plate.

Determine the anchor rod size and location

Because no anchor rod forces exist in the completed structure, the anchor rod size and embedment should be determined based on the OSHA requirements, erection considerations such as wind during construction, and practical considerations.

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

Determine the column to base plate welds

The axial force will be transferred through bearing from the column to the base plate. Only minimum welding needs to be provided as discussed in Section 4.5.2.

EXAMPLE 4.7-2—Base Connection for Concentric Axial Compression Load (Using Concrete Confinement)

A base connection for a wide-flange column is designed for a concentric compression load. The dimensions of the base plate (width, length, and thickness) are determined considering the concrete bearing strength and flexural yielding strength of the plate. An increase in concrete bearing strength resulting from concrete confinement is considered. The anchor rod quantity and configuration are determined and the column to base plate weld designed.

Given:

Using the criteria from Example 4.7-1, determine the base plate plan dimensions considering the effect of concrete confinement in determining the available concrete bearing strength (Case III).

The required strength due to axial loads is:

LRFD	ASD
$P_u = 700\text{ kips}$	$P_a = 466\text{ kips}$

Solution:

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

LRFD	ASD
$A_{1(req)} = \dfrac{P_u}{2\phi_c 0.85 f_c'}$ (4-7a) $= \dfrac{700\text{ kips}}{2(0.65)(0.85)(3\text{ ksi})}$ $= 211\text{ in.}^2$	$A_{1(req)} = \dfrac{\Omega_c P_a}{2(0.85 f_c')}$ (4-7b) $= \dfrac{(2.31)(466\text{ kips})}{(2)(0.85)(3\text{ ksi})}$ $= 211\text{ in.}^2$

Optimize the base plate dimensions, N *and* B

Setting $m = n$ and $\Delta = B - N$ will yield:

$$\Delta = \frac{0.95d - 0.8b_f}{2} \tag{4-17}$$

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

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

$$N \approx \sqrt{A_{1(req)}} + \Delta \tag{4-16}$$

$$\approx \sqrt{211 \text{ in.}^2} + 1.15 \text{ in.}$$

$$\approx 15.7 \text{ in.}$$

Round N up to its nearest whole number, $N = 16.0$ in.

$$B = \frac{A_{1(req)}}{N} \tag{4-18}$$

$$= \frac{211 \text{ in.}^2}{16.0 \text{ in.}}$$

$$= 13.2 \text{ in.}$$

Round B up to its nearest whole number, $B = 14.0$ in.

$$A_1 = BN$$

$$= (14.0 \text{ in.})(16.0 \text{ in.})$$

$$= 224 \text{ in.}^2 > 211 \text{ in.}^2 \quad \textbf{o.k.}$$

Calculate A_2 *geometrically similar to* A_1

The geometrically similar area is calculated based on the 24.0 in. pier as:

$$N_2 = 24.0 \text{ in.}$$

$$B_2 = 14.0 \text{ in.} + (24.0 \text{ in.} - 16.0 \text{ in.})$$

$$= 22.0 \text{ in.}$$

$$A_2 = N_2 B_2$$

$$= (24.0 \text{ in.})(22.0 \text{ in.})$$

$$= 528 \text{ in.}^2$$

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

$$= 896 \text{ in.}^2 \geq 528 \text{ in.}^2$$

Case III applies and, because full confinement is not possible, a larger plate size will be tried.

Trial dimensions $N = 20.0$ in. and $B = 18.0$ in. are considered, which yields:

$$A_1 = BN$$

$$= (18.0 \text{ in.})(20.0 \text{ in.})$$

$$= 360 \text{ in.}^2$$

$$N_2 = 24.0 \text{ in.}$$

$$B_2 = 18.0 \text{ in.} + (24.0 \text{ in.} - 20.0 \text{ in.})$$
$$= 22.0 \text{ in.}$$

$$A_2 = N_2 B_2$$
$$= (24.0 \text{ in.})(22.0 \text{ in.})$$
$$= 528 \text{ in.}^2$$

Compare the required bearing strength to the available bearing strength

If the required bearing strength is less than the available bearing strength, revise B and N until criteria is satisfied.

LRFD	ASD
$\phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\dfrac{A_2}{A_1}}$ (4-20a) $= (0.65)(0.85)(3 \text{ ksi})(360 \text{ in.}^2)\sqrt{\dfrac{528 \text{ in.}^2}{360 \text{ in.}^2}}$ $= 723 \text{ kips} > 700 \text{ kips}$ **o.k.** Use $N = 20.0$ in., $B = 18.0$ in.	$\dfrac{P_p}{\Omega_c} = \dfrac{0.85 f_c'}{\Omega_c} A_1 \sqrt{\dfrac{A_2}{A_1}}$ (4-20b) $= \dfrac{(0.85)(3 \text{ ksi})}{2.31}(360 \text{ in.}^2)\sqrt{\dfrac{528 \text{ in.}^2}{360 \text{ in.}^2}}$ $= 481 \text{ kips} > 466 \text{ kips}$ **o.k.** Use $N = 20.0$ in., $B = 18.0$ in.

Calculate required base plate thickness

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$
$$= \frac{20.0 \text{ in.} - 0.95(12.7 \text{ in.})}{2}$$
$$= 3.97 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$
$$= \frac{18.0 \text{ in.} - 0.8(12.2 \text{ in.})}{2}$$
$$= 4.12 \text{ in.}$$

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{P_u}{\phi_c P_p}$ (4-14a) $= \left[\dfrac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\dfrac{700 \text{ kips}}{723 \text{ kips}}$ $= 0.968$	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\dfrac{\Omega_c P_a}{P_p}$ (4-14b) $= \left[\dfrac{4(12.7 \text{ in.})(12.2 \text{ in.})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2}\right]\dfrac{466 \text{ kips}}{481 \text{ kips}}$ $= 0.968$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1 \tag{4-13}$$
$$= \frac{2\sqrt{0.968}}{1 + \sqrt{1 - 0.968}}$$
$$= 1.67 > 1$$
$$= 1$$

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} \tag{4-12}$$

$$= (1)\frac{\sqrt{(12.7 \text{ in.})(12.2 \text{ in.})}}{4}$$

$$= 3.11 \text{ in.}$$

$$l = \max(m, n, \lambda n')$$

$$= \max(3.97 \text{ in.}, 4.12 \text{ in.}, 3.11 \text{ in.})$$

$$= 4.12 \text{ in.}$$

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{\phi_b F_y BN}}$ $= (4.12 \text{ in.})\sqrt{\dfrac{(2)(700 \text{ kips})}{(0.90)(50 \text{ ksi})(18.0 \text{ in.})(20.0 \text{ in.})}}$ $= 1.21 \text{ in.}$	$t_{min} = l\sqrt{\dfrac{2\Omega_b P_a}{F_y BN}}$ (4-15b) $= (4.12 \text{ in.})\sqrt{\dfrac{(2)(1.67)(466 \text{ kips})}{(50 \text{ ksi})(18.0 \text{ in.})(20.0 \text{ in.})}}$ $= 1.21 \text{ in.}$

Use a 1¼-in.-thick base plate.

The anchor rods are the same as Example 4.7-1.

EXAMPLE 4.7-3—Base Connection for Concentric Axial Tension Load

A base connection for a wide-flange column is designed for a concentric tension load. The type and number of anchor rods, base plate dimensions, welding, and concrete anchorage are designed.

Given:

A W10×45 column is subjected to a net uplift load. The column will be anchored to the foundation using an ASTM A572/A572M Grade 50 base plate and ASTM F1554 Grade 36 anchor rods. The column is attached to a large spread footing with a specified compressive strength of concrete, $f_c' = 4{,}000$ psi. Use 70 ksi weld electrodes.

The required strengths due to axial tensile loads are:

LRFD	ASD
$P_u = 70.0$ kips (uplift)	$P_a = 45.0$ kips (uplift)

Solution:

From AISC *Manual* Tables 2-4, 2-5, and 2-6, the material properties are as follows:

W10×45
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi

Anchor rods
ASTM F1554 Grade 36

F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties of the column are as follows:

W10×45
b_f = 8.02 in.
d = 10.1 in.
t_w = 0.350 in.

Procedure

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

Select the type and number of anchor rods

Per OSHA requirements, a minimum of four anchor rods are required. Determine the tension per anchor rod considering that the anchor rod group is concentric with the applied uplift load such that the tension load is equally distributed to all anchor rods.

LRFD	ASD
$r_u = \dfrac{P_u}{\text{number of rods}}$ $= \dfrac{70.0 \text{ kips}}{4 \text{ rods}}$ $= 17.5 \text{ kips/rod}$	$r_a = \dfrac{P_a}{\text{number of rods}}$ $= \dfrac{45.0 \text{ kips}}{4 \text{ rods}}$ $= 11.3 \text{ kips/rod}$

Using ⅞-in.-diameter anchor rods and the tensile stress area determined from Table 4-1, the nominal tensile strength of each anchor rod is:

$$R_n = F_u A_{se,N} \tag{4-22}$$
$$= (58 \text{ ksi})(0.462 \text{ in.}^2)$$
$$= 26.8 \text{ kips}$$

The available tensile strength of each rod may then be calculated as:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(26.8 \text{ kips})$ $= 20.1 \text{ kips} > 17.5 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{26.8 \text{ kips}}{2.00}$ $= 13.4 \text{ kips} > 11.3 \text{ kips}$ **o.k.**

Alternatively, these values could also have been determined from Table 4-1.

The four ⅞-in.-diameter ASTM F1554 Grade 36 anchor rods have adequate tensile capacity to resist the required strength.

Determine the appropriate base plate thickness and weld to transfer the uplift forces from the column to the anchor rods

The rods are positioned inside the column profile with a 4.00 in. square pattern (g = 4.00 in.). Prying forces are considered negligible for this example but could be considered if deemed appropriate based on the engineer's judgment. To simplify the analysis, conservatively consider that the tensile loads in the anchor rods generate one-way bending in the base plate about the web of the column. This consideration is illustrated by the bending lines shown in Figure 4-22. If the column web strength controls the design, then consider distributing the forces to the flanges as well as the web using relative stiffness and two-way bending. For bolts located outside of the flange, the 45° load distribution can be used to distribute the forces to the flanges only. A yield line analysis may also be used to design the plate if the welds are properly designed to account for the assumed yield line.

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

$$\begin{aligned} a &= \frac{g - t_w}{2} \\ &= \frac{4.00 \text{ in.} - 0.350 \text{ in.}}{2} \\ &= 1.83 \text{ in.} \end{aligned}$$

LRFD	ASD
$\begin{aligned} M_u &= r_u a \\ &= (17.5 \text{ kips})(1.83 \text{ in.}) \\ &= 32.0 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_a &= r_a a \\ &= (11.3 \text{ kips})(1.83 \text{ in.}) \\ &= 20.7 \text{ kip-in.} \end{aligned}$

The effective width, b_{eff}, of the base plate for resisting the required moment strength at the face of the web is determined from a 45° distribution for the rod loads (width shown between the dashed lines in Figure 4-22),

$$\begin{aligned} b_{eff} &= 2a \\ &= 2(1.83 \text{ in.}) \\ &= 3.66 \text{ in.} \end{aligned}$$

The plastic section modulus, Z, of the effective section can then be calculated as:

$$Z = \frac{b_{eff} t^2}{4}$$

Setting the available strength equal to the required strength and solving for the required thickness yields:

LRFD	ASD
$\begin{aligned} t_{req} &= \sqrt{\frac{4M_u}{b_{eff} \phi_b F_y}} \\ &= \sqrt{\frac{4(32.0 \text{ kip-in.})}{(3.66 \text{ in.})(0.90)(50 \text{ ksi})}} \\ &= 0.882 \text{ in.} \end{aligned}$	$\begin{aligned} t_{req} &= \sqrt{\frac{4\Omega_b M_a}{b_{eff} F_y}} \\ &= \sqrt{\frac{4(1.67)(20.7 \text{ kip-in.})}{(3.66 \text{ in.})(50 \text{ ksi})}} \\ &= 0.869 \text{ in.} \end{aligned}$

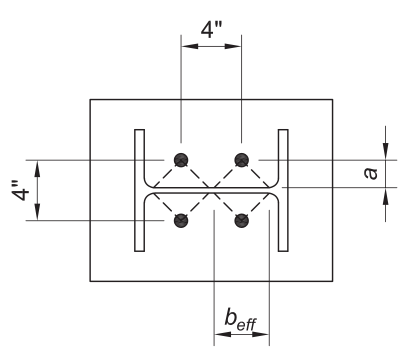


Fig. 4-22. Rod load distribution.

Use a 1-in.-thick ASTM A572/A572M Grade 50 base plate.

For the column to base plate weld on each side of the column web, consider the anchor rod tensile force active only on an effective width, b_{eff}, of the weld.

LRFD	ASD
$r_{uw} = \dfrac{r_u}{b_{eff}}$ $= \dfrac{17.5 \text{ kips}}{3.66 \text{ in.}}$ $= 4.78 \text{ kips/in.}$	$r_{aw} = \dfrac{r_a}{b_{eff}}$ $= \dfrac{11.3 \text{ kips}}{3.66 \text{ in.}}$ $= 3.09 \text{ kips/in.}$

From AISC *Specification* Table J2.4, the minimum fillet weld size for the 0.350 in. column web is 3⁄16 in.

The welds are placed on each side of the column web and are therefore loaded through its center of gravity. Therefore, a directional strength increase may be utilized. From AISC *Specification* Section J2.4(a), the increase factor for an angle of 90° between the line of action of the required force and weld longitudinal axis is calculated by:

$$\theta = 90°$$

$$k_{ds} = (1.0 + 0.50 \sin^{1.5}\theta) \quad \text{(\textit{Spec}. Eq. J2-5)}$$

$$= \left[1.0 + 0.50 \sin^{1.5}(90°)\right]$$

$$= 1.50$$

From AISC *Specification* Section J2.4(a) and Table J2.5, the nominal weld strength per in. for a 3⁄16 in. fillet weld with E70 electrode is:

$$R_n = F_{nw} A_{we} k_{ds} \quad \text{(\textit{Spec}. Eq. J2-4)}$$

$$= \left[0.60(70 \text{ ksi})\right]\left(\frac{3/16 \text{ in.}}{\sqrt{2}}\right)(1.50)$$

$$= 8.35 \text{ kip/in.}$$

The available strength is then calculated as follows:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(8.35 \text{ kip/in.})$ $= 6.26 \text{ kip/in.} > 4.78 \text{ kip/in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{8.35 \text{ kip/in.}}{2.00}$ $= 4.18 \text{ kip/in.} > 3.09 \text{ kip/in.}$ **o.k.**

Check the local stress at the web at the weld:

LRFD	ASD
$\phi F_y = 0.90(50 \text{ ksi})$ $= 45.0 \text{ ksi}$ $f_{web} = \dfrac{2r_{uw}}{t_w}$ $= \dfrac{2(4.78 \text{ kip/in.})}{0.350 \text{ in.}}$ $= 27.3 \text{ ksi} < 45.0 \text{ ksi}$ **o.k.**	$\dfrac{F_y}{\Omega} = \dfrac{50 \text{ ksi}}{1.67}$ $= 29.9 \text{ ksi}$ $f_{web} = \dfrac{2r_{aw}}{t_w}$ $= \dfrac{2(3.09 \text{ kip/in.})}{0.350 \text{ in.}}$ $= 17.7 \text{ ksi} < 29.9 \text{ ksi}$ **o.k.**

Determine the design concrete anchorage strength for developing the required strength of the anchor rods in the concrete spread footing

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 side-face blowout of the concrete.

Try using a 3½ in. hook on the embedded end of the anchor rod to develop the required strength of 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. Refer to AISC *Manual* Part 14 for recommended limitations of use. Because no analysis was performed to confirm there will be no cracking at service load levels, $\psi_{c,P} = 1.0$. Note that no equivalent ASD solution exists for concrete pullout capacity within ACI 318.

The design pullout strength of a single cast-in hooked anchor is calculated according to ACI 318, Section 17.6.3.

Calculate the distance from the inner surface of the shaft of the anchor to the tip of the hook, e_h, and confirm that the hook geometry conforms to the requirements of ACI 318, Section 17.6.3.2.2(b).

$$d_a = \tfrac{7}{8} \text{ in.}$$

$$\begin{aligned} e_h &= \text{hook length} - d_a \\ &= 3\tfrac{1}{2} \text{ in.} - \tfrac{7}{8} \text{ in.} \\ &= 2.63 \text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{e_h}{d_a} &= \frac{2.63 \text{ in.}}{\tfrac{7}{8} \text{ in.}} \\ &= 3.01 \end{aligned}$$

Because e_h is at least $3d_a$ and not greater than $4.5d_a$, the hook geometry is acceptable.

$$\begin{aligned} N_p &= 0.9 f'_c e_h d_a && \text{(ACI 318, Eq. 17.6.3.2.2b)} \\ &= 0.9(4{,}000 \text{ psi})(2.63 \text{ in.})(\tfrac{7}{8} \text{ in.})\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right) \\ &= 8.28 \text{ kips} \end{aligned}$$

$$\begin{aligned} N_{pn} &= \psi_{c,P} N_p && \text{(ACI 318, Eq. 17.6.3.1)} \\ &= 1.0(8.28 \text{ kips}) \\ &= 8.28 \text{ kips} \end{aligned}$$

$$\phi = 0.70 \qquad \text{[ACI 318, Table 17.5.3(c)]}$$

$$\begin{aligned} \phi N_{pn} &= 0.70(8.28 \text{ kips}) \\ &= 5.80 \text{ kips} < 17.5 \text{ kips} \quad \textbf{n.g.} \end{aligned}$$

Thus, a 3.50 in. hook is not capable of developing the required tensile force in the rod.

Therefore, use a heavy hex nut and a threaded rod to develop the required tensile force in the rod.

Concrete pullout strength

The design pullout strength of a ⅞-in.-diameter rod from Table 4-2 is $\phi N_{pn} = 26.7$ kips, which is greater than the required strength per anchor rod of $r_u = 17.5$ kips.

Concrete breakout strength

The required embedment depth to achieve a concrete breakout strength, ϕN_{cbg}, that exceeds the required uplift of 70.0 kips (LRFD) can be determined by trial and error. The final trial with an embedment length, h_{ef}, of 15.0 in. follows. The design concrete breakout strength of the cast-in anchor group is calculated according to ACI 318, Section 17.6.2. Because the tension load is concentric with the anchor group, $e'_N = 0$ in.

λ_a = 1.0 for normal-weight concrete

The projected concrete failure area of a group of anchors with 4.00 in. by 4.00 in. spacing (s_1, s_2) and unaffected by edge distance is calculated according to ACI 318, Section 17.6.2.1.1, and Figure 4-23.

$$\begin{aligned} A_{Nc} &= (1.5h_{ef} + s_1 + 1.5h_{ef})(1.5h_{ef} + s_2 + 1.5h_{ef}) \\ &= [1.5(15.0 \text{ in.}) + 4.00 \text{ in.} + 1.5(15.0 \text{ in.})][1.5(15.0 \text{ in.}) + 4.00 \text{ in.} + 1.5(15.0 \text{ in.})] \\ &= 2{,}400 \text{ in.}^2 \end{aligned}$$

The projected failure area of a single anchor with an edge distance of at least $1.5h_{ef}$ is:

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 & \text{(ACI 318, Eq. 17.6.2.1.4)} \\ &= 9(15.0 \text{ in.})^2 \\ &= 2{,}030 \text{ in.}^2 \end{aligned}$$

The modification factor for eccentric loading on anchor groups is given by:

$$\begin{aligned} \psi_{ec,N} &= \frac{1}{\left(1 + \dfrac{e'_N}{1.5h_{ef}}\right)} \leq 1 & \text{(ACI 318, Eq. 17.6.2.3.1)} \\ &= \frac{1}{\left[1 + \dfrac{0 \text{ in.}}{1.5(15.0 \text{ in.})}\right]} \leq 1 \\ &= 1.0 \end{aligned}$$

Because the edge distance exceeds $1.5h_{ef}$,

$$\psi_{ed,N} = 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.4.1a)}$$

Because no analysis was performed, consider the concrete to be cracked at service load levels, use $\psi_{c,N}$ = 1.0, in accordance with ACI 318, Section 17.6.2.5.1(b).

For cast-in anchors, the factor representing breakout splitting is determined as $\psi_{cp,N}$ = 1.0 per ACI 318, Section 17.6.2.6.2.

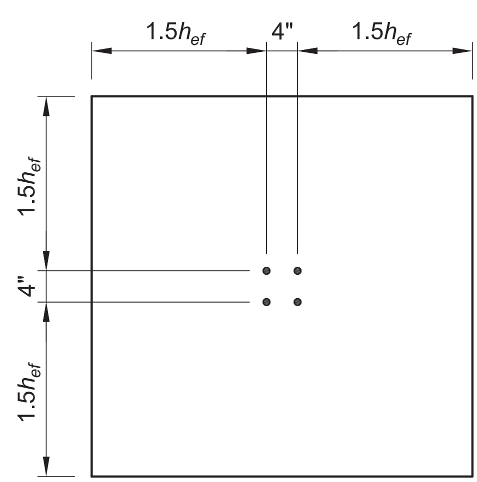


Fig. 4-23. Breakout cone for Example 4.7-3.

For 11.0 in. ≤ h_{ef} ≤ 25.0 in.,

$$N_b = 16\lambda_a \sqrt{f_c'} h_{ef}^{5/3} \qquad \text{(ACI 318, Eq. 17.6.2.2.3)}$$

$$= \left[16(1.0)\sqrt{\frac{4{,}000 \text{ psi}}{\text{psi}}}\left(\frac{15.0 \text{ in.}}{\text{in.}}\right)^{5/3}\right](\text{lbf})\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right)$$

$$= 92.3 \text{ kips}$$

The nominal breakout strength of the anchor group in tension is determined according to ACI 318, Section 17.6.2.1 as:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

$$= \left(\frac{2{,}400 \text{ in.}^2}{2{,}030 \text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(92.3 \text{ kips})$$

$$= 109 \text{ kips}$$

The resulting available strength considering that supplemental reinforcement will not be provided to restrain concrete tension breakout is:

$$\phi = 0.70 \qquad \text{[ACI 318, Table 17.5.3(b)]}$$

$$\phi N_{cbg} = 0.70(109 \text{ kips})$$

$$= 76.3 \text{ kips} > 70.0 \text{ kips} \quad \textbf{o.k.}$$

With the ⅞-in.-diameter rods, a 15.0 in. embedment is adequate to achieve the required strength considering the breakout strength.

Concrete side-face blowout strength is not applicable because the anchors are away from an edge.

EXAMPLE 4.7-4—Base Connection for Concentric Shear Load (Limited by Edge Distance)

The concrete edge distances parallel, c_{a1}, and perpendicular, c_{a2}, to the force to develop the shear force for a group of anchor rods are determined in this example. The perpendicular edge distance, c_{a2}, will be set such that the concrete breakout capacity in shear is not reduced by the edge distance, c_{a2}.

Given:

A 4.00 in. × 4.00 in. pattern is used for the ¾-in.-diameter ASTM F1554 Grade 36 rods shown in Figure 4-24. The concrete strength is $f_c' = 4{,}000$ psi, and no supplemental reinforcement will be considered.

Solution:

Case 3 from ACI 318, Commentary Figure R17.7.2.1b, will be applicable because the spacing, s, of the anchor rod group is less than the expected edge distance, c_{a1}, and additionally, the anchor rods are not welded to a common plate. In this case, although four anchors are provided, only the strength of two of the anchors adjacent to the edge can be considered when determining the available strength of the connection, including steel and concrete limit states.

The nominal shear strength of a single anchor rod is given in AISC *Specification* Section J3.7 as:

$$R_{nv} = F_{nv} A_b \qquad (\textit{Spec}. \text{ Eq. J3-1})$$

$$= 0.450(58 \text{ ksi})(0.442 \text{ in.}^2)$$

$$= 11.5 \text{ kips}$$

The available shear strength of a single anchor is therefore,

$$\phi R_{nv} = 0.75(11.5 \text{ kips})$$

$$= 8.63 \text{ kips}$$

Case 3 from ACI 318, Commentary Figure R17.7.2.1b, governs, the available shear strength of two of the four rods is:

$$\begin{aligned}\phi R_{nv} &= 2(8.63 \text{ kips})\\ &= 17.3 \text{ kips}\end{aligned}$$

Find the concrete breakout strength of the anchor group

The resistance factor for concrete breakout strength is given by ACI 318, Table 17.5.3(b), as $\phi = 0.70$ for the case where no supplementary reinforcement is present. The nominal concrete breakout strength in shear is determined according to ACI 318, Section 17.7.2.

Try a preliminary edge distance from the center of the closest anchor rod in the direction of the shear force, c_{a1}, of 12.0 in. Ensure that the edge distance from the center of the closest anchor rod perpendicular to the force, c_{a2}, and the depth of concrete, h_a, exceed $1.5c_{a1} = 18.0$ in.

The projected concrete failure area on the side face of the concrete foundation is calculated per ACI 318, Section 17.7.2.1.1. The total breakout shear area for the group of anchors is calculated by:

$$\begin{aligned}A_{Vc} &= 1.5c_{a1}(1.5c_{a1} + s + 1.5c_{a1})\\ &= 1.5(12.0 \text{ in.})[1.5(12.0 \text{ in.}) + 4.00 \text{ in.} + 1.5(12.0 \text{ in.})]\\ &= 720 \text{ in.}^2\end{aligned}$$

The projected area for a single anchor in a deep member is given by:

$$\begin{aligned}A_{Vco} &= 4.5(c_{a1})^2 \qquad \text{(ACI 318, Eq. 17.7.2.1.3)}\\ &= 4.5(12.0 \text{ in.})^2\\ &= 648 \text{ in.}^2\end{aligned}$$

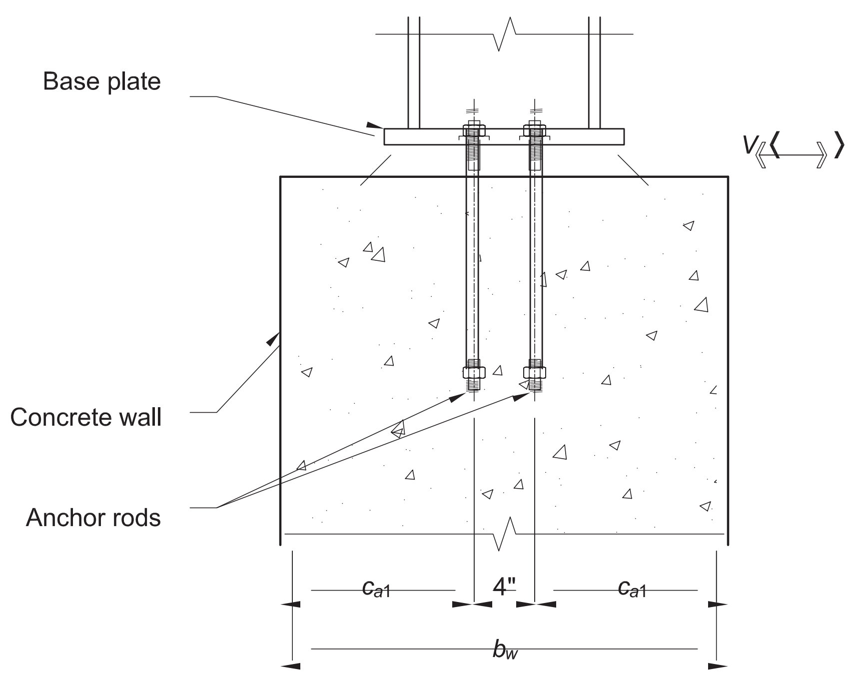


Fig. 4-24. Base connection section for configuration used in Example 4.7-4.

The basic concrete breakout strength of a single anchor, with $d_a = 0.750$ in. and $h_{ef} \geq 8d_a$ is given by:

$$V_b = 9\lambda_a \sqrt{f'_c}(c_{a1})^{1.5} \qquad \text{(ACI 318, Eq. 17.7.2.2.1b)}$$

$$= 9(1.0)\sqrt{4{,}000 \text{ psi}}(12.0 \text{ in.})^{1.5}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right)$$

$$= 23.7 \text{ kips}$$

Because the anchor group is not loaded eccentrically, $\psi_{ec,V} = 1.0$ per ACI 318, Section 17.7.2.3.

Because $c_{a2} \geq 1.5c_{a1}$, an edge distance reduction is not required per ACI 318, Section 17.7.2.4:

$$\psi_{ed,V} = 1.0$$

Because the bottom of the concrete shear breakout cone does not extend past a concrete edge, $\psi_{h,V} = 1.0$ per ACI 318, Section 17.7.2.6.1.

Use $\psi_{c,V} = 1.0$ per ACI 318, Section 17.7.2.5, for cracked concrete without adequate supplementary reinforcement.

The nominal concrete breakout strength of the anchor group is then given by:

$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b \qquad \text{(ACI 318, Eq. 17.7.2.1b)}$$

$$= \left(\frac{720 \text{ in.}^2}{648 \text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(23.7 \text{ kips})$$

$$= 26.3 \text{ kips}$$

The available shear breakout capacity may then be determined using, $\phi = 0.70$:

$$\phi V_{cbg} = 0.70(26.3 \text{ kips})$$

$$= 18.4 \text{ kips} > 17.3 \text{ kips} \quad \textbf{o.k.}$$

To develop the available strength (17.3 kips) of the two rods resisting the shear load, use a distance from the center of the closest anchor rod in the direction of the shear force, c_{a1}, of 12.0 in. Ensure that the edge distance from the center of the closest anchor rod perpendicular to the force, c_{a2}, and the depth of concrete, h_a, exceed $1.5c_{a1} = 18.0$ in.

Additionally, the embedment length, h_{ef}, must be determined to satisfy the concrete pryout limit state. The concrete shear breakout strength in this example considers an embedment equal to at least eight times the anchor rod diameter.

Design the shear lugs

Shear forces can be transferred in bearing by the use of shear lugs welded to the base plate as illustrated in Figure 4-25. When shear lugs are used, ACI 318, Section 17.11, provisions are used for the design of concrete limit states and the AISC *Specification* for the steel limit states. When used, shear lugs must be designed to transfer the entire required shear strength.

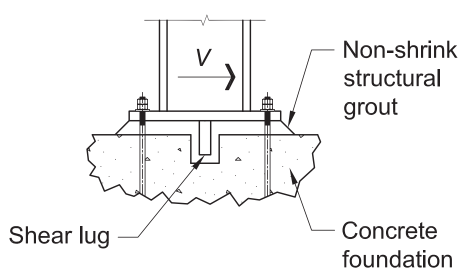


Fig. 4-25. Shear lug detail.

The steel limit states are the welds required between the shear lug and the base plate as well as the bending of the shear lug. The concrete limit states are the bearing strength and the concrete breakout strength of the shear lug in shear.

The concrete bearing strength of a shear lug in shear is:

$$\phi V_{brg,sl} = \phi(1.7)f_c' A_{ef,sl}\psi_{brg,sl} \qquad \text{(from ACI 318, Equation 17.11.2.1)}$$

where

$A_{ef,sl}$ = effective bearing area of shear lug, in.^{2}

f_c' = concrete compressive strength, psi

ϕ = 0.65

$\psi_{brg,sl} = 1 + \dfrac{P_u}{nN_{sa}} \leq 1.0$ for applied axial tension (ACI 318, Eq. 17.11.2.2.1a)

= 1 for no applied axial load (ACI 318, Eq. 17.11.2.2.1b)

$= 1 + 4\dfrac{P_u}{A_{bp}f_c'} \leq 2.0$ for applied axial compression (ACI 318, Eq. 17.11.2.2.1c)

A_{bp} = area of the attachment base plate in contact with concrete or grout when loaded in compression, in.^{2}

N_{sa} = nominal strength of a single anchor or individual anchor in group of anchors as governed by the steel strength, lbf

P_u = factored axial force, positive for compression and negative for tension, lbf

n = number of anchors in tension

The concrete breakout strength of a shear lug in shear is:

$$\phi V_{cb} = \phi \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b \qquad \text{(ACI 318, Equation 17.7.2.1a)}$$

where

A_{Vc} = projected concrete failure area calculated per ACI 318, Section 17.11.3.1.1, in.^{2}

A_{Vco} = projected concrete failure area of a single anchor if not limited by corner influences, spacing, or member thicknesses, in.^{2}

V_b = basic concrete breakout strength in shear of a single anchor in cracked concrete, lbf

ϕ = 0.65 per ACI 318, Section 17.11.1.1.6

$\psi_{c,V}$ = breakout cracking factor used to modify shear strength of anchors based on the influence of cracks in concrete and presence or absence of supplementary reinforcement

$\psi_{ed,V}$ = breakout edge effect factor used to modify shear strength of anchors based on proximity to edges of concrete member

$\psi_{h,V}$ = breakout thickness factor used to modify shear strength of anchors in concrete members with $h_a \leq 1.5c_{a1}$

c_{a1} = distance from the bearing surface of the shear lug to the free edge of concrete, in.

h_a = thickness of member in which an anchor is located, measured parallel to anchor axis, in.

Additional considerations related to the use of shear lugs:

1. A minimum of four anchor rods must be provided when a shear lug connection is used. The anchor rods are not required to be designed to carry any shear unless welded to a common plate. Therefore, these anchors are not designed for steel strength in shear, concrete breakout strength in shear, and concrete pryout strength in shear.
2. The base plate and the anchor rods must be designed for the eccentricity resulting from bearing forces in the shear lug to the base plate. This can be of special concern when the base shears (most likely due to bracing forces) are large and bending from the bearing 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 thickness.
3. Multiple shear lugs may be used to resist large shear forces. ACI 318, Section 17.11.3.4, indicates the concrete breakout strength is to be determined for each potential breakout surface.

4. Base plates with shear lugs must have a minimum 1-in.-diameter hole along each of the long sides of the shear lug. This is to ensure proper concrete or grout consolidation around the shear lug. Nonshrink grout of flowable consistency should be used.
5. Typically, no interaction is considered between the anchor rods design and the shear lug design within the same base connection unless the anchor rods are welded to the baseplate.

The design of a shear lug is illustrated in Example 4.7-5.

EXAMPLE 4.7-5—Base Connection for Concentric Shear Load (Shear Lug Design)

A shear lug for a base connection is designed in this example for a concentric shear load.

Given:

The W14×90 column shown in Figure 4-26 is subjected only to a shear load due to wind. A shear lug will be designed to resist the shear force, and anchor rods will be provided to meet ACI 318 requirements. The column is supported on a concrete wall with 30 in. width and concrete compressive strength, f_c', equal to 4 ksi. The column is ASTM A992/A992M (2022c), and the plate is ASTM A572/A572M Grade 50 material. The anchor rods are ASTM F1554 Grade 36 material.

The required strength due to shear loads is:

LRFD	ASD
$V_u = 25.0$ kips	$V_a = 16.0$ kips

Solution:

From AISC *Manual* Tables 2-4, 2-5, and 2-6, the material properties are as follows:

W14×90
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi

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

From AISC *Manual* Table 1-1, the geometric properties of the column are as follows:

W14×90
$t_w = 0.440$ in.

Determine the anchor rod requirements

Although there is no externally applied axial tension at the baseplate, ACI 318, Section 17.11.1.1.2, requires that a minimum of four anchor rods be provided and that they be designed for the eccentricity present from the location of the externally applied shear to the bearing reaction on the shear lug. Try an 8-in.-wide shear lug with an effective embedment depth of 2.00 in. and four ¾-in.-diameter threaded rods for anchorage.

The eccentricity from the location of the applied shear is taken as the V_u from the bottom of the baseplate to the center of the effective embedment depth as follows:

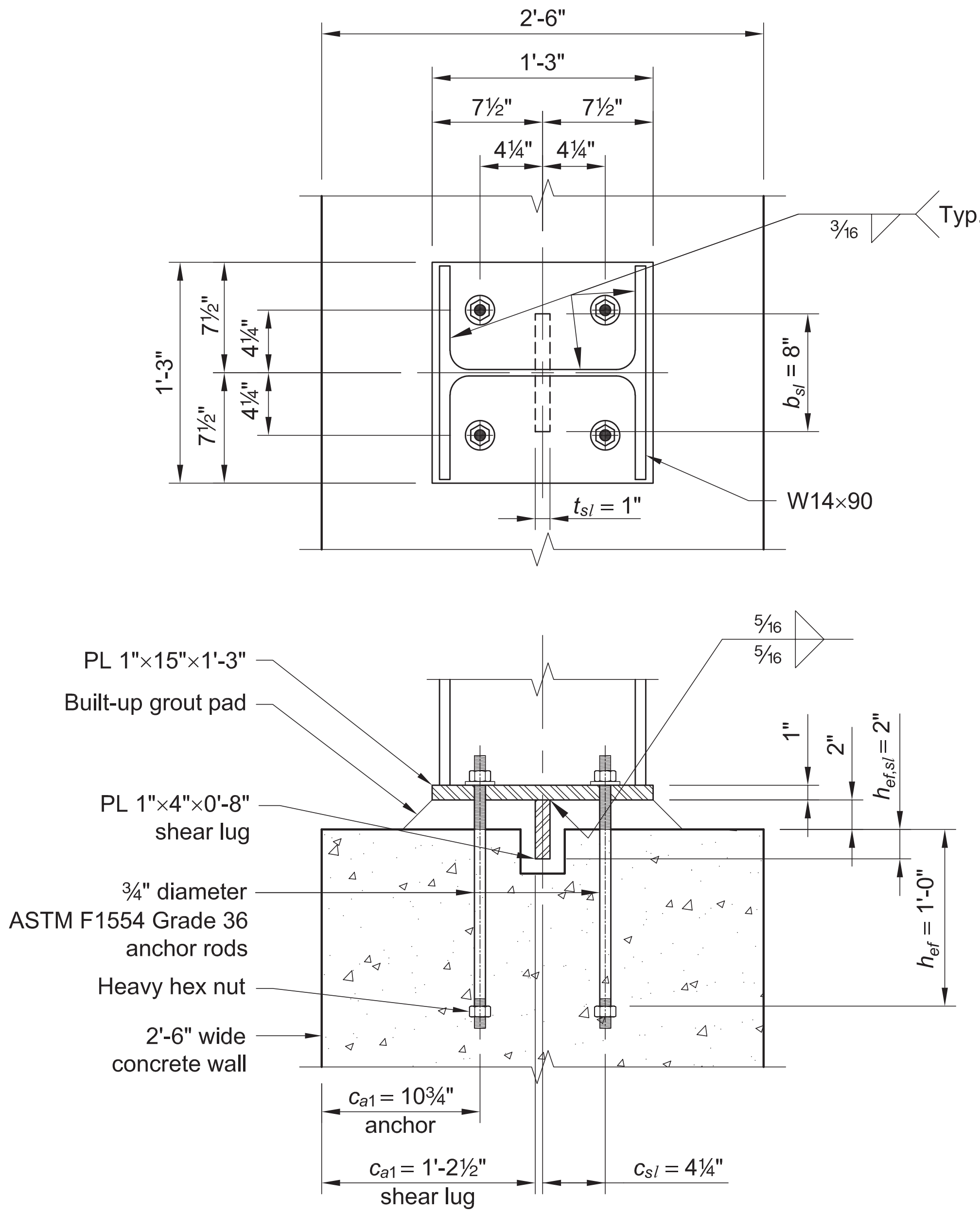


Fig. 4-26. Base connection as detailed in Example 4.7-5.

$$e = t_{grout} + \frac{h_{ef,sl}}{2}$$
$$= 2.00\text{ in.} + \frac{2.00\text{ in.}}{2}$$
$$= 3.00\text{ in.}$$

Use the large-moment baseplate procedure to determine the anchor rod tension from Section 4.3.6. Use baseplate dimensions with B = 15.0 in. and N = 15.0 in., and use an 8½-in.-square anchor rod pattern (s = 8.50 in.).

The moment resulting from the shear eccentricity is calculated by:

LRFD	ASD
$M_u = V_u e$ $= (25.0\text{ kips})(3.00\text{ in.})$ $= 75.0\text{ kip-in.}$	$M_a = V_a e$ $= (16.0\text{ kips})(3.00\text{ in.})$ $= 48.0\text{ kip-in.}$

Consider that $A_2/A_1 = 4.00$ and verify after forces are determined that this ratio exceeds 4.00. The nominal bearing strength may then be calculated as:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \qquad \text{(\textit{Spec}. Eq. J8-2)}$$

And the available bearing strength is then:

LRFD	ASD
$\frac{\phi_c P_p}{A_1} = \phi_c 0.85 f_c' \sqrt{A_2/A_1} \le \phi_c 1.7 f_c'$ $= 0.65(0.85)(4\text{ ksi})\sqrt{4.00} \le 0.65(1.7)(4\text{ ksi})$ $= 4.42\text{ ksi} \le 4.42\text{ ksi}$ $= 4.42\text{ ksi}$	$\frac{P_p}{\Omega_c A_1} = \frac{0.85 f_c' \sqrt{A_2/A_1}}{\Omega_c} \le \frac{1.7 f_c'}{\Omega_c}$ $= \frac{(0.85)(4\text{ ksi})\sqrt{4.00}}{2.31} \le \frac{(1.7)(4\text{ ksi})}{2.31}$ $= 2.94\text{ ksi} \le 2.94\text{ ksi}$ $= 2.94\text{ ksi}$

The distance between the center of the base plate and the tension anchor rods, f, is shown in Figure 4-8 and calculated by:

$$f = \frac{s}{2}$$
$$= \frac{8.50\text{ in.}}{2}$$
$$= 4.25\text{ in.}$$

LRFD	ASD
$q_{max} = f_{pu} B$ (from Eq. 4-37) $= (4.42\text{ ksi})(15.0\text{ in.})$ $= 66.3\text{ kip/in.}$	$q_{max} = f_{pa} B$ (from Eq. 4-37) $= (2.94\text{ ksi})(15.0\text{ in.})$ $= 44.1\text{ kip/in.}$

The bearing length, Y, is given by Equation 4-32:

LRFD	ASD
$Y = \left(f + \dfrac{N}{2}\right) - \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2M_u}{q_{max}}}$ $= \left(4.25 \text{ in.} + \dfrac{15.0 \text{ in.}}{2}\right)$ $- \sqrt{\left(4.25 \text{ in.} + \dfrac{15.0 \text{ in.}}{2}\right)^2 - \dfrac{2(75.0 \text{ kip-in.})}{66.3 \text{ kip/in.}}}$ $= 0.0967 \text{ in.}$ The resulting total anchor rod tension is then: $T_u = q_{max}Y$ $= (66.3 \text{ kips/in.})(0.0967 \text{ in.})$ $= 6.41 \text{ kips}$	$Y = \left(f + \dfrac{N}{2}\right) - \sqrt{\left(f + \dfrac{N}{2}\right)^2 - \dfrac{2M_a}{q_{max}}}$ $= \left(4.25 \text{ in.} + \dfrac{15.0 \text{ in.}}{2}\right)$ $- \sqrt{\left(4.25 \text{ in.} + \dfrac{15.0 \text{ in.}}{2}\right)^2 - \dfrac{2(48.0 \text{ kip-in.})}{44.1 \text{ kip/in.}}}$ $= 0.0930 \text{ in.}$ The resulting total anchor rod tension is then: $T_a = q_{max}Y$ $= (44.1 \text{ kips/in.})(0.0930 \text{ in.})$ $= 4.10 \text{ kips}$

The total anchor rod tension is distributed to two anchor rods:

LRFD	ASD
$r_u = \dfrac{T_u}{2 \text{ anchor rods}}$ $= \dfrac{6.41 \text{ kips}}{2 \text{ anchor rods}}$ $= 3.21 \text{ kips}$	$r_a = \dfrac{T_a}{2 \text{ anchor rods}}$ $= \dfrac{4.10 \text{ kips}}{2 \text{ anchor rods}}$ $= 2.05 \text{ kips}$

From Table 4-1, confirm the available rod tensile strength:

LRFD	ASD
$\phi R_n = 14.5 \text{ kips} > 3.21 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 9.69 \text{ kips} > 2.05 \text{ kips}$ **o.k.**

From Table 4-2, confirm the available anchor rod concrete pullout strength:

$\phi N_{pn} = 20.4 \text{ kips} > 3.21 \text{ kips}$ **o.k.**

Determine the minimum embedment of the anchor rods based on the requirements of ACI 318, Section 17.11.1.1.8, where h_{sl} is the shear lug embedment depth, and c_{sl} is determined considering that the shear lug and anchor rods are centered on the base plate and column:

$$\begin{aligned} h_{ef} &\geq 2.5h_{sl} \\ &= 2.5(2.00 \text{ in.}) \\ &= 5.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} c_{sl} &= \frac{s}{2} \\ &= \frac{8.50 \text{ in.}}{2} \\ &= 4.25 \text{ in.} \end{aligned}$$

$$
\begin{aligned}
h_{ef} &\geq 2.5c_{sl} \\
&= 2.5(4.25 \text{ in.}) \\
&= 10.6 \text{ in.}
\end{aligned}
$$

Use $h_{ef} = 12.0$ in.

Using ACI 318, Section 17.6.2, determine the concrete breakout capacity of the two anchors in tension.

$$
\begin{aligned}
N_b &= 16\lambda_a\sqrt{f_c'}\left(h_{ef}\right)^{5/3} && \text{(ACI 318, Eq. 17.6.2.2.3)} \\
&= 16(1.0)\sqrt{4{,}000 \text{ psi}}\,(12.0 \text{ in.})^{5/3}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right) \\
&= 63.6 \text{ kips}
\end{aligned}
$$

Because the tensile load is applied concentrically to the two anchors that are in tension, $e_N' = 0$ in. and:

$$
\begin{aligned}
\psi_{ec,N} &= \frac{1}{\left(1+\dfrac{e_N'}{1.5h_{ef}}\right)} \leq 1 && \text{(ACI 318, Eq. 17.6.2.3.1)} \\
&= \frac{1}{\left[1+\dfrac{0 \text{ in.}}{1.5(12.0 \text{ in.})}\right]} \\
&= 1.0
\end{aligned}
$$

The edge distance factor is calculated according to ACI 318, Section 17.6.2.4, as:

$$
\begin{aligned}
1.5h_{ef} &= 1.5(12.0 \text{ in.}) \\
&= 18.0 \text{ in.} \\
c_{a,min} &= \frac{(30.0 \text{ in.} - 8.50 \text{ in.})}{2} \\
&= 10.8 \text{ in.} < 18.0 \text{ in.}
\end{aligned}
$$

Therefore,

$$
\begin{aligned}
\psi_{ed,N} &= 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} && \text{(ACI 318, Eq. 17.6.2.4.1b)} \\
&= 0.7 + 0.3\frac{10.8 \text{ in.}}{1.5(12.0 \text{ in.})} \\
&= 0.880
\end{aligned}
$$

Because no analysis was performed to confirm if there will be cracking at service load levels, per ACI 318, Section 17.6.2.5:

$$\psi_{c,N} = 1.0$$

For cast-in-place concrete anchors, per ACI 318, Section 17.6.2.6:

$$\psi_{cp,N} = 1.0$$

The projected concrete failure area of a single anchor is given by:

$$
\begin{aligned}
A_{Nco} &= 9h_{ef}^2 && \text{(ACI 318, Eq. 17.6.2.1.4)} \\
&= 9(12.0 \text{ in.})^2 \\
&= 1{,}300 \text{ in.}^2
\end{aligned}
$$

The projected concrete failure area of the group is given by:

$$A_{Nc} = (c_{a1} + 1.5h_{ef})(1.5h_{ef} + s + 1.5h_{ef})$$
$$= [10.8 \text{ in.} + 1.5(12.0 \text{ in.})][1.5(12.0 \text{ in.}) + 8.50 \text{ in.} + 1.5(12.0 \text{ in.})]$$
$$= 1{,}280 \text{ in.}^2$$

The resulting concrete breakout strength in tension is given by:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{(ACI 318, Eq. 17.6.2.1b)}$$
$$= \frac{1{,}280 \text{ in.}^2}{1{,}300 \text{ in.}^2}(1.0)(0.880)(1.0)(1.0)(63.6 \text{ kips})$$
$$= 55.1 \text{ kips}$$

Without considering the addition of supplementary reinforcement to restrain concrete breakout, $\phi = 0.70$ per ACI 318, Table 17.5.3(b), and:

$$\phi N_{cbg} = 0.70(55.1 \text{ kips})$$
$$= 38.6 \text{ kips} > T_u = 6.41 \text{ kips} \quad \textbf{o.k.}$$

Because $h_{ef} \le 2.5c_{a1}$, side-face blowout is not applicable per ACI 318, Section 17.6.4.

Therefore, the four ¾-in.-diameter ASTM F1554 Grade 36 anchor rods with heavy hex nuts and 12.0 in. embedment meet the design requirements.

Determine the shear lug available strength limited by concrete bearing

The shear lug embedment must provide adequate area such that the concrete bearing strength exceeds the required strength from the applied wind loading. The resistance factor for bearing against shear lugs is given by ACI 318, Section 17.11.1.1.4, as $\phi = 0.65$ and the nominal bearing strength in shear is determined according to ACI 318, Sections 17.11.1.1.5 and 17.11.2.

The bearing area of the shear lug, $A_{ef,sl}$, is limited by the requirements of ACI 318, Section 17.11.2.1.1, as only extending a depth of two times the shear lug thickness. With a plate thickness greater than or equal to 1 in., the entire embedment depth will be effective for bearing.

The effective bearing area based on the trial lug size is given by:

$$A_{ef,sl} = b_{sl} h_{ef,sl}$$
$$= (8.00 \text{ in.})(2.00 \text{ in.})$$
$$= 16.0 \text{ in.}^2$$

Because there is no external axial load applied in this example:

$$\psi_{brg,sl} = 1.0 \quad \text{(ACI 318, Eq. 17.11.2.2.1b)}$$

The nominal bearing strength in shear of the shear lug is given by:

$$V_{brg,sl} = 1.7 f'_c A_{ef,sl} \psi_{brg,sl} \quad \text{(ACI 318, Eq. 17.11.2.1)}$$
$$= 1.7(4 \text{ ksi})(16.0 \text{ in.}^2)(1.0)$$
$$= 109 \text{ kips}$$

The available strength is therefore,

$$\phi V_{brg,sl} = 0.65(109 \text{ kips})$$
$$= 70.9 \text{ kips} > 25.0 \text{ kips} \quad \textbf{o.k.}$$

Verify the concrete breakout strength of the shear lug with the chosen shear lug dimensions

The resistance factor for concrete breakout strength of shear lugs is given by ACI 318, Section 17.11.1.1.6, as $\phi = 0.65$, and the nominal concrete breakout strength in shear is determined according to ACI 318, Sections 17.11.1.1.7 and 17.11.3.

Use a preliminary thickness of 1 in. and locate the shear lug in the center of a 30-in.-wide concrete wall. The edge distance from the face of the shear lug to the face of the wall in the direction of the shear force is calculated by:

$$
\begin{aligned}
c_{a1} &= \frac{b_w - t_{sl}}{2} \\
&= \frac{30.0\text{ in.} - 1.00\text{ in.}}{2} \\
&= 14.5\text{ in.}
\end{aligned}
$$

The projected concrete failure area, exclusive of the shear lug area, on the side face of the concrete wall is calculated per ACI 318, Section 17.11.3.1.1

$$
\begin{aligned}
A_{Vc} &= \underbrace{\left(h_{ef,sl} + 1.5c_{a1}\right)\left(b_{sl} + 1.5c_{a1} + 1.5c_{a1}\right)}_{\text{Gross concrete failure area}} - \underbrace{h_{ef,sl}b_{sl}}_{A_{ef,sl}} \\
&= \left[2.00\text{ in.} + 1.5(14.5\text{ in.})\right]\left[8.00\text{ in.} + 1.5(14.5\text{ in.}) + 1.5(14.5\text{ in.})\right] - (2.00\text{ in.})(8.00\text{ in.}) \\
&= 1{,}210\text{ in.}^2
\end{aligned}
$$

The projected area for a single anchor in a deep member with a distance from the edge of at least $1.5c_{a1}$ in the direction perpendicular to the shear is given by:

$$
\begin{aligned}
A_{Vco} &= 4.5\left(c_{a1}\right)^2 && \text{(ACI 318, Eq. 17.7.2.1.3)} \\
&= 4.5(14.5\text{ in.})^2 \\
&= 946\text{ in.}^2
\end{aligned}
$$

The basic concrete breakout strength of the shear lug is given by:

$$
\begin{aligned}
V_b &= 9\lambda_a\sqrt{f_c'}\left(c_{a1}\right)^{1.5} && \text{(ACI 318, Eq. 17.7.2.2.1b)} \\
&= 9(1.0)\sqrt{4{,}000\text{ psi}}\,(14.5\text{ in.})^{1.5}\left(\frac{1\text{ kip}}{1{,}000\text{ lbf}}\right) \\
&= 31.4\text{ kips}
\end{aligned}
$$

Because $c_{a2} \geq 1.5c_{a1}$, a reduction is not required per ACI 318, Section 17.7.2.4:

$$
\psi_{ed,V} = 1.0 \qquad \text{(ACI 318, Eq. 17.7.2.4.1a)}
$$

Because the bottom of the concrete shear breakout cone does not extend past a concrete edge, $\psi_{h,V} = 1.0$ per ACI 318, Section 17.7.2.6.1.

Because an analysis was not performed confirming that there will be no cracking at service load levels, and because supplementary reinforcement was not considered, $\psi_{c,V} = 1.0$ per ACI 318, Section 17.7.2.5.

The nominal concrete breakout strength of the shear lug is then given by:

$$
\begin{aligned}
V_{cb} &= \frac{A_{Vc}}{A_{Vco}}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b && \text{(ACI 318, Eq. 17.7.2.1a)} \\
&= \left(\frac{1{,}210\text{ in.}^2}{946\text{ in.}^2}\right)(1.0)(1.0)(1.0)(31.4\text{ kips}) \\
&= 40.2\text{ kips}
\end{aligned}
$$

The available shear breakout capacity may then be determined using $\phi = 0.65$ per ACI 318, Section 17.11.1.1.6:

$$\begin{aligned}\phi V_{cb} &= 0.65(40.2 \text{ kips})\\ &= 26.1 \text{ kips} > 25.0 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Determine the strength of the steel shear lug and the weld to the baseplate

The required strength of the shear lug was determined earlier as:

LRFD	ASD
$V_u = 25.0$ kips $M_u = 75.0$ kip-in.	$V_a = 16.0$ kips $M_a = 48.0$ kip-in.

The nominal shear strength of the connected shear lug element is given in AISC *Specification* Section J4.2. For the limit state of shear yielding:

$$R_n = 0.60F_yA_{gv} \qquad (\textit{Spec.}\text{ Eq. J4-3})$$

$$\begin{aligned}&= 0.60(50 \text{ ksi})(1.00 \text{ in.})(8.00 \text{ in.})\\ &= 240 \text{ kips}\end{aligned}$$

The available strength is then determined by:

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(240 \text{ kips})$ $= 240 \text{ kips} > 25.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{240 \text{ kips}}{1.50}$ $= 160 \text{ kips} > 16.0 \text{ kips}$ **o.k.**

For the limit state of shear rupture:

$$R_n = 0.60F_uA_{nv} \qquad (\textit{Spec.}\text{ Eq. J4-4})$$

$$\begin{aligned}&= 0.60(65 \text{ ksi})(1.00 \text{ in.})(8.00 \text{ in.})\\ &= 312 \text{ kips}\end{aligned}$$

The available strength is then determined by:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(312 \text{ kips})$ $= 234 \text{ kips} > 25.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{312 \text{ kips}}{2.00}$ $= 156 \text{ kips} > 16.0 \text{ kips}$ **o.k.**

The nominal flexural strength of the connected shear lug element is given in AISC *Specification* Section J4.5. For the limit state of flexural yielding:

$$\begin{aligned}M_n &= F_yZ\\ &= (50 \text{ ksi})\frac{(8.00 \text{ in.})(1.00 \text{ in.})^2}{4}\\ &= 100 \text{ kip-in.}\end{aligned}$$

The available strength is then determined by:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(100 \text{ kip-in.})$ $= 90.0 \text{ kip-in.} > 75.0 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{100 \text{ kip-in.}}{1.67}$ $= 59.9 \text{ kip-in.} > 48.0 \text{ kip-in.}$ **o.k.**

For the limit state of flexural rupture:

$$M_n = F_u Z_{net}$$
$$= (65 \text{ ksi})\frac{(8.00 \text{ in.})(1.00 \text{ in.})^2}{4}$$
$$= 130 \text{ kip-in.}$$

The available strength is then determined by:

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(130 \text{ kip-in.})$ $= 97.5 \text{ kip-in.} > 75.0 \text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{130 \text{ kip-in.}}{2.00}$ $= 65.0 \text{ kip-in.} > 48.0 \text{ kip-in.}$ **o.k.**

Use AISC *Manual* Equation 9-1 to check the flexural and shear yielding interaction:

LRFD	ASD
$\dfrac{M_r}{M_c} + \left(\dfrac{P_r}{P_c}\right)^2 + \left(\dfrac{V_r}{V_c}\right)^4 \le 1.0$ $\dfrac{75.0 \text{ kip-in.}}{90.0 \text{ kip-in.}} + 0 + \left(\dfrac{25.0 \text{ kips}}{240 \text{ kips}}\right)^4 \le 1.0$ $0.833 < 1.0$ **o.k.**	$\dfrac{M_r}{M_c} + \left(\dfrac{P_r}{P_c}\right)^2 + \left(\dfrac{V_r}{V_c}\right)^4 \le 1.0$ $\dfrac{48.0 \text{ kip-in.}}{59.9 \text{ kip-in.}} + 0 + \left(\dfrac{16.0 \text{ kips}}{160 \text{ kips}}\right)^4 \le 1.0$ $0.801 < 1.0$ **o.k.**

Determine the weld size from the shear lug to the base plate

The shear load will be distributed equally to the fillet welds on each side of the shear lug. The moment will be resolved as a force couple and applied at the centroids of the two welds. The minimum fillet weld size per AISC *Specification* Table J2.4 is ⁵⁄₁₆ in. Determine the required strength of the weld.

$$w = \tfrac{5}{16} \text{ in. (fillet weld)}$$
$$a = t_{sl} + \frac{2w}{3}$$
$$= 1.00 \text{ in.} + \frac{2(\tfrac{5}{16} \text{ in.})}{3}$$
$$= 1.21 \text{ in.}$$

LRFD	ASD
$r_{uv} = \dfrac{V_u}{(2\text{ welds})b_{sl}}$ $= \dfrac{25.0\text{ kips}}{(2\text{ welds})(8.00\text{ in.})}$ $= 1.56\text{ kip/in.}$ $r_{um} = \dfrac{M_u}{ab_{sl}}$ $= \dfrac{75.0\text{ kip-in.}}{(1.21\text{ in.})(8.00\text{ in.})}$ $= 7.75\text{ kip/in.}$ $r_u = \sqrt{r_{uv}^2 + r_{um}^2}$ $= \sqrt{(1.56\text{ kip/in.})^2 + (7.75\text{ kip/in.})^2}$ $= 7.91\text{ kip/in.}$	$r_{av} = \dfrac{V_a}{(2\text{ welds})b_{sl}}$ $= \dfrac{16.0\text{ kips}}{(2\text{ welds})(8.00\text{ in.})}$ $= 1.00\text{ kip/in.}$ $r_{am} = \dfrac{M_a}{ab_{sl}}$ $= \dfrac{48.0\text{ kip-in.}}{(1.21\text{ in.})(8.00\text{ in.})}$ $= 4.96\text{ kip/in}$ $r_a = \sqrt{r_{av}^2 + r_{am}^2}$ $= \sqrt{(1.00\text{ kip/in.})^2 + (4.96\text{ kip/in.})^2}$ $= 5.06\text{ kip/in.}$

Determine the directional strength increase and available strength for welds loaded at $\theta = 90°$ to its longitudinal axis.

$$k_{ds} = (1.0 + 0.50\sin^{1.5}\theta) \qquad (Spec.\ \text{Eq. J2-5})$$

$$= (1.0 + 0.50\sin^{1.5}90°)$$

$$= 1.50$$

$$R_n = F_{nw}A_{we}k_{ds} \qquad (Spec.\ \text{Eq. J2-4})$$

$$= 0.6(70\text{ ksi})\left(\frac{5/16\text{ in.}}{\sqrt{2}}\right)(1.50)$$

$$= 13.9\text{ kip/in.}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(13.9\text{ kip/in.})$ $= 10.4\text{ kip/in.} > 7.91\text{ kip/in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{13.9\text{ kip/in.}}{2.00}$ $= 6.95\text{ kip/in.} > 5.06\text{ kip/in.}$ **o.k.**

Alternatively, the welds may be designed using the instantaneous center of rotation method or a plastic mechanism type analysis that accounts for bearing of the shear lug against the base plate in addition to the welds. Accounting for bearing of the plate may result in a reduction in weld strength when the connection is analyzed elastically because the neutral axis shifts toward the weld in compression increasing the stress in the tension weld.

Check the base plate for local bending due to shear lug

In this example, a concentrated moment is applied at the shear lug location that is resisted by tension at two of the anchor rods and a compression bearing block at the edge of the base plate. The maximum bending moment occurs at the location of the shear lug and is calculated as follows. In this example, the stiffening effect of the column cross section is conservatively neglected.

LRFD	ASD
$M_u = \max\begin{cases} T_u f, \\ (q_{max}Y)\left(\dfrac{N}{2} - \dfrac{Y}{2}\right) \end{cases}$ $= \max\begin{cases} (6.41 \text{ kips})(4.25 \text{ in.}), \\ \left[\begin{array}{l}(66.3 \text{ kips/in.})(0.0967 \text{ in.})\times \\ \qquad \left(\dfrac{15.0 \text{ in.}}{2} - \dfrac{0.0967 \text{ in.}}{2}\right)\end{array}\right] \end{cases}$ $= \max\begin{cases} 27.2 \text{ kip-in.}, \\ 47.8 \text{ kip-in.} \end{cases}$ $= 47.8 \text{ kip-in.}$	$M_a = \max\begin{cases} T_a f, \\ (q_{max}Y)\left(\dfrac{N}{2} - \dfrac{Y}{2}\right) \end{cases}$ $= \max\begin{cases} (4.10 \text{ kips})(4.25 \text{ in.}), \\ \left[\begin{array}{l}(44.1 \text{ kips/in.})(0.0930 \text{ in.})\times \\ \qquad \left(\dfrac{15.0 \text{ in.}}{2} - \dfrac{0.0930 \text{ in.}}{2}\right)\end{array}\right] \end{cases}$ $= \max\begin{cases} 17.4 \text{ kip-in.}, \\ 30.6 \text{ kip-in.} \end{cases}$ $= 30.6 \text{ kip-in.}$

Using an effective width equal to the shear lug width, the flexural strength of the baseplate can conservatively be determined according to AISC *Specification* Section J4.5. It is recommended that, at a minimum, the baseplate thickness equal the thickness of the shear lug.

$$\begin{aligned} Z_y &= \frac{b_{sl}t_{bp}}{4} \\ &= \frac{(8.00 \text{ in.})(1.00 \text{ in.})^2}{4} \\ &= 2.00 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} M_n &= F_y Z_y \\ &= (50 \text{ ksi})(2.00 \text{ in.}^3) \\ &= 100 \text{ kip-in.} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(100 \text{ kip-in.})$ $= 90.0 \text{ kip-in.} > 47.8 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{100 \text{ kip-in.}}{1.67}$ $= 59.9 \text{ kip-in.} > 30.6 \text{ kip-in.}$ **o.k.**

Design column web-to-base plate weld

The shear force will be transferred through the weld from the web of the column to the base plate. For a W14×90 column with a web thickness of 0.440 in., the minimum weld per AISC *Specification* Table J2.4 is 3⁄16 in. With a weld on each side of the web with a length equal to the T dimension of 10 in., the required weld strength is determined as follows:

LRFD	ASD
$r_u = \dfrac{V_u}{T(2 \text{ welds})}$ $= \dfrac{25.0 \text{ kips}}{(10 \text{ in.})(2 \text{ welds})}$ $= 1.25 \text{ kip/in.}$	$r_a = \dfrac{V_a}{T(2 \text{ welds})}$ $= \dfrac{16.0 \text{ kips}}{(10 \text{ in.})(2 \text{ welds})}$ $= 0.800 \text{ kip/in.}$

Because the applied weld force is parallel to the longitudinal weld axis, $k_{ds} = 1.0$ and:

$$R_n = F_{nw} A_{we} k_{ds} \qquad (Spec.\ \text{Eq. J2-4})$$

$$= 0.6(70\text{ ksi})\left(\frac{\frac{3}{16}\text{ in.}}{\sqrt{2}}\right)(1.0)$$

$$= 5.57\text{ kip/in.}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(5.57\text{ kip/in.})$ $= 4.18\text{ kip/in.} > 1.25\text{ kip/in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{5.57\text{ kip/in.}}{2.00}$ $= 2.79\text{ kip/in.} > 0.800\text{ kip/in.}$ **o.k.**

The local shear rupture capacity of the column web is given by:

$$R_n = 0.60 F_u A_{nv} \qquad (Spec.\ \text{Eq. J4-4})$$

$$= 0.60(65\text{ ksi})(0.440\text{ in.})(10.0\text{ in.})$$

$$= 172\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(172\text{ kips})$ $= 129\text{ kips} > 25.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{172\text{ kips}}{2.00}$ $= 86.0\text{ kips} > 16.0\text{ kips}$ **o.k.**

EXAMPLE 4.7-6—Base Connection for Anchor Rods Resisting Combined Tension and Shear

An exposed base connection is designed in this example that considers anchor rods subjected to combined tension and shear. The anchor rods are checked considering anchor rod bending that results from the transfer of the shear force through plate washers.

Given:

Determine the required size of four anchor rods for the W10×45 column shown in Figure 4-27, using the anchor rods to resist the wind shear. Use a base plate thickness of 1 in. Only the steel limit states are evaluated in this example. The base plate is ASTM A572/A572M Grade 50, and the anchor rods are ASTM 1554 Grade 36 material.

The nominal wind shear force, 1.0*W*, is 38.0 kips.

Solution:

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

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

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.0(38.0 \text{ kips})$ $= 38.0 \text{ kips}$	$V_a = 0.6(38.0 \text{ kips})$ $= 22.8 \text{ kips}$

From Chapter 2 of ASCE/SEI 7, the required strength due to uplift on the column, N_u or N_a, is:

LRFD	ASD
$N_u = -0.9P_{DL} + 1.0P_{1.0W}$ $= -0.9(22.0 \text{ kips}) + 1.0(93.0 \text{ kips})$ $= 73.2 \text{ kips}$	$N_a = -0.6P_{DL} + 0.6P_{1.0W}$ $= -0.6(22.0 \text{ kips}) + 0.6(93.0 \text{ kips})$ $= 42.6 \text{ kips}$

A total of four anchor rods are used. Plate washers with standard holes are welded to the top of the base plate, and the concrete is reinforced for shear breakout so that the shear can be transferred to all four anchor rods. Try four 1⅛-in.-diameter anchors. For combined shear and tension, the anchor rods must meet the provisions of AISC *Specification* Section J3.8.

LRFD	ASD
$f_u \le \phi F'_{nt} = \phi\left(1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{uv}\right) \le \phi F_{nt}$ where $\phi = 0.75$	$f_a \le \dfrac{F'_{nt}}{\Omega} = \dfrac{\left(1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{av}\right)}{\Omega} \le \dfrac{F_{nt}}{\Omega}$ where $\Omega = 2.00$

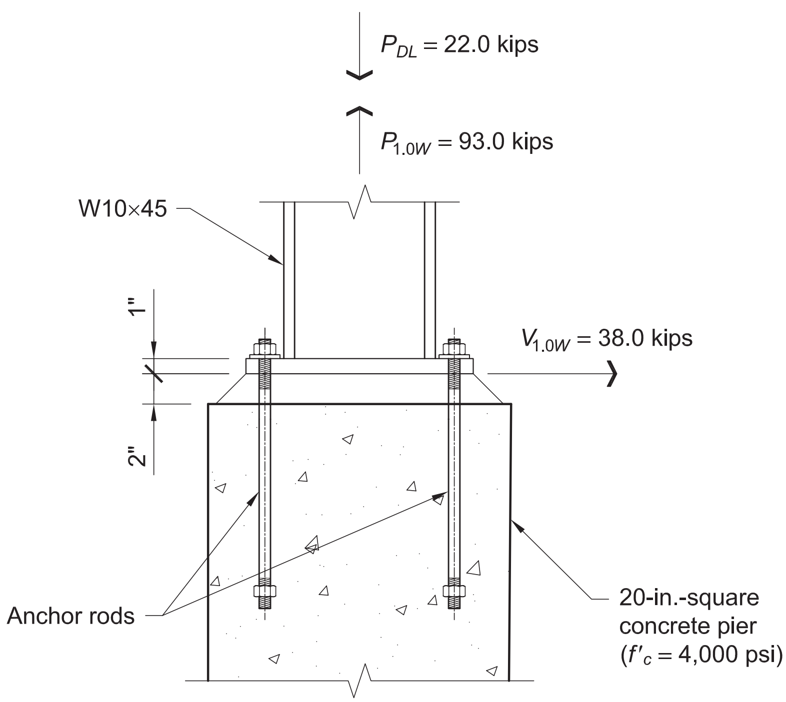


Fig. 4-27. Applied loading and base connection configuration used in Example 4.7-6.

Using the anchor area, $A_b = 0.994$ in.^{2} from Table 4-1, the shear stress in the anchor rods is calculated by:

LRFD	ASD
$f_{uv} = \dfrac{38.0 \text{ kips}}{4(0.994 \text{ in.}^2)}$ $= 9.56$ ksi	$f_{av} = \dfrac{22.8 \text{ kips}}{4(0.994 \text{ in.}^2)}$ $= 5.73$ ksi

The tensile stress in the anchor rods comes from both the axial tensile force and the tension from bending.

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

Determine the anchor rod diameter

The lever arm can be taken as half the distance from the center of the plate washer to the top of the grout. The base plate is 1.00 in. thick. Try a plate washer thickness of ⅜ in. and an anchor rod diameter of 1⅛ in.

$$\text{Lever arm} = \frac{1.00 \text{ in.} + (3/8 \text{ in.}/2)}{2}$$
$$= 0.594 \text{ in.}$$

Thus,

LRFD	ASD
$M_u = \dfrac{(38.0 \text{ kips})(0.594 \text{ in.})}{4}$ $= 5.64$ kip-in.	$M_a = \dfrac{(22.8 \text{ kips})(0.594 \text{ in.})}{4}$ $= 3.39$ kip-in.

The stress in the rod due to bending equals

$$f_{rb} = \frac{M_r}{Z}$$

where

$$Z = \frac{d^3}{6}$$
$$= \frac{(1\tfrac{1}{8} \text{ in.})^3}{6}$$
$$= 0.237 \text{ in.}^3$$

LRFD	ASD
$f_{ub} = \dfrac{5.64 \text{ kip-in.}}{0.237 \text{ in.}^3}$ $= 23.8$ ksi	$f_{ab} = \dfrac{3.39 \text{ kip-in.}}{0.237 \text{ in.}^3}$ $= 14.3$ ksi

The combined shear and tensile strength of the anchor rods is determined by AISC *Specification* Section J3.8 as follows.

LRFD	ASD
The axial stress in the rods is: $$f_{ua} = \frac{N_u}{A} = \frac{73.2 \text{ kips}}{4(0.994 \text{ in.}^2)} = 18.4 \text{ ksi}$$	The axial stress in the rods is: $$f_{aa} = \frac{N_a}{A} = \frac{42.6 \text{ kips}}{4(0.994 \text{ in.}^2)} = 10.7 \text{ ksi}$$
The total tensile stress is: $$f_{ut} = 23.8 \text{ ksi} + 18.4 \text{ ksi} = 42.2 \text{ ksi}$$	The total tensile stress is: $$f_{at} = 14.3 \text{ ksi} + 10.7 \text{ ksi} = 25.0 \text{ ksi}$$
Combined shear and tensile strength: $$F_{nt} = 0.75F_u = (0.75)(58 \text{ ksi}) = 43.5 \text{ ksi}$$ $$F_{nv} = 0.450F_u = (0.450)(58 \text{ ksi}) = 26.1 \text{ ksi}$$ $$\phi F'_{nt} = \phi\left(1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv}\right) \le \phi F_{nt}$$ $$= 0.75\left[(1.3)(43.5 \text{ ksi}) - \frac{43.5 \text{ ksi}(9.56 \text{ ksi})}{(0.75)(26.1 \text{ ksi})}\right] = 26.5 \text{ ksi} \le (0.75)(43.5 \text{ ksi}) = 32.6 \text{ ksi}$$ 42.2 ksi > 26.5 ksi **n.g.**	Combined shear and tensile strength: $$F_{nt} = 0.75F_u = (0.75)(58 \text{ ksi}) = 43.5 \text{ ksi}$$ $$F_{nv} = 0.450F_u = (0.450)(58 \text{ ksi}) = 26.1 \text{ ksi}$$ $$\frac{F'_{nt}}{\Omega} = \frac{\left(1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv}\right)}{\Omega} \le \frac{F_{nt}}{\Omega}$$ $$= \frac{\left[(1.3)(43.5 \text{ ksi}) - \frac{2.00(43.5 \text{ ksi})(5.73 \text{ ksi})}{(26.1 \text{ ksi})}\right]}{2.00} = 18.7 \text{ ksi} \le \frac{(43.5 \text{ ksi})}{2.00} = 21.8 \text{ ksi}$$ 25.0 ksi > 18.7 ksi **n.g.**

Try four 1½-in.-diameter rods. The plastic section modulus of the rod is given by:

$$Z = \frac{d^3}{6} = \frac{(1\tfrac{1}{2} \text{ in.})^3}{6} = 0.563 \text{ in.}^3$$

LRFD	ASD
Shear stress: $$f_{uv} = \frac{38.0\text{ kips}}{4\left(1.77\text{ in.}^2\right)} = 5.37\text{ ksi}$$ Flexural stress: $$f_{ub} = \frac{5.64\text{ kip-in.}}{0.563\text{ in.}^3} = 10.0\text{ ksi}$$	Shear stress: $$f_{av} = \frac{22.8\text{ kips}}{4\left(1.77\text{ in.}^2\right)} = 3.22\text{ ksi}$$ Flexural stress: $$f_{ab} = \frac{3.39\text{ kip-in.}}{0.563\text{ in.}^3} = 6.02\text{ ksi}$$

LRFD	ASD
The axial stress in the rods is: $$f_{ua} = \frac{N_u}{A} = \frac{73.2\text{ kips}}{4\left(1.77\text{ in.}^2\right)} = 10.3\text{ ksi}$$ The total tensile stress is: $$f_{ut} = 10.0\text{ ksi} + 10.3\text{ ksi} = 20.3\text{ ksi}$$ Combined shear and tensile strength: $$\phi F'_{nt} = 0.75\left[(1.3)(43.5\text{ ksi}) - \frac{43.5\text{ ksi}(5.37\text{ ksi})}{(0.75)(26.1\text{ ksi})}\right] = 33.5\text{ ksi} \leq (0.75)(43.5\text{ ksi}) = 32.6\text{ ksi}$$ $$f_{ut} = 20.3\text{ ksi} < \phi F'_{nt} = 32.6\text{ ksi} \quad \textbf{o.k.}$$	The axial stress in the rods is: $$f_{aa} = \frac{N_a}{A} = \frac{42.6\text{ kips}}{4\left(1.77\text{ in.}^2\right)} = 6.02\text{ ksi}$$ The total tensile stress is: $$f_{at} = 6.02\text{ ksi} + 6.02\text{ ksi} = 12.0\text{ ksi}$$ Combined shear and tensile strength: $$\frac{F'_{nt}}{\Omega} = \frac{\left[(1.3)(43.5\text{ ksi}) - \frac{2.00(43.5\text{ ksi})(3.22\text{ ksi})}{(26.1\text{ ksi})}\right]}{2.00} = 22.9\text{ ksi} \leq \frac{(43.5\text{ ksi})}{2.00} = 21.8\text{ ksi}$$ $$f_{at} = 12.0\text{ ksi} < \frac{F'_{nt}}{\Omega} = 21.8\text{ ksi} \quad \textbf{o.k.}$$

Use four 1½-in.-diameter rods, ASTM F1554 Grade 36.

Determine the plate washer thickness

The bearing force per rod is:

LRFD	ASD
$$R_u = \frac{38.0\text{ kips}}{4} = 9.50\text{ kips}$$	$$R_a = \frac{22.8\text{ kips}}{4} = 5.70\text{ kips}$$

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

$$R_n = 3.0dtF_u \qquad (Spec.\ \text{Eq. J3-6b})$$

$$R_n = 1.5l_c tF_u \qquad (Spec.\ \text{Eq. J3-6d})$$

Use an ASTM A572/A572M Grade 50, ⅜-in.-thick plate by 4-in.-diameter washer per Table 4-3 in Section 4.5.3, using the recommendation for a 1½-in.-diameter anchor rod with a standard hole (1⅝ in.).

The bearing strength per anchor rod is therefore:

$$\begin{aligned} R_n &= 3.0dtF_u \qquad (Spec.\ \text{Eq. J3-6b}) \\ &= 3.0(1\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.})(65\text{ ksi}) \\ &= 110\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(110\text{ kips})$ $= 82.5\text{ kips} > 9.50\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{110\text{ kips}}{2.00}$ $= 55.0\text{ kips} > 5.70\text{ kips}$ **o.k.**

$$\begin{aligned} l_c &= \frac{d_w - d_h}{2} \\ &= \frac{(4.00\text{ in.}) - (1\tfrac{5}{8}\text{ in.})}{2} \\ &= 1.19\text{ in.} \end{aligned}$$

The nominal tearout strength per anchor rod is therefore:

$$\begin{aligned} R_n &= 1.5l_c tF_u \qquad (Spec.\ \text{Eq. J3-6d}) \\ &= 1.5(1.19\text{ in.})(\tfrac{3}{8}\text{ in.})(65\text{ ksi}) \\ &= 43.5\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(43.5\text{ kips})$ $= 32.6\text{ kips} > 9.50\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{43.5\text{ kips}}{2.00}$ $= 21.8\text{ kips} > 5.70\text{ kips}$ **o.k.**

Due to the size of the rods, they will have to be positioned beyond the column flanges. In locating the anchor rods, consideration should be given to the tolerances of the anchor rod placement, weld access, and weld shelf dimensions. Additionally, when welds are used to transfer shear from the plate washers to the base plate, they should be designed for the required shear strength and the minimum weld requirements of the AISC *Specification*.

EXAMPLE 4.7-7—Base Connection at Brace Producing Combined Tension and Shear

An exposed base connection is designed in this example that considers shear transfer through a welded setting plate to preclude anchor rod bending. The concrete anchorage capacity is confirmed using ACI 318.

Given:

A base connection for a W21×83 column is subjected to wind forces from a tension-only brace as illustrated in Figure 4-28. The base plate plan view is shown in Figure 4-29. Determine the anchorage requirements and confirm the capacity of the base plate and column-to-base plate weld. The connection is located away from any concrete edges. The concrete compressive strength, f_c', is 5,000 psi. The plate is ASTM A572/A572M Grade 50, and anchor rods are ASTM F1554 Grade 36 material.

Solution:

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

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

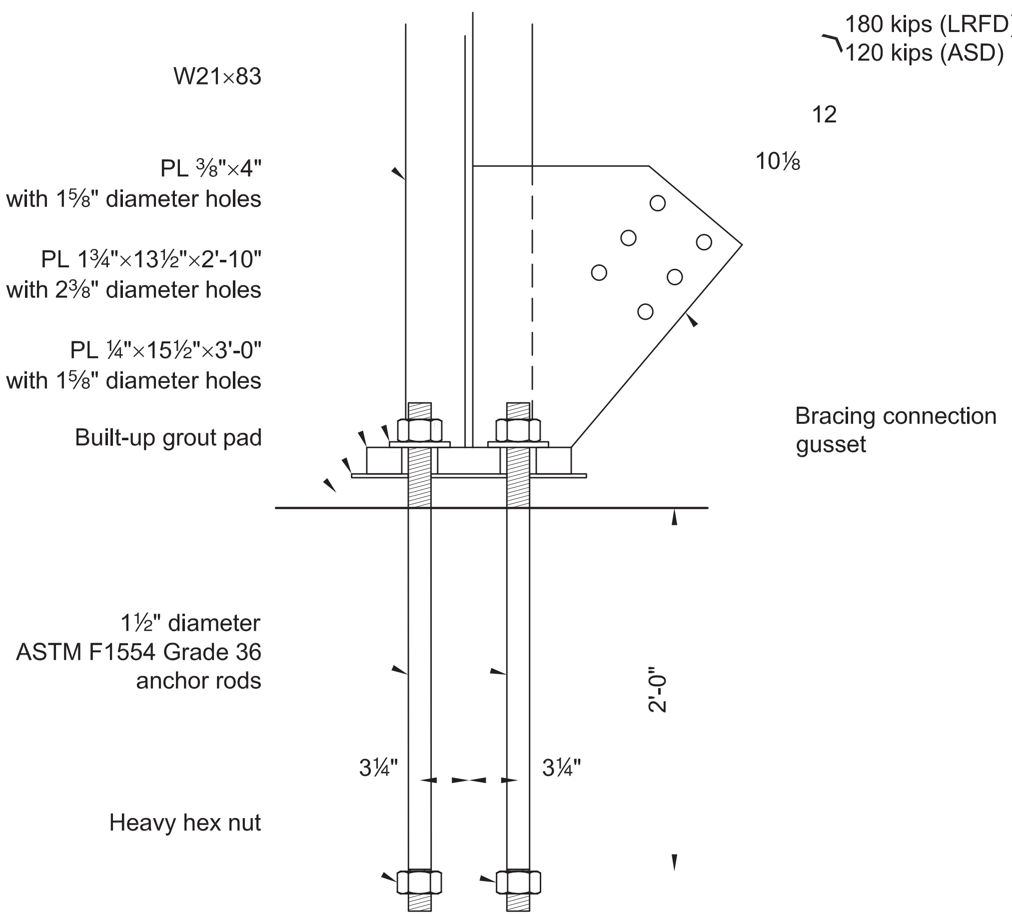


Fig. 4-28. Base connection as detailed in Example 4.7-7.

From AISC *Manual* Table 1-1, the geometric properties of the column are as follows:

W21×83
$t_w = 0.515$ in.

Determine the required strength for the anchor group

Resolve the brace force into shear and tensile forces on the anchor group.

LRFD	ASD
$V_u = (180 \text{ kips})\left(\dfrac{10.125}{\sqrt{10.125^2+12^2}}\right)$ $= 116$ kips $N_u = (180 \text{ kips})\left(\dfrac{12}{\sqrt{10.125^2+12^2}}\right)$ $= 138$ kips	$V_a = (120 \text{ kips})\left(\dfrac{10.125}{\sqrt{10.125^2+12^2}}\right)$ $= 77.4$ kips $N_a = (120 \text{ kips})\left(\dfrac{12}{\sqrt{10.125^2+12^2}}\right)$ $= 91.7$ kips

Because a setting plate with standard holes will be field welded to the baseplate and there are no adjacent edges that need to be considered for concrete breakout in shear, the shear will be distributed equally to all eight anchor rods. Similarly, because the anchor rods are concentric with the forces, the tension loading will also be equally distributed to all eight anchor rods. Determine the required strength in tension and shear for the anchor rods:

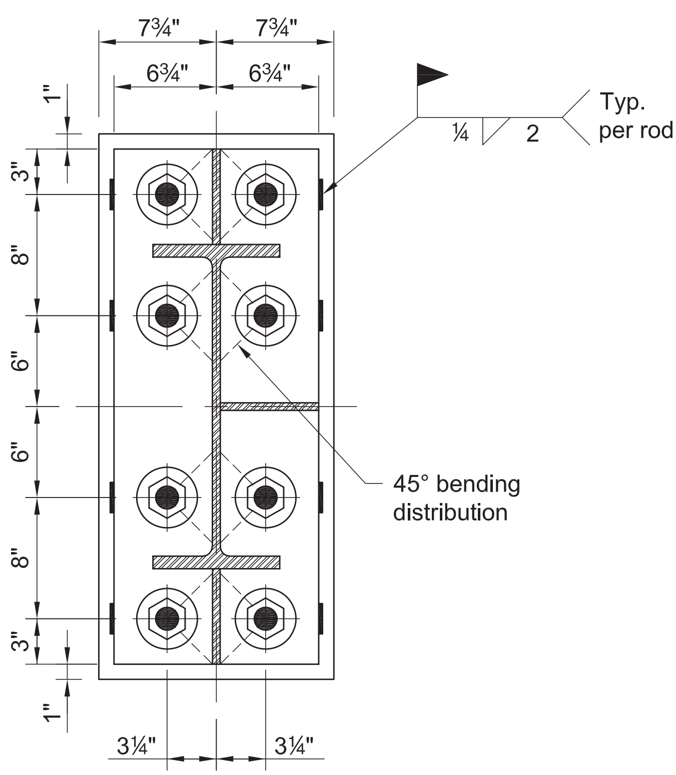


Fig. 4-29. Base connection plan view as detailed in Example 4.7-7.

LRFD	ASD
$V_{ua,i} = \dfrac{116 \text{ kips}}{8 \text{ anchor rods}}$ $= 14.5 \text{ kips}$ $N_{ua,i} = \dfrac{138 \text{ kips}}{8 \text{ anchor rods}}$ $= 17.3 \text{ kips}$	$V_{aa,i} = \dfrac{77.4 \text{ kips}}{8 \text{ anchor rods}}$ $= 9.68 \text{ kips}$ $N_{aa,i} = \dfrac{91.7 \text{ kips}}{8 \text{ anchor rods}}$ $= 11.5 \text{ kips}$

Determine the available steel strength in tension of the anchor rods

The available strength of the steel in tension is determined according to ACI 318, Section 17.6.1. As discussed previously, the AISC *Specification* provides similar capacities.

From Table 4-1, $A_{se,N} = 1.41 \text{ in.}^2$

$$N_{sa} = A_{se,N} f_{uta} \qquad \text{(ACI 318, Eq. 17.6.1.2)}$$
$$= (1.41 \text{ in.}^2)(58 \text{ ksi})$$
$$= 81.8 \text{ kips}$$

For a ductile steel element per ACI 318, Table 17.5.3(a), $\phi = 0.75$ and

$$\phi N_{sa} = 0.75(81.8 \text{ kips})$$
$$= 61.4 \text{ kips} > 17.3 \text{ kips} \quad \textbf{o.k.}$$

Determine the concrete breakout strength in tension

The available concrete breakout strength in tension is determined according to ACI 318, Section 17.6.2. Because the maximum spacing among any anchor in the group is less than $3.0h_{ef}$, all the anchors will act as a group for concrete breakout in tension. Because there is no edge within $1.5h_{ef}$ of any anchor, no reduction for edge distance will occur.

$$A_{Nc} = (1.5h_{ef} + 8.00 \text{ in.} + 6.00 \text{ in.} + 6.00 \text{ in.} + 8.00 \text{ in.} + 1.5h_{ef})(1.5h_{ef} + 3.25 \text{ in.} + 3.25 \text{ in.} + 1.5h_{ef})$$
$$= \left\{ \begin{array}{l} [1.5(24.0 \text{ in.}) + 8.00 \text{ in.} + 6.00 \text{ in.} + 6.00 \text{ in.} + 8.00 \text{ in.} + 1.5(24.0 \text{ in.})] \times \\ [1.5(24.0 \text{ in.}) + 3.25 \text{ in.} + 3.25 \text{ in.} + 1.5(24.0 \text{ in.})] \end{array} \right\}$$
$$= 7{,}850 \text{ in.}^2$$

$$A_{Nco} = 9h_{ef}^2 \qquad \text{(ACI 318, Eq. 17.6.2.1.4)}$$
$$= 9(24.0 \text{ in.})^2$$
$$= 5{,}180 \text{ in.}^2$$

For an anchor with an embedment 11.0 in. $\le h_{ef} \le$ 25.0 in.,

$$N_b = 16\lambda_a \sqrt{f'_c}\,(h_{ef})^{5/3} \qquad \text{(ACI 318, Eq. 17.6.2.2.3)}$$
$$= 16(1.0)\sqrt{5{,}000 \text{ psi}}\,(24.0 \text{ in.})^{5/3}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right)$$
$$= 226 \text{ kips}$$

For an anchor group concentrically loaded,

$$e'_N = 0 \text{ in.}$$

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{e'_N}{1.5h_{ef}}\right)} \le 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.3.1)}$$

$$= \frac{1}{\left[1+\dfrac{0 \text{ in.}}{1.5(24.0 \text{ in.})}\right]}$$

$$= 1.0$$

Because there are no adjacent edges, $c_{a,min} \ge 1.5h_{ef}$ and

$$\psi_{ed,N} = 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.4.1a)}$$

Because no analysis was performed to confirm there would be no cracking at service load levels per ACI 318, Section 17.6.2.5,

$$\psi_{c,N} = 1.0$$

For a cast-in-place anchor rod, the breakout splitting factor is determined per ACI 318, Section 17.6.2.6,

$$\psi_{cp,N} = 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.6.1a)}$$

The resulting concrete breakout strength of the anchor group in tension is determined by ACI 318, Section 17.6.2,

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

$$= \left(\frac{7{,}850 \text{ in.}^2}{5{,}180 \text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(226 \text{ kips})$$

$$= 342 \text{ kips}$$

Because no supplementary reinforcement was specified, $\phi = 0.70$ per ACI 318, Table 17.5.3(b), and

$$\phi N_{cbg} = (0.70)(342 \text{ kips})$$

$$= 239 \text{ kips} > 138 \text{ kips} \quad \textbf{o.k.}$$

Determine the concrete pullout strength in tension

The available concrete pullout strength in tension is determined according to ACI 318, Section 17.6.3.

$$N_p = 8A_{brg}f'_c \qquad \text{(ACI 318, Eq. 17.6.3.2.2a)}$$

$$= 8(3.12 \text{ in.}^2)(5 \text{ ksi})$$

$$= 125 \text{ kips}$$

Because no analysis was performed to confirm there would be no cracking at service levels per ACI 318, Section 17.6.3.3.1(b),

$$\psi_{c,P} = 1.0$$

The nominal pullout strength of a single anchor in tension is determined per ACI 318, Section 17.6.3.1, as

$$N_{pn} = \psi_{c,P}N_p \qquad \text{(ACI 318, Eq. 17.6.3.1)}$$

$$= 1.0(125 \text{ kips})$$

$$= 125 \text{ kips}$$

The resistance factor from ACI 318, Table 17.5.3(c), is $\phi = 0.70$, and the resulting available strength is determined as:

$$\begin{aligned}\phi N_{pn} &= 0.70(125 \text{ kips}) \\ &= 87.5 \text{ kips} > 17.3 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Determine the concrete side-face blowout strength in tension of the anchor group

The available concrete side-face blowout strength in tension of the anchor group is determined according to ACI 318, Section 17.6.4.

Because there are no cases with anchor rods close to an edge ($h_{ef} > 2.5c_{a1}$), side-face blowout is not applicable.

Determine the available steel strength in shear

The available strength of the steel in shear is determined according to ACI 318, Section 17.7.1. Because a built-up grout pad is present, the nominal strength is multiplied by 0.80 per ACI 318, Section 17.7.1.2.1.

$$V_{sa} = 0.80(0.6A_{se,V}f_{uta}) \qquad \text{(ACI 318, Eq. 17.7.1.2b)}$$

$$\begin{aligned}&= (0.80)(0.6)(1.41 \text{ in.}^2)(58 \text{ ksi}) \\ &= 39.3 \text{ kips}\end{aligned}$$

For a ductile steel element per ACI 318, Table 17.5.3(a), $\phi = 0.65$ and

$$\begin{aligned}\phi V_{sa} &= 0.65(39.3 \text{ kips}) \\ &= 25.5 \text{ kips} > 14.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Determine the available concrete breakout strength in shear

Because there are no edges adjacent to the base connection, the available concrete breakout strength in shear is not applicable.

Determine the available concrete pryout strength in shear

The available concrete pryout strength in shear is determined according to ACI 318, Section 17.7.3.

$$\begin{aligned}N_{cpg} &= N_{cbg} \\ &= 342 \text{ kips}\end{aligned}$$

For $h_{ef} \geq 2.5$ in.

$$k_{cp} = 2.0$$

$$V_{cpg} = k_{cp}N_{cpg} \qquad \text{(ACI 318, Eq. 17.7.3.1b)}$$

$$\begin{aligned}&= (2.0)(342 \text{ kips}) \\ &= 684 \text{ kips}\end{aligned}$$

The resistance factor from ACI 318, Table 17.5.3(c), is $\phi = 0.70$, and the resulting available strength is determined as:

$$\begin{aligned}\phi V_{cpg} &= 0.70(684 \text{ kips}) \\ &= 479 \text{ kips} > 116 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Determine the anchorage utilization considering tension and shear interaction

The tension and shear interaction is considered according to ACI 318, Section 17.8. The available tensile strength is limited by concrete breakout failure and the available shear strength is limited by the strength of the steel considering the reduction for a built-up grout pad.

$V_u \;\; = 14.5 \text{ kips}$
$\phi V_n = 25.5 \text{ kips}$

$N_u \;\; = 138 \text{ kips}$
$\phi N_n = 239 \text{ kips}$

$$\frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} \le 1.2 \qquad \text{[ACI 318, Eq. 17.8.3]}$$

$$\frac{138 \text{ kips}}{239 \text{ kips}} + \frac{14.5 \text{ kips}}{25.5 \text{ kips}} \le 1.2$$

$$1.15 \le 1.2 \quad \textbf{o.k.}$$

Evaluate the load path from the baseplate through the setting plate into the anchor rods

To preclude anchor rod bending and to facilitate distribution of the shear force to all the anchor rods, a setting plate with standard holes is used. The shear force is transferred through field welds from the base plate to the setting plate and then through bearing against the anchor rods. Use a ¼-in.-thick setting plate and provide a ¼ in. × 2 in. fillet weld at each anchor rod.

The bearing strength of the setting plate is determined per AISC *Specification* Section J3.11a.

$$R_n = 2.4dtF_u \qquad (\textit{Spec.} \text{ Eq. J3-6a})$$
$$= 2.4(1\tfrac{1}{2} \text{ in.})(\tfrac{1}{4} \text{ in.})(65 \text{ ksi})$$
$$= 58.5 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(58.5 \text{ kips})$ $= 43.9 \text{ kips} > 14.5 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{58.5 \text{ kips}}{2.00}$ $= 29.3 \text{ kips} > 9.68 \text{ kips}$ **o.k.**

Because the clear distance from the edge of the hole to the edge of the plate, l_c, is more than twice the rod diameter, bolt tearout of the anchor rod will not govern.

Because the welds are symmetrically applied to the plate, the force will be loaded through the welds' center of gravity, and a directional strength increase may be utilized. The capacity of the eight field welds is determined per AISC *Specification* Section J2.4a for a transversely loaded weld ($\theta = 90°$):

$$k_{ds} = (1.0 + 0.50 \sin^{1.5} \theta) \qquad (\textit{Spec.} \text{ Eq. J2-5})$$
$$= (1.0 + 0.50 \sin^{1.5} 90°)$$
$$= 1.50$$

$$R_n = F_{nw} A_{we} k_{ds} \qquad (\textit{Spec.} \text{ Eq. J2-4})$$
$$= 0.6(70 \text{ ksi})\left(\frac{\tfrac{1}{4} \text{ in.}}{\sqrt{2}}\right)(1.50)(2.00 \text{ in.})(8 \text{ welds})$$
$$= 178 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(178 \text{ kips})$ $= 134 \text{ kips} > 116 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{178 \text{ kips}}{2.00}$ $= 89.0 \text{ kips} > 77.4 \text{ kips}$ **o.k.**

Continuous welding between the base plate and the setting plate or coatings such as paint may be specified to assist with corrosion protection.

Check base plate bending for anchor rods in tension

A 45° distribution will be used to determine the flexural strength of the base plate using the rod spacing, s, and web thickness, t_w, to calculate the moment arm, a. A more refined approach considering two-way bending or yield line analysis may also be used.

$$
\begin{aligned}
a &= \frac{s - t_w}{2} \\
&= \frac{2(3\tfrac{1}{4} \text{ in.}) - \tfrac{1}{2} \text{ in.}}{2} \\
&= 3.00 \text{ in.}
\end{aligned}
$$

LRFD	ASD
$M_r = N_{ua,i}a$ $= (17.3 \text{ kips})(3.00 \text{ in.})$ $= 51.9 \text{ kip-in.}$	$M_r = N_{aa,i}a$ $= (11.5 \text{ kips})(3.00 \text{ in.})$ $= 34.5 \text{ kip-in.}$

$$
\begin{aligned}
b_e &= 2a \\
&= 2(3.00 \text{ in.}) \\
&= 6.00 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
M_n &= F_y Z \\
&= (50 \text{ ksi})\left[\frac{(6.00 \text{ in.})(1\tfrac{3}{4} \text{ in.})^2}{4}\right] \\
&= 230 \text{ kip-in.}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(230 \text{ kip-in.})$ $= 207 \text{ kip-in.} > 51.9 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{230 \text{ kip-in.}}{1.67}$ $= 138 \text{ kip-in.} > 34.5 \text{ kip-in.}$ **o.k.**

In this case, because the available flexural strength is much larger than the required flexural strength, iteration may be applied to reduce the thickness of the base plate and provide a design with additional economy.

Check weld from the stiffener and column web to the base plate for the tensile loading in the anchors

The tensile load is distributed to the welds along the effective length, b_e. For plates with little flexibility, engaging more weld in resisting the anchor rod tensile loading could be justified.

The total weld length effective in resisting the anchor rod tension is therefore:

$$
\begin{aligned}
L &= (8 \text{ anchor rods})(6.00 \text{ in.}) \\
&= 48.0 \text{ in.}
\end{aligned}
$$

For $\theta = 90°$, $k_{ds} = 1.5$ as determined previously. Based on the thickness of the web and the base plate, the minimum fillet weld size permitted by AISC *Specification* Table J2.4 is ¼ in. The nominal strength of the weld is:

$$
\begin{aligned}
R_n &= F_{nw} A_{we} k_{ds} \qquad \text{(}\textit{Spec}\text{. Eq. J2-4)} \\
&= 0.6(70 \text{ ksi})\left(\frac{\frac{1}{4} \text{ in.}}{\sqrt{2}}\right)(48.0 \text{ in.})(1.50) \\
&= 535 \text{ kips}
\end{aligned}
$$

The available strength of the weld is then:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(535 \text{ kips})$ $= 401 \text{ kips} > 138 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{535 \text{ kips}}{2.00}$ $= 268 \text{ kips} > 91.7 \text{ kips}$ **o.k.**

Check weld from the column web and brace gusset to the base plate for the shear loading

For the design of these welds, and all other connection limits states, see AISC Design Guide 29, *Vertical Bracing Connections—Analysis and Design* (Muir and Thornton, 2014), Example 5.12.2, which contains a similar connection configuration.

EXAMPLE 4.7-8—Base Connection at Brace Producing Combined Compression and Shear

This example illustrates a base connection design subjected to combined compression and shear in a non-seismic application.

Given:

A W24×104 column base connection is subjected to wind forces from a brace in compression and additional permanent column dead loads as illustrated in Figure 4-30. Confirm the bearing strength of the base plate, calculate the required base plate thickness, and design the weld. Confirm that the friction present under the applied compression load is adequate to resist the applied shear load. The concrete compressive strength, f'_c, is 4,000 psi, and there are no adjacent edges. The column is ASTM A992/A992M, and the plate is ASTM A572/A572M Grade 50 material.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the column properties are as follows:

W24×104
d = 24.1 in.
b_f = 12.8 in.
t_w = 0.500 in.
T = 20 in.

Determine the required strength at the baseplate-to-grout interface

Resolve the brace force into shear and compression forces. Because there is no eccentricity between the brace workpoint and the center of the base plate, the applied forces will consist of concentric compression and shear. Determine the maximum and minimum compression forces according to the load combinations of ASCE/SEI 7.

The horizontal shear at the base plate from the brace compression is:

$$
\begin{aligned}
V_{1.0W} &= (180\text{ kips})\left(\frac{12}{\sqrt{12^2+12^2}}\right) \\
&= 127\text{ kips}
\end{aligned}
$$

The axial compression at the base plate from the brace compression is:

$$
\begin{aligned}
P_{1.0W} &= (180\text{ kips})\left(\frac{12}{\sqrt{12^2+12^2}}\right) \\
&= 127\text{ kips}
\end{aligned}
$$

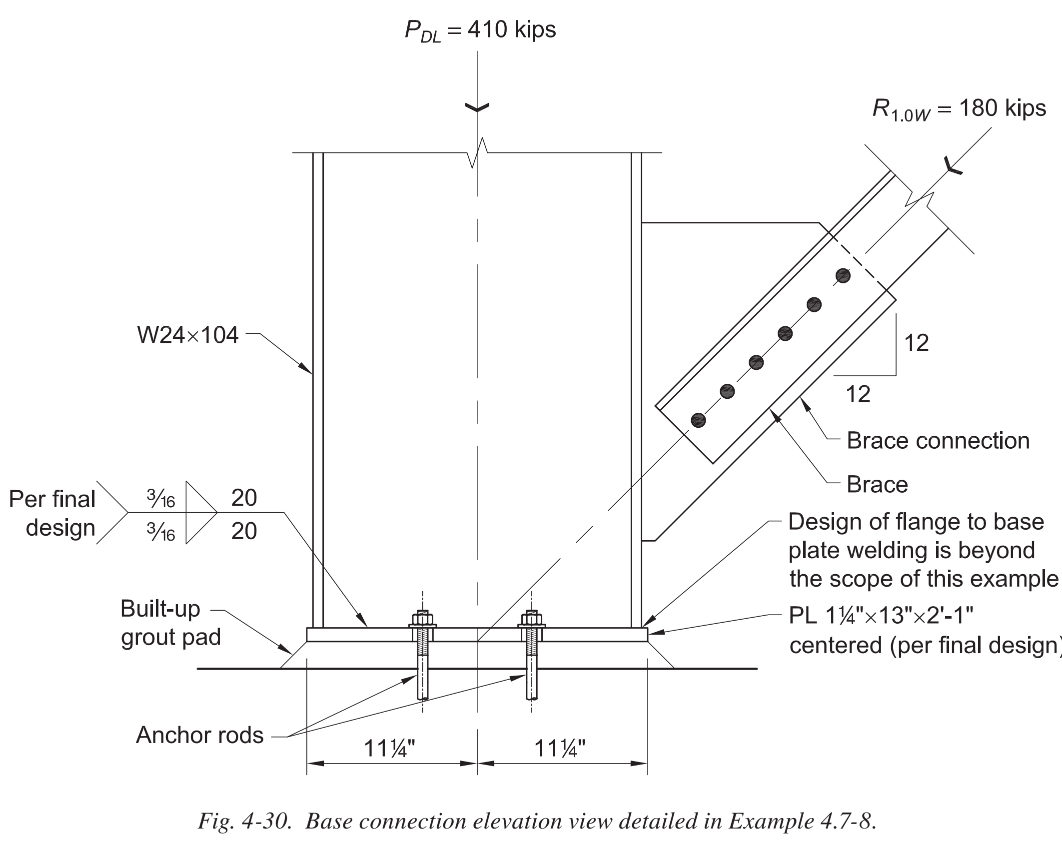


Fig. 4-30. Base connection elevation view detailed in Example 4.7-8.

LRFD	ASD
Load combination 1a from ASCE/SEI 7, Section 2.3.1: $V_u = 0$ kips; $P_u = 1.4D = 1.4(410 \text{ kips}) = 574$ kips	Load combination 1a from ASCE/SEI 7, Section 2.4.1: $V_a = 0$ kips; $P_a = D = 410$ kips
Load combination 4a from ASCE/SEI 7, Section 2.3.1: $V_u = 1.0W = 1.0(127 \text{ kips}) = 127$ kips; $P_u = 1.2D + 1.0W = 1.2(410 \text{ kips}) + 1.0(127 \text{ kips}) = 619$ kips	Load combination 5a from ASCE/SEI 7, Section 2.4.1: $V_a = 0.6W = 0.6(127 \text{ kips}) = 76.2$ kips; $P_a = D + 0.6W = 410 \text{ kips} + 0.6(127 \text{ kips}) = 486$ kips
Load combination 5a from ASCE/SEI 7, Section 2.3.1: $V_u = 1.0W = 1.0(127 \text{ kips}) = 127$ kips; $P_u = 0.9D + 1.0W = 0.9(410 \text{ kips}) + 1.0(127 \text{ kips}) = 496$ kips	Load combination 7a from ASCE/SEI 7, Section 2.4.1: $V_a = 0.6W = 0.6(127 \text{ kips}) = 76.2$ kips; $P_a = 0.6D + 0.6W = 0.6(410 \text{ kips}) + 0.6(127 \text{ kips}) = 322$ kips
The maximum base plate compression combined with maximum shear occurs from load combination 4a, and the minimum base plate compression combined with maximum shear occurs from load combination 5a.	The maximum base plate compression combined with maximum shear occurs from load combination 5a, and the minimum base plate compression combined with maximum shear occurs from load combination 7a.

Verify the base plate bearing on the concrete

To facilitate welding and to minimize the base plate footprint, use a base plate with $B = 13.0$ in. and $N = 25.0$ in. Provide a minimum of four anchor rods to satisfy OSHA requirements. In the absence of other requirements, it is beneficial to provide a square anchor pattern to reduce the probability of the pattern being inadvertently rotated during installation or fabrication. Note that if the base connection will also be subject to tension and shear, the anchor rods should be designed according to Section 4.3.4.

Because the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area exceeds four times the area of the base plate, the nominal bearing strength of the base plate is given by AISC *Specification* Equation J8-2:

$$
\begin{aligned}
A_1 &= BN \\
&= (13.0 \text{ in.})(25.0 \text{ in.}) \\
&= 325 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
P_p &= 1.7 f'_c A_1 \\
&= 1.7(4 \text{ ksi})(325 \text{ in.}^2) \\
&= 2{,}210 \text{ kips}
\end{aligned}
$$

(from *Spec.* Eq. J8-2)

LRFD	ASD
$\phi_c = 0.65$ $\phi_c P_p = 0.65(2{,}210 \text{ kips})$ $= 1{,}440 \text{ kips} > 619 \text{ kips}$ **o.k.**	$\Omega_c = 2.31$ $\dfrac{P_p}{\Omega_c} = \dfrac{2{,}210 \text{ kips}}{2.31}$ $= 957 \text{ kips} > 486 \text{ kips}$ **o.k.**

The base plate footprint provides adequate concrete bearing capacity.

Determine the minimum thickness of the base plate

The minimum thickness of the base plate is determined as follows:

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$= \frac{25.0 \text{ in.} - 0.95(24.1 \text{ in.})}{2}$$

$$= 1.05 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$= \frac{13.0 \text{ in.} - 0.8(12.8 \text{ in.})}{2}$$

$$= 1.38 \text{ in.}$$

$$n' = \frac{\sqrt{db_f}}{4} \qquad \text{(from Eq. 4-12)}$$

$$= \frac{\sqrt{(24.1 \text{ in.})(12.8 \text{ in.})}}{4}$$

$$= 4.39 \text{ in.}$$

LRFD	ASD
$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\left(\dfrac{P_u}{\phi_c P_p}\right)$ (4-14a) $= \left[\dfrac{4(24.1 \text{ in.})(12.8 \text{ in.})}{(24.1 \text{ in.} + 12.8 \text{ in.})^2}\right]\left(\dfrac{619 \text{ kips}}{1{,}440 \text{ kips}}\right)$ $= 0.390$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ (4-13) $= \dfrac{2\sqrt{0.390}}{1+\sqrt{1-0.390}}$ $= 0.701$ $l = \max(m,\ n,\ \lambda n')$ $= \max[1.05 \text{ in.},\ 1.38 \text{ in.},\ 0.701(4.39 \text{ in.})]$ $= 3.08 \text{ in.}$	$X = \left[\dfrac{4db_f}{(d+b_f)^2}\right]\left(\dfrac{P_a}{P_p/\Omega_c}\right)$ (4-14b) $= \left[\dfrac{4(24.1 \text{ in.})(12.8 \text{ in.})}{(24.1 \text{ in.} + 12.8 \text{ in.})^2}\right]\left(\dfrac{486 \text{ kips}}{957 \text{ kips}}\right)$ $= 0.460$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ (4-13) $= \dfrac{2\sqrt{0.460}}{1+\sqrt{1-0.460}}$ $= 0.782$ $l = \max(m,\ n,\ \lambda n')$ $= \max[1.05 \text{ in.},\ 1.38 \text{ in.},\ 0.782(4.39 \text{ in.})]$ $= 3.43 \text{ in.}$

LRFD	ASD
$$t_{min} = l\sqrt{\frac{2P_u}{\phi_b F_y BN}} \quad \text{(4-15a)}$$ $$= (3.08\text{ in.})\sqrt{\frac{2(619\text{ kips})}{0.90(50\text{ ksi})(13.0\text{ in.})(25.0\text{ in.})}}$$ $$= 0.896\text{ in.}$$	$$t_{min} = l\sqrt{\frac{2\Omega_b P_a}{F_y BN}} \quad \text{(4-15b)}$$ $$= (3.43\text{ in.})\sqrt{\frac{2(1.67)(486\text{ kips})}{(50\text{ ksi})(13.0\text{ in.})(25.0\text{ in.})}}$$ $$= 1.08\text{ in.}$$

Use a 1¼-in.-thick base plate.

Determine if friction is adequate to resist the required shear strength

The case with minimum compression load and maximum shear governs (P_u = 496 kips, V_u = 127 kips). The methodology contained in Section 4.3.5 is used with $\mu = 0.4$, $A_c = A_1$, $\phi_{friction} = 0.65$, and $\phi = 0.75$. Only LRFD is applicable. The contribution of the anchors in shear is not considered.

$$\phi V_n = \min\left[\phi_{friction}(\mu P_u),\ \phi 0.2 f'_c A_c,\ \phi(0.8\text{ ksi})A_c\right] \quad \text{(4-30)}$$

$$= \min\left[0.65(0.4)(496\text{ kips}),\ 0.75(0.2)(4\text{ ksi})(325\text{ in.}^2),\ 0.75(0.8\text{ ksi})(325\text{ in.}^2)\right]$$

$$= \min(129\text{ kips},\ 195\text{ kips},\ 195\text{ kips})$$

$$= 129\text{ kips} > 127\text{ kips} \quad \textbf{o.k.}$$

Determine the weld requirements from the column web to the base plate

The weld from the column web to the base plate resists the shear, V_u or V_a. The column-to-base connection will be fit-to-bear for compression. For the 1¼-in.-thick base plate and t_w = 0.500 in., the minimum required fillet weld size per AISC *Specification* Table J2.4 is 3⁄16 in. Determine the strength of two 3⁄16 in. fillet welds applied along the full T = 20 in. dimension of the column.

The strength of the two welds is determined per AISC *Specification* Section J2.4a for a longitudinally loaded weld ($\theta = 0°$)

$$k_{ds} = (1.0 + 0.50\sin^{1.5}\theta) \quad \text{(\textit{Spec.} Eq. J2-5)}$$

$$= (1.0 + 0.50\sin^{1.5}0°)$$

$$= 1.00$$

$$R_n = F_{nw}A_{we}k_{ds} \quad \text{(\textit{Spec.} Eq. J2-4)}$$

$$= 0.6(70\text{ ksi})\left(\frac{3/16\text{ in.}}{\sqrt{2}}\right)(20.0\text{ in.})(2\text{ welds})(1.00)$$

$$= 223\text{ kips}$$

LRFD	ASD
$$\phi = 0.75$$ $$\phi R_n = 0.75(223\text{ kips})$$ $$= 167\text{ kips} > 127\text{ kips} \quad \textbf{o.k.}$$	$$\Omega = 2.00$$ $$\frac{R_n}{\Omega} = \frac{223\text{ kips}}{2.00}$$ $$= 112\text{ kips} > 76.2\text{ kips} \quad \textbf{o.k.}$$

The nominal shear rupture strength of the column at the welds is calculated using AISC *Specification* Section J4.2(b)

$$R_n = 0.60F_uA_{nv} \quad \text{(\textit{Spec.} Eq. J4-4)}$$

$$= 0.60(65\text{ ksi})(0.500\text{ in.})(20.0\text{ in.})$$

$$= 390\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(390\text{ kips})$ $= 293\text{ kips} > 127\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{390\text{ kips}}{2.00}$ $= 195\text{ kips} > 76.2\text{ kips}$ **o.k.**

Welding to the flanges may also be required for other loading and/or other minimum requirements.

EXAMPLE 4.7-9—Base Connection for Bending

An exposed base connection is designed in this example that considers a base subjected to only flexural forces producing compression at the toe of the base plate and tension in the anchor rods.

Given:

A base connection of a W18×76 column has a moment from nonseismic forces as illustrated in Figure 4-31. Determine the anchorage and base plate requirements. The concrete compressive strength, f_c', is 4,000 psi, and there are no adjacent edges. The column is ASTM A992/A992M, and the plate is ASTM A572/A572M Grade 50 material.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the column properties are as follows:

W18×76
$d = 18.2$ in.
$b_f = 11.0$ in.
$t_f = 0.680$ in.

Determine the available concrete bearing stress limit, $f_{p(max)}$

Because there are no adjacent edges, $A_2/A_1 > 4$ and:

LRFD	ASD
$f_{p(max)} = \phi_c 1.7 f_c'$ (from Eq. 4-2) $= (0.65)(1.7)(4\text{ ksi})$ $= 4.42\text{ ksi}$	$f_{p(max)} = 1.7 f_c'/\Omega_c$ (from Eq. 4-2) $= (1.7)(4\text{ ksi})/2.31$ $= 2.94\text{ ksi}$

Determine the anchor rod tension

The anchor rod tension is determined per the previously described procedure for flexure in absence of axial load.

LRFD	ASD
$q_{max} = f_{p(max)}B$ (4-37) $= (4.42 \text{ ksi})(12.0 \text{ in.})$ $= 53.0 \text{ kip/in.}$ $Y = \left(f + \frac{N}{2}\right) - \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2M_r}{q_{max}}}$ (4-32) $= \left(12.0 \text{ in.} + \frac{28.0 \text{ in.}}{2}\right) - \sqrt{\left(12.0 \text{ in.} + \frac{28.0 \text{ in.}}{2}\right)^2 - \left[\frac{2(1{,}200 \text{ kip-in.})}{53.0 \text{ kip/in.}}\right]}$ $= 0.886 \text{ in.}$ $T_u = q_{max}Y$ (from Eq. 4-31) $= (53.0 \text{ kip/in.})(0.886 \text{ in.})$ $= 47.0 \text{ kips}$ Because the tension from the applied moment is resisted only by two anchor rods, $n = 2$ and: $r_u = \frac{T_u}{n}$ $= \frac{47.0 \text{ kips}}{2 \text{ rods}}$ $= 23.5 \text{ kips/rod}$	$q_{max} = f_{p(max)}B$ (4-37) $= (2.94 \text{ ksi})(12.0 \text{ in.})$ $= 35.3 \text{ kip/in.}$ $Y = \left(f + \frac{N}{2}\right) - \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2M_r}{q_{max}}}$ (4-32) $= \left(12.0 \text{ in.} + \frac{28.0 \text{ in.}}{2}\right) - \sqrt{\left(12.0 \text{ in.} + \frac{28.0 \text{ in.}}{2}\right)^2 - \left[\frac{2(816 \text{ kip-in.})}{35.3 \text{ kip/in.}}\right]}$ $= 0.905 \text{ in.}$ $T_a = q_{max}Y$ (from Eq. 4-31) $= (35.3 \text{ kip/in.})(0.905 \text{ in.})$ $= 31.9 \text{ kips}$ Because the tension from the applied moment is resisted only by two anchor rods, $n = 2$ and: $r_a = \frac{T_a}{n}$ $= \frac{31.9 \text{ kips}}{2 \text{ rods}}$ $= 16.0 \text{ kips/rod}$

Determine the available steel strength in tension of the anchor rods

The available strength of the steel in tension is determined according to ACI 318, Section 17.6.1. As discussed previously, the AISC *Specification* provides similar capacities.

From Table 4-1, $A_{se,N} = 0.606 \text{ in.}^2$

$$N_{sa} = A_{se,N} f_{uta} \qquad \text{(ACI 318, Eq. 17.6.1.2)}$$
$$= (0.606 \text{ in.}^2)(58 \text{ ksi})$$
$$= 35.1 \text{ kips}$$

For a ductile steel element per ACI 318, Table 17.5.3(a), $\phi = 0.75$ and

$$\phi N_{sa} = 0.75(35.1 \text{ kips})$$
$$= 26.3 \text{ kips} > 23.5 \text{ kips} \quad \textbf{o.k.}$$

Determine the concrete breakout strength in tension

The available concrete breakout strength in tension is determined according to ACI 318, Section 17.6.2. The anchor group is only comprised of the two anchors in tension. Because there is no edge within $1.5h_{ef}$ of any anchor, no reduction for edge distance will occur.

$$A_{Nc} = (1.5h_{ef} + 6.00 \text{ in.} + 1.5h_{ef})(1.5h_{ef} + 1.5h_{ef})$$
$$= [1.5(12.0 \text{ in.}) + 6.00 \text{ in.} + 1.5(12.0 \text{ in.})][1.5(12.0 \text{ in.}) + 1.5(12.0 \text{ in.})]$$
$$= 1{,}510 \text{ in.}^2$$

$$A_{Nco} = 9h_{ef}^2 \qquad \text{(ACI 318, Eq. 17.6.2.1.4)}$$
$$= 9(12.0 \text{ in.})^2$$
$$= 1{,}300 \text{ in.}^2$$

For an anchor with an embedment 11.0 in. ≤ h_{ef} ≤ 25.0 in.

$$N_b = 16\lambda_a \sqrt{f_c'}(h_{ef})^{5/3} \qquad \text{(ACI 318, Eq. 17.6.2.2.3)}$$
$$= 16(1.0)\sqrt{4{,}000 \text{ psi}}(12.0 \text{ in.})^{5/3}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right)$$
$$= 63.6 \text{ kips}$$

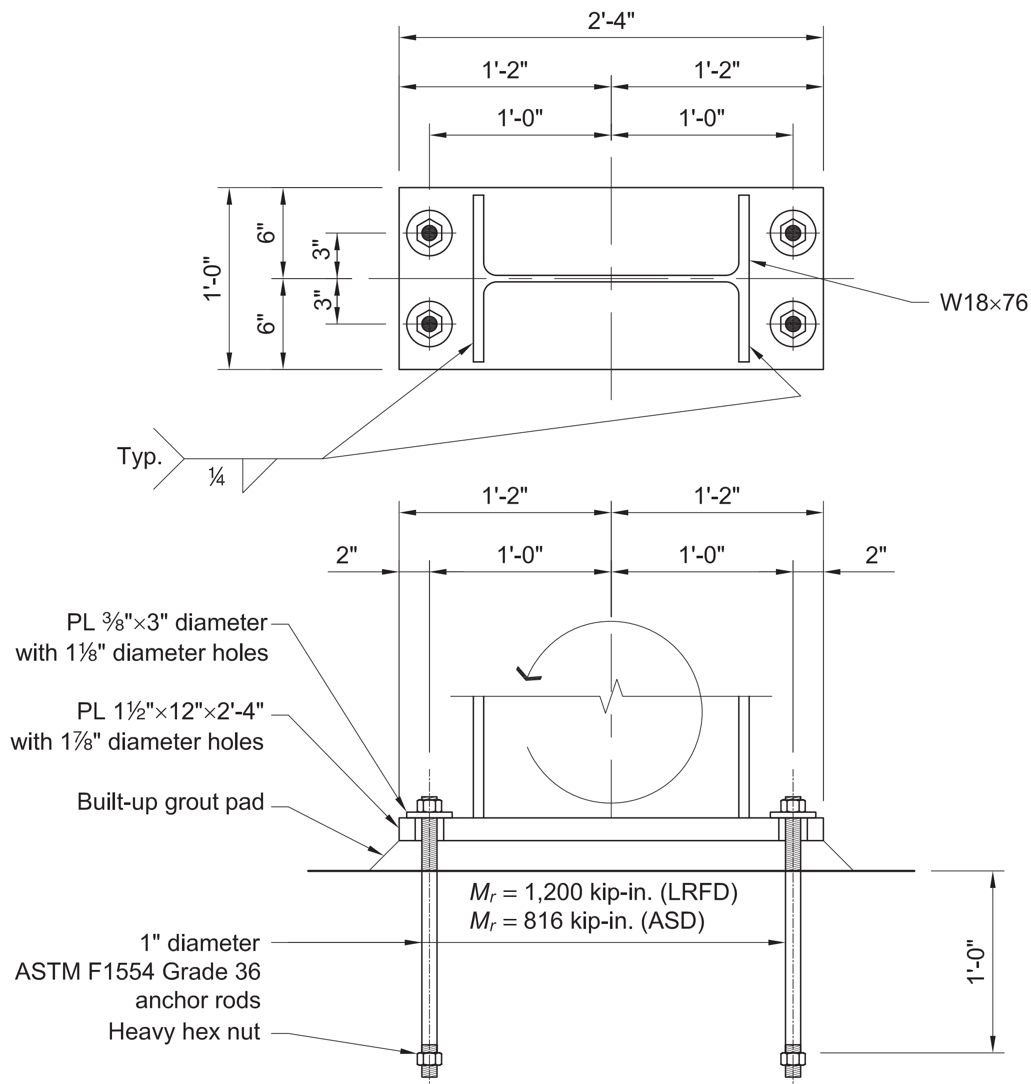


Fig. 4-31. Base configuration and applied forces used in Example 4.7-9.

For an anchor group concentrically loaded,

$$e'_N = 0 \text{ in.}$$

$$\psi_{ec,N} = \frac{1}{\left(1+\dfrac{e'_N}{1.5h_{ef}}\right)} \le 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.3.1)}$$

$$= \frac{1}{\left[1+\dfrac{0 \text{ in.}}{1.5(12.0 \text{ in.})}\right]}$$

$$= 1.0$$

Because there are no adjacent edges, $c_{a,min} \ge 1.5h_{ef}$ and

$$\psi_{ed,N} = 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.4.1a)}$$

Because no analysis was performed to confirm there would be no cracking at service load levels per ACI 318, Section 17.6.2.5,

$$\psi_{c,N} = 1.0$$

For a cast-in-place anchor rod, the breakout splitting factor is determined per ACI 318, Section 17.6.2.6.2,

$$\psi_{cp,N} = 1.0$$

The resulting concrete breakout strength of the anchor group in tension is determined by ACI 318, Section 17.6.2,

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

$$= \left(\frac{1{,}510 \text{ in.}^2}{1{,}300 \text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(63.6 \text{ kips})$$

$$= 73.9 \text{ kips}$$

Because no supplementary reinforcement was specified, $\phi = 0.70$ per ACI 318, Table 17.5.3(b), and

$$\phi N_{cbg} = (0.70)(73.9 \text{ kips})$$

$$= 51.7 \text{ kips} > 47.0 \text{ kips} \quad \textbf{o.k.}$$

Determine the concrete pullout strength in tension

The available concrete pullout strength in tension is determined according to ACI 318, Section 17.6.3.

$$N_p = 8A_{brg}f'_c \qquad \text{(ACI 318, Eq. 17.6.3.2.2a)}$$

$$= 8(1.50 \text{ in.}^2)(4 \text{ ksi})$$

$$= 48.0 \text{ kips}$$

Because no analysis was performed to confirm there would be no cracking at service levels per ACI 318, Section 17.6.3.3,

$$\psi_{c,P} = 1.0$$

The nominal pullout strength of a single anchor in tension is determined per ACI 318, Section 17.6.3.1, as

$$N_{pn} = \psi_{c,P} N_p \qquad \text{(ACI 318, Eq. 17.6.3.1)}$$

$$= 1.0(48.0 \text{ kips})$$

$$= 48.0 \text{ kips}$$

The resistance factor from ACI 318, Table 17.5.3(c), is $\phi = 0.70$, and the resulting available strength is determined as

$$\begin{aligned}\phi N_{pn} &= (0.70)(48.0 \text{ kips})\\ &= 33.6 \text{ kips} > 23.5 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Determine the concrete side-face blowout strength in tension of the anchor group

The available concrete side-face blowout strength in tension of the anchor group is determined according to ACI 318, Section 17.6.4.

Because $h_{ef} \le 2.5c_{a1}$, side-face blowout is not applicable.

Determine the minimum plate thickness for bending with the anchor rod in tension

Because the plate and column flange have similar widths, consider that the full width of the plate, B, will be effective in bending. The distance from the anchor rod to the center of the column flange is given by:

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{4-61}$$

$$\begin{aligned}&= 12.0 \text{ in.} - \frac{18.2 \text{ in.}}{2} + \frac{0.680 \text{ in.}}{2}\\ &= 3.24 \text{ in.}\end{aligned}$$

The required thickness is given by the following as derived in Section 4.3.7:

LRFD	ASD
$t_{p(req)} = 2.11\sqrt{\dfrac{T_u x}{BF_y}}$ (4-62a) $= 2.11\sqrt{\dfrac{(47.0 \text{ kip})(3.24 \text{ in.})}{(12.0 \text{ in.})(50 \text{ ksi})}}$ $= 1.06$ in.	$t_{p(req)} = 2.58\sqrt{\dfrac{T_a x}{BF_y}}$ (4-62b) $= 2.58\sqrt{\dfrac{(31.9 \text{ kip})(3.24 \text{ in.})}{(12.0 \text{ in.})(50 \text{ ksi})}}$ $= 1.07$ in.

Determine the minimum plate thickness for bending with the concrete compression

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$\begin{aligned}&= \frac{28.0 \text{ in.} - 0.95(18.2 \text{ in.})}{2}\\ &= 5.36 \text{ in.}\end{aligned}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$\begin{aligned}&= \frac{12.0 \text{ in.} - 0.8(11.0 \text{ in.})}{2}\\ &= 1.60 \text{ in.}\end{aligned}$$

Because $Y < m$ and $m > n$, and using Equation 4-52a and 4-52b:

LRFD	ASD
$t_{p(req)} = 2.11\sqrt{\dfrac{f_{p(max)}Y\left(m-\dfrac{Y}{2}\right)}{F_y}}$ $= 2.11\sqrt{\dfrac{\left[\begin{array}{l}(4.42\text{ ksi})(0.886\text{ in.})\\ \times\left(5.36\text{ in.}-\dfrac{0.886\text{ in.}}{2}\right)\end{array}\right]}{50\text{ ksi}}}$ $= 1.31\text{ in.}$	$t_{p(req)} = 2.58\sqrt{\dfrac{f_{p(max)}Y\left(m-\dfrac{Y}{2}\right)}{F_y}}$ $= 2.58\sqrt{\dfrac{\left[\begin{array}{l}(2.94\text{ ksi})(0.905\text{ in.})\\ \times\left(5.36\text{ in.}-\dfrac{0.905\text{ in.}}{2}\right)\end{array}\right]}{50\text{ ksi}}}$ $= 1.32\text{ in.}$

Compression governs, therefore use a 1½-in.-thick plate.

Determine the flange welding requirements

With a column flange thickness of 0.680 in. and a 1½-in.-thick plate, the minimum weld size per AISC *Specification* Table J2.4 is ¼ in. The weld along the outside of the flange will be the stiff load path to transfer the forces from the column to the anchor rods.

The required strength is determined by decoupling the moment into a force couple separated by the depth of the column, *d*.

LRFD	ASD
$R_u = \dfrac{M_u}{d}$ $= \dfrac{1{,}200\text{ kip-in.}}{18.2\text{ in.}}$ $= 65.9\text{ kips}$	$R_a = \dfrac{M_a}{d}$ $= \dfrac{816\text{ kip-in.}}{18.2\text{ in.}}$ $= 44.8\text{ kips}$

From AISC *Specification* Section J2.4(a), the directional strength increase factor for an angle of 90° between the line of action of the required force and weld longitudinal axis is calculated by:

$$\theta = 90°$$

$$k_{ds} = (1.0 + 0.50\sin^{1.5}\theta) \qquad (Spec.\text{ Eq. J2-5})$$

$$= \left[1.0 + 0.50\sin^{1.5}(90°)\right]$$

$$= 1.50$$

From AISC *Specification* Section J2.4(a) and Table J2.5, the nominal weld strength per in. for a ¼ in. fillet weld with E70 electrodes is given by:

$$R_n = F_{nw}A_{we}k_{ds} \qquad (Spec.\text{ Eq. J2-4})$$

$$= \left[0.60(70\text{ ksi})\right]\left(\frac{1/4\text{ in.}}{\sqrt{2}}\right)(11.0\text{ in.})(1.50)$$

$$= 123\text{ kips}$$

The available weld strength is then:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(123\text{ kips})$ $= 92.3\text{ kips} > 65.9\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{123\text{ kips}}{2.00}$ $= 61.5\text{ kips} > 44.8\text{ kips}$ **o.k.**

Additional welding may be required for other loading scenarios or for minimum attachment of the base plate to the column.

EXAMPLE 4.7-10—Base Connection for Bending without Anchor Rod Tension (Low Moment)

A base connection for a wide-flange column subject to compression and moment is designed in this example. The ratio of flexure to compression in this example is such that the moment can be resisted without producing tension in the anchor rods.

Given:

A W12×96 column is subjected to the axial and moment dead and live loads shown in Figure 4-32. Bending is about the strong axis of the W12×96. The ratio of the concrete-to-base plate area is unity ($A_1 = A_2$). The column will be anchored to the foundation using ASTM F1554 Grade 36 anchor rods. The column is attached to a concrete foundation with a specified compressive strength of concrete, $f_c' = 4{,}000$ psi.

Solution:

From AISC *Manual* Tables 2-4, 2-5, and 2-6, the material properties are as follows:

W12×96
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

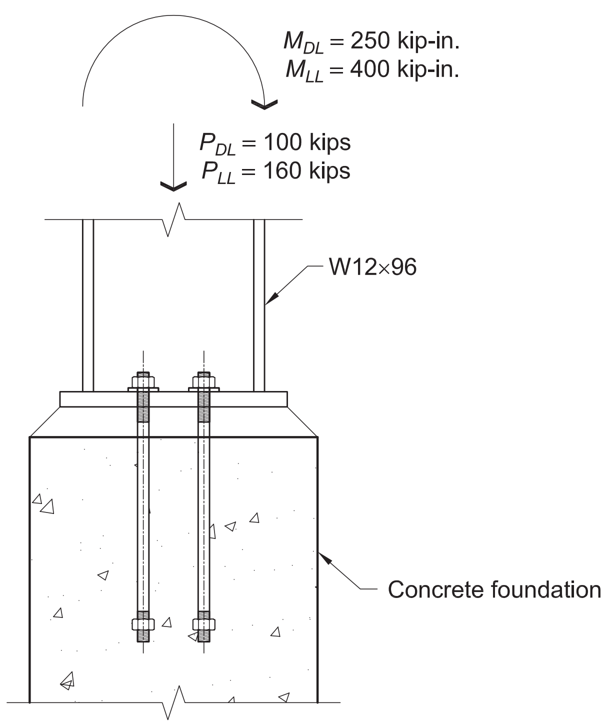


Fig. 4-32. Base detail section as used in Example 4.7-10.

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

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
$b_f = 12.2$ in.
$d = 12.7$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

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

Choose trial base plate size

The base plate dimensions $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA. Consider N and B to be at least 3 in. larger than the outside column dimensions.

$$N > d + (2)(3.00 \text{ in.}) = 18.7 \text{ in.}$$

$$B > b_f + (2)(3.00 \text{ in.}) = 18.2 \text{ in.}$$

Try $N = 19.0$ in. and $B = 19.0$ in.

Determine e *and* e_{crit}

LRFD	ASD
$e = \dfrac{M_u}{P_u}$ (from Eq. 4-39) $= \dfrac{940 \text{ kip-in.}}{376 \text{ kips}}$ $= 2.50 \text{ in.}$ $f_{p(max)} = \phi_c (0.85 f'_c)\sqrt{\dfrac{A_2}{A_1}}$ (from Eq. 4-2) $= (0.65)(0.85)(4 \text{ ksi})(1)$ $= 2.21 \text{ ksi}$	$e = \dfrac{M_a}{P_a}$ (from Eq. 4-39) $= \dfrac{650 \text{ kip-in.}}{260 \text{ kips}}$ $= 2.50 \text{ in.}$ $f_{p(max)} = \dfrac{(0.85 f'_c)}{\Omega_c}\sqrt{\dfrac{A_2}{A_1}}$ (from Eq. 4-2) $= \dfrac{(0.85)(4 \text{ ksi})(1)}{2.31}$ $= 1.47 \text{ ksi}$

LRFD	ASD
$q_{max} = f_{p(max)}B$ (4-37) $= (2.21 \text{ ksi})(19.0 \text{ in.})$ $= 42.0 \text{ kips/in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_u}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{19.0 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{2(42.0 \text{ kips/in.})}$ $= 5.02 \text{ in.}$	$q_{max} = f_{p(max)}B$ (4-37) $= (1.47 \text{ ksi})(19.0 \text{ in.})$ $= 27.9 \text{ kips/in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_a}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{19.0 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{2(27.9 \text{ kips/in.})}$ $= 4.84 \text{ in.}$

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

Determine bearing length, Y*, and verify bearing pressure*

The bearing length, Y, can be determined using Equation 4-42.

$$Y = N - 2e \qquad \text{(from Eq. 4-42)}$$
$$= 19.0 \text{ in.} - (2)(2.50 \text{ in.})$$
$$= 14.0 \text{ in.}$$

The bearing pressure can then be determined as follows:

LRFD	ASD
$q = \dfrac{P_u}{Y}$ (from Eq. 4-43) $= \dfrac{376 \text{ kips}}{14.0 \text{ in.}}$ $= 26.9 \text{ kips/in.} < 42.0 \text{ kips/in.} = q_{max}$ **o.k.**	$q = \dfrac{P_a}{Y}$ (from Eq. 4-43) $= \dfrac{260 \text{ kips}}{14.0 \text{ in.}}$ $= 18.6 \text{ kips/in.} < 27.9 \text{ kips/in.} = q_{max}$ **o.k.**

Determine the minimum plate thickness

At the bearing interface:

$$m = \frac{N - 0.95d}{2} \qquad \text{(4-10)}$$
$$= \frac{19.0 \text{ in.} - 0.95(12.7 \text{ in.})}{2}$$
$$= 3.47 \text{ in.}$$

The bearing stress between the plate and concrete is calculated as follows:

LRFD	ASD
$f_p = \dfrac{P_u}{BY}$ (4-44) $= \dfrac{376 \text{ kips}}{(19.0 \text{ in.})(14.0 \text{ in.})}$ $= 1.41 \text{ ksi}$	$f_p = \dfrac{P_a}{BY}$ (4-44) $= \dfrac{260 \text{ kips}}{(19.0 \text{ in.})(14.0 \text{ in.})}$ $= 0.977 \text{ ksi}$

Because $Y \geq m$, the minimum plate thickness may be calculated using Equation 4-51:

LRFD	ASD
$t_{p(req)} = 1.49m\sqrt{\dfrac{f_p}{F_y}}$ (4-51a) $= (1.49)(3.47 \text{ in.})\sqrt{\dfrac{1.41 \text{ ksi}}{50 \text{ ksi}}}$ $= 0.868 \text{ in.}$	$t_{p(req)} = 1.83m\sqrt{\dfrac{f_p}{F_y}}$ (4-51b) $= (1.83)(3.47 \text{ in.})\sqrt{\dfrac{0.977 \text{ ksi}}{50 \text{ ksi}}}$ $= 0.888 \text{ in.}$

Check the thickness using the value of n.

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$= \frac{19.0 \text{ in.} - (0.8)(12.2 \text{ in.})}{2}$$

$$= 4.62 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = (1.49)(4.62 \text{ in.})\sqrt{\dfrac{1.41 \text{ ksi}}{50 \text{ ksi}}}$ (from Eq. 4-51a) $= 1.16 \text{ in.}$ **controls**	$t_{p(req)} = (1.83)(4.62 \text{ in.})\sqrt{\dfrac{0.977 \text{ ksi}}{50 \text{ ksi}}}$ (from Eq. 4-51b) $= 1.18 \text{ in.}$ **controls**

Use a 1¼ in. × 19 in. × 19 in. base plate.

Determine the anchor rod size

Because 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 ASTM F1554 Grade 36 rods.

EXAMPLE 4.7-11—Base Connection for Bending with Anchor Rod Tension (Large Moment)

A base connection for a wide-flange column subject to compression and moment is designed in this example. The ratio of flexure to compression in this example is such that the moment produces tension in the anchor rods.

Given:

A W12×96 column is subjected to compressive 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 of the W12×96. Consider the ratio of the concrete-to-base plate area as unity ($A_1 = A_2$). The column is anchored to the foundation using ASTM F1554 Grade 36 anchor rods. The column is attached to a concrete foundation with a specified compressive strength of concrete, $f'_c = 4{,}000$ psi. The column is ASTM A992/A992M and the plate is ASTM A572/A572M Grade 50 material.

From AISC *Manual* Tables 2-4, 2-5, and 2-6, the material properties are as follows:

W12×96
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

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

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
$b_f = 12.2$ in.
$d = 12.7$ in.
$t_f = 0.900$ in.

Solution:

From ASCE/SEI 7, Chapter 2, the required strength is:

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

Choose trial base plate size

The base plate dimensions $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA. Consider N and B to be at least 3 in. larger than the outside column dimensions. For reference, see Figure 4-33 on page 124 that illustrates the final configuration.

$$N > d + (2)(3.00 \text{ in.}) = 18.7 \text{ in.}$$
$$B > b_f + (2)(3.00 \text{ in.}) = 18.2 \text{ in.}$$

Try $N = 19.0$ in. and $B = 19.0$ in.

Determine e *and* e_{crit}

Check the inequality in Equation 4-53 to determine if a low or high moment case exists.

LRFD	ASD
$q_{max} = 42.0$ kips/in. (from Example 4.7-10) $e = \dfrac{M_u}{P_u}$ (from Eq. 4-39) $= \dfrac{3{,}600 \text{ kip-in.}}{376 \text{ kips}}$ $= 9.57 \text{ in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_u}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{19.0 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{2(42.0 \text{ kips/in.})}$ $= 5.02 \text{ in.}$	$q_{max} = 27.9$ kips/in. (from Example 4.7-10) $e = \dfrac{M_a}{P_a}$ (from Eq. 4-39) $= \dfrac{2{,}500 \text{ kip-in.}}{260 \text{ kips}}$ $= 9.62 \text{ in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_a}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{19.0 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{2(27.9 \text{ kips/in.})}$ $= 4.84 \text{ in.}$

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

Check the inequality of Equation 4-59. Assume that the anchor rod edge distance is 1.50 in. Therefore, from the geometry in Figure 4-33:

$$\begin{aligned} f &= \frac{N}{2} - 1.50 \text{ in.} \\ &= \frac{19.0 \text{ in.}}{2} - 1.50 \text{ in.} \\ &= 8.00 \text{ in.} \end{aligned}$$

$$\begin{aligned} \left(f + \frac{N}{2}\right)^2 &= \left(8.00 \text{ in.} + \frac{19.0 \text{ in.}}{2}\right)^2 \\ &= 306 \text{ in.}^2 \end{aligned}$$

LRFD	ASD
$\dfrac{2P_u(e+f)}{q_{max}} = \dfrac{(2)(376 \text{ kips})(9.57 \text{ in.} + 8.00 \text{ in.})}{42.0 \text{ kips/in.}}$ $= 315 \text{ in.}^2$ Because 315 in.^{2} > 306 in.^{2}, the inequality is not satisfied. Hence, larger plate plan dimensions are required.	$\dfrac{2P_a(e+f)}{q_{max}} = \dfrac{(2)(260 \text{ kips})(9.62 \text{ in.} + 8.00 \text{ in.})}{27.9 \text{ kips/in.}}$ $= 328 \text{ in.}^2$ Because 328 in.^{2} > 306 in.^{2}, the inequality is not satisfied. Hence, larger plate plan dimensions are required.

As the second iteration, try $N = 24.0$ in. $\times$ $B = 22.0$ 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
$f_{p(max)} = 2.21$ ksi (from Example 4.7-10) $q_{max} = f_{p(max)}B$ $= (2.21 \text{ ksi})(22.0 \text{ in.})$ $= 48.6 \text{ kips/in.}$ $f = \dfrac{24.0 \text{ in.}}{2} - 1.50 \text{ in.}$ $= 10.5 \text{ in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_u}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{24.0 \text{ in.}}{2} - \dfrac{376 \text{ kips}}{(2)(48.6 \text{ kips/in.})}$ $= 8.13 \text{ in.}$ The eccentricity, e, still exceeds e_{crit}, therefore, the load combination is for large moments. Also: $\left(f + \dfrac{N}{2}\right)^2 = \left(10.5 \text{ in.} + \dfrac{24.0 \text{ in.}}{2}\right)^2$ $= 506 \text{ in.}^2$ $\dfrac{2P_u(e+f)}{q_{max}} = \dfrac{(2)(376 \text{ kips})(9.57 \text{ in.} + 10.5 \text{ in.})}{48.6 \text{ kips/in.}}$ $= 311 \text{ in.}^2$ 311 in.^{2} < 506 in.^{2}, therefore the inequality in Equation 4-59 is satisfied and a real solution for Y exists.	$F_{p(max)} = 1.47$ ksi (from Example 4.7-10) $q_{max} = f_{p(max)}B$ $= (1.47 \text{ ksi})(22.0 \text{ in.})$ $= 32.3 \text{ kips/in.}$ $f = \dfrac{24.0 \text{ in.}}{2} - 1.50 \text{ in.}$ $= 10.5 \text{ in.}$ $e_{crit} = \dfrac{N}{2} - \dfrac{P_a}{2q_{max}}$ (from Eq. 4-40) $= \dfrac{24.0 \text{ in.}}{2} - \dfrac{260 \text{ kips}}{(2)(32.3 \text{ kips/in.})}$ $= 7.98 \text{ in.}$ The eccentricity, e, still exceeds e_{crit}, therefore, the load combination is for large moments. Also: $\left(f + \dfrac{N}{2}\right)^2 = \left(10.5 \text{ in.} + \dfrac{24.0 \text{ in.}}{2}\right)^2$ $= 506 \text{ in.}^2$ $\dfrac{2P_a(e+f)}{q_{max}} = \dfrac{(2)(260 \text{ kips})(9.62 \text{ in.} + 10.5 \text{ in.})}{32.3 \text{ kips/in.}}$ $= 324 \text{ in.}^2$ 324 in.^{2} < 506 in.^{2}, therefore the inequality in Equation 4-59 is satisfied and a real solution for Y exists.

Determine the bearing length, Y*, and anchor rod tension,* T_u *or* T_a

Use Equation 4-58 and Equation 4-55.

LRFD	ASD
$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}}$ $= \left(10.5\text{ in.} + \frac{24.0\text{ in.}}{2}\right) \pm \sqrt{506\text{ in.}^2 - 311\text{ in.}^2}$ $= 22.5\text{ in.} \pm 14.0\text{ in.}$ $= 8.50\text{ in.}$ $T_u = q_{max}Y - P_u$ $= (48.6\text{ kips/in.})(8.50\text{ in.}) - 376\text{ kips}$ $= 37.1\text{ kips}$	$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_a(e+f)}{q_{max}}}$ $= \left(10.5\text{ in.} + \frac{24.0\text{ in.}}{2}\right) \pm \sqrt{506\text{ in.}^2 - 324\text{ in.}^2}$ $= 22.5\text{ in.} \pm 13.5\text{ in.}$ $= 9.00\text{ in.}$ $T_a = q_{max}Y - P_a$ $= (32.3\text{ kips/in.})(9.00\text{ in.}) - 260\text{ kips}$ $= 30.7\text{ kips}$

Determine the anchor rod size and embedment (LRFD only)

From previous calculations, $T_u = 37.1$ kips. If two anchor rods are used on each face of the column, the force per rod equals 18.6 kips. From Table 4-1, the available design strength of 1-in.-diameter ASTM F1554 Grade 36 anchor rods is 26.4 kips. The recommended hole size for the 1-in.-diameter rod is 1⅞ in. per Table 4-3. Using an edge distance to the center of the hole of 2¾ in., the initial assumption of 1½ in. edge distance must be adjusted. Clearance from the 3-in.-diameter plate washer to the column weld and to the edge of the base plate should be considered, allowing for anchor rods not centered within the oversized base plate hole.

$$
\begin{aligned}
f &= \frac{24.0\text{ in.}}{2} - 2.75\text{ in.} \\
&= 9.25\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
\left(f + \frac{N}{2}\right)^2 &= \left(9.25\text{ in.} + \frac{24.0\text{ in.}}{2}\right)^2 \\
&= 452\text{ in.}^2
\end{aligned}
$$

LRFD	ASD
$\frac{2P_u(e+f)}{q_{max}} = \frac{(2)(376\text{ kips})(9.57\text{ in.} + 9.25\text{ in.})}{48.6\text{ kips/in.}}$ $= 291\text{ in.}^2$ $Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}}$ (from Eq. 4-58) $= \left(9.25\text{ in.} + \frac{24.0\text{ in.}}{2}\right) \pm \sqrt{452\text{ in.}^2 - 291\text{ in.}^2}$ $= 21.3\text{ in.} \pm 12.7\text{ in.}$ $= 8.60\text{ in.}$ $T_u = q_{max}Y - P_u$ (from Eq. 4-55) $= (48.6\text{ kips/in.})(8.60\text{ in.}) - 376\text{ kips}$ $= 42.0\text{ kips}$	$\frac{2P_a(e+f)}{q_{max}} = \frac{(2)(260\text{ kips})(9.62\text{ in.} + 9.25\text{ in.})}{32.3\text{ kips/in.}}$ $= 304\text{ in.}^2$ $Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_a(e+f)}{q_{max}}}$ (from Eq. 4-58) $= \left(9.25\text{ in.} + \frac{24.0\text{ in.}}{2}\right) \pm \sqrt{452\text{ in.}^2 - 304\text{ in.}^2}$ $= 21.3\text{ in.} \pm 12.2\text{ in.}$ $= 9.10\text{ in.}$ $T_a = q_{max}Y - P_a$ (from Eq. 4-55) $= (32.3\text{ kips/in.})(9.10\text{ in.}) - 260\text{ kips}$ $= 33.9\text{ kips}$

The required force per anchor rod after adjusting the edge distance assumption is 21.0 kips. The 1-in.-diameter anchor rods are still adequate.

The design pullout strength of each anchor rod with a heavy hex nut is selected from Table 4-2 as 33.6 kips, which is greater than the required strength per rod of 21.0 kips.

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

Try 18.0 in. of embedment.

The projected concrete failure area on the top face of the concrete foundation is calculated using ACI 318, Section 17.6.2.1.1. If the anchor rods are spaced 12.0 in. apart, the total breakout area for the two anchors considered in tension is calculated by:

$$\begin{aligned} A_{Nc} &= 2(1.5h_{ef})(1.5h_{ef} + s + 1.5h_{ef}) \\ &= 2(1.5)(18.0\text{ in.})[1.5(18.0\text{ in.}) + 12.0\text{ in.} + 1.5(18.0\text{ in.})] \\ &= 3{,}560\text{ in.}^2 \end{aligned}$$

The projected area for a single anchor in a deep member is given by:

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 \\ &= 9(18.0\text{ in.})^2 \\ &= 2{,}920\text{ in.}^2 \end{aligned} \qquad \text{(ACI 318, Eq. 17.6.2.1.4)}$$

The basic concrete breakout strength of a single anchor with 11.0 in. $\leq h_{ef} \leq$ 25.0 in. is given by:

$$\begin{aligned} N_b &= 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3} \\ &= 16(1.0)\sqrt{4{,}000\text{ psi}}\,(18.0\text{ in.})^{5/3}\left(\frac{1\text{ kip}}{1{,}000\text{ lbf}}\right) \\ &= 125\text{ kips} \end{aligned} \qquad \text{(ACI 318, Eq. 17.6.2.2.3)}$$

Because the anchor group is not loaded eccentrically, $\psi_{ec,N}$ = 1.0 per ACI 318, Section 17.6.2.3.

Because $c_{a,min} \geq 1.5h_{ef}$, a reduction is not required per ACI 318, Section 17.6.2.4:

$$\psi_{ed,N} = 1.0 \qquad \text{(ACI 318, Eq. 17.6.2.4.1a)}$$

Use $\psi_{c,N}$ = 1.0 per ACI 318, Section 17.6.2.5, for cracked concrete.

Because the anchors are cast-in-place, $\psi_{cp,N}$ = 1.0 per ACI 318, Section 17.6.2.6.2.

The nominal concrete breakout strength of the anchor group is then given by:

$$\begin{aligned} N_{cbg} &= \frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \\ &= \left(\frac{3{,}560\text{ in.}^2}{2{,}920\text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(125\text{ kips}) \\ &= 152\text{ kips} \end{aligned} \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

The available tension breakout capacity may then be determined using, ϕ = 0.70 per ACI 318, Table 17.5.3(b):

$$\begin{aligned} \phi N_{cbg} &= 0.70(152\text{ kips}) \\ &= 106\text{ kips} > 42.0\text{ kips} \quad \textbf{o.k.} \end{aligned}$$

Determine the minimum base plate thickness

At the bearing interface:

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$= \frac{24.0 \text{ in.} - 0.95(12.7 \text{ in.})}{2}$$

$$= 5.97 \text{ in.}$$

The bearing stress between the plate and concrete is as follows:

LRFD	ASD
From Example 4.7-10: $f_p = f_{p(max)}$ $= 2.21 \text{ ksi}$	From Example 4.7-10: $f_p = f_{p(max)}$ $= 1.47 \text{ ksi}$

Because $Y \geq m$, the minimum plate thickness may be calculated using Equation 4-51:

LRFD	ASD
$t_{p(req)} = 1.49m\sqrt{\frac{f_{p(max)}}{F_y}}$ $= (1.49)(5.97 \text{ in.})\sqrt{\frac{2.21 \text{ ksi}}{50 \text{ ksi}}}$ $= 1.87 \text{ in.}$	$t_{p(req)} = 1.83m\sqrt{\frac{f_{p(max)}}{F_y}}$ $= (1.83)(5.97 \text{ in.})\sqrt{\frac{1.47 \text{ ksi}}{50 \text{ ksi}}}$ $= 1.87 \text{ in.}$

Check the thickness required using the value of n:

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$= \frac{22.0 \text{ in.} - 0.8(12.2 \text{ in.})}{2}$$

$$= 6.12 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = (1.49)(6.12 \text{ in.})\sqrt{\frac{2.21 \text{ ksi}}{50 \text{ ksi}}}$ $= 1.92 \text{ in.}$ **controls**	$t_{p(req)} = (1.83)(6.12 \text{ in.})\sqrt{\frac{1.47 \text{ ksi}}{50 \text{ ksi}}}$ $= 1.92 \text{ in.}$ **controls**

At the tension interface:

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{4-61}$$

$$= 9.25 \text{ in.} - \frac{12.7 \text{ in.}}{2} + \frac{0.900 \text{ in.}}{2}$$

$$= 3.35 \text{ in.}$$

LRFD	ASD
$t_{p(req)} = 2.11\sqrt{\dfrac{T_u x}{BF_y}}$ (4-62a) $= 2.11\sqrt{\dfrac{(42.0\text{ kips})(3.35\text{ in.})}{(22.0\text{ in.})(50\text{ ksi})}}$ $= 0.755$ in.	$t_{p(req)} = 2.58\sqrt{\dfrac{T_a x}{BF_y}}$ (4-62b) $= 2.58\sqrt{\dfrac{(33.9\text{ kips})(3.35\text{ in.})}{(22.0\text{ in.})(50\text{ ksi})}}$ $= 0.829$ in.

The bearing interface governs the design of the base plate thickness. Use a plate thickness of 2 in.

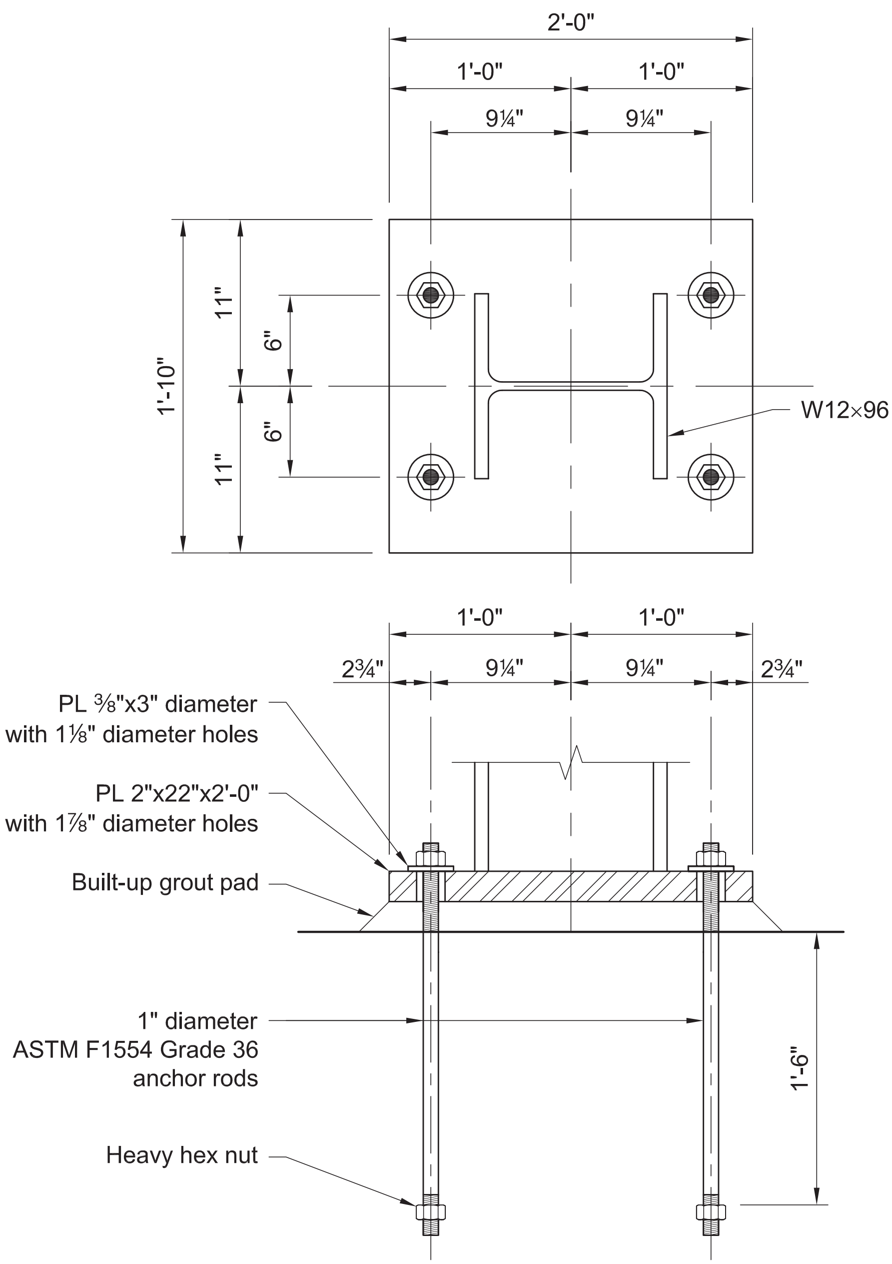


Fig. 4-33. Base connection as detailed in Example 4.7-11.

EXAMPLE 4.7-12—Base Connection for Bending with Anchor Rod Tension (Large Moment)

A base connection for a wide-flange column subject to tension and moment is designed in this example. The ratio of flexure to tension in this example is such that the moment cannot be resisted without producing compression bearing against the supporting concrete.

Given:

A W12×96 column is subjected to axial wind uplift equal to $1.0W = 100$ kips and moment from wind load equal to $1.0W = 1{,}000$ kip-in., as shown in Figure 4-34. Bending is about the strong axis of the W12×96. Consider the ratio of the concrete to base plate area as unity. The column is anchored to the foundation using ASTM F1554 Grade 36 anchor rods. The column is attached to a concrete foundation with a specified compressive strength of concrete, $f_c' = 4{,}000$ psi. Determine the anchor rod tension and compression bearing length.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
$b_f = 12.2$ in.
$d = 12.7$ in.

Solution:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.0(100 \text{ kips})$ $= 100 \text{ kips}$ $M_u = 1.0(1{,}000 \text{ kip-in.})$ $= 1{,}000 \text{ kip-in.}$	$P_a = 0.6(100 \text{ kips})$ $= 60.0 \text{ kips}$ $M_a = 0.6(1{,}000 \text{ kip-in.})$ $= 600 \text{ kip-in.}$

Choose trial base plate size

The base plate dimensions $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA. Consider N and B to be at least 3 in. larger than the outside column dimensions.

$$N > d + (2)(3.00 \text{ in.}) = 18.7 \text{ in.}$$
$$B > b_f + (2)(3.00 \text{ in.}) = 18.2 \text{ in.}$$

Try $N = 19.0$ in. and $B = 19.0$ in.

Determine e *and confirm force distribution model*

Check the inequality $e > f$ to determine if the moment is large enough to form a compression block under the base plate.

LRFD	ASD
$e = \dfrac{M_u}{P_u}$ $= \dfrac{1{,}000 \text{ kip-in.}}{100 \text{ kips}}$ $= 10.0 \text{ in.}$	$e = \dfrac{M_a}{P_a}$ $= \dfrac{600 \text{ kip-in.}}{60.0 \text{ kips}}$ $= 10.0 \text{ in.}$

Assume that the anchor rod edge distance is 1.50 in. Therefore, from the geometry in Figure 4-34:

$$\begin{aligned} f &= \frac{N}{2} - 1.50 \text{ in.} \\ &= \frac{19.0 \text{ in.}}{2} - 1.50 \text{ in.} \\ &= 8.00 \text{ in.} \end{aligned}$$

Because $e > f$, this is the case of a base plate with a large moment such that there will be a compression block under the base plate and tension in two of the four anchor rods.

Check the inequality of Equation 4-66.

$$\begin{aligned} \left(f + \frac{N}{2}\right)^2 &= \left(8.00 \text{ in.} + \frac{19.0 \text{ in.}}{2}\right)^2 \\ &= 306 \text{ in.}^2 \end{aligned}$$

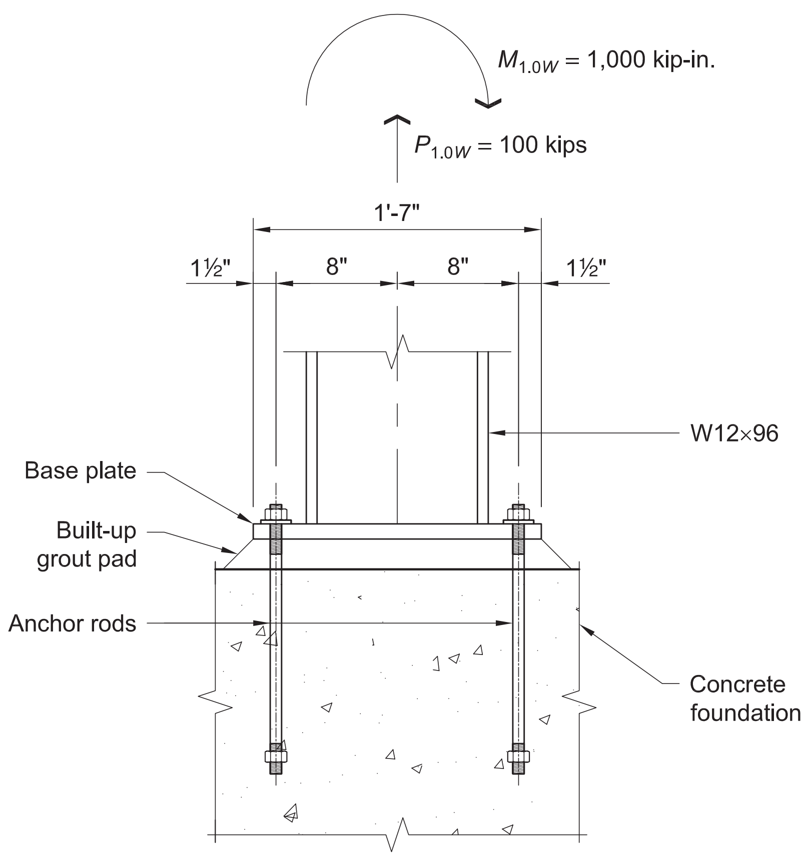


Fig. 4-34. Base connection configuration as detailed in Example 4.7-12.

LRFD	ASD
$q_{max} = 42.0$ kips/in. (see Example 4.7-10) $\dfrac{2P_u(e-f)}{q_{max}} = \dfrac{(2)(100\text{ kips})(10.0\text{ in.}-8.00\text{ in.})}{42.0\text{ kips/in.}}$ $= 9.52\text{ in.}^2$ Because 306 in.^{2} > 9.52 in.^{2}, the inequality is satisfied.	$q_{max} = 27.9$ kips/in. (see Example 4.7-10) $\dfrac{2P_a(e-f)}{q_{max}} = \dfrac{(2)(60.0\text{ kips})(10.0\text{ in.}-8.00\text{ in.})}{27.9\text{ kips/in.}}$ $= 8.60\text{ in.}^2$ Because 306 in.^{2} > 8.60 in.^{2}, the inequality is satisfied.

Determine the bearing length, Y*, and anchor rod tension,* T_u *or* T_a

LRFD	ASD
$Y = \left(f+\dfrac{N}{2}\right) \pm \sqrt{\left(f+\dfrac{N}{2}\right)^2 - \dfrac{2P_u(e-f)}{q_{max}}}$ (from Eq. 4-65) $= \left(8.00\text{ in.}+\dfrac{19.0\text{ in.}}{2}\right) \pm \sqrt{306\text{ in.}^2 - 9.52\text{ in.}^2}$ $= 17.5\text{ in.} \pm 17.2\text{ in.}$ $= 0.300$ in. From Equation 4-63: $T_u = q_{max}Y + P_u$ $= (42.0\text{ kips/in.})(0.300\text{ in.}) + 100\text{ kips}$ $= 113$ kips	$Y = \left(f+\dfrac{N}{2}\right) \pm \sqrt{\left(f+\dfrac{N}{2}\right)^2 - \dfrac{2P_a(e-f)}{q_{max}}}$ (from Eq. 4-65) $= \left(8.00\text{ in.}+\dfrac{19.0\text{ in.}}{2}\right) \pm \sqrt{306\text{ in.}^2 - 8.60\text{ in.}^2}$ $= 17.5\text{ in.} \pm 17.2\text{ in.}$ $= 0.300$ in. From Equation 4-63: $T_a = q_{max}Y + P_a$ $= (27.9\text{ kips/in.})(0.300\text{ in.}) + 60.0\text{ kips}$ $= 68.4$ kips

EXAMPLE 4.7-13—Base Connection for Bending with Anchor Rod Tension (Low Moment)

A base connection for a wide-flange column subject to tension and moment is designed in this example. The ratio of flexure to tension in this example is such that the moment can be resisted without producing compression bearing against the supporting concrete, and all anchor rods will be in varying levels of tension.

Given:

A W12×96 column is subjected to axial wind uplift equal to $1.0W = 100$ kips and moment from wind load equal to $1.0W = 100$ kip-in., as shown in Figure 4-35. Bending is about the major axis of the column. The column is anchored to the foundation using ASTM F1554 Grade 36 anchor rods. Determine the anchor rod tension.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
$b_f = 12.2$ in.
$d = 12.7$ in.

Solution:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.0(100 \text{ kips})$ $= 100 \text{ kips}$ $M_u = 1.0(100 \text{ kip-in.})$ $= 100 \text{ kip-in.}$	$P_a = 0.6(100 \text{ kips})$ $= 60.0 \text{ kips}$ $M_a = 0.6(100 \text{ kip-in.})$ $= 60.0 \text{ kip-in.}$

Choose trial base plate size

The base plate dimensions $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA. Consider N and B to be at least 3 in. larger than the outside column dimensions.

$$N > d + (2)(3.00 \text{ in.}) = 18.7 \text{ in.}$$
$$B > b_f + (2)(3.00 \text{ in.}) = 18.2 \text{ in.}$$

Try N = 19.0 in. and B = 19.0 in.

Determine e *and confirm force distribution model*

Check the inequality to confirm the moment is not large enough to form a compression block under the base plate.

LRFD	ASD
$e = \dfrac{M_u}{P_u}$ (from Eq. 4-39) $= \dfrac{100 \text{ kip-in.}}{100 \text{ kips}}$ $= 1.00 \text{ in.}$	$e = \dfrac{M_a}{P_a}$ (from Eq. 4-39) $= \dfrac{60.0 \text{ kip-in.}}{60.0 \text{ kips}}$ $= 1.00 \text{ in.}$

Assume that the anchor rod edge distance is 1.5 in. Therefore, from the geometry in Figure 4-35:

$$\begin{aligned} f &= \frac{N}{2} - 1.50 \text{ in.} \\ &= \frac{19.0 \text{ in.}}{2} - 1.50 \text{ in.} \\ &= 8.00 \text{ in.} \end{aligned}$$

Because $e < f$, this is the case of a base plate with a low moment such that there will be no compression block under the base plate and tension in all four anchor rods. The resulting anchor rod tension consists of the axial tension split equally to all four anchors and the moment resolved as a force couple. The anchors are symmetrically located with the center of the column resulting in each anchor being a distance, f, from the centroid of the anchor group. Therefore, the coordinates of each anchor and the resulting moment of inertia of the group is given by:

$$y = [8.00 \text{ in.}, 8.00 \text{ in.}, -8.00 \text{ in.}, -8.00 \text{ in.}]$$

$$
\begin{aligned}
I_x &= \Sigma(y_i)^2 \\
&= (8.00\text{ in.})^2 + (8.00\text{ in.})^2 + (-8.00\text{ in.})^2 + (-8.00\text{ in.})^2 \\
&= 256\text{ in.}^4/\text{in.}^2
\end{aligned}
$$

Using Equation 4-67:

LRFD	ASD
$r_{u,i} = \dfrac{P_u}{n} + \dfrac{(P_u e)y_i}{I_x}$ $= \dfrac{100\text{ kips}}{4\text{ rods}} + \dfrac{(100\text{ kips})(1.00\text{ in.})(\pm 8.00\text{ in.})}{256\text{ in.}^4/\text{in.}^2}$ $= 25.0\text{ kips} \pm 3.13\text{ kips}$	$r_{a,i} = \dfrac{P_a}{n} + \dfrac{(P_a e)y_i}{I_x}$ $= \dfrac{60.0\text{ kips}}{4\text{ rods}} + \dfrac{(60.0\text{ kips})(1.00\text{ in.})(\pm 8.00\text{ in.})}{256\text{ in.}^4/\text{in.}^2}$ $= 15.0\text{ kips} \pm 1.88\text{ kips}$
Therefore, there is 28.1 kips each in two of the rods and 21.9 kips each in the other two rods.	Therefore, there is 16.9 kips each in two of the rods and 13.1 kips each in the other two rods.

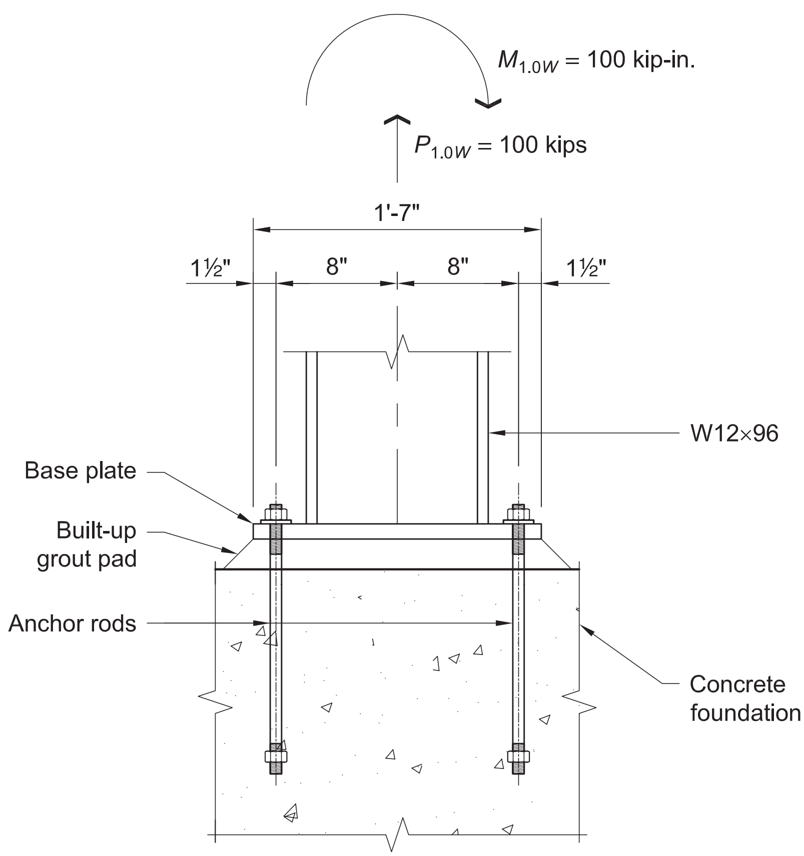


Fig. 4-35. Base configuration and dimensions used in Example 4.7-13.

EXAMPLE 4.7-14—Base Connection for Biaxial Bending with Axial Compression

Given:

Design a base plate for the factored compressive axial load $P_u = 90.0$ kips and the design moments $M_{ux} = 700$ kip-in. about the strong axis and $M_{uy} = 450$ kip-in. about the weak axis of a W8×48 column. Assume that the ratio of the footing to base plate area is equal to 4.00. The base plate is ASTM A572/A572M Grade 50 material, and the compressive strength of concrete, f_c', is 4 ksi. Use ASTM F1554 Grade 55 anchor rods.

Solution:

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

Anchor rods
ASTM F1554 Grade 55
$F_y = 55$ ksi
$F_u = 75$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W8×48
$d = 8.50$ in.
$b_f = 8.11$ in.
$t_f = 0.685$ in.

1. Choose the trial plate size.

Try $N = 14.0$ in. and $B = 14.0$ in.

Assume that the anchor rod edge distance is 1.50 in. in each direction. Therefore:

$$\begin{aligned} f &= \frac{N}{2} - 1.50 \text{ in.} \\ &= \frac{14.0 \text{ in.}}{2} - 1.50 \text{ in.} \\ &= 5.50 \text{ in.} \end{aligned}$$

Because the plate is square, the value of $f = 5.50$ in. is true for both strong- and weak-axis bending.

2. Determine e and e_{crit}; check the inequality in Equation 4-53 to determine if this is a low or high moment case.

First estimate $f_{p(max)}$:

$$\begin{aligned} f_{p(max)} &= \phi_c(0.85 f_c')\sqrt{\frac{A_2}{A_1}} \qquad \text{(from Eq. 4-2)} \\ &= 0.65(0.85)(4 \text{ ksi})\sqrt{4.00} \\ &= 4.42 \text{ ksi} \end{aligned}$$

$$\begin{aligned} q_{max} &= f_{p(max)}B \qquad \text{(4-37)} \\ &= (4.42 \text{ ksi})(14.0 \text{ in.}) \\ &= 61.9 \text{ kip/in.} \end{aligned}$$

$$e_{crit,x} = \frac{N}{2} - \frac{P}{2q_{max}} \qquad \text{(from Eq. 4-53)}$$

$$e_{crit,y} = \frac{B}{2} - \frac{P}{2q_{max}} \quad \text{(from Eq. 4-53)}$$

Using $N = B = 14.0$ in.,

$$\begin{aligned} e_{crit} &= \frac{N}{2} - \frac{P}{2q_{max}} \quad \text{(from Eq. 4-53)} \\ &= \frac{14.0 \text{ in.}}{2} - \frac{90.0 \text{ kip}}{(2)(61.9 \text{ kip/in.})} \\ &= 6.27 \text{ in.} \end{aligned}$$

The eccentricity in the strong-axis direction may be calculated as:

$$\begin{aligned} e_x &= \frac{M_{ux}}{P_u} \quad \text{(from Eq. 4-37)} \\ &= \frac{700 \text{ kip-in.}}{90.0 \text{ kips}} \\ &= 7.78 \text{ in.} \end{aligned}$$

The eccentricity in the weak-axis direction may be calculated as:

$$\begin{aligned} e_y &= \frac{M_{uy}}{P_u} \quad \text{(from Eq. 4-37)} \\ &= \frac{450 \text{ kip-in.}}{90.0 \text{ kips}} \\ &= 5.00 \text{ in.} \end{aligned}$$

This indicates that this is a high-moment condition in the strong-axis direction, and a low-moment condition in the weak-axis direction.

3. Determine bearing length, Y, and anchor rod tension, T_u, due to bending in the strong-axis direction.

$$\begin{aligned} Y &= \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e_x + f)}{q_{max}}} \quad \text{(from Eq. 4-58)} \\ &= \left(5.50 \text{ in.} + \frac{14.0 \text{ in.}}{2}\right) \pm \sqrt{\left(5.50 \text{ in.} + \frac{14.0 \text{ in.}}{2}\right)^2 - \frac{2(90.0 \text{ kips})(7.78 \text{ in.} + 5.50 \text{ in.})}{61.9 \text{ kip/in.}}} \\ &= 12.5 \text{ in.} \pm 10.8 \text{ in.} \\ &= 1.70 \text{ in.} \end{aligned}$$

$$\begin{aligned} T_u &= q_{max}Y - P_u \quad \text{(from Eq. 4-55)} \\ &= (61.9 \text{ kip/in.})(1.70 \text{ in.}) - 90.0 \text{ kips} \\ &= 15.2 \text{ kips} \end{aligned}$$

A trial anchor size and base plate thickness may be estimated for this condition, and then upsized anticipating additional loading from weak-axis bending.

4. Determine trial anchor rod size.

If two anchor rods are used on each face of the column, the force per rod is 7.60 kips. From Table 4-1, the design tensile strength of a ⅝-in.-diameter ASTM F1554 Grade 55 anchor rod is 12.7 kips. This size may be used conservatively, recognizing that the calculated anchor force does not include weak-axis bending, whose magnitude is approximately equal to that of strong-axis bending. It is assumed here that the embedment of the anchor rod is designed to prevent pullout and other concrete limit states.

5. Determine trial base plate thickness.

For this, consider base plate yielding at both the bearing and tension interface due to strong-axis bending. For the bearing interface, determine m and n:

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$= \frac{14.0 \text{ in.} - 0.95(8.50 \text{ in.})}{2}$$

$$= 2.96 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} \tag{4-11}$$

$$= \frac{14.0 \text{ in.} - 0.8(8.11 \text{ in.})}{2}$$

$$= 3.76 \text{ in.}$$

Because $n > m$ and for Grade 50 material,

$$t_{p(req)} = 1.49n\sqrt{\frac{f_{p(max)}}{F_y}} \qquad \text{(from Eq. 4-51a)}$$

This indicates bending along a yield line parallel to the web, without considering the additional effective width outlined in Appendix B.

$$t_{p(req)} = 1.49(3.76 \text{ in.})\sqrt{\frac{4.42 \text{ ksi}}{50 \text{ ksi}}}$$

$$= 1.67 \text{ in.}$$

For the tension interface,

$$t_{p(req)} = 2.11\sqrt{\frac{T_u x}{BF_y}} \tag{4-62a}$$

where

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{4-61}$$

$$= 5.50 \text{ in.} - \frac{8.50 \text{ in.}}{2} + \frac{0.685 \text{ in.}}{2}$$

$$= 1.59 \text{ in.}$$

Thus,

$$t_{p(req)} = 2.11\sqrt{\frac{(15.2 \text{ kips})(1.59 \text{ in.})}{(14.0 \text{ in.})(50 \text{ ksi})}}$$

$$= 0.392 \text{ in.}$$

Base plate yielding at the bearing interface governs. Select a base plate of the following dimensions.

$B = N = 14.0$ in. and $t_p = 1\frac{3}{4}$ in.

6. Estimate the moment strength of the base connection in each direction of bending.

Note that because the anchors and plate thickness are selected conservatively with respect to the induced loading in them, the strength of the connection in each direction needs to be determined based on the selected dimensions.

For strong-axis bending:

Connection capacity due to anchor rod failure will be achieved when:

$$T_u = 2(12.7 \text{ kips})$$
$$= 25.4 \text{ kips}$$

$$T_u = q_{max} Y - P_u \qquad \text{(from Eq. 4-55)}$$

Thus,

$$Y = \frac{T_u + P_u}{q_{max}}$$
$$= \frac{25.4 \text{ kips} + 90.0 \text{ kips}}{61.9 \text{ kip/in.}}$$
$$= 1.86 \text{ in.}$$

Because,

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e_x + f)}{q_{max}}} \qquad \text{(from Eq. 4-58)}$$

A value of $e_x = 9.30$ in. may be determined by setting $Y = 1.86$ in.

This results in the moment capacity in the strong-axis direction due to yielding of the anchors as:

$$M_{x,P_u}^{Anchors} = e_x P_u$$
$$= (9.30 \text{ in.})(90.0 \text{ kips})$$
$$= 837 \text{ kip-in.}$$

Note that although the connection may be classified as low moment for weak-axis bending for the given moment, the moment capacity in weak-axis bending assumes that axial load is held constant and the moment is increased to its capacity. In this context, because the plate is square with a symmetrical anchor layout, failure will be obtained under the high-moment condition such that $M_{x,P_u}^{Anchors} = M_{y,P_u}^{Anchors} = 837$ kip-in.

In strong-axis bending for the bearing interface, the maximum possible moment in the base plate is:

$$\frac{f_{p(max)} N n^2}{2} = \frac{(4.42 \text{ ksi})(14.0 \text{ in.})(3.76 \text{ in.})^2}{2}$$
$$= 437 \text{ kip-in.}$$

This assumes the stress, $f_{p(max)}$, is developed under the entire base plate. The moment capacity of the yield line is:

$$\phi F_y \frac{N t_p^2}{4} = (0.90)(50 \text{ ksi}) \frac{(14.0 \text{ in.})(1\tfrac{3}{4} \text{ in.})^2}{4}$$
$$= 482 \text{ kip-in.}$$

This indicates that base plate yielding at the bearing interface is not possible.

The connection capacity due to base plate yielding at the tension interface may be calculated by setting $t_{p(req)} = 1\tfrac{3}{4}$ in. in the following equation:

$$t_{p(req)} = 2.11\sqrt{\frac{T_u x}{B F_y}} \qquad \text{(4-62a)}$$

This results in T_u = 303 kips, which is significantly greater than the capacity of the anchors (25.4 kips), indicating that plate bending at the tension interface will not govern. The moment capacity in the strong-axis direction is thus governed by anchor rod failure, such that:

$$M_{x,P_u} = M_{x,P_u}^{Anchors} = 837 \text{ kip-in.}$$

In the weak-axis direction, the moment capacity due to failure of the anchors has already been determined as $M_{y,P_u}^{Anchors}$ = 837 kip-in. Also, as for strong-axis bending, yielding of the base plate at the bearing interface is not possible, because the yield line is identical to that for strong-axis bending. The weak-axis strength due to yielding of the base plate at the tension interface may be determined by setting $t_{p(req)}$ = 1¾ in. in the following equation:

$$t_{p(req)} = 2.11\sqrt{\frac{T_u y}{NF_y}}$$

The term y corresponds to the cantilever distance from the yield line to the anchors and may be taken as $y = n - 1.50$ in., where 1.50 in. is the edge distance of the anchor holes. Consequently,

$$1.75 \text{ in.} = 2.11\sqrt{\frac{T_u(3.76 \text{ in.} - 1.50 \text{ in.})}{(14.0 \text{ in.})(50 \text{ ksi})}}$$

This results in T_u = 213 kips, which is significantly higher than the capacity of the anchors (25.4 kips), indicating that yielding of the base plate will not govern. As a result, yielding of the anchors will control the connection strength in the weak-axis direction.

$$M_{y,P_u} = M_{y,P_u}^{Anchors} = 837 \text{ kip-in.}$$

Once the moment strength in each direction is determined, the interaction equation may be used.

$$\left(\frac{M_{ux}}{M_{x,P_u}}\right)^2 + \left(\frac{M_{uy}}{M_{y,P_u}}\right)^2 = \left(\frac{700 \text{ kip-in.}}{837 \text{ kip-in.}}\right)^2 + \left(\frac{450 \text{ kip-in.}}{837 \text{ kip-in.}}\right)^2 \qquad \text{(from Eq. 4-69)}$$
$$= 0.988 \le 1$$

This is an acceptable design. Note that other limit states for concrete have not been considered here and must be addressed as they are for base connections under uniaxial bending and compression.

EXAMPLE 4.7-15—Anchor Reinforcement Design

Anchor reinforcement to preclude concrete breakout in tension is designed in this example. The anchor reinforcement is designed to transfer the entire required strength across the concrete breakout cone plane.

Given:

Four ⅞-in.-diameter ASTM F1554 Grade 36 anchor rods with a heavy hex nut and 4.00 in. × 4.00 in. spacing are embedded in the center of a 20.0-in.-square concrete column. The concrete column has a specified compressive strength of concrete, f_c' = 4,000 psi. Any required anchorage reinforcement will consist of ASTM A615/A615M Grade 60 (f_y = 60,000 psi) deformed bars.

It is required to confirm if the concrete will have adequate concrete breakout strength in tension to resist the required strengths. If the concrete breakout strength in tension is less than the required strength, determine the anchor reinforcement configuration necessary to preclude concrete breakout in tension and to resist the required strength. Finally, confirm that the anchorage will have adequate side-face blowout strength.

Verification of the steel anchor rod capacity and pullout capacity are addressed in Example 4.7-3.

The required strengths due to axial tensile loads is:

P_u = 70.0 kips (uplift)

Solution:

Determine the concrete breakout strength in tension

As the anchors are installed in a 20.0-in.-square concrete column, the concrete breakout strength would be limited by the column cross section. With an 8.00 in. maximum edge distance, the effective h_{ef} need only be 8.00 in./1.5 = 5.33 to have the breakout cone area equal the column cross-sectional area. Based on the procedure in ACI 318, Section 17.6.2.1.2, this leads to:

$$\begin{aligned} h_{ef} &= \max\left(c_{a,max}/1.5,\ s/3\right) \\ &= \max\left(8.00 \text{ in.}/1.5,\ 4 \text{ in.}/3\right) \\ &= \max\left(5.33 \text{ in.},\ 1.33 \text{ in.}\right) \\ &= 5.33 \text{ in.} \end{aligned}$$

$$\begin{aligned} A_{Nc} &= \left(1.5h_{ef} + s_1 + 1.5h_{ef}\right)\left(1.5h_{ef} + s_2 + 1.5h_{ef}\right) \\ &= \left[1.5(5.33 \text{ in.}) + (4.00 \text{ in.}) + 1.5(5.33 \text{ in.})\right]\left[1.5(5.33 \text{ in.}) + (4.00 \text{ in.}) + 1.5(5.33 \text{ in.})\right] \\ &= 400 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 && \text{(ACI 318, Eq. 17.6.2.1.4)} \\ &= 9(5.33 \text{ in.})^2 \\ &= 256 \text{ in.}^2 \end{aligned}$$

Because the tensile load is concentric with the anchor group, $e'_N = 0$ in.

$$\begin{aligned} \psi_{ec,N} &= \frac{1}{\left(1 + \dfrac{e'_N}{1.5h_{ef}}\right)} \le 1 && \text{(ACI 318, Eq. 17.6.2.3.1)} \\ &= \frac{1}{\left[1 + \dfrac{0 \text{ in.}}{1.5(5.33 \text{ in.})}\right]} \le 1 \\ &= 1.00 \end{aligned}$$

Because the edge distance equals $1.5h_{ef}$, the edge distance factor is calculated per ACI 318, Section 17.6.2.4.1, as:

$$\psi_{ed,N} = 1.0$$

Because no analysis was performed, consider the concrete to be cracked at service load levels, use $\psi_{c,N} = 1.0$, in accordance with ACI 318, Section 17.6.2.5.1(b).

For cast-in anchors, the factor representing breakout splitting is determined as $\psi_{cp,N} = 1.0$ per ACI 318, Section 17.6.2.6.2.

From ACI 318, Section 17.6.2.2, $k_c = 24$ for cast-in anchors and for $h_{ef} < 11.0$ in.,

$$\begin{aligned} N_b &= k_c \lambda_a \sqrt{f'_c}\, {h_{ef}}^{1.5} && \text{(ACI 318, Eq. 17.6.2.2.1)} \\ &= (24)(1.0)\sqrt{4{,}000 \text{ psi}}\,(5.33 \text{ in.})^{1.5}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right) \\ &= 18.7 \text{ kips} \end{aligned}$$

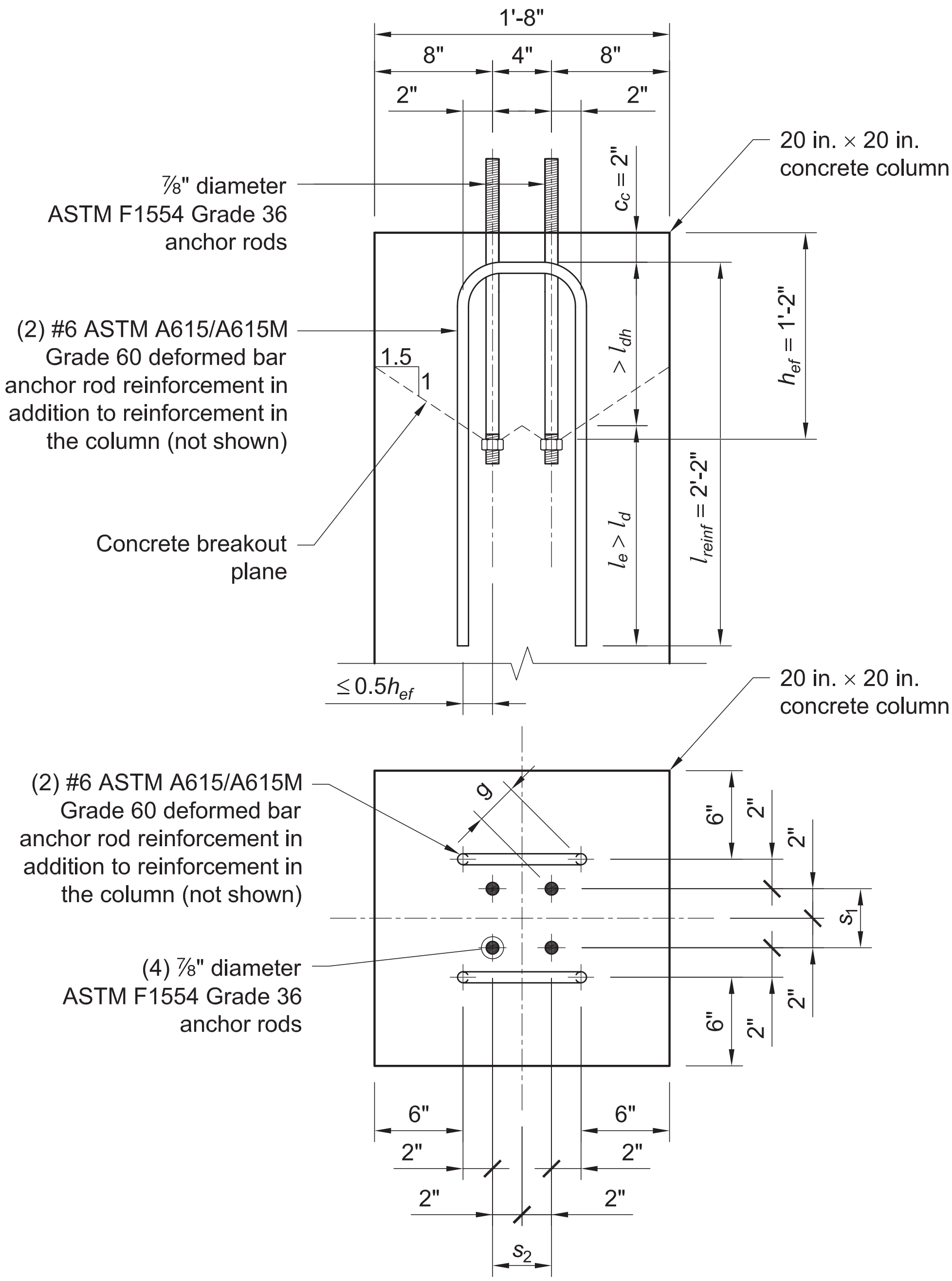


Fig. 4-36. Anchor and reinforcement detailed in Example 4.7-15.

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

$$= \left(\frac{400 \text{ in.}^2}{256 \text{ in.}^2} \right)(1.0)(1.0)(1.0)(1.0)(18.7 \text{ kips})$$

$$= 29.2 \text{ kips}$$

Because no supplementary reinforcement was specified, $\phi = 0.70$ per ACI 318, Table 17.5.3(b), and

$$\phi N_{cbg} = 0.70(29.2 \text{ kips})$$

$$= 20.4 \text{ kips} < 70.0 \text{ kips} \quad \textbf{n.g.}$$

Thus, it is necessary to transfer the anchor load to the column using anchor reinforcement.

Determine the anchor reinforcement required to preclude concrete breakout in tension

The required area of steel is determined according to ACI 318, Sections 17.5.2.1 and 17.5.3, as:

$$\phi = 0.75 \qquad \text{(ACI 318, Section 17.5.3)}$$

$$A_{s,req} = \frac{R_u}{\phi f_y}$$

$$= \frac{70.0 \text{ kips}}{0.75(60 \text{ ksi})}$$

$$= 1.56 \text{ in.}^2$$

Use 4-#6 bars, and consider these bars are only being used and designed as anchor reinforcement.

$$A_s = (4 \text{ bars})\left(0.44 \frac{\text{in.}^2}{\text{bar}}\right)$$

$$= 1.76 \text{ in.}^2 > 1.56 \text{ in.}^2 \quad \textbf{o.k.}$$

With the bars located as shown in Figure 4-36, the horizontal distance, g, from the center of the anchor to the center of the reinforcing steel is determined by:

$$g = (2.00 \text{ in.})\sqrt{2}$$

$$= 2.83 \text{ in.}$$

The reinforcing steel used as anchor reinforcement must be developed in accordance with ACI 318, Section 17.5.2.1(a), on both sides of the concrete breakout surface using the development length calculated per ACI 318, Chapter 25. The development length for hooks, l_{dh}, will be used above the breakout plane, and the development length for unhooked bars, l_d, will be used below the breakout plane.

For normal-weight concrete, and #6 ASTM A615/A615M Grade 60 uncoated, hooked reinforcement, with a center-to-center spacing greater than $6d_b$ and side cover normal to the plane of the hook greater than or equal to $6d_b$, the development factors are given in ACI 318, Table 25.4.3.2.

The basic development length for bars with standard hooks is then given by ACI 318, Section 25.4.3.1.

$$l_{dh} = \left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}} \right) d_b^{1.5}$$

$$\lambda = 1.0$$

$$\psi_e = 1.0$$

$$\psi_r = 1.0$$

$$\psi_o = 1.0$$

$$\psi_c = \frac{f_c'}{15{,}000} + 0.6$$
$$= \frac{4{,}000 \text{ psi}}{15{,}000 \text{ psi}} + 0.6$$
$$= 0.867$$

Therefore,

$$l_{dh} = \left[\frac{(60{,}000\,\text{psi})(1.0)(1.0)(1.0)(0.867)}{55(1.0)\sqrt{4{,}000 \text{ psi}}}\right](0.750 \text{ in.})^{1.5}$$
$$= 9.71 \text{ in.}$$

The additional limits of ACI 318, Section 25.4.3.1, items (b) and (c), do not govern and are given by:

$$l_{dh} = 8d_b$$
$$= 8(0.750 \text{ in.})$$
$$= 6.00 \text{ in.}$$

$$l_{dh} = 6.00 \text{ in.}$$

Therefore, the minimum required embedment length is illustrated in Figure 4-36 and calculated by:

$$h_{ef} = l_{dh} + g\left(\frac{1}{1.5}\right) + c_c$$
$$= 9.71 \text{ in} + (2.83 \text{ in.})\left(\frac{1}{1.5}\right) + 2.00 \text{ in.}$$
$$= 13.6 \text{ in.}$$

Select a 14.0 in. embedment for the anchors.

For normal-weight concrete, with the effect of transverse reinforcement neglected, and #6 ASTM A615/A615M Grade 60 uncoated, vertical reinforcement, the development factors are given in ACI 318, Table 25.4.2.5.

The basic development length is then given by ACI 318-19(22), Section 25.4.2.4.

$$L_d = \left[\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s\psi_g}{\left(\dfrac{c_b + K_{tr}}{d_b}\right)}\right]d_b \qquad \text{(ACI 318, Eq. 25.4.2.4a)}$$

$$\lambda = 1.0$$
$$\psi_g = 1.0$$
$$\psi_e = 1.0$$
$$\psi_s = 0.8$$
$$\psi_t = 1.0$$
$$K_{tr} = 0$$

The confinement term based on the spacing and cover dimensions shown in Figure 4-36 is calculated by:

$$c_b = \min\begin{Bmatrix}6.00 \text{ in.,}\\ (2.00 \text{ in.} + 2.00 \text{ in.} + 2.00 \text{ in.} + 2.00 \text{ in.})/2\end{Bmatrix}$$
$$= \min\begin{Bmatrix}6.00 \text{ in.,}\\ 4.00 \text{ in.}\end{Bmatrix}$$
$$= 4.00 \text{ in.}$$

$$\left(\frac{c_b + K_{tr}}{d_b}\right) = \left(\frac{4.00 \text{ in.} + 0}{0.750 \text{ in.}}\right) \le 2.5$$
$$= 5.33 \le 2.5$$
$$= 2.5$$

Therefore,

$$l_d = \left[\frac{3}{40}\left(\frac{60{,}000 \text{ psi}}{1.00\sqrt{4{,}000 \text{ psi}}}\right)\left(\frac{(1.0)(1.0)(0.8)(1.0)}{2.5}\right)\right](0.750 \text{ in.})$$
$$= 17.1 \text{ in.}$$

The required development length may be reduced in accordance with ACI 318, Section 25.4.10, in cases where the requirements contained therein are satisfied. For this example, the prohibitions contained in ACI 318, Section 25.4.10.2, are not applicable. Therefore, a reduction in development length may be considered if the development length is not—in any case— reduced to less than 12 in. per ACI 318, Section 25.4.2.1(b).

$$l_e = l_d \frac{A_{s,required}}{A_{s,provided}}$$
$$= (17.1 \text{ in.})\left(\frac{1.56 \text{ in.}^2}{1.76 \text{ in.}^2}\right)$$
$$= 15.2 \text{ in.} > 12.0 \text{ in.} \quad \textbf{o.k.}$$

where l_e is the effective steel reinforcement development length required below the potential concrete failure plane.

The total length of the anchor rod reinforcement can then be calculated based on $l_{\hat{a}}$ and the dimensions shown in Figure 4-36 as:

$$l_{reinf} = h_{ef} - c_c - g\left(\frac{1}{1.5}\right) + l_e$$
$$= 14.0 \text{ in.} - 2.00 \text{ in.} - 2.83 \text{ in}\left(\frac{1}{1.5}\right) + 15.2 \text{ in.}$$
$$= 25.3 \text{ in.}$$

Select a 26.0 in. length for the anchor rod reinforcement. The anchor reinforcement shown in Figure 4-36 is adequate to preclude the concrete breakout in tension.

Confirm the anchorage concrete side face blowout capacity.

$$h_{ef} = 14.0 \text{ in.}$$

$$c_{a1} = \left(\frac{20.0 \text{ in.} - 4.00 \text{ in.}}{2}\right)$$
$$= 8.00 \text{ in.}$$

$$2.5c_{a1} = 2.5(8.00 \text{ in.})$$
$$= 20.0 \text{ in.}$$

Because $h_{ef} < 2.5c_{a1}$, concrete side-face blowout is not applicable per ACI 318, Section 17.6.4.

Chapter 5
Design of Embedded Base Connections

5.1 CONTEXT FOR USE OF EMBEDDED BASE CONNECTIONS

Exposed base plate connections, such as the ones discussed previously in Chapter 4, are commonly used when the axial tension in the anchor rods is not too high; this usually is the case in low- to mid-rise structures or in columns loaded predominantly in axial compression. In taller structures or when the base moments are high, it is not feasible to resist the applied loads through exposed base plate connections. This is because these situations require the use of multiple, large anchor rods to resist tension. This in turn creates additional expense and inconvenience, including (1) the use of deeper anchorage lengths and footing depths to develop the tensile forces; (2) overcrowding of anchors in the base plate; and (3) the use of thicker or stiffened base plates to resist the moment introduced by the anchors—which is not only costly from a fabrication standpoint, but also problematic from the standpoint of fracture vulnerability.

In these situations, it can be preferrable to specify embedded column base (ECB) connections such as the one shown in Figure 5-1. In these connections, the column is embedded into the foundation. The ECB connections resist moment (primarily) through the bearing of the column flanges in the horizontal direction against the footing, such that heavy anchorage is not required. Supplemental mechanisms of bearing, depending on the specific configuration or detail include (1) resistance to uplift of the embedded base plate due to the concrete, (2) resistance due to horizontal reinforcement—if any—attached to the column flanges, and (3) resistance provided by anchor rods attached to the embedded base plate (if provided for erection stability or strength) in addition to the embedment.

Research between 2010 and 2022 has resulted in new data on ECBs and informs the development of this chapter. The previous edition of the Guide does not address the design of these connections. The focus of this chapter is on ECB connections in which the embedment is explicitly provided for moment resistance, rather than for situations where this embedment is incidental (e.g., due to an overtopping slab on grade), and provides resistance supplementary to exposed base plate connections. These overtopped connections are addressed in the context of their simulation in Appendix C. Section 5.2 outlines common connection details, summarizing their key attributes and failure modes. This is followed by a discussion of failure modes, strength characterization methods, and design approaches in Section 5.3 and considerations for fabrication and installation in Section 5.4.

5.2 CONNECTION CONFIGURATIONS AND LOAD RESISTANCE MECHANISMS

Figure 5-2 shows some details commonly used for ECB connections, along with the attached foundation systems. Referring to these figures, while there is some variation in these details, they share some common features. Typically, they include a base plate welded to the bottom of the column in a manner similar to exposed bases for setting the column. The

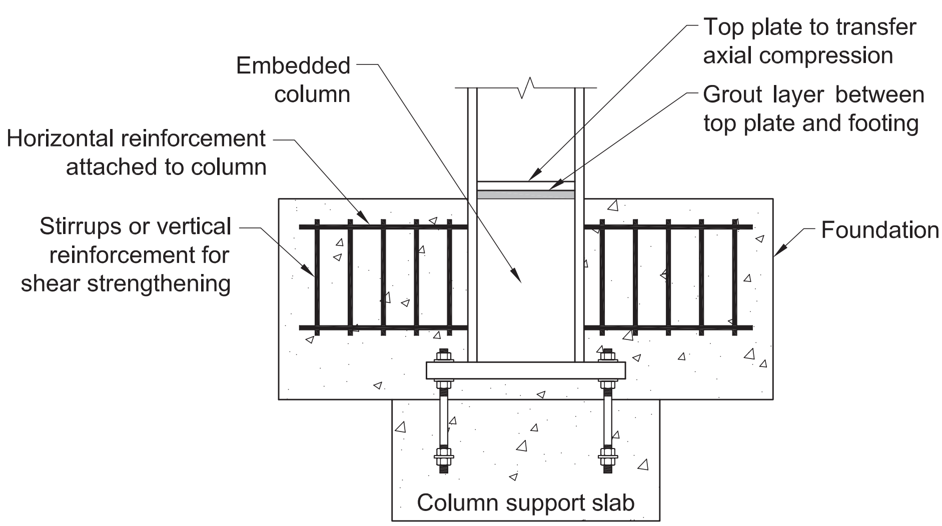
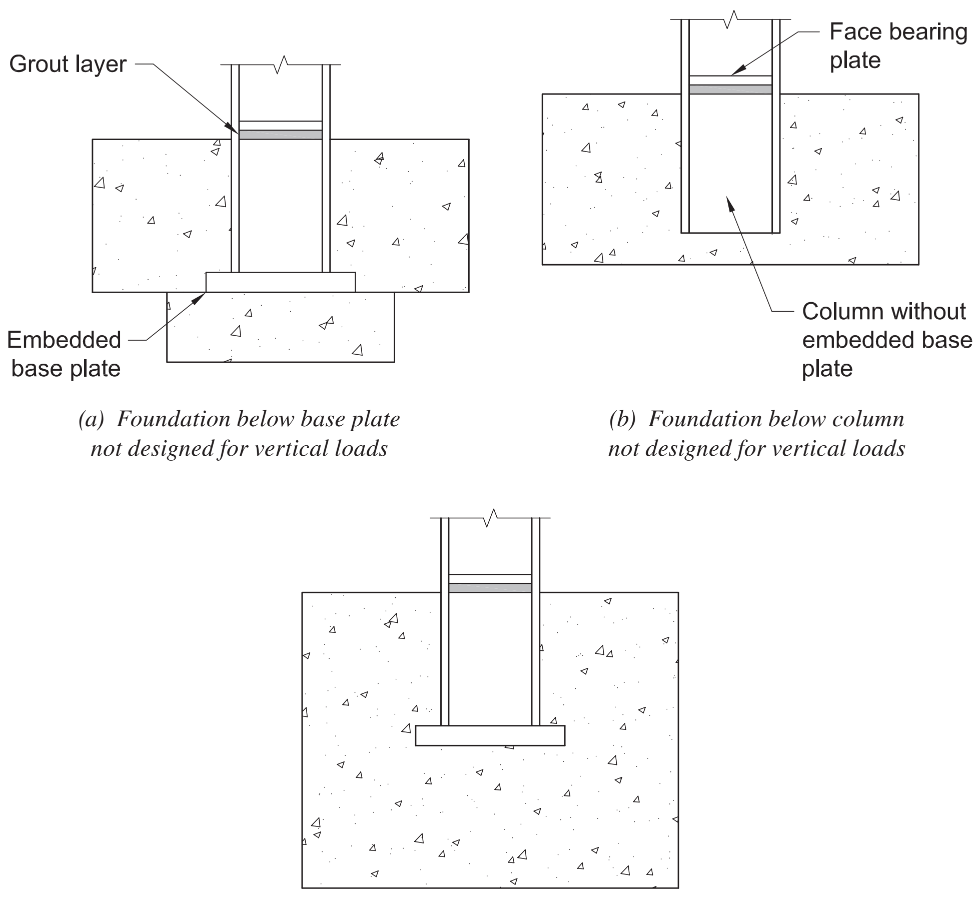


Fig. 5-1. Embedded base connection showing details.

base plate may rest on a supporting slab constructed exclusively for construction purposes (i.e., not designed to carry structural loads) or on the portion of the foundation below if designed to carry structural loads (e.g., in a pile cap). Connections may also be constructed without an embedded base plate, wherein the column is supported externally during erection, provided that compliance with OSHA regulations is ensured—for example, as shown in Figure 5-2(b). Typically, face bearing plates are provided at the top surface of the foundation, similar to a stiffener between the flanges of the column. The primary purpose of these face bearing plates is to transmit axial compression into the foundation. Other variations in the details pertain to the inclusion of attached horizontal or vertical reinforcement and foundation configurations—which may include grade beams, pile caps, isolated footings, or mat foundations.

The focus in this chapter is on the connection between the column and the footing and not the foundation outside of it. Consequently, the failure modes and strength models discussed herein refer only to this portion of the connection, assuming that the remainder of the foundation system will be designed appropriately to provide stress/load paths from the column into the soil or attached elements, such as the grade beams. From the standpoint of load resisting mechanisms, the connections may be divided into two broad categories:

1. *Type I connections:* These are connections in which the slab or footing below the embedded plate is not explicitly designed to carry forces and is provided only for the purpose of column erection, or such a plate does not exist, for example, in the detail shown in Figures 5-2(a) and (b) respectively. In these connections, the moment is resisted only through horizontal forces—bearing stresses on the column flanges and tensile forces in attached reinforcement, if present. Compressive axial load is transferred through the face bearing plate at the top of the connection, whereas the tensile column force is resisted by the embedded base plate bearing upward on the footing.
2. *Type II connections:* These are connections in which the slab or the footing below the base plate, as shown in Figure 5-2(c), is explicitly designed to resist vertical



(a) Foundation below base plate not designed for vertical loads

(b) Foundation below column not designed for vertical loads

(c) Supporting foundation below base plate designed for vertical loads

Fig. 5-2. Embedded column base connection configurations.

stresses and forces imposed by the embedded base plate. In these cases, the moment is resisted by a combination of the horizontal stresses as in Type I connections, as well as vertical stresses that restrain the rotation of the embedded base plate. The axial compressive forces may be resisted in a manner similar to Type I connections (i.e., through face bearing plates for compression) or even through the embedded base plate, whereas the axial tension in the column is resisted through the embedded base plate.

Other variations in detailing may include the use of headed studs to transfer vertical forces from the column into the foundation. These details are not addressed in this Guide, primarily due to the lack of research in the area.

5.2.1 Type I Connections

When the slab or foundation below the embedded base plate does not resist vertical stresses or forces, the applied moment and shear are resisted through the development of horizontal stresses in the foundation along with tensile forces in the attached horizontal reinforcement, as shown in Figure 5-3. The column compression is resisted by downward bearing of the face bearing plates on the top of the foundation, whereas column tension is resisted by upward bearing of the embedded base plate on the underside of the foundation, as shown in Figure 5-4. The transfer of axial forces is considered independently of the transfer of moment and shear. The transfer of moment and shear is discussed first.

The total resisted moment, M_{HB}, and the entire shear, V, is resisted through the development of bearing stresses on both sides of the embedded column flanges. A modified version of the approach developed by Grilli and Kanvinde (2017) may be used to estimate the moment resistance provided by the horizontal bearing mechanism while adding the contribution of horizontal reinforcement. The bearing stresses are idealized such that a uniform stress distribution is assumed for the top stress, f_b^{top}, and the bottom stress, f_b^{bottom} (see Figure 5-3), such that:

$$f_b = f_b^{top} = f_b^{bottom} = 1.54\sqrt{f'_c}\left(\frac{b_w}{b_f}\right)^n \le 1.7 f'_c \tag{5-1}$$

The term b_w/b_f accounts for the effect of confinement, where b_w (in inches) is the width of the foundation (perpendicular to the plane of bending), and b_f (in inches) is the width of the flange. The stresses f_b^{top}, f_b^{bottom}, and f'_c in Equation 5-1 are in ksi units. The exponent n may be taken as 0.66. Referring to Figure 5-3, the resultant bearing forces on the embedment, C_{top} and C_{bottom}, may be determined as:

$$C_{top} = f_b^{top}\beta_1 c b_j \tag{5-2}$$

$$C_{bottom} = f_b^{bottom}\beta_1(d_{embed} - c)b_j \tag{5-3}$$

In Equations 5-2 and 5-3, c is the neutral axis depth and $\beta_1 = 0.85$ is the factor relating the depth of equivalent rectangular stress block to c. This value assumes that $f'_c = 4$ ksi or lower. However, the value of β_1 may be determined through linear interpolation, assuming a 0.65 value when $f'_c = 8$ ksi and higher concrete strength (ACI, 2022). The term $b_j = (b_f + B)/2$ reflects the effective width of the concrete panel, in which B is the width of the embedded base plate at the bottom end of the column. If no embedded base plate is provided, then b_j may be taken as b_f. The attached reinforcement may be assumed to act in both tension and compression if welded directly to the column flanges, as shown in Figure 5-3. However, if alternate details (e.g., U-bar hairpins that wrap around the column flange rather than being welded to it) are used, then they may be assumed to act only in tension because the flanges may not effectively engage the reinforcement on the compression side. The reinforcement bars are assumed to be elastic perfectly plastic and fully developed in tension as per ACI 318—that is, $F_{rebar} = A_{sr}F_{ysr}$. The resultant from each rebar row is directly

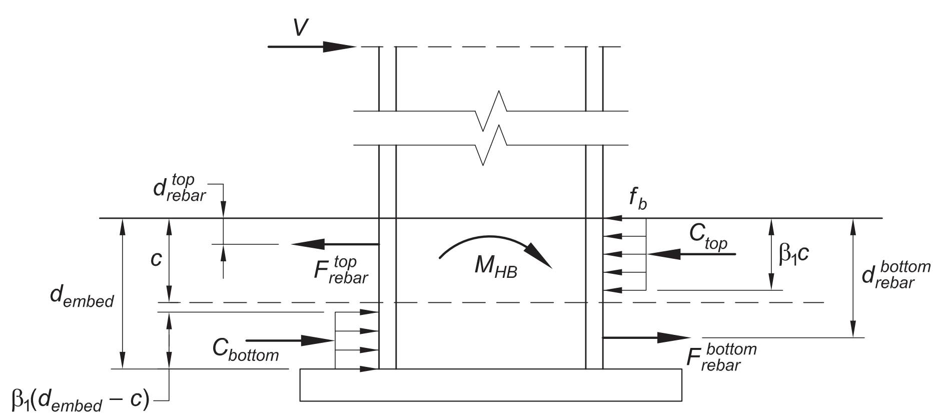


Fig. 5-3. Moment and shear transfer in Type I connections.

added to the resultants from the stress distributions such that the force and equilibrium equations may be written as:

$$V - C_{top} + C_{bottom} - F_{rebar}^{top} + F_{rebar}^{bottom} = 0 \tag{5-4}$$

$$M_{HB} = -C_{top}\frac{\beta_1 c}{2} + C_{bottom}\left[d_{embed} - \frac{\beta_1(d_{embed} - c)}{2}\right] - F_{rebar}^{top} d_{rebar}^{top} + F_{rebar}^{bottom} d_{rebar}^{bottom} \tag{5-5}$$

In Equations 5-4 and 5-5, F_{rebar}^{top} and F_{rebar}^{bottom} are the resultant forces from the engaged reinforcement rods, and d_{rebar}^{top} and d_{rebar}^{bottom} are the distances from the rebar location to the top of the foundation surface, for the top and bottom rebar, respectively. The term c may be eliminated from these equations, resulting in the following equation:

$$M_{HB} = \frac{(F_{rebar}^{top} - F_{rebar}^{bottom} - V)d_{embed}}{2} - \frac{(F_{rebar}^{bottom} - F_{rebar}^{top} + V)^2}{4b_j f_b} - \frac{\beta_1 b_j d_{embed}^2 f_b(\beta_1 - 2)}{4} - F_{rebar}^{top} d_{rebar}^{top} + F_{rebar}^{bottom} d_{rebar}^{bottom} \tag{5-6}$$

Equation 5-6 represents an interaction equation between the shear force, V, and the moment, M_{HB}, such that for any given shear force, V, the maximum moment may be determined using Equation 5-6. The equation assumes that the "neutral axis"—that is, the transition in bearing stress direction occurs between the upper and lower layers of horizontal reinforcement.

For Type I connections, axial compression must be transferred through the top of the foundation through the face bearing plates, as shown in Figure 5-4(a). This is associated with the following possible limit states: (1) flexural yielding of the face bearing plates, (2) fracture of welds between the face bearing plates and the column, (3) bearing failure of the grout or foundation, and (4) punching or other failure in the foundation. The first three are addressed in this Guide and the design example, whereas item 4 is outside the scope of this Guide—similar to exposed base plates where only bearing under the base plate is considered, whereas the effects of this bearing on overall foundation failure are not. These limit states are discussed next.

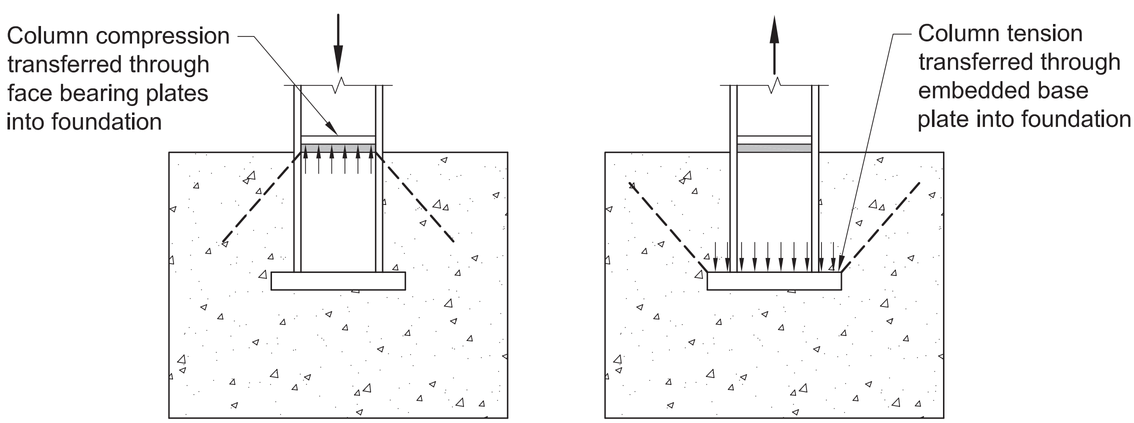
The column axial force is distributed from the column into the face bearing plates and then to the foundation in direct bearing. The bearing strength of the concrete in this case may be determined in a manner similar to base plates in compression as indicated in Section 4.3.1. The critical face plate cantilever dimension, l, may be determined as $\lambda n'$ assuming the face bearing plate to be similar to a base plate loading in compression (see Section 4.3.1), wherein:

$$l = \lambda n' = \frac{\lambda\sqrt{db_f}}{4} \tag{5-7}$$

in which, λ is conservatively taken as 1.0. Using these, the thickness of the face bearing plate may be calculated (in LRFD) as follows:

$$t_{min} = l\sqrt{\frac{2P_u}{0.9F_y(b_f - t_w)(d - 2t_f)}} \tag{5-8}$$

The welds between the face bearing plate and the column webs and flanges may conservatively be specified as CJP groove welds because flexural yielding at the interface of the face bearing plate and the column section will likely govern. Alternatively, fillet or PJP groove welds may be specified such that they develop the flexural strength of the



(a) Compression in column *(b) Tension in column*

Fig. 5-4. Column axial force transfer in Type I connections.

face bearing plate. Type I connections without an embedded base plate are ill-suited for resisting axial tension. However, if such a base plate is provided, then axial tension may be resisted through the upward bearing of the base plate against the lower surface of the foundation [Figure 5.4(b)]. In this case, the following limit states are of interest: (1) flexural yielding of the base plate in the region between the flanges—similar to face bearing plates and (2) flexural yielding of the base plate in the regions outside the flange. For each of these, equations similar to those for base plates loaded in compression (see Section 4.3.1) may be applied to determine strength.

5.2.2 Type II Connections

When the slab or foundation below the embedded base plate is capable of (i.e., designed to) resisting vertical stresses, then the mechanisms outlined previously for Type I connections are supplemented by additional mechanisms due to these vertical stresses. Referring to Figure 5-5, these additional vertical stresses are imposed on the upper and lower surface of the base plate, resisting the moment transferred to the base through the column flanges. For Type II connections, axial tension may be transferred in a manner similar to Type I connections—that is, by the embedded base plate bearing upward on the foundation above it. Axial compression may be transferred either through face bearing plates or, more realistically, through the embedded base plate bearing on the portion of the foundation below it because it is designed for vertical forces. Experimental data (Grilli et al., 2017; Hassan and Kanvinde, 2023), suggests that the latter (i.e., transfer through the embedded base plate) is the favored mechanism because (1) the column provides a stiff load path to the bottom of the foundation, if the underlying portion of the foundation is designed for these stresses, and (2) if large lateral deformations are present, then separation between the column flanges and the foundation reduce the efficacy of the face bearing plates. In either case, the base plate is subjected to vertical stresses due to both the moment (wherein these stresses resist rotation of the plate) and column axial force.

The additional strength provided by the vertical bearing mechanisms depends on the resistance to rotation of the embedded base plate at the bottom of the column embedment. This resistance is active on the compression side of the connection where the plate bears downward on the underlying foundation, as well as on the tension side of the connection where the uplift of the plate is restrained by the foundation above it. The former is controlled by the design of the portion of the foundation below the embedded base plate. The latter (i.e., the restraint provided to the uplift of the base plate) is highly sensitive to the reinforcement detailing of the foundation. Specifically, research has shown that the use of horizontal reinforcement attached to the column produces a tension field in the foundation above the base plate greatly diminishing the resistance to its uplift (Hassan and Kanvinde, 2023). This may have the result of reducing the overall moment capacity of the connection even below that of a similar connection in which no horizontal reinforcement is attached. The same research further indicates that the use of vertical reinforcement (i.e., stirrups) may mitigate this issue to a limited extent, although not fully. As a result, the overall strength of the connection is highly sensitive to the amount as well as the patterns of both the horizontal and vertical reinforcement.

Given these sensitivities, a definitive design method for Type II connections is not presented in this Guide. Rather, basic modes of resistance and failure observed in experiments are presented here, directing the user to the Hassan and Kanvinde (2023) research for comprehensive insights and equations that may be adapted toward the design of specific connections. The base plate at the bottom is subjected to bearing stresses on the lower as well as the upper surfaces, resisting the moment transferred to the base through the column flanges, as well as the net axial force transferred to the base plate. The base plate is assumed to resist the total axial force (through upward bearing in case of compressive load or downward bearing in case of tensile load) in addition to the moment resisted through the vertical bearing mechanism. Under these stresses, the moment strength due to vertical bearing stresses may be controlled through one of the three limit states (i.e., Scenario 1, 2, and 3) outlined in the following that may occur under different reinforcing details. These scenarios pertain to "tension side" failures—that is, the base plate uplifting the concrete above it on the tension side of the connection. In addition to these, the moment capacity may also be reached due to failure of the foundation under the toe of the embedded base plate on the compression side; as discussed previously, this is not addressed in this Guide primarily because no test data exists for this type of failure.

Scenario 1: Breakout of concrete failure cone in the absence of attached horizontal reinforcement

When no horizontal reinforcement is attached to the column flanges, the region above the base plate on the tension side fails in a conventional pryout type failure with a 35° failure cone emanating from the tension side flap of the base plate. This failure mode occurs only when no horizontal reinforcement is attached because there is no tension field above the base plate. The process for determination of the force associated with this failure cone, and then its use for estimating the moment, M_{VB}, is outlined in Hassan and Kanvinde (2023). This failure mode is applicable only when no horizontal reinforcement is attached (tests by Grilli et al., 2017); that is, the tension field produced by the reinforcement does not affect the development of such a cone. Note that cone formation in the absence of horizontal reinforcement is associated with a higher breakout strength than when horizontal reinforcement attached to the column flange is present. In such a case, the total breakout force, F_t, (in kips) may be calculated as (see Figure 5-6):

$$F_t = \frac{40}{9} A_{35} \sqrt{\frac{f'_c}{d_{cover}}} \tag{5-9}$$

Equation 5-9 is based on the concrete capacity design (CCD) method proposed by Fuchs et al. (1995), such that d_{cover} is the thickness of the material (in inches) that must be ruptured for breakout, which is equal to d_{embed} for tension breakout. The concrete strength, f'_c, is in psi units. The term A_{35} (in in.^{2}) is the projected area of a 35° failure cone emanating from the edges of the stress block on the tension side of width $0.3N$, where N is the length of the base plate.

Scenario 2: Breakout of concrete failure cone in the presence of attached horizontal reinforcement and no vertical reinforcement.

When horizontal reinforcement is attached to the column flanges, a tension field is created above the uplifting end of the base plate, reducing the resistance to vertical motion. In these cases, the resistance to uplift is negligible, and the response of the connection approaches that of a Type I connection with no moment resistance due to vertical stresses. This scenario motivates a tradeoff between the use of horizontal reinforcement to enhance moment strength due to increase in the moment capacity, M_{HB}, as calculated previously (for Type I connections) and the loss in vertical strength due to the tension field.

Scenario 3: Shear failure of concrete in the presence of horizontal reinforcement and vertical reinforcement.

The third scenario is associated with the presence of vertical reinforcement/stirrups supplementary to the attached horizontal reinforcement as shown in Figure 5-1. The intent of the stirrups is to increase the vertical bearing resistance by mitigating the breakout failure mode noted in Scenario 2 and shown in Figure 5-7. The stirrups add vertical resistance while inducing direct shear failure at a weak point in the foundation. This results in a significant increase in uplift strength and moment capacity, M_{VB}. Hassan and Kanvinde (2023) outline the procedure for calculating the force, F_t, in this situation.

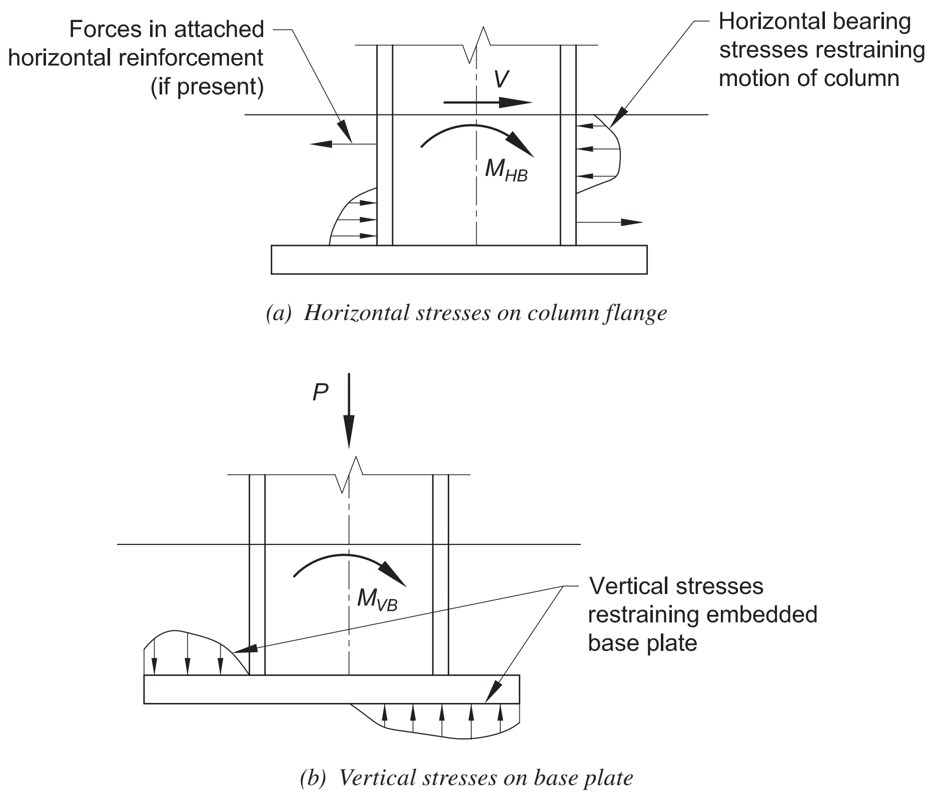


Fig. 5-5. Load transfer in Type II connections.

5.3 DESIGN METHOD FOR COMBINED BENDING, SHEAR, AND AXIAL FORCE

The design approach is presented here only for Type I connections, along with a design example. To aid the design process, an online tool is provided at www.aisc.org/DG1 that may be used to conveniently estimate the capacity of Type I embedded column base connections given all the connection parameters. The following steps may be used for design, under a combination of moment, axial force, and shear:

1. Estimate embedment depth, d_{embed}, through trial and error, given the design moment and shear, the column flange width, and the foundation dimensions. For this purpose, use Equation 5-6 or the online tool to determine if the moment and shear capacity are adequate. In this context, it is important to note that the moment capacity, M_{HB}, determined using these equations, is an ultimate strength value that is accompanied by significant damage in the concrete foundation and deformation. If such deformation and damage is to be prevented, it is recommended to use the capacity of the connection, $M_n = 0.8M_{HB}$, and the corresponding shear. The 0.8 factor reflects experimental observations across numerous tests (Grilli et al., 2017; Hassan and Kanvinde, 2023) that nonlinear response (and damage) initiates at around 80% of the peak moment.
2. If embedment depth is acceptable given foundation dimensional constraints, advance to Step 3. If not, consider attaching horizontal reinforcement to enhance embedment depth or using a Type II connection.
3. Design the face bearing plates and the embedded base plate for axial compression or tension as the case may be.
4. For LRFD, a resistance factor $\phi = 0.75$ is recommended for the overall connection strength, based on the design of similar connections outlined in the *PCI Design Handbook* (2017).

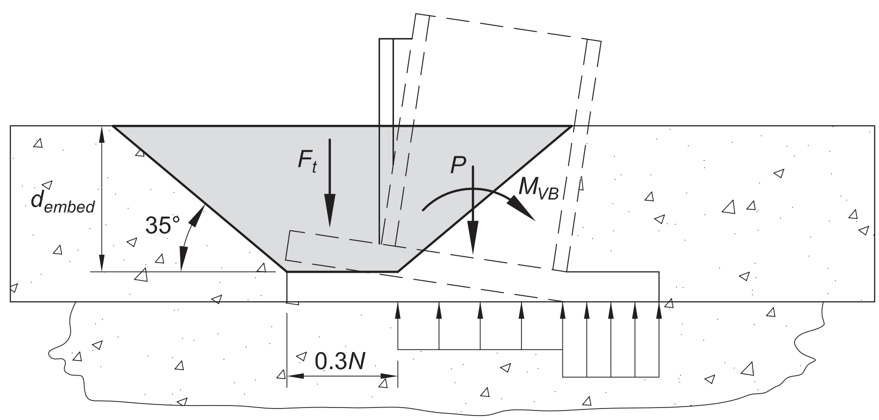


Fig. 5-6. Concrete breakout force above the tension side of the connection in the absence of horizontal reinforcement.

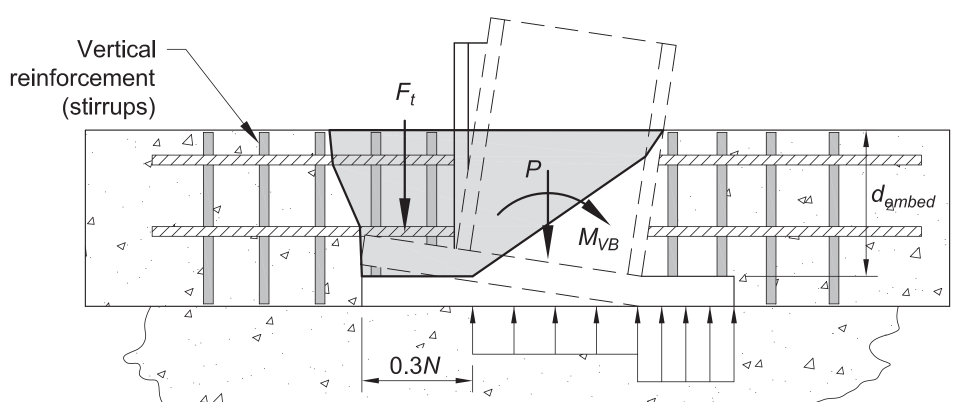


Fig. 5-7. Breakout force of concrete above the tension side of the connection.

EXAMPLE 5.3-1—Embedded Base Connection for Bending, Shear, and Axial Compression

Given:

Design an embedded column base connection for a W14×176 column. The factored compressive axial load is $P_u = 250$ kips, the shear force is $V_u = 96.0$ kips, and the design moment is $M_u = 700$ kip-ft. Bending is about the strong axis of the column. Assume that the foundation below the embedded column base plate cannot resist vertical stresses. The assumed width of the foundation is 60 in. The column is ASTM A992/A992M material, the base plate is ASTM A572/A572M Grade 50 material, and the concrete compressive strength is $f_c' = 4$ ksi.

Solution:

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Column
ASTM A992/A992M
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Column
W14×176
$d = 15.2$ in.
$b_f = 15.7$ in.
$t_w = 0.830$ in.
$t_f = 1.31$ in.

1. Estimate the embedment depth, d_{embed}.

A good starting estimate for the embedment depth is $d_{embed} = 1.5d$.

$$d_{embed} = 1.5(15.2 \text{ in.})$$
$$= 22.8 \text{ in.}$$

Try $d_{embed} = 22.0$ in.

Assuming a Type I connection—that is, no vertical stresses—as well as no attached reinforcement, the moment capacity may be determined using Equation 5-6. By substituting the value of shear $V_u = 96.0$ kips, all values corresponding to the rebar are zero, and the width of the joint, $b_j = b_f = 15.7$ in. because no embedded plate is provided.

$$f_b = f_b^{top} = f_b^{bottom} \tag{5-1}$$
$$= 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^n \leq 1.7 f_c'$$
$$= 1.54\sqrt{4 \text{ ksi}}\left(\frac{60 \text{ in.}}{15.7 \text{ in.}}\right)^{0.66} \leq (1.7)(4 \text{ ksi})$$
$$= 7.46 \text{ ksi} \leq 6.80 \text{ ksi}$$
$$= 6.80 \text{ ksi}$$

From Equation 5-6:

$$M_{HB} = \frac{\left(F_{rebar}^{top} - F_{rebar}^{bottom} - V\right)d_{embed}}{2} - \frac{\left(F_{rebar}^{bottom} - F_{rebar}^{top} + V\right)^2}{4b_f f_b} - \frac{\beta_1 b_f d_{embed}^2 f_b(\beta_1 - 2)}{4} - F_{rebar}^{top} d_{rebar}^{top} + F_{rebar}^{bottom} d_{rebar}^{bottom}$$

$$= \frac{(0 - 0 - 96.0 \text{ kips})(22.0 \text{ in.})}{2} - \frac{(0 - 0 + 96.0 \text{ kips})^2}{4(15.7 \text{ in.})(6.80 \text{ ksi})} - \frac{(0.85)(15.7 \text{ in.})(22.0 \text{ in.})^2(6.80 \text{ ksi})(0.85 - 2)}{4} - 0 + 0$$

$$= -1{,}060 \text{ kip-in.} - 21.6 \text{ kip-in.} + 12{,}600 \text{ kip-in.}$$

$$= 11{,}500 \text{ kip-in.}$$

$$= 958 \text{ kip-ft}$$

$$\phi M_{HB} = (0.7)(958 \text{ kip-ft})$$

$$= 719 \text{ kip-ft} > 700 \text{ kip-ft} \quad \textbf{o.k.}$$

Thus, the embedment is satisfactory. An unsatisfactory embedment may be addressed by either increasing the value of d_{embed} or by providing additional reinforcement. Note that this design process is intended to prevent failure in a reliable way but does not prevent damage in a reliable way. To achieve the latter, it is recommended to use an additional reduction factor of 0.8 as discussed in the preceding section.

2. Design the face bearing plates for axial compression.

The axial compression is transferred from the face bearing plates into the foundation below. Similar to exposed base plate connections, it is assumed that the foundation as a whole is designed to resist the effects of this axial compression (e.g., those shown in Figure 5-4). With this assumption, two design checks remain: (1) bearing failure under the face bearing plates and (2) yielding of the face bearing plates themselves.

Bearing failure under the face bearing plates may be checked in a manner similar to base plates under axial compression.

$$P_u \le \phi P_p = \phi f_{p(max)} A_1$$

Where ϕ = 0.65, A_1 is the bearing area, and $f_{p(max)}$ is the maximum bearing stress such that $f_{p(max)} = 1.7 f_c'$, assuming confined concrete and grout strength exceeding twice the specified concrete compressive strength. For a W14×176, the bearing area may be determined as:

$$A_1 = (b_f - t_w)(d - 2t_f)$$

$$= (15.7 \text{ in.} - 0.830 \text{ in.})[15.2 \text{ in.} - (2)(1.31 \text{ in.})]$$

$$= 187 \text{ in.}^2$$

Thus, the available strength is:

$$\phi P_p = \phi f_{p(max)} A_1$$

$$= (0.65)(1.7)(4 \text{ ksi})(187 \text{ in.}^2)$$

$$= 827 \text{ kips}$$

Because P_u = 250 kips < 827 kips, the bearing check is satisfied.

The face bearing plate thickness may be initially selected to meet the stiffener requirements of AISC *Seismic Provisions* Section F3.5b.4, such that:

$$t_{min} = 0.75 t_w \ge 3/8 \text{ in.}$$

$$= 0.75(0.830 \text{ in.})$$

$$= 0.623 \text{ in.}$$

As outlined in Chapter 4, the critical face plate cantilever dimension, l, may be determined as $\lambda n'$, such that:

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4} \tag{5-7}$$

where λ may be conservatively taken as 1.0. Thus,

$$\begin{aligned} l &= \lambda \frac{\sqrt{db_f}}{4} \\ &= 1.0 \frac{\sqrt{(15.2\text{ in.})(15.7\text{ in.})}}{4} \\ &= 3.86\text{ in.} \end{aligned}$$

The minimum plate thickness may then be determined as:

$$t_{min} = l\sqrt{\frac{2P_u}{0.9F_yA_1}} \qquad \text{(from Eq. 5-8)}$$

$$\begin{aligned} &= (3.86\text{ in.})\sqrt{\frac{2(250\text{ kips})}{0.9(50\text{ ksi})(187\text{ in.}^2)}} \\ &= 0.941\text{ in.} \end{aligned}$$

Select 1-in.-thick face bearing plates. Flexural yielding at the face bearing plate design controls, consequently the face bearing should be welded to the columns with CJP groove welds.

5.4 FABRICATION AND INSTALLATION

The fabrication of the embedded column base assemblies should follow the same requirements as for exposed base plate connections in terms of (1) base plate fabrication and finishing if an embedded base plate is present for erection purposes, as shown in Figures 5-2(a) and 5-2(c); (2) base plate welding; and (3) anchor rod holes and washers, as well as anchor placement, if anchors are used during erection; see Sections 4.5.1, 4.5.2, 4.5.3, and 4.5.4. Grout will usually be provided below the embedded plate as well as between the face bearing plates and the top of the foundation; for both of these recommendations, see Section 4.5.6.

Column erection will normally follow procedures similar to exposed base plates (Section 4.5.5), except that the column will be set on top of either a supporting slab cast on top of the soil or the underlying foundation. The setting nut and washer method, the setting plate method, or the shim stack method may be used as appropriate. Unlike exposed base plate connections, the installation of concrete above the base plate (for the purposes of embedment) will introduce additional lateral forces and instabilities on the erection arrangement (e.g., nut and washer, shim stacks, or setting plate). The erection arrangement should consequently consider these forces in its design.

Chapter 6
Design of Column Base Connections for Seismic Loading

6.1 OVERVIEW AND ORGANIZATION

Chapters 4 and 5 address the design of exposed and embedded base connections, respectively. In these chapters, the design procedure is focused on force-based design, such that the connection is designed to resist the given combinations of applied loads without failure. This type of design is adequate for static loads and wind loads (where inelastic action is not anticipated). However, under seismic conditions, inelastic actions may be expected either in the base connection itself, in its close vicinity, or elsewhere in the seismic force-resisting system (SFRS). Additionally, as discussed previously in Chapter 3, the rotational flexibility of the base has the potential to influence overall structural response. As a result, seismic conditions introduce additional considerations to ensure acceptable response of both the connection as well as the structure. This chapter addresses these additional considerations, supplementing the force-based design methods presented in Chapters 4 and 5.

Chapter 1, Section 1.1, and Figure 1-2 establish the relationships between various design documents and standards, including this Guide. In the context of seismic design, the 2022 AISC *Seismic Provisions* are a companion to the AISC *Specification* that extend coverage to connection detailing and member design requirements for structural steel and composite systems in high-seismic applications. The AISC *Seismic Provisions* (and the discussion and examples in this chapter) are applicable to base connections that are part of the SFRS, as well as those that are not part of it. In the context of base connections, the AISC *Seismic Provisions* establish strength (axial, shear, and moment) requirements, in addition to detailing requirements regarding the welds between the column and base plate. Additionally, the AISC *Seismic Provisions* (and the Commentary) suggest details acceptable for seismic design. While the AISC *Seismic Provisions* represent a mandatory design standard, this Design Guide, along with the AISC *Seismic Design Manual*, represent nonmandatory resources to facilitate design that meets the requirements of the AISC *Seismic Provisions* along with the AISC *Specification*. It is emphasized that the aim of this Design Guide (and this chapter) is not to establish strength or ductility requirements, but to provide guidance for the design of connections that meet these requirements (or the intent) as established in these mandatory resources.

Research over the last two decades (Hassan et al., 2022; Trautner et al., 2017b; Gomez et al., 2010; Falborski et al., 2020a, 2020b) indicates that exposed column base plates in moment frames may be designed to achieve ductile performance; this results in the use of a reduced seismic load for their design as discussed in the next section. Following this, the main focus in this chapter is on exposed base plate type connections in moment frames because other types of connections—that is, embedded base connections in moment frames or exposed and embedded base connections in braced frames—are typically designed to remain elastic under seismic loading, such that they may be designed following the force-based design procedures outlined in Chapters 4 and 5. Section 6.2 outlines acceptable performance characteristics of seismic base connections, followed by a discussion of overall foundation and grade beam effects that are relevant in the context of the seismic design in Section 6.3. Section 6.4 presents a design method for base connections in steel moment frames subjected to seismic loading. Section 6.5 provides discussion regarding base plate connections for braced frames subjected to seismic loading.

6.2 SEISMIC PERFORMANCE REQUIREMENTS FOR COLUMN BASES

Generally, the seismic loads and load combinations including load overstrength are determined using the IBC and ASCE/SEI 7 codes. Both documents defer to the material codes to establish when the overstrength combinations are to be considered along with any material ductility requirements. The AISC *Seismic Provisions* and ACI 318 provide additional seismic performance requirements for the design of exposed column bases and their anchorage. The AISC *Seismic Provisions* do not apply to the seismic design of buildings composed solely of steel systems not specifically detailed for seismic resistance ($R = 3$); certain categories of nonbuilding structures noted in ASCE/SEI 7-22, Chapter 15; and nonstructural components, except for certain categories of penthouses and rooftop structures, designed according to ASCE/SEI 7-22, Chapter 13. Only the provisions of ACI 318, Chapter 17, are applicable in Seismic Design Category C–F in these instances. A discussion of the requirements in both documents follows.

AISC *Seismic Provisions* Section D2.6 and associated Commentary provide general guidance for the required strength of column bases. Of these, the strength requirements for axial (AISC *Seismic Provisions* Section D2.6a) and shear (AISC *Seismic Provisions* Section D2.6b) force are fairly straightforward to design for because these actions are assumed force controlled, wherein the forces themselves are calculated as per seismic considerations. Once these forces have been determined in accordance with the AISC *Seismic Provisions*, the base connections may be designed using the procedures outlined previously in Chapters 4 and 5. The considerations particular to seismic design of the connections themselves arise in the context of flexural strength and design. Specifically, AISC *Seismic Provisions* Section D2.6c allow for the column bases to be designed using the lesser of conditions 1 and 2:

1. For a moment corresponding to $1.1R_yF_yZ/\alpha_s$ of the attached column, which reflects the moment corresponding to the fully yielded and strain hardened column. This may be termed the "strong-base" condition.
2. For a moment corresponding to the overstrength (i.e., Ω_0) seismic loads, provided that a ductile limit state in either the column base or the foundation controls the design. This may be termed the "weak-base" condition. The ductility requirement recognizes that the moment corresponding to overstrength seismic load may be lower than the flexural capacity of the column, such that the base connection and foundation system will be required to accommodate inelastic rotation. An exception to this is for ordinary moment frames (OMFs) where, according to AISC *Seismic Provisions* Commentary Section E1.2, the connections, including the base, must be strong enough (i.e., strong base) so that "…significant inelastic action in response to earthquake loading occurs in frame elements rather than connections."
3. In addition to conditions 1 and 2, the base connections may also be designed as pinned, such that they are designed only for axial force and shear, without consideration of moment, provided they can sustain the expected rotations without failure—that is, loss of shear capacity.

The design corresponding to case 1—that is, the strong-base condition—is fairly straightforward. This is because conducting capacity design of the base connection in this manner forces plastic hinging into the column, and the base connection itself remains elastic. As a result, the design of the base connection is similar in concept to force-controlled base connections (as outlined in Chapters 4 and 5), with the exception of additional detailing and material toughness requirements for the welds between the base plate and the column.

Design corresponding to cases 2 and 3 requires inelastic rotation capacity in the bases. However, there are two issues in the implementation of these cases in design. First, the AISC *Seismic Provisions* do not explicitly mention the degree of rotation capacity required. Second, the AISC *Seismic Provisions* only qualitatively discuss detailing that may be used to achieve such capacity; specifically, "This can be achieved through flexural bending of the base plate similar to an end-plate connection, bending of elements used as anchor chairs, ductile yielding of the foundation, uplift of the foundation, or elongation of the anchor rods." These guidelines are qualitative and generic, rather than prescriptive. Both these issues have been addressed through research. The key findings are:

1. Nonlinear response history analysis (NLRHA) by Falborski et al. (2020a, 2020b) indicates that a rotation capacity of 0.04–0.05 rad in weak-base connections (designed for moments corresponding to overstrength seismic loads) in steel moment frames (SMF) is sufficient to achieve performance (i.e., collapse probabilities) that are similar to those achieved by SMFs designed with strong bases. At the time of this writing, research is under way to establish acceptable rotation capacities for base connections designed for other levels of moment (e.g., for lower levels of base moment, including for pinned connections). However, from the standpoint of kinematics, it may be conservatively assumed that the rotation demand in base connections (regardless of the moment they are designed for) is on the order of the rotation demands in beam-to-column connections in SMFs, which are expected to maintain their moment capacity up to a rotation of 0.04 rad under cyclic loading as required by the AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, ANSI/AISC 358 (AISC, 2022d).
2. Numerous experimental research programs (e.g., Hassan et al., 2022; Trautner et al., 2017b; Gomez et al., 2010) have examined the seismic performance and rotation capacity of exposed base plate connections. These studies indicate that base connections generally show good ductility and rotation capacity. However, with the exception of two studies (Hassan et al., 2022; Trautner et al., 2017b), the observed ductility of these specimens was incidental; that is, the specimens in these test programs were not intentionally detailed to achieve ductility. As a result, the detailing of these connections cannot be replicated in a controlled manner or used to develop prescriptive detailing practices. Details tested in the other studies provide excellent rotation capacities, well in excess of 0.05 rad under cyclic loading. Recommendations from these studies are used to inform the design procedures and examples in this chapter.

It is important to note here that the aforementioned performance requirements pertain only to the response of the base connection itself. However, as discussed previously, the rotational flexibility of the base connection influences the force distribution and deformations in the entire structure. This influence is increased if the base connections are designed as weak bases, because weaker bases are, in general, more flexible. Consequently, it is important (for strong-base designs and more so for weak-base designs) to conduct structural analysis incorporating the rotational flexibility of the designed base connections (as outlined in Appendix C), to ensure that the design is adequate considering calculated forces and moments. This is similar to design iterations for member design where structural analysis is conducted using the most updated member sizes.

ACI 318, Section 17.10, provides the anchorage design requirements for tension and shear loading in seismic design categories C, D, E, and F. When the strength-level earthquake-induced forces do not exceed 20% of total factored anchor forces associated with the same load combination, no additional seismic requirements need to be considered. Otherwise, Sections 17.10.5 and 17.10.6 provide additional design requirements for tensile and shear load, respectively.

ACI 318, Section 17.10.5.3, lists four options for the design of the anchors of steel base connections subjected to seismic tensile loads.

In option a, the anchorage design is controlled by yielding of a ductile anchor. The material overstrength and strain hardening are considered by increasing the steel strength to 1.2 times the nominal steel strength of the anchor. A stretch length equal to 8 times the anchor diameter must also be provided. Fully threaded rods can be used if the ratio f_{uta}/f_{ya} exceeds 1.3, unless the threaded portions are upset. To ensure that a brittle concrete failure mode does not govern, the strength of all concrete limit states must be higher than the steel strength of the anchors, including material overstrength and strain hardening.

In option b, the anchors must be designed for the maximum forces that can be transmitted by the development of a ductile yield mechanism in the attachment. This must include material overstrength and strain hardening.

In option c, the anchors must be designed for the maximum tension that can be transmitted by a nonyielding attachment. The ACI commentary suggests this option can be used for cases where the AISC *Seismic Provisions* specify design loads based on the member strength.

In option d, the anchors are designed for load overstrength with no regard to any ductility requirements.

In addition, ACI 318, Section 17.10.5.4, requires the design tensile strength for concrete breakout, concrete pullout, and concrete side-face blowout be further reduced by a 0.75 seismic factor.

ACI 318, Section 17.10.6.3, lists three options for the design of the anchors of steel base connections subjected to seismic shear loads.

In option a, the anchors must be designed for the maximum forces that can be transmitted by the development of a ductile yield mechanism in the attachment. This must include material overstrength and strain hardening.

In option b, the anchors must be designed for the maximum shear that can be transmitted by a nonyielding attachment. The ACI commentary suggests the bearing strength at holes in a steel attachment can be considered for this option.

In option c, the anchors are designed for load overstrength with no regard to any ductility requirements.

The main difference in the requirements between the AISC *Seismic Provisions* and ACI 318 is that AISC *Seismic Provisions* (2016) Section D2.6c(b)(2) requires the moment be determined using load overstrength and a ductile limit state in either the column base or the foundation to control the design at this overstrength level. This additional ductility requirement in combination with the load overstrength was added in the 2016 Edition of the AISC *Seismic Provisions* (2016). ACI 318 does not have such a requirement for anchorage design. When load overstrength is used, it is not required that the failure mode controlling the design be a ductile limit state.

Noting the dichotomy between the two approaches, the design approach and example presented in this Guide follows the AISC philosophy, focusing on weak base (with appropriate ductility) for overstrength seismic loads. This acknowledges the current state of research (which is evolving) and its interpretation by code and guidance bodies (which is ongoing), directed toward future editions of various codes and standards. Specifically, the following points have been taken into consideration:

1. Recent research by Hassan et al., (2022) was focused on qualification-type testing of base connections with ductile anchors, directed specifically toward the AISC requirements (i.e., the design of weak bases for overstrength loads). This has resulted in a base connection detail and design approach that provides adequate ductility without significant fabrication costs.

2. Guidance in ACI 318 regarding the stretch length is based on observations in earthquakes and only notionally suggests how this stretch length should be achieved in design. Nonetheless, recent research by Trautner et al. (2017a, 2017b) provides support to the approach outlined in ACI 318. This research (along with previously conducted tests by Gomez et al., 2010) suggests that (1) even without special detailing, commonly used base connection and anchor rod details provide excellent ductility; (2) notwithstanding the preceding observation, some specific details do not perform well; and (3) the findings of these programs (e.g., Gomez et al., 2010) are incidental, without controlled variation of key parameters such as the anchor stretch length or base plate size. Consequently, the results need additional interpretation and analysis.

6.3 INFLUENCE OF GRADE BEAMS AND OTHER FOOTING EFFECTS

The scope of this Design Guide includes the connection of the column to the concrete footing or grade beam. Nonetheless, the expected seismic performance requirements of the column bases (as outlined in the AISC *Seismic Provisions*) pertain to the column base as well as the foundation system. A similar dichotomy was noted in Chapters 4 and 5 in the context of force-based design of column bases. For force-controlled design, it is important to provide a load path from the column into the foundation system and the soil. Although the Design Guide addresses load transfer in the immediate vicinity of the column (e.g., anchor rod embedment and detailing) the specific nature of this load path as well as the design considerations will, in general, depend on the configuration of the foundation (e.g., mat foundation, grade beam, or individual footing). Similar considerations arise in the context of seismic performance, wherein both the force as well as deformation characteristics of the entire base connection and foundation system are important. In this regard, the following points are noted:

1. The AISC *Seismic Provisions* do not prescribe where the base inelastic rotations should be accommodated (for weak-base design) and provides multiple options for this—including the column to footing connection or the grade beams.
2. The NLRHA simulations mentioned previously, as well as the kinematic considerations for column base rotation demands, do not distinguish between rotation in the column-footing connection or other parts of the foundation, as long as the total rotation capacity is achieved.
3. It is important to identify the specific mechanism and part of the foundation system used to accommodate these rotations, such that it may be detailed for ductility, while the surrounding elements of the foundation system are capacity designed to remain elastic.
4. The design procedure and examples summarized in this chapter (for weak-base design) assume that all inelastic rotations are accommodated in the column to footing connection. Within this, anchor rod yielding is assumed to be the primary dissipative or ductile mechanism. The remainder of the foundation system—that is, the grade beams and footing—are assumed to remain elastic. As a result, these must be designed for the expected moment capacity of the base connection. Other ductile modes identified in the AISC *Seismic Provisions* include formation of plastic hinges in the grade beams. Such a design may be conducted using ACI 318 and is outside the scope of this Guide.

6.4 DESIGN METHOD FOR SEISMIC DESIGN OF COLUMN BASE CONNECTIONS IN MOMENT FRAMES

It is expected that users of this Guide will determine the applicable design loads (i.e., the combinations of axial force, moment, and shear) from the seismic load combinations outlined in ASCE/SEI 7-22, the AISC *Seismic Provisions*, and ACI 318. The design method and examples provided in this Guide assume these loads and illustrate the procedure to design the base connection given these loads. Commentary for contextualization of this loading is provided as needed. This section is divided into two subsections: one focuses on strong-base design for seismic applications (Section 6.4.1), whereas the other focuses on weak-base design (Section 6.4.2). The scope of these sections is limited to base connections subjected to axial force, uniaxial bending, and uniaxial shear. Biaxial bending and shear are not considered.

6.4.1 Strong-Base Design for Seismic Conditions

The objective of strong-base design for seismic loading is to ensure that the base connection remains elastic under design level seismic shaking. To achieve this, the design moment for the base connection is determined as the fully yielded and strain hardened capacity of the column—that is, $M_u = 1.1R_yF_yZ$ (for LRFD). The axial load, P_u, is determined according to the combination of dead, live, and overstrength seismic load (i.e., corresponding to the Ω_0 factor). The shear force, V_u, may be determined in accordance with the AISC *Seismic Provisions*. Once the load (i.e., a set of P_u, M_u, and V_u) is determined in this fashion, the force-based design procedures outlined in Chapter 4 (for exposed base plate connections) and Chapter 5 (for embedded base connections) may be followed; these are not repeated here. In terms of construction, too, strong-base details are identical to those shown in Chapter 4 (see Figure 4-21) or Chapter 5 (see Figure 5-1). However, some specific considerations are emphasized:

- The AISC *Seismic Provisions* list specific weld requirements for column base connections (pertaining to welds between the column and the base plate); these depend on the specific type of SFRS used.
- Axial forces must include consideration of overstrength seismic load in both directions (resulting in effective tensile and compressive axial forces in the base connection). This is important for two reasons: (1) net tension in the base connection triggers additional weld requirements (e.g., designation as demand critical welds) in some SFRS, as well as strength requirements for axial tension, and (2) lower compressive force in the connection may result in the more critical design condition for both the anchor rod as well as the base plate.

- For embedded base connections, the procedures outlined in Chapter 5 may be followed directly.

6.4.2 Weak-Base Design for Seismic Conditions

Weak-base design implies that the base connection will accommodate plastic rotations, while the column will remain elastic. Following previous discussion, only exposed base plate type connections may be used for this type of design. Based on experimental as well as analytical research summarized earlier, a reliable mode of accommodating these plastic deformations is through cyclic yielding of the anchor rods, while the base plate remains elastic. Base plate yielding (especially on the tension side of the connection) is undesirable because it results in kinking of the plate under the column weld, raising the likelihood of fracture. To achieve ductile behavior through anchor rod yielding, the following behavioral assumptions, and design and detailing considerations are used (see Example 6.4-1 for application):

1. Sufficient axial deformation capacity of the rod should be ensured. A recommended detail is the upset thread (UT) detail such as the one shown in Figure 6-1. Referring to the figure, the UT anchor rods in which the threads are milled to a smooth shank providing a designated stretch length $L_{stretch}$ over which the diameter is reduced to $d_{reduced}$, such that inelastic deformations may be concentrated over this length (Hassan and Kanvinde, 2023). The shank is frictionally isolated from the footing using polyethylene tape. The material grades may be ASTM F1554 Grade 36, 55, or 105. UT details entail additional fabrication cost and also decrease the strength of the rod over the reduced diameter, necessitating that other details (which are demonstrated to have adequate deformation capacity) may be specified as well. These may include, for example, welded chairs on top of the base plate to extend the rod length (see Soules et al., 2016).
2. It is assumed that the rectangular stress block (RSB) method outlined previously for force-based design (Chapter 4) provides appropriate characterization of internal stresses and forces in the connection.
3. It is further assumed that the applied loads result in the “large-moment” condition requiring engagement of the anchor rod. It is highly unlikely that the moment due to seismic loading is not large enough to cause uplift of the base plate; in this case, there will be no inelastic action in the base connection.
4. Under the applied axial load, P_u, and moment, M_u, the base plate plan dimensions are first sized using the method provided in Section 4.3.7.
5. The RSB method is used to determine the anchor rod force under the applied loads. The anchor rod is then sized (i.e., the diameter of the threaded section and the reduced diameter $d_{reduced}$) are chosen such that (a) the tensile yield strength of the reduced section is greater than the computed anchor force, and (b) the tensile yield strength of the reduced section is lower than the tensile strength of the unreduced, threaded section of the rod. This ensures that yielding occurs over the stretch length, rather than in the threaded region of the rod.

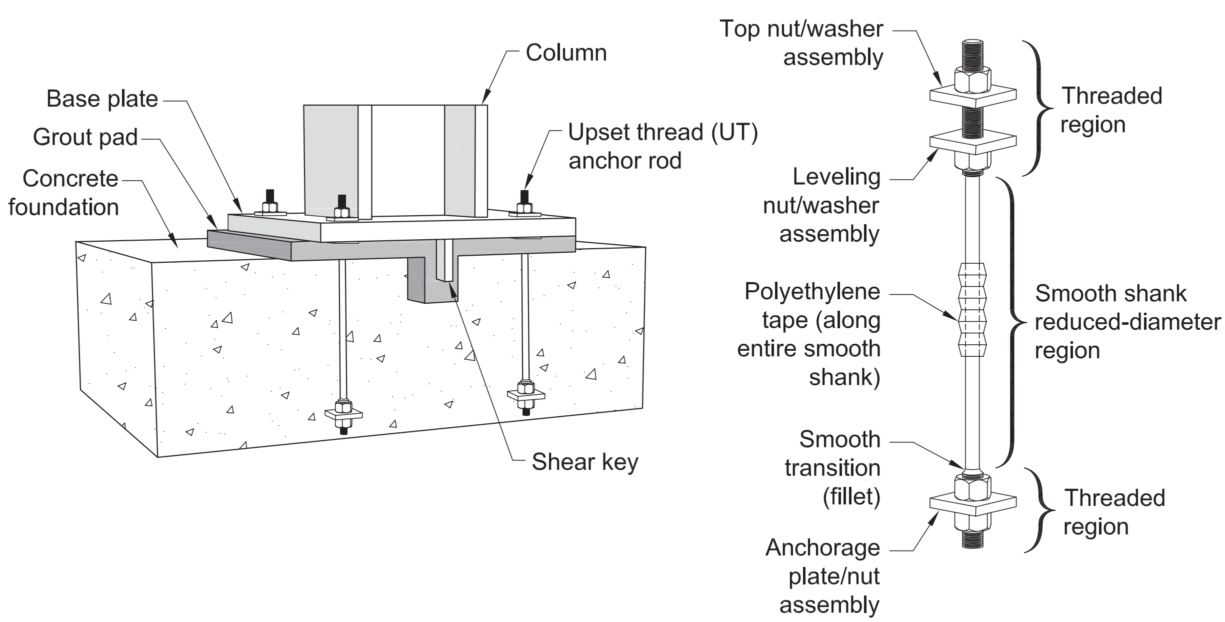


Fig. 6-1. Upset thread detail for weak-base seismic design.

6. The base plate is checked for two flexural limit states: (a) on the compression side of the connection due to the development of the bearing stresses in the footing as per the procedure outlined in Section 4.3.7 and (b) on the tension side of the connection due to the downward bending induced by the fully yielded and strain hardened anchor rods.
7. Additional detailing considerations include the following:
   - Ensuring that the stretch length of the rods is equal to at least half the distance between the sets of rods on either side of the connection—that is, $L_{stretch} \geq f$, where f is illustrated in Figure 4-18. This ensures that the deformations are distributed over a sufficient length, thereby controlling the strains in this region, regardless of the anchor length provisions outlined in ACI 318, Section 17.10.5.3 (Hassan et al., 2022).
   - A smooth transition utilizing fillets between the reduced section and the threaded section to mitigate sharp corners.
   - Application of polyethylene tape to the reduced shank to minimize friction.
   - A shear lug (designed using Section 4.3.3) must be provided to transfer shear. Shear transfer through anchor rod bearing or friction may introduce additional strains in the anchors, compromising their ductility.
   - A leveling nut should be included under the base plate (along with a washer plate), even if shim stacks are used to level the plate. This nut enables transfer of compressive force into the anchors.
   - Sufficient cover should be provided to prevent punching shear failure of the concrete below the anchors. However, experiments indicate that such failure is not detrimental to the overall response of the connection.

Based on the aforementioned considerations, a design example is now presented. The example is presented only in LRFD format because it is expected that weak-base seismic design will very possibly favor the LRFD approach. The UT detail shown herein is not the only means by which ductility may be achieved in the base connections; other solutions may include increasing the stretch length by using chairs on the top surface of the base plate (Soules et al., 2016). Nonetheless, demonstrating the effectiveness of such details may be challenging because there are no prequalification standards for ductile column base connections similar to ANSI/AISC 358.

EXAMPLE 6.4-1—Weak-Base Design of a Base Plate Connection with Ductile Anchor Rods

Design a base plate consistent with AISC *Seismic Provisions* Section D2.6 using the given material properties, member size, and loading.

Given:

The following loads are given, and correspond to the overstrength (i.e., Ω_0) factor.

Column axial compressive force, $P_u = 376$ kips
Shear force, $V_u = 50$ kips
Design moment, $M_u = 3{,}600$ kip-in.

Bending is about the strong axis for a W12×96 wide-flange column. Assume that the ratio of the footing to base plate area is equal to 4. The base plate is ASTM A572/A572M Grade 50 material, and the compressive strength, f_c', of the concrete is 4 ksi. Use ASTM F1554, Grade 55 anchor rods.

Solution:

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
Column
$d = 12.7$ in.
$b_f = 12.2$ in.

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

Anchor rods
ASTM F1554 Grade 55
$F_y = 55$ ksi
$F_u = 75$ ksi

The base plate dimension $N \times B$ should be large enough for the installation of four anchor rods, as required by OSHA. Select dimensions 3 in. larger than the column outside dimensions.

$$N > d + 2(3.00 \text{ in.}) = 18.7 \text{ in.}$$

$$N > b_f + 2(3.00 \text{ in.}) = 18.2 \text{ in.}$$

Try $N = 20.0$ in. and $B = 20.0$ in.

Assume that the anchor rod edge distance is 2 in. Therefore,

$$\begin{aligned} f &= \frac{N}{2} - 2 \text{ in.} \\ &= \frac{20.0 \text{ in.}}{2} - 2 \text{ in.} \\ &= 8.00 \text{ in.} \end{aligned}$$

Determine e and e_{crit}; check the inequality in Equation 4-53 to determine if this is a large or small moment case. For this, first estimate $f_{p(max)}$:

$$\begin{aligned} f_{p(max)} &= \phi_c(0.85 f_c')\sqrt{\frac{A_2}{A_1}} \\ &= 0.65(0.85)(4 \text{ ksi})\sqrt{4} \\ &= 4.42 \text{ ksi} \end{aligned} \qquad \text{(from Eq. 4-2)}$$

$$\begin{aligned} q_{max} &= f_{p(max)}B \\ &= (4.42 \text{ ksi})(20.0 \text{ in.}) \\ &= 88.4 \text{ kip/in.} \end{aligned} \qquad \text{(4-37)}$$

The eccentricity may be calculated as:

$$\begin{aligned} e &= \frac{M_u}{P_u} \\ &= \frac{3{,}600 \text{ kip-in.}}{376 \text{ kips}} \\ &= 9.57 \text{ in.} \end{aligned} \qquad \text{(from Eq. 4-39)}$$

$$\begin{aligned} e_{crit} &= \frac{N}{2} - \frac{P_u}{2q_{max}} \\ &= \frac{20.0 \text{ in.}}{2} - \frac{376 \text{ kips}}{(2)(88.4 \text{ kip/in.})} \\ &= 7.87 \text{ in.} \end{aligned} \qquad \text{(from Eq. 4-40)}$$

Because $e > e_{crit}$,

1. Determine bearing length, Y, and tension in the anchor rod group, T_u.

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_u(e+f)}{q_{max}}} \qquad \text{(from Eq. 4-58)}$$

$$= \left(8.00 \text{ in.} + \frac{20.0 \text{ in.}}{2}\right) \pm \sqrt{\left(8.00 \text{ in.} + \frac{20.0 \text{ in.}}{2}\right)^2 - \frac{2(376 \text{ kips})(9.57 \text{ in.} + 8.00 \text{ in.})}{88.4 \text{ kip/in.}}}$$

$$= 18.0 \text{ in.} \pm 13.2 \text{ in.}$$

$$= 4.80 \text{ in.}$$

$$T_u = q_{max}Y - P_u \qquad \text{(from Eq. 4-55)}$$

$$= (88.4 \text{ kips/in.})(4.80 \text{ in.}) - 376 \text{ kips}$$

$$= 48.3 \text{ kips}$$

2. Determine anchor rod size, stretch length, and embedment.

From previous calculations, $T_u = 48.3$ kips. Referring to discussion in Section 6.4.2, for ductile anchors using the UT detail, the summation of the cross-sectional area of the reduced diameters of the anchors may be calculated as:

$$T_u \le n\phi_t F_{y_{anchor}} A_{reduced}$$

where n is the number of rods on each side of the connection, and $A_{reduced}$ is the cross-sectional area of the UT region—see Figure 6-1. If three rods are used on either side of the connection, then $n = 3$, and:

$$A_{reduced} \ge \frac{T_u}{n\phi_t F_{y_{anchor}}}$$

$$\ge \frac{48.3 \text{ kips}}{(3)(0.9)(55 \text{ ksi})}$$

$$\ge 0.325 \text{ in.}^2$$

This implies a minimum diameter in the UT region $d_{reduced} > 0.644$ in. Specify $d_{reduced} = 0.750$ in. It is important to note here that because the anchors are expected to be fully yielded, the unequal distribution of anchor forces (as discussed previously in Section 4.3.8) is not of concern here. The fully yielded and strain hardened strength of this anchor (assuming ASTM F1554 Grade 55) may be determined as:

$$T_{reduced} = \frac{\pi d_{reduced}^2}{4} F_{u_{anchor}}$$

$$= \frac{\pi (0.750 \text{ in.})^2}{4}(75 \text{ ksi})$$

$$= 33.1 \text{ kips}$$

This is the maximum force associated with fully yielding and strain hardening of the anchor over its stretch length. Consequently, the threaded region of the anchor must have strength that can resist this force. Based on Table 4-1, the design strength of 1-in.-diameter ASTM F1554 Grade 55 anchor rods is 34.1 kips. The recommended hole size for this rod is 1⅞ in., and the minimum washer thickness is ⅜ in. (following recommendations in Table 4-3 in this Guide). It is relevant to make two points here:

- This diameter ensures that the threaded region of the rod (with strength 34.1 kips) will be able to sustain the yielding and strain hardening of the reduced region of the rods.
- The R_t factor (which accounts for the ratio between the expected and specified ultimate strength) is not used here because it is assumed that it is present in both (UT as well as threaded) regions of the rod, which are of identical material.

Referring to Figure 6-1, a minimum stretch length $L_{stretch} \geq f$ must be provided to distribute strains. Thus, a minimum $L_{stretch}$ = 8.00 in. should be specified. Including the end threaded regions, the total embedment of the rod may be selected as h_{ref} = 24.0 in. The concrete limit states, as well as the base plate, should be designed for the fully yielded rods and incorporate the possible difference between the expected and specified ultimate strength, using the R_t factor. Assuming an R_t value of 1.2 (based on ACI 318):

$$\begin{aligned} R_t T_{reduced} &= (1.2)(33.1 \text{ kips}) \\ &= 39.7 \text{ kips} \end{aligned}$$

Referring to Table 4-2, the pullout strength for a 1-in.-diameter anchor (assuming heavy hex heads and nuts) is 33.6 kips, which is lower than the $R_t T_{reduced}$, which is 39.7 kips. Consequently, washer plates must be provided at the lower end. Provide 2-in.-square washer plates and check various limit states using ACI 318, Chapter 17. For headed anchors and including the 0.75 pullout seismic reduction factor per ACI 318, Section 17.10.5.4(c), the pullout strength may be determined as:

$$0.75\phi N_{pn} = 0.75\left(\phi\psi_{c,P} A_{brg} 8 f_c'\right)$$

in which the bearing area A_{brg} is:

$$\begin{aligned} A_{brg} &= (2.00 \text{ in.})^2 - \frac{\pi(1.00 \text{ in.})^2}{4} \\ &= 3.21 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} 0.75\phi N_{pn} &= 0.75\left(\phi\psi_{c,P} A_{brg} 8 f_c'\right) \\ &= (0.75)(0.70)(1.0)(3.21 \text{ in.}^2)(8)(4 \text{ ksi}) \\ &= 53.9 \text{ kips} > 39.7 \text{ kips} \end{aligned}$$

Therefore, the rods are not susceptible to pullout failure. The concrete breakout strength for the anchor group is determined according to ACI 318, Section 17.6.2. In the CCD method, the concrete cone is considered to form at a slope of 1.5 to 1 as discussed in Section 4.3.2.2 of this Design Guide. If the outside rods are placed 16.0 in. apart (s = 16.0 in.), the plan area of the failure cone is given by:

$$\begin{aligned} A_{Nc} &= 2(1.5h_{ef})\left[2(1.5h_{ef}) + s\right] \\ &= 2(1.5)(24.0 \text{ in.})\left[2(1.5)(24.0 \text{ in.}) + 16.0 \text{ in.}\right] \\ &= 6{,}340 \text{ in.}^2 \end{aligned}$$

The plan area of the failure cone for a single rod is:

$$\begin{aligned} A_{Nco} &= 9h_{ef}^2 && \text{(ACI 318, Eq. 17.6.2.1.4)} \\ &= 9(24.0 \text{ in.})^2 \\ &= 5{,}180 \text{ in.}^2 \end{aligned}$$

Because 11.0 in. $\leq h_{ef} \leq$ 25.0 in., ACI 318, Equation 17.6.2.2.3 applies. Use λ_a = 1.0 for normal weight concrete:

$$\begin{aligned} N_b &= 16\lambda_a \sqrt{f_c'}\, h_{ef}^{5/3} && \text{(ACI 318, Eq. 17.6.2.2.3)} \\ &= 16(1.0)\sqrt{4{,}000 \text{ psi}}\,(24.0 \text{ in.})^{5/3}\left(\frac{1 \text{ kip}}{1{,}000 \text{ lbf}}\right) \\ &= 202 \text{ kips} \end{aligned}$$

For a tension load applied concentrically with the anchor rod group, $\psi_{ec,N}$ = 1.0 per ACI 318, Section 17.6.2.3.

Because all edges are located more than $1.5h_{ef}$ from any anchor rod, $\psi_{ed,N}$ = 1.0 per ACI 318, Section 17.6.2.4.

In accordance with ACI 318, Section 17.10.5.4, and because no analysis was performed to confirm that the concrete will remain uncracked, $\psi_{c,N}$ = 1.0 per ACI 318, Section 17.6.2.5.

For cast-in anchors, $\psi_{cp,N}$ = 1.0 per ACI 318, Section 17.6.2.6.

The resulting tension breakout nominal capacity is then given by ACI 318, Section 17.6.2.1, as:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \qquad \text{(ACI 318, Eq. 17.6.2.1b)}$$

$$= \left(\frac{6{,}340 \text{ in.}^2}{5{,}180 \text{ in.}^2}\right)(1.0)(1.0)(1.0)(1.0)(202 \text{ kips})$$

$$= 247 \text{ kips}$$

Because supplementary reinforcement is not provided to restrain the concrete breakout cone, ϕ = 0.70. The capacity must be further reduced by the 0.75 seismic reduction factor per ACI 318, Section 17.10.5.4(b). The resulting available concrete breakout strength of the anchor group in tension is given by:

$$0.75\phi N_{cbg} = 0.75(0.70)(247 \text{ kips})$$

$$= 130 \text{ kips}$$

The required strength to confirm that the nonductile concrete breakout limit state will not govern is:

$$N_u = 3(R_t T_{reduced})$$

$$= 3(1.2)(33.1 \text{ kips})$$

$$= 119 \text{ kip} < 130 \text{ kips} \quad \textbf{o.k.}$$

Provide three 1-in.-diameter ASTM F1554 Grade 55 anchor rods, located 2.00 in. from the edge of the plate. Provide a minimum stretch length $L_{stretch}$ = 8.00 in., with a reduced diameter $d_{reduced}$ = 0.750 in. and a total embedment depth h_{ef} = 24.0 in

Because there are no adjacent concrete edges, sideface blowout is not applicable. Other limit states indicated in ACI 318, Chapter 17 (e.g., steel strength in shear or breakout in shear), are not applicable.

3. Determine the plate thickness.

Following research by Gomez et al. (2010) and others, as well as the AISC *Seismic Provisions* Commentary, only the tension side interface is checked for flexural yielding. This assumes that base plate flexural yielding on the bearing interface will not affect overall performance in a significant manner. Three anchor rods are provided on the tension interface, indicating that a single straight yield line will form parallel to the column flange. The bending length associated with plate flexure per Equation 4-61 is x = 2.10 in. Therefore,

$$t_{p(req)} = 2.11\sqrt{\frac{T_u x}{BF_y}} \qquad \text{(4-62a)}$$

$$= 2.11\sqrt{\frac{(119 \text{ kips})(2.10 \text{ in.})}{(20.0 \text{ in.})(50 \text{ ksi})}}$$

$$= 1.05 \text{ in.}$$

Provide a 1¼-in.-thick ASTM A572/A572M Grade 50 base plate.

4. Design the welds.

All welds in the base connection are considered demand critical as defined in the AISC *Seismic Provisions*. For the column flange-to-base plate welds, provide CJP groove welds following the detailing guidelines in the AISC *Seismic Provisions*, specifically, “Where columns are welded to base plates with groove welds, weld tabs and weld backing shall be removed, except that weld backing located on the inside of flanges and weld backing on the web of I-shaped sections need not be removed if backing is attached to the column base plate with a continuous 5⁄16 in. (8 mm) fillet weld. Fillet welds of backing to the inside of column flanges are prohibited. Weld backing located on the inside of HSS and box-section columns need not be removed."

The welds between the web and the base plate may be designed for the shear. The effective length of the weld available on both sides of the web, excluding the "*k*" region is:

$$\begin{aligned} l_e &= d - 2k_{des} \\ &= 12.7 \text{ in.} - 2(1.50 \text{ in.}) \\ &= 9.70 \text{ in.} \end{aligned}$$

The weld size in sixteenths of an inch (for E70 weld material) is:

$$\begin{aligned} D_{req} &= \frac{V_u}{1.392(2l_e)} \qquad \text{(from AISC } \textit{Manual} \text{ Eq. 8-2a)} \\ &= \frac{50.0 \text{ kips}}{(1.392 \text{ kip/in.})(2)(9.70 \text{ in.})} \\ &= 1.85 \text{ sixteenths} \end{aligned}$$

Provide a minimum weld size of ¼ in. required for the 0.550-in.-thick web of the W12×96.

5. Design the shear lug.

Referring to the preceding discussion, when anchor rods are used as the yielding element in a weak base, a shear lug is required to transfer the shear force into the footing. The design of the shear lug (i.e., shear lug dimensions, embedment, edge distance, and welds) will depend on the dimensions of the footing. The procedure outlined in Chapter 4, Section 4.3.3, may be used for this purpose. If shear is transferred through the lug, then it is important to check the design for the additional moment due to the shear on the base connection. Assuming the shear lug protrudes 3.00 in. from the bottom of the base plate, and the grout layer is 1.00 in. thick, the resulting lever arm is 2.00 in. This results in an additional moment of 100 kip-in. applied to the base plate, such that the total moment is:

$$\begin{aligned} M_{u_{total}} &= M_u + M_{shear\text{–}lug} \\ &= 3{,}600 \text{ kip-in} + (50.0 \text{ kips})(1.00 \text{ in.} + 2.00 \text{ in.}/2) \\ &= 3{,}700 \text{ kip-in.} \end{aligned}$$

The anchor diameter is checked against this updated value of moment, resulting in a requirement that $d_{reduced} \geq 0.687$ in. This is still lower than the specified $d_{reduced} = 0.750$ in. Thus, the design does not need to be revised.

6.5 SEISMIC DESIGN OF BRACED FRAME BASE PLATE CONNECTIONS

The seismic design of base connections in braced frames has received minimal attention in research. Moreover, a braced frame base connection in seismic situations is typically designed as a strong base connection. Astaneh-Asl (2008) provides some additional guidance regarding such design, although it is not based on research. For the purposes of this Guide, the design procedure outlined in Chapter 4 for exposed base plate connections in braced frames may be used for seismic design as well.

Appendix A
Special Considerations for Double-Nut Joints, Pretension Joints, and Special Structures

A.1 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 anchor rod chairs.

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 nonredundant 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 et al., 1996). The base plates of light and sign standards are not grouted after erection, and the rod carries all 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 unless the sleeves are grouted and designed to transfer the forces to 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 with a length of 24 in., it only takes concrete creep/shrinkage of 0.050 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 A-1.

Large mill building columns that must be set accurately and have large moments at the base can be designed using an anchor rod chair detail as shown in Figure A-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 anchor rod chair avoids having to use a CJP groove weld between the column base and the heavy base plate. If the column and base plate are over 2 in. thick, using a CJP weld detail would require special material toughness. The use of the anchor rod chair has the added advantage that the extended anchor rod length will allow easier adjustment to meet the holes in the anchor rod chair cap plate.

A.1.1 Compression Limit State for Anchor Rods

With the typical 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:

$$R_c = F_y A_g \tag{A-1}$$

$$\phi_c = 0.90$$

$$\Omega_c = 1.67$$

where

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}

F_y = specified minimum yield stress, ksi

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

Typically, the clear distance under the base plate should not exceed 2.50 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 in 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 ACI 318 criteria.

A.1.2 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 per AISC

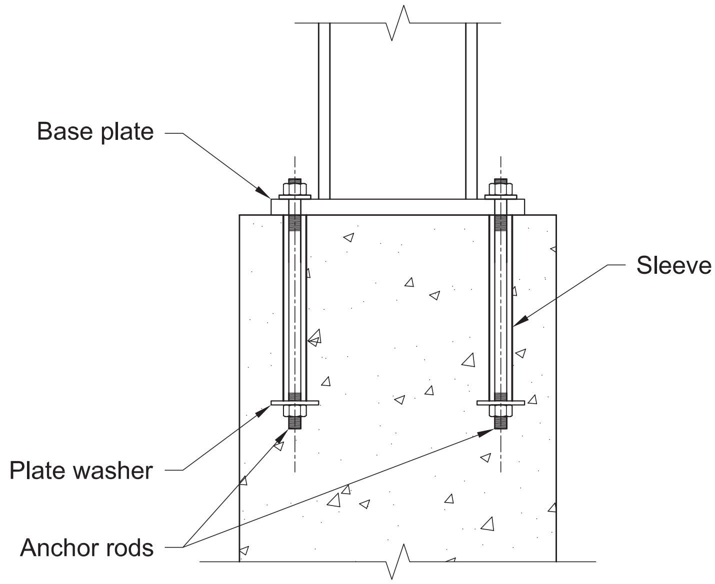


Fig. A-1. Anchor rods with sleeves.

Fig. A-2. Column moment base using an anchor rod chair.

Specification Appendix 3, 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, as well as 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 nonredundant 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 nonredundant. 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 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 (Dexter and Ricker, 2002).

Corrosion protection is particularly important for fatigue-critical anchor rods because 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.

Based on AISC *Specification* Appendix 3, 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. In the AISC *Specification,* this condition of using Category E′ with a fatigue threshold of 7 ksi is represented as Category G.

Because 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 multiplied by 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 of 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:

$$F_{SR} = 1{,}000\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \quad (Spec.\ \text{Eq. A-3-1})$$

where

C_f = constant for stress category, equal to 0.39 for stress category G

F_{SR} = allowable stress range, ksi

F_{TH} = threshold allowable stress range, maximum stress range for indefinite design life, equal to 7 ksi for stress category G

n_{SR} = number of stress range fluctuations in design life

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 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 thicknesses. 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 1½ 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 et al., 2003). In these tests, a socket joint detail with a 2-in.-thick base plate performed much better in fatigue than one with a 1½-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 A-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" 1¼-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. For a base plate 2¼ in. thick, the outer stress at this location decreases to about 65% of what it would be for a 1¼-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 1¼-in.-thick base plate.

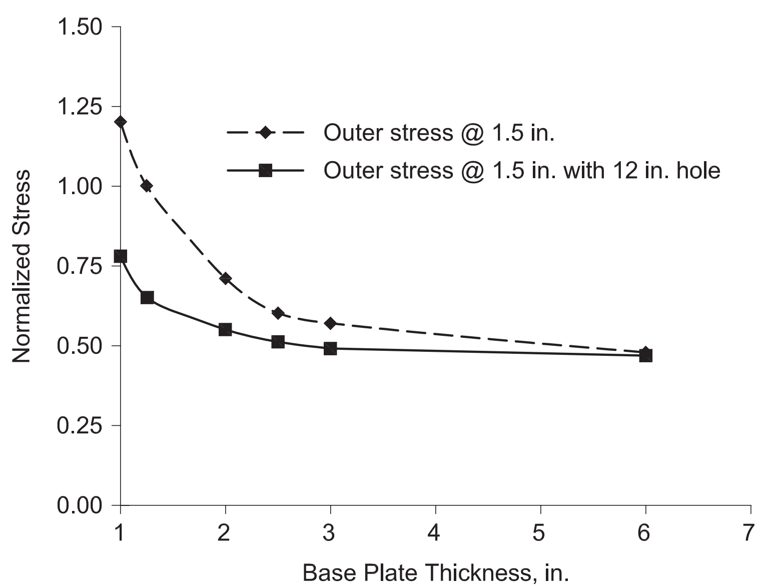


Fig. A-3. Stresses in base plate.

A.2 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 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 comprised of 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 ASCE/SEI 7, 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 to 30% of the final tension. For anchor rods, this is defined as a function of torque, as:

$$T_v = 0.12 d_b T_m \tag{A-2}$$

where

T_m = minimum installation pretension, kips, given in Table A-1

T_v = verification torque, kip-in.

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

Till and Lefke (1994) have 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.

A.2.1 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 retighten 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 AWS D1.1/D1.1M. While tack welding to the unstressed top of

Table A-1. Minimum Anchor Rod Pretension for Double-Nut Moment Joints					
Anchor Rod Diameter, in.	Rebar Designation	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/A615M and A706/A706M Bars Grade 60[b]
½	#4	4.00	6.00	11.0	7.00
⅝	#5	7.00	10.0	17.0	11.0
¾	#6	10.0	15.0	25.0	16.0
⅞	#7	13.0	21.0	35.0	22.0
1	#8	18.0	27.0	45.0	28.0
1⅛	—	22.0	34.0	57.0	—
—	#9	—	—	—	36.0
1¼	—	28.0	44.0	73.0	—
—	#10	—	—	—	46.0
—	#11	—	—	—	56.0
1½	—	41.0	63.0	106	—
—	#14	—	—	—	81.0
1¾	—	55.0	85.0	143	—
2	—	73.0	113	187	—
2¼	—	94.0	146	244	—
—	#18	—	—	—	144
2½	—	116	180	300	—
2¾	—	143	222	370	—
3	—	173	269	448	—
3¼	—	206	319	—	—
3½	—	242	375	—	—
3¾	—	280	434	—	—
4	—	322	499	—	—

[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.

the anchor rod is relatively harmless, under no circumstance should any nut be tack welded to the washer or the base plate.

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 Equation A-3:

$$T_v = 0.12 d_b T_m \tag{A-3}$$

where

T_m = minimum installation pretension, kips, given in Table A-1

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

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 (because there is only one anchor rod). The nut should be rotated to at least the required rotation given in Table A-2.

Table A-2. Nut Rotation for Turn-of-Nut Pretensioning of Unified National Coarse (UNC) Threads		
Anchor Rod Diameter, in.	**Nut Rotation[a],[b],[c]**	
	ASTM F1554 Rod Grade 36	**ASTM F1554 Grades 55 and 105, A615/A615M Grades 60 and 75, and A706/A706M 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°.
[b] Applicable only to UNC threads.
[c] Beveled washer should be used if: (a) the nut is not in firm contact with the base plate or (b) the outer face of the base plate is sloped more than 1:40.

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.
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 hr 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 within the tolerances specified in the AISC *Code of Standard Practice*. 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 hr 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 and 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 cannot 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 20 to 30% of the verification torque following a star pattern.
16. Leveling nuts should be tightened to 20 to 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 A-2 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 hr, 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 hr 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 ASCE/SEI 7, or designed for fatigue, the nut should be prevented from loosening unless a maintenance plan is in place to verify at least every 4 yr that a torque equal to at least 110% of the verification torque is required to additionally tighten the leveling nuts and the top nuts.

A.2.2 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 centerhole 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 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 hr 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*. 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 an 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 hr 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.

A.3 INSPECTION AND MAINTENANCE AFTER INSTALLATION

Regular inspection and maintenance should be conducted for joints that are designed for fatigue. All joints designed for Seismic Design Category D or greater, according to ASCE/SEI 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 because the zinc from the galvanized parts will attempt to protect the bare steel. This may result in a weakening of the galvanizing, furthering the corrosion of the carbon steel (NSBA, 2022). 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 A-1 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 study 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 the AISC *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 no 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 the 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
Alternate Methods for Design

B.1 CONTEXT AND MOTIVATION

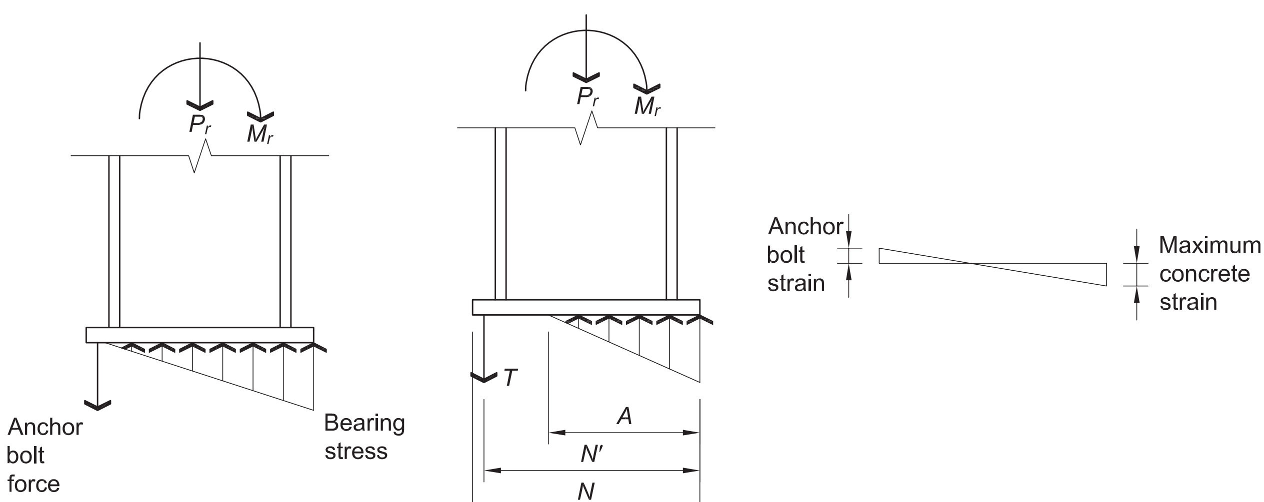
This appendix is focused on alternate methods of designing exposed column base plate connections. These alternate methods are presented for three specific contexts: (1) design of base plates under flexure and axial compression using the triangular stress block method, (2) design of base plates under axial compression considering base flexibility, and (3) design of the base plate bearing interface under two-way bending. Experimental data as well as simulations indicate these alternate methods provide performance comparable to that produced by the design methods presented in this Guide but may offer either slightly conservative designs (in the case of the triangular stress block approach) or significantly more economical designs for some cases (in the case of the base plate under compression and two-way bending). Each of these is described in detail in the following sections.

B.2 TRIANGULAR PRESSURE DISTRIBUTION

B.2.1 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 that 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 because the centroid of the pressure distribution is closer to the cantilevered edge of the plate.



(a) Resultant compressive bearing stress under column flange *(b) General case* *(c) Strain distribution*

Fig. B-1. Triangular stress distribution for axial load plus moment.

B.2.2 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 strength over the width of the base plate equal to the available flexural strength and solving for t:

LRFD	ASD
$$t_{req} = \sqrt{\frac{4M_{u,pl}}{\phi_b B F_y}} \quad \text{(B-1a)}$$ where ϕ_b = resistance factor in bending = 0.90	$$t_{req} = \sqrt{\frac{4M_{a,pl}\Omega_b}{B F_y}} \quad \text{(B-1b)}$$ where Ω_b = safety factor in bending = 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.2.3 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 are then combined by superposition to calculate the pressure distribution across the plate.

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

$$f_{pa} = \frac{P_r}{A_1} \tag{B-2}$$

$$f_{pb} = \frac{M_r}{S_{pl}} \tag{B-3}$$

where

A_1 = area of base plate plan dimensions ($B \times N$), in.^{2}

M_r = applied bending moment, kip-in.

P_r = applied axial compressive load, kips

S_{pl} = section modulus of base plate area with respect to direction of applied moment, in.^{3}

$= \dfrac{BN^2}{6}$ for bending of a rectangular plate

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:

$$e = \frac{M_r}{P_r} \tag{B-4}$$

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:

$$\frac{P_r}{A_1} = \frac{M_r}{S_{pl}} \tag{B-5}$$

$$\frac{P_r}{BN} = \frac{P_r e}{\left(\dfrac{BN^2}{6}\right)} \tag{B-6}$$

$$e = \frac{N}{6} \tag{B-7}$$

This point, where $e = N/6$, is commonly called the kern of the base plate.

B.2.4 Design Procedure

Design Procedure for a Small Moment Base

1. Choose trial base plate sizes (B and N) based on the geometry of the column and anchor rod layout.
2. Determine plate cantilever dimension, m or n, in the direction of the applied moment (see Figure 4-1).

$$m = \frac{N - 0.95d}{2} \tag{4-10}$$

$$n = \frac{B - 0.80b_f}{2} \tag{4-11}$$

3. Determine applied loads, P_r and M_r (P_u and M_u for LRFD, P_a and M_a for ASD) based on ASCE/SEI 7 load combinations.
4. Determine eccentricity e and e_{kern}.

$$e = \frac{M_r}{P_r} \tag{B-8}$$

$$e_{kern} = \frac{N}{6} \tag{B-9}$$

   If $e \le e_{kern}$, this is a small moment base, and no tension exists between the base plate and the foundation. See Figure B-2(a).

   If $e > e_{kern}$, this is a large moment base and must be designed for tension anchorage per the subsequent large moment design procedure [see Figure B-2(b)].

5. Determine base pressures.

   Due to axial compression:

$$f_{p(ax)} = \frac{P_r}{A_1} = \frac{P_r}{BN} \tag{B-10}$$

   Due to applied moment:

$$f_{p(b)} = \frac{M_r}{S_{pl}} = \frac{6P_r e}{BN^2} \tag{B-11}$$

Combined pressure:

$$f_{p(max)} = f_{p(ax)} + f_{p(b)} \tag{B-12}$$
$$= \frac{P_r}{BN}\left(1+\frac{6e}{N}\right) \le f_{p\ avail}$$

where

LRFD	ASD
$f_{p\ avail} = \phi_c 0.85 f_c'$ (B-13a)	$f_{p\ avail} = \dfrac{0.85 f_c'}{\Omega_c}$ (B-13b)

Additionally, $f_{p\ avail}$ may be calculated accounting for concrete confinement when $A_2 > A_1$.

If $f_{p(max)} \ge f_p\ avail$, adjust the base plate dimensions.

$$f_{p(min)} = f_{p(ax)} - f_{p(b)} \tag{B-14}$$
$$= \frac{P_r}{BN}\left(1-\frac{6e}{N}\right)$$

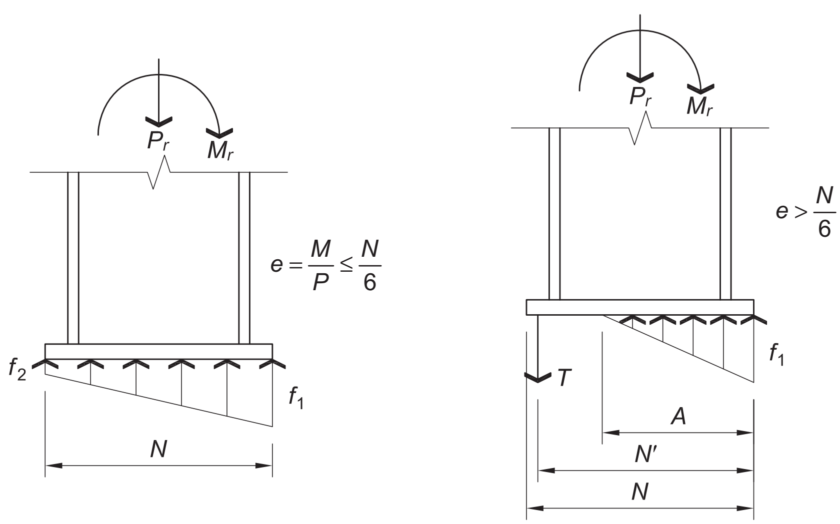
6. Determine pressure at m distance from $f_{p(max)}$.

$$f_{p(m)} = f_{p(max)} - 2f_{p(b)}\left(\frac{m}{N}\right) \tag{B-15}$$

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:

$$M_{pl} = \left(f_{p(m)}\right)\left(\frac{m^2}{2}\right) + \left(f_{p(max)} - f_{p(m)}\right)\left(\frac{m^2}{3}\right) \tag{B-16}$$



(a) Small eccentricity—bearing on full plate

(b) Large eccentricity—bearing on partial plate

Fig. B-2. Effect of eccentricity on bearing.

For bending about a plane at m, perpendicular to the applied moment:

$$M_{pl} = f_{p(ax)}\left(\frac{n^2}{2}\right) \tag{B-17}$$

The critical moment is the larger of M_{pl} about the m and n critical planes.

8. Determine required plate thickness based on the required flexural strength per inch of plate:

LRFD	ASD
$$t_{req} = \sqrt{\frac{4M_{u\,pl}}{\phi_b F_y}} \quad \text{(B-18a)}$$ where $\phi_b = 0.90$	$$t_{req} = \sqrt{\frac{4M_{a\,pl}\Omega_b}{F_y}} \quad \text{(B-18b)}$$ where $\Omega_b = 1.67$

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, as shown in Figure B-2(b). 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:

$$T_r + P_r = \frac{f_p AB}{2} \tag{B-19}$$

$$P_r A' + M_r = \frac{f_p AB}{2}\left(N' - \frac{A}{3}\right) \tag{B-20}$$

where

A' = the distance between the anchor rod and the column center, in.

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:

$$A = \frac{3N' \pm \sqrt{(3N')^2 - \dfrac{24(P_r A' + M_r)}{f_p B}}}{2} \tag{B-21}$$

The resulting tensile force in the anchor rods is then:

$$T_r = \frac{f_p AB}{2} - P_r \tag{B-22}$$

The design procedure is as follows:

1. Determine the available bearing strength, $\phi_c P_p$ or P_p/Ω_c, using AISC *Specification* Section J8:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \le 1.7 f_c' A_1 \qquad \text{(}Spec.\text{ Eq. J8-2)}$$

$$\phi_c = 0.65 \text{ (LRFD)}$$

$$\Omega_c = 2.31 \text{ (ASD)}$$

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 Equation B-21. 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 because 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 a larger plate size.
4. Determine the resultant anchor rod force, T_r, from Equation B-22. If it is reasonable, go to the next step. Otherwise return to Step 2.
5. Determine the required flexural strength per in. 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
$t_p = \sqrt{\dfrac{4M_{u\,pl}}{\phi_b F_y}}$ (B-23a)	$t_p = \sqrt{\dfrac{4M_{a\,pl}\Omega_b}{F_y}}$ (B-23b)

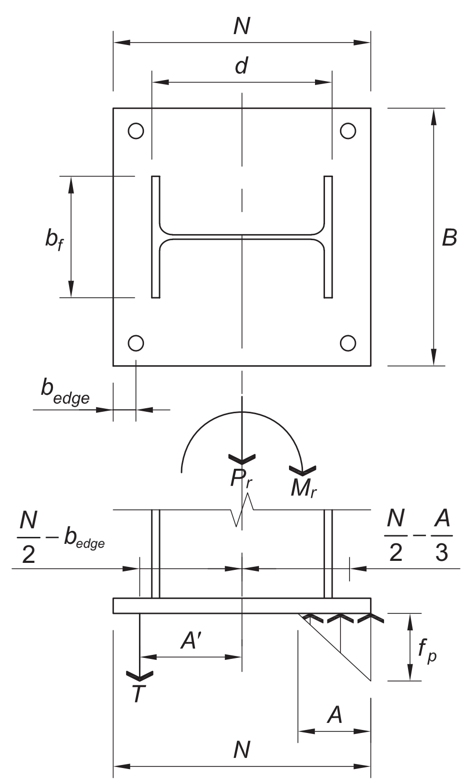


Fig. B-3. General definition of variables.

EXAMPLE B.2-1—Base Connection for Bending without Anchor Rod Tension (Low Moment), Triangular Pressure Distribution

A base connection for a wide-flange column subject to compression and moment is designed in this example. The ratio of flexure to compression is such that the moment can be resisted without producing tension in the anchor rods. A triangular pressure distribution is considered.

Given:

A W12×96 column is subject to an axial dead load of 100 kips and an axial live load equal to 160 kips and moments from the dead and live loads equal to 250 kip-in. and 400 kip-in., respectively. Bending is about the strong axis of the column. The ratio of the concrete to base plate area is unity. The base plate is ASTM A572/A572M Grade 50 material, and the compressive strength, f'_c, of the concrete is 4 ksi.

Solution:

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×96
Column
d = 12.7 in.
b_f = 12.2 in.

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

Base plate
ASTM A572/A572M Grade 50
F_y = 50 ksi
F_u = 65 ksi

1. Choose a trial base plate size (B and N) based on geometry of the column and the four-anchor-rod requirement.

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

2. Determine plate cantilever dimension, m or n, based on the procedure outlined in the previous section.

$$m = \frac{(N - 0.95d)}{2} \tag{4-10}$$

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

$$= 3.47 \text{ in.}$$

$$n = \frac{B - 0.80b_f}{2} \tag{4-11}$$

$$= \frac{19 \text{ in.} - 0.80(12.2 \text{ in.})}{2}$$

$$= 4.62 \text{ in.}$$

3. From ASCE/SEI 7, Chapter 2, determine the required strength:

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

4. Determine eccentricity e and e_{kern}.

LRFD	ASD
$e_u = \dfrac{M_u}{P_u}$ (B-8) $= \dfrac{940\text{ kip-in.}}{376\text{ kips}}$ $= 2.50\text{ in.}$	$e_a = \dfrac{M_a}{P_a}$ (B-8) $= \dfrac{650\text{ kip-in.}}{260\text{ kips}}$ $= 2.50\text{ in.}$

$$e_{kern} = \frac{N}{6} \tag{B-9}$$
$$= \frac{19\text{ in.}}{6}$$
$$= 3.17\text{ in.}$$

Because $e = 2.50$ in. < $e_{kern} = 3.17$ in., 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:

LRFD	ASD
$f_{pu(ax)} = \dfrac{P_u}{BN}$ (B-10) $= \dfrac{376\text{ kips}}{(19\text{ in.})(19\text{ in.})}$ $= 1.04\text{ ksi}$	$f_{pa(ax)} = \dfrac{P_a}{BN}$ (B-10) $= \dfrac{260\text{ kips}}{(19\text{ in.})(19\text{ in.})}$ $= 0.720\text{ ksi}$

Due to applied moment:

LRFD	ASD
$f_{pu(b)} = \dfrac{6P_u e_u}{BN^2}$ (from Eq. B-11) $= \dfrac{6(376\text{ kips})(2.50\text{ in.})}{(19\text{ in.})(19\text{ in.})^2}$ $= 0.822\text{ ksi}$	$f_{pa(b)} = \dfrac{6P_a e_a}{BN^2}$ (from Eq. B-11) $= \dfrac{6(260\text{ kips})(2.50\text{ in.})}{(19\text{ in.})(19\text{ in.})^2}$ $= 0.569\text{ ksi}$

Combined pressure:

LRFD	ASD
$f_{pu(max)} = f_{pu(ax)} + f_{pu(b)}$ (B-12) $= 1.04\text{ ksi} + 0.822\text{ ksi}$ $= 1.86\text{ ksi}$	$f_{pa(max)} = f_{pa(ax)} + f_{pa(b)}$ (B-12) $= 0.720\text{ ksi} + 0.569\text{ ksi}$ $= 1.29\text{ ksi}$
$f_{pu(min)} = f_{pu(ax)} - f_{pu(b)}$ (B-14) $= 1.04\text{ ksi} - 0.822\text{ ksi}$ $= 0.218\text{ ksi}$	$f_{pa(min)} = f_{pa(ax)} - f_{pa(b)}$ (B-14) $= 0.720\text{ ksi} - 0.569\text{ ksi}$ $= 0.151\text{ ksi}$

The maximum available bearing strength is then confirmed by Equation B-13.

LRFD	ASD
$f_{p\ avail} = \phi 0.85 f'_c$ (B-13a) $= \dfrac{0.65(0.85)(4\text{ ksi})}{2.31}$ $= 2.21\text{ ksi} > 1.86\text{ ksi}$ **o.k.**	$f_{p\ avail} = \dfrac{0.85 f'_c}{\Omega_c}$ (B-13b) $= \dfrac{0.85(4\text{ ksi})}{2.31}$ $= 1.47\text{ ksi} > 1.29\text{ ksi}$ **o.k.**

6. Determine pressure at critical bending plane (m distance from $f_{p(max)}$)

LRFD	ASD
$f_{pu(m)} = f_{pu(max)} - 2f_{pu(b)}\left(\dfrac{m}{N}\right)$ (B-15) $= 1.86\text{ ksi} - 2(0.822\text{ ksi})\left(\dfrac{3.47\text{ in.}}{19.0\text{ in.}}\right)$ $= 1.56\text{ ksi}$	$f_{pa(m)} = f_{pa(max)} - 2f_{pa(b)}\left(\dfrac{m}{N}\right)$ (B-15) $= 1.29\text{ ksi} - 2(0.569\text{ ksi})\left(\dfrac{3.47\text{ in.}}{19.0\text{ in.}}\right)$ $= 1.08\text{ ksi}$

7. Determine M_{pl} for bending about critical planes m and n:

Bending of a 1-in.-wide strip of plate about a plane at m, in the direction of the applied moment, is determined using Equation B-16:

LRFD	ASD
$M_{u\ pl} = \left(f_{pu(m)}\right)\left(\dfrac{m^2}{2}\right) + \left(f_{pu(max)} - f_{pu(m)}\right)\left(\dfrac{m^2}{3}\right)$ $= (1.56\text{ ksi})\dfrac{(3.47\text{ in.})^2}{2}$ $+ (1.86\text{ ksi} - 1.56\text{ ksi})\dfrac{(3.47\text{ in.})^2}{3}$ $= 10.6\text{ kip-in./in.}$	$M_{a\ pl} = \left(f_{pa(m)}\right)\left(\dfrac{m^2}{2}\right) + \left(f_{pa(max)} - f_{pa(m)}\right)\left(\dfrac{m^2}{3}\right)$ $= (1.08\text{ ksi})\dfrac{(3.47\text{ in.})^2}{2}$ $+ (1.29\text{ ksi} - 1.08\text{ ksi})\dfrac{(3.47\text{ in.})^2}{3}$ $= 7.34\text{ kip-in./in.}$

Because this is a case of axial loads plus small moments, bending about a plane at n, perpendicular to the applied moment, can be determined using the following procedure using the axial load only. Note that for axial loads plus large moments, a more refined analysis is required.

LRFD	ASD
$M_{u\ pl} = f_{pu(ax)}\left(\dfrac{n^2}{2}\right)$ (B-17) $= 1.04\text{ ksi}\dfrac{(4.62\text{ in.})^2}{2}$ $= 11.1\text{ kip-in./in.}$	$M_{a\ pl} = f_{pa(ax)}\left(\dfrac{n^2}{2}\right)$ (B-17) $= 0.720\text{ ksi}\dfrac{(4.62\text{ in.})^2}{2}$ $= 7.68\text{ kip-in./in.}$

The critical moment is the larger of M_{pl} about m and n critical planes.

LRFD	ASD
$M_{u\ crit} = 11.1\text{ kip-in./in.}$	$M_{a\ crit} = 7.68\text{ kip-in./in.}$

8. Determine required plate thickness.

   Note: Because 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
$t_{u\,req} = \sqrt{\dfrac{4M_{u\,crit}}{\phi_b F_y}}$ (from Eq. B-18a) $= \sqrt{\dfrac{(4)(11.1 \text{ kip-in./in.})}{(0.90)(50 \text{ ksi})}}$ $= 0.993$ in.	$t_{a\,req} = \sqrt{\dfrac{4M_{a\,crit}\Omega_b}{F_y}}$ (from Eq. B-18b) $= \sqrt{\dfrac{(4)(7.68 \text{ kip-in./in.})(1.67)}{50 \text{ ksi}}}$ $= 1.01$ in.

   Use a plate 19 in. × 19 in. × 1 in. (LRFD) or 1¼ in. (ASD).

EXAMPLE B.2-2—Base Connection for Bending with Anchor Rod Tension (Large Moment), Triangular Pressure Distribution

A base connection for a wide-flange column subject to compression and moment is designed in this example. The ratio of flexure to compression is such that the moment produces tension in the anchor rods. A triangular pressure distribution is considered.

Given:

A W8×31 column is subject to the loads shown in Figure B-4. The ratio of the concrete to base plate area (A_2/A_1) is 4.00. Bending is about the strong axis of the column. The base plate is ASTM A572/A572M Grade 50 material, the anchor rods are ASTM F1554 Grade 36, and the compressive strength, f_c', of the concrete is 3 ksi.

Solution:

From AISC *Manual* Table 1-1, the geometric properties of the column are as follows:

W8×31
$d = 8.00$ in.
$b_f = 8.00$ in.

From AISC *Manual* Tables 2-5 and 2-6, the material properties are as follows:

Base plate
ASTM A572/A572M Grade 50
$F_y = 50$ ksi
$F_u = 65$ ksi

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

1. Determine the available bearing strength.

LRFD	ASD
$\dfrac{\phi_c P_p}{A_1} = \phi_c 0.85 f'_c \sqrt{A_2/A_1} \le \phi_c 1.7 f'_c$ $= (0.65)(0.85)(3\text{ ksi})\sqrt{4.00}$ $= 3.32\text{ ksi}$ $\le (0.65)(1.7)(3\text{ ksi}) = 3.32\text{ ksi}$ $f_{pu} = 3.32\text{ ksi}$	$\dfrac{P_p}{\Omega_c A_1} = \dfrac{0.85 f'_c \sqrt{A_2/A_1}}{\Omega_c} \le \dfrac{1.7 f'_c}{\Omega_c}$ $= \dfrac{(0.85)(3\text{ ksi})\sqrt{4.00}}{2.31}$ $= 2.21\text{ ksi}$ $\le \dfrac{(1.7)(3\text{ ksi})}{2.31} = 2.21\text{ ksi}$ $f_{pa} = 2.21\text{ ksi}$

2. Assume a 14 in. × 14 in. base plate. The effective eccentricity is:

LRFD	ASD
$P_u = 90.0\text{ kips}$ $M_u = 720\text{ kip-in.}$ $e = \dfrac{M_u}{P_u}$ (B-10) $= \dfrac{720\text{ kip-in.}}{90.0\text{ kips}}$ $= 8.00\text{ in.}$	$P_a = 60.0\text{ kips}$ $M_a = 480\text{ kip-in.}$ $e = \dfrac{M_a}{P_a}$ (B-10) $= \dfrac{480\text{ kip-in.}}{60.0\text{ kips}}$ $= 8.00\text{ in.}$

Because $e > e_{kern} = N/6 = 2.33$ in., anchor rods are required to resist the tensile force. The anchor rods are assumed to be 1.50 in. from the plate edge.

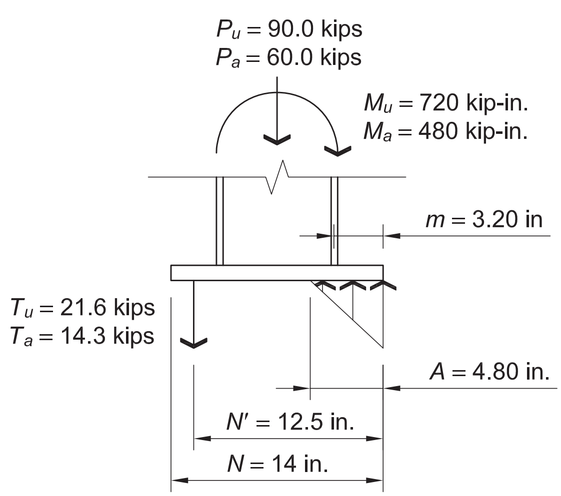


Fig. B-4. Design example with large eccentricity.

3. Determine the length of bearing using Equation B-21:

LRFD	ASD
$A = \dfrac{3N' \pm \sqrt{(3N')^2 - \dfrac{24(P_u A' + M_u)}{f_{pu} B}}}{2}$ $= \dfrac{\left(3(12.5 \text{ in.}) \pm \sqrt{[(3)(12.5 \text{ in.})]^2 - \left\{\dfrac{24[(90.0 \text{ kips})(5.50 \text{ in.}) + 720 \text{ kip-in.}]}{(3.32 \text{ ksi})(14 \text{ in.})}\right\}}\right)}{2}$ $= 4.80 \text{ in.}$	$A = \dfrac{3N' \pm \sqrt{(3N')^2 - \dfrac{24(P_a A' + M_a)}{f_{pa} B}}}{2}$ $= \dfrac{\left(3(12.5 \text{ in.}) \pm \sqrt{[(3)(12.5 \text{ in.})]^2 - \left\{\dfrac{24[(60.0 \text{ kips})(5.50 \text{ in.}) + 480 \text{ kip-in.}]}{(2.21 \text{ ksi})(14 \text{ in.})}\right\}}\right)}{2}$ $= 4.80 \text{ in.}$

In the calculation of A, the minus sign before the radical controls the solution.

4. Determine the required tensile strength of the anchor rod using Equation B-22, and distribute to the two anchor rods per side.

LRFD	ASD
$T_u = \dfrac{f_{pu} AB}{2} - P_u$ $= \dfrac{(3.32 \text{ ksi})(4.80 \text{ in.})(14 \text{ in.})}{2} - 90.0 \text{ kips}$ $= 21.6 \text{ kips}$ $T_{rod} = \dfrac{T_u}{2}$ $= \dfrac{21.6 \text{ kips}}{2}$ $= 10.8 \text{ kips}$	$T_a = \dfrac{f_{pa} AB}{2} - P_a$ $= \dfrac{(2.21 \text{ ksi})(4.80 \text{ in.})(14 \text{ in.})}{2} - 60.0 \text{ kips}$ $= 14.3 \text{ kips}$ $T_{rod} = \dfrac{T_a}{2}$ $= \dfrac{14.3 \text{ kips}}{2}$ $= 7.15 \text{ kips}$

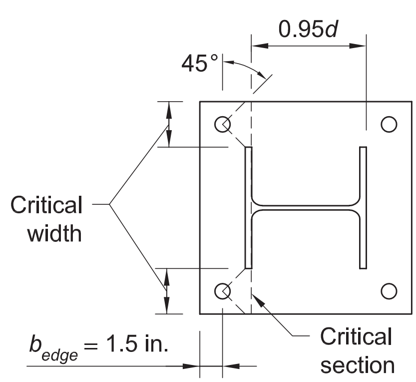


Fig. B-5. Critical plate width for anchor rod (tension side).

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 45.0° 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:

$$\begin{aligned} m &= \frac{N - 0.95d}{2} \\ &= \frac{14 \text{ in.} - (0.95)(8.00 \text{ in.})}{2} \\ &= 3.20 \text{ in.} \end{aligned} \tag{4-10}$$

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
$f_{pu(m)} = (3.32 \text{ ksi})\left(\dfrac{4.80 \text{ in.} - 3.20 \text{ in.}}{4.80 \text{ in.}}\right)$ $= 1.11 \text{ ksi}$ $M_{u\,pl} = \dfrac{(1.11 \text{ ksi})(3.20 \text{ in.})^2}{2} + \dfrac{(3.32 \text{ ksi} - 1.11 \text{ ksi})(3.20 \text{ in.})^2}{3}$ $= 13.2 \text{ kip-in./in.}$	$f_{pa(m)} = (2.21 \text{ ksi})\left(\dfrac{4.80 \text{ in.} - 3.20 \text{ in.}}{4.80 \text{ in.}}\right)$ $= 0.737 \text{ ksi}$ $M_{a\,pl} = \dfrac{(0.737 \text{ ksi})(3.20 \text{ in.})^2}{2} + \dfrac{(2.21 \text{ ksi} - 0.737 \text{ ksi})(3.20 \text{ in.})^2}{3}$ $= 8.80 \text{ kip-in./in.}$

Anchor rods are placed at a 1½ in. edge distance. Using the moment arm and effective width per anchor from Figure B-5, 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
$M_{u\,pl} = \dfrac{(10.8 \text{ kips})(3.20 \text{ in.} - 1.50 \text{ in.})}{2(3.20 \text{ in.} - 1.50 \text{ in.})}$ $= 5.40 \text{ kip-in./in.}$	$M_{a\,pl} = \dfrac{(7.15 \text{ kips})(3.20 \text{ in.} - 1.50 \text{ in.})}{2(3.20 \text{ in.} - 1.50 \text{ in.})}$ $= 3.58 \text{ kip-in./in.}$

The required moment strength due to the bearing stress distribution is critical. The required plate thickness is:

LRFD	ASD
$t_p = \sqrt{\dfrac{4M_{u\,pl}}{\phi_b F_y}}$ (B-23a) $= \sqrt{\dfrac{4(13.2 \text{ kip-in./in.})}{(0.90)(50 \text{ ksi})}}$ $= 1.08 \text{ in.}$	$t_p = \sqrt{\dfrac{4M_{a\,pl}\Omega_b}{F_y}}$ (B-23b) $= \sqrt{\dfrac{4(8.80 \text{ kip-in./in.})(1.67)}{50 \text{ ksi}}}$ $= 1.08 \text{ in.}$

Use a 14.0 × 14.0 × 1¼ in. base plate.

B.3 DESIGN OF BASE PLATES UNDER AXIAL COMPRESSION CONSIDERING FLEXIBILITY

Chapter 4, Section 4.3.1, presents a design approach for the design of base plates subjected to axial compression. Two limit states—concrete bearing and base plate yielding are considered. Of these, the plate yielding limit state corresponds to upward bending of the base plate as shown in Figure 4-1. Specifically, the base plate is assumed to yield at the suggested locations of the yield lines [Figure 4-1(b)] under the upward bearing pressure. The upward bearing pressure in turn is assumed to be constant, which implicitly suggests that the base plate itself is rigid. However, this assumption can result in extremely large moments on the base plate yield lines if the base plate has a large footprint (or large in-plane dimensions), resulting in very thick base plates. Experimental and simulation data by Steenhuis et al. (2008) and Denavit (2022) suggests this is conservative because a large base plate is also flexible, such that the bearing stresses concentrate under the column flanges and webs. This stress distribution results in significantly lower moments in the base plate. Under such a situation, the base plate may be designed by assuming it to be rigid but with an effective area as shown in Figure B-6. Specifically, the effective area extends a distance $c = 1.5t_p$ outside the webs and the flanges. The base plate may simply be designed by checking the bearing stresses over this area (termed A_{eff}; see Figure B-6) against the bearing capacity of the footing. The effective area of the rigid base plate (i.e., the distance c) is calibrated such that it results in stresses in the base plate equivalent to a flexible base plate; there is no need to independently conduct the check for base plate yielding.

Thus, under an applied load P_u, the only design check to be conducted is:

$$\frac{P_u}{A_{eff}} \le \phi f_{p(max)} \tag{B-24}$$

where, $\phi = 0.65$, and the bearing strength of the footing may be determined as outlined previously in Chapter 4—that is, on the full area of a concrete support:

$$f_{p(max)} = 0.85 f_c' \tag{B-25}$$

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

$$f_{p(max)} = 0.85 f_c' \sqrt{A_2/A_1} \le 1.7 f_c' \tag{B-26}$$

Further, if grout is used under the base plate, the grout compressive strength should always be higher than the concrete compressive strength. Because the grout compressive strength is always specified higher than the concrete strength, the concrete compressive strength, f_c', must be used in the preceding equations. It is recommended that the grout strength be specified as two times the concrete strength. Lower grout strengths may be justified if the bearing strength of the grout (treated as unconfined) is checked against the required strength.

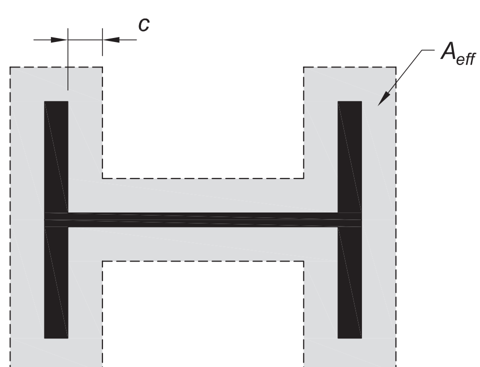


Fig. B-6. Effective bearing area to account for plate flexibility.

EXAMPLE B.3-1—Base Connection for Concentric Axial Compression Load (with Concrete Confinement)

Given:

A W12×96 column bears on a large concrete pedestal (such that concrete is fully confined) as shown in Figure B-7. The minimum concrete compressive strength is $f'_c = 4$ ksi. The base plate is A572/A572M Grade 50 material. The bearing force is $P_u = 700$ kips.

Solution:

From AISC *Manual* Table 1-1, the geometric properties of the column are:

W12×96
d = 12.7 in.
b_f = 12.2 in.
t_f = 0.900 in.
t_w = 0.550 in.

Try a 1¼-in.-thick base plate.

$$c = 1.5t_p$$
$$= (1.5)(1.25 \text{ in.})$$
$$= 1.88 \text{ in.}$$

A base plate with dimensions 18 in. × 18 in. fully accommodates this effective area on each side (see Figure B-7). The area of the shaded portion in Figure B-7 may be readily calculated as $A_{eff} = 178$ in.^{2} As a result, the strength in bearing may be determined as:

$$\phi P_n = \phi f_{p(max)} A_{eff}$$
$$= (0.65)(1.7)(4 \text{ ksi})(178 \text{ in.}^2)$$
$$= 787 \text{ kips} > 700 \text{ kips} \quad \textbf{o.k.}$$

This is an acceptable design.

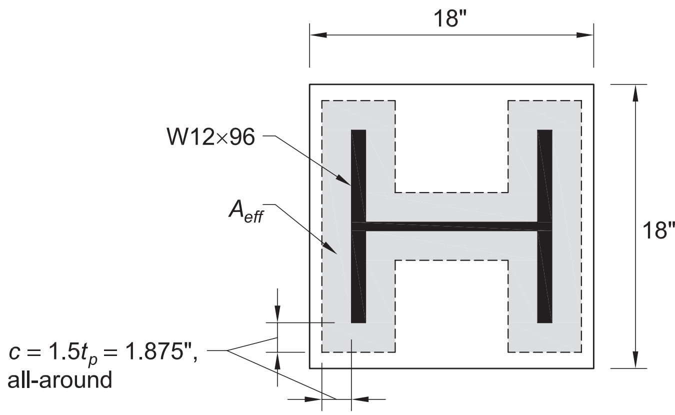


Fig. B-7. Design of base plate under axial compression.

B.4 DESIGN OF BASE PLATE BEARING INTERFACE UNDER TWO-WAY BENDING

Two-way bending may often govern the required thickness of base plates under combinations of axial load and applied moment, even if the moment is applied in one direction. Two-way bending refers to the bending of the column base plate perpendicular to the primary direction of bending. Previous discussion on the topic in this Guide is included in the notes in Section 4.3.7, which indicates that "When n is larger than m, the thickness will be governed by n."

In effect, this sets the effective bearing width of the base plate in out of plane bending, b_{eff}, equal to the compression length Y [see Figure B-8(a)]. As per research by Haninger and Tong (2014), this is a reasonable assumption when the base plate is in compression along its entire length or when Y is large relative to n [Figure B-8(a)]. However, when Y is small relative to n [Figure B-8(b)], a greater width of the plate (than Y) is engaged in bending because of two-way bending effects. This effect is qualitatively shown in Figure B-8(b). In these situations, disregarding this additional bearing width may be conservative, leading to thicker base plates.

For these situations, the following may be used to determine the effective width.

For $Y < 2n$:

$$b_{eff} = \frac{Y}{2} + n \tag{B-27}$$

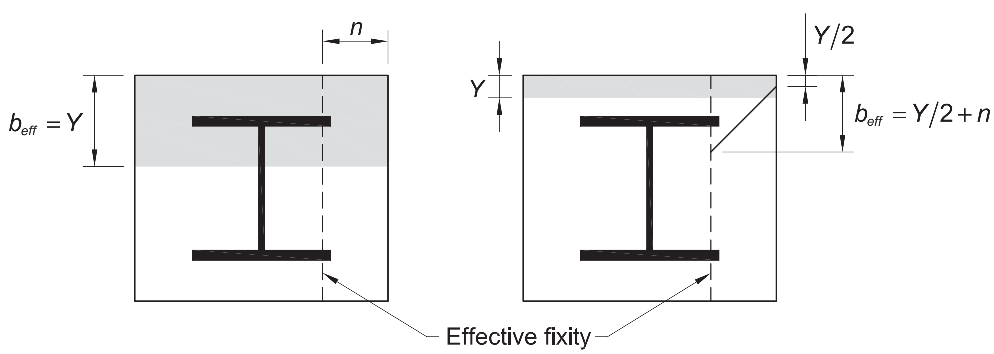
For $Y \geq 2n$:

$$b_{eff} = Y \tag{B-28}$$

Once the effective width is determined, the plate thickness required to resist two-way bending may be determined as:

$$t_{p(req)} = n\sqrt{\frac{2 f_{p(max)} Y}{\phi F_y b_{eff}}} \tag{B-29}$$

where $f_{p(max)}$ is the bearing strength of the footing calculated as discussed in Chapter 4, and $\phi = 0.90$ for plate bending.



(a) Effective bending width equal to bearing length

(b) Effective bending width with two-way bending

Fig. B-8. Effective width for two-way bending of base plates.

Appendix C
Guidance for Simulating Column Base Connections in Structural Analysis

C.1 INTRODUCTION

Referring to Chapter 3, base connections (and, more generally, the foundation systems) interact structurally with the frame, influencing its performance in multiple ways. In a vast majority of cases (with the exception of weak-base seismic design), base connections are expected to remain elastic under design level loads. In these cases, the main characteristic of these base connections that interacts with the structure is the elastic stiffness. In the minority of cases when the base connection is expected to yield, post-yield properties of the base connection (such as strength degradation and ductility) become important. In this appendix, the first condition is addressed in detail in Section C.2, whereas brief commentary is provided regarding the hysteretic (post-yield) properties in Section C.3. The focus is on the in-plane rotational properties (stiffness and hysteretic) of base connections in the context of moment frames or cantilever columns when a brace is not present. Other modes of deformation of the base connection (i.e., shear and axial) are not considered in this appendix. More specifically, this appendix provides guidance for estimating the rotational stiffness of base connections given the configuration of the base connection. In general, it is good practice to represent the rotational stiffness of the connection as a flexible spring in all cases. However, when such calculation is not feasible from a practical standpoint, it is recommended that the structural analysis solutions are bound with both fixed and pinned base solutions.

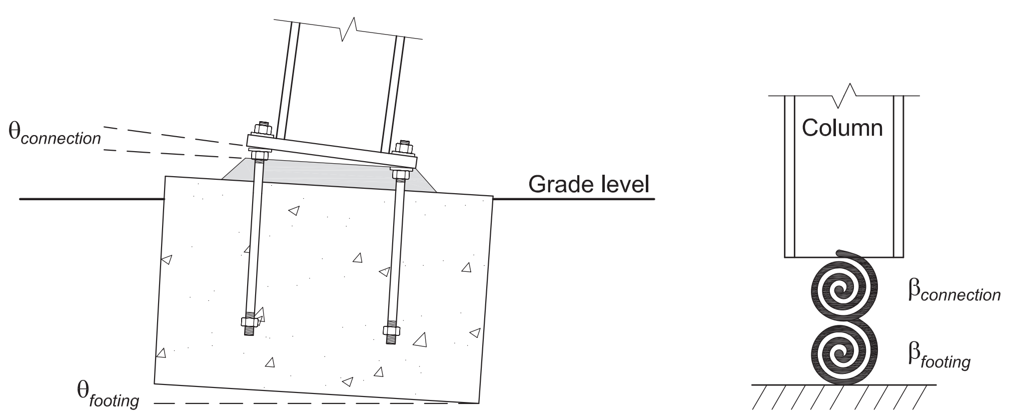
C.2 ROTATIONAL STIFFNESS MODELS

Base connections are typically designed to remain elastic under design loads. Thus, from a structural simulation standpoint, rotational stiffness of these connections is the primary characteristic of interest. Figure C-1(a) schematically illustrates a typical exposed base connection discussed previously along with the supporting footing. Figure C-1(b) shows the idealization of the base connection stiffness within the context of structural analysis. Referring to these figures, it is important to note that the overall flexibility of the base connection includes the flexibility of the column to footing connection and the flexibility of the footing and foundation system itself, including its interaction with the soil.

Each of these subcomponents may be represented as a rotational spring with stiffness $\beta_{connection}$ and $\beta_{footing}$, respectively. The effective stiffness of the system may be determined as the series stiffness of the two springs, considering that their rotations are additive:

$$\beta_{base} = \frac{\beta_{connection}\,\beta_{footing}}{\beta_{connection} + \beta_{footing}} \tag{C-1}$$

The main focus of this appendix is estimating the stiffness $\beta_{connection}$, which reflects the stiffness of the connection between the column and the footing, recognizing that the stiffness $\beta_{footing}$ will be controlled by footing and soil characteristics that are outside the scope of this Guide. Zareian and Kanvinde (2013) and Melchers (1992) provide some



(a) *Schematic illustration* (b) *Idealization as spring in series*

Fig. C-1. Deformation at column base.

guidance on how to estimate this. The stiffness models for column base connections depend on the type of base connection. Consequently, this section is divided into three subsections, each addressing the three types of connections—exposed, blockout, and embedded connections.

C.2.1 Estimation of Rotational Stiffness for Exposed Column Base Connections

The method presented in this section may be used to characterize the rotational stiffness of exposed base connections. Referring to discussion in Chapter 3 (Section 3.3.1), the moment-rotation response of the base connection exhibits slight nonlinearity even in the initial stages of loading, due to uplift of the base plate and nonlinearity of the concrete footing. Consequently, it is expedient to compute a secant stiffness of the connection at a predetermined level of moment—for example, the design moment M_u, given the axial compression P_u. Once this moment is selected, a series of steps may be followed to determine the rotational stiffness $\beta_{connection}$; the background and validation of this approach is presented in Kanvinde et al. (2012), with subsequent validation against other datasets in Trautner et al. (2017b) and against instrumented buildings in Falborski and Kanvinde (2022). Figure C-2 shows the underlying assumptions regarding the deformations that are included in the approach presented herein.

The process involves the following steps:

1. Determine if the moment M_u corresponds to a low- or high-moment condition as implied in the design process outlined previously in Chapter 4. Specifically, the eccentricity may be calculated as:

$$e = \frac{M_u}{P_u} \tag{C-2}$$

and compared to the critical value:

$$e_{crit} = \frac{N}{2} - \frac{P_u}{2q_{max}} \tag{C-3}$$

where the symbols carry their typical meanings as outlined in Chapter 4.

2. If $e \geq e_{crit}$, then the moment corresponds to the high-eccentricity condition with uplift of the base plate and engagement of the anchor rods. This implies that the total deformation of the base connection will be due to elongation of the anchors, in addition to the bending of the base plate and compressive deformations of the footing under the toe of the base plate; this condition is shown in Figure C-2. In this case, the rotational stiffness of the base connection may be calculated as:

$$\beta_{connection} = \frac{M_u}{\theta} \tag{C-4}$$

In the Equation C-4, the rotation, θ, may be determined through enforcement of compatibility on the deformations in the various components of the connections, as shown in Figure C-2. Specifically:

$$\theta = \frac{\left(\Delta_{rod} + \Delta_{plate}^{tension} + \Delta_{plate}^{compression} + \Delta_{footing}\right)}{\left(f + N/2\right)} \tag{C-5}$$

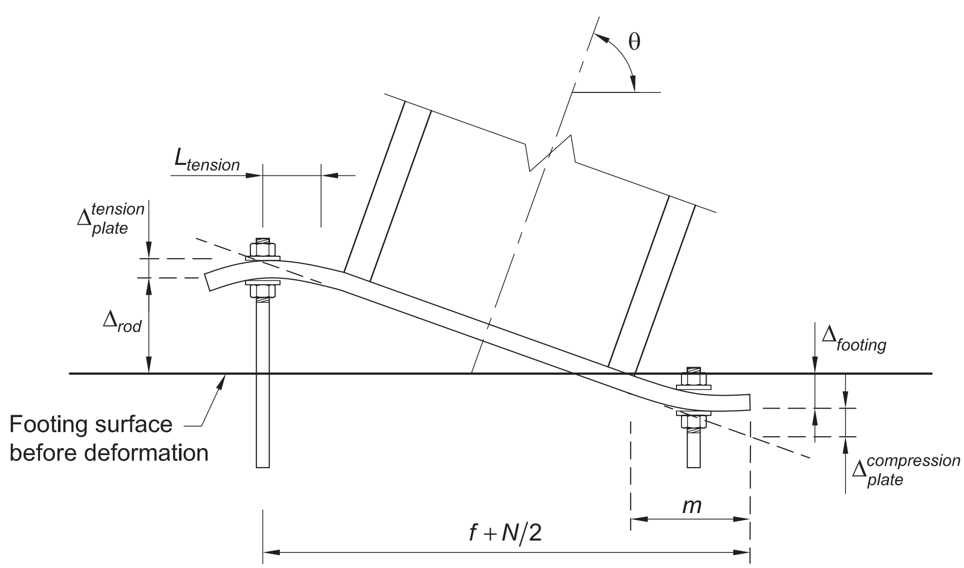


Fig. C-2. Contributors to deformation of exposed base plate connection.

Determination of the four deformation components requires estimation of the internal forces in the base connection. This may be accomplished in a straightforward manner by using the method outlined in Chapter 4 to calculate two key quantities under the applied combination of M_u and P_u—the tension in the anchor rod, T, and the width of the bearing interface, Y. Once these are determined, the four deformation components may be conveniently estimated as shown in the following. The axial elongation of the anchor rod may be determined as:

$$\Delta_{rod} = \frac{TL_{rod}}{A_{rod}E_{rod}} \tag{C-6}$$

In Equation C-6, the term L_{rod} refers to the total length of the anchor rod, from the top of the base plate to the top of the nut at the bottom anchorage assembly. The terms A_{rod} and E_{rod} refer to the gross cross-sectional area and the modulus of elasticity of the rod, respectively.

The flexural deflection of the plate on the tension side of the connection may be determined as:

$$\Delta_{plate}^{tension} = \frac{TL_{tension}^3}{3E_{plate}I_{plate}} + \frac{TL_{tension}}{A_{plate}^{shear}G_{plate}} \tag{C-7}$$

In Equation C-7 , the term $L_{tension}$ denotes the cantilever bending length of the base plate on the tension side of the connection. Specifically,

$$L_{tension} = f - \frac{d}{2} \tag{C-8}$$

where d is the column section depth, such that $L_{tension}$ is the distance between the edge of the column section and the centerline of the anchor rods. The term A_{plate}^{shear} is the effective shear area of the base plate, which may be determined as:

$$A_{plate}^{shear} = \left(\frac{5}{6}\right)Bt_p \tag{C-9}$$

The 5/6 factor accounts for the effective shear area of a rectangular cross section, while B and t_p are the base plate width (out of plane) and thickness, respectively. The term $I_{plate} = Bt_p^3/12$ is the cross-sectional moment of inertia of the plate. The flexural deflection of the plate on the compression side of the connection may be determined depending on whether the bearing width Y is greater or lesser than the plate compression-side flap, m. Thus, two equations arise:

If $Y \geq m$

$$\Delta_{plate}^{compression} = f_{max}B\left(\frac{m^4}{8E_{plate}I_{plate}} + \frac{m^2}{2A_{plate}^{shear}G_{plate}}\right) \tag{C-10}$$

If $Y < m$

$$\Delta_{plate}^{compression} = \frac{f_{max}B}{8E_{plate}I_{plate}}\left[m^4 - \frac{(m-Y)^3(3m+Y)}{3}\right] + \frac{f_{max}BY}{A_{plate}^{shear}G_{plate}}\left(m - \frac{Y}{2}\right) \tag{C-11}$$

Finally, the deformation in the footing may be determined as:

$$\Delta_{bearing} = \frac{f_{max}d_{footing}}{E_{concrete}} \tag{C-12}$$

In Equation C-12, the term $d_{footing}$ represents the total depth of the footing, whereas the elastic modulus of concrete may be determined as $E_{concrete} = w_c^{1.5}\sqrt{f_c'}$, where both f_c' and $E_{concrete}$ are in ksi units, and w_c is the weight of concrete per unit volume in lb/ft^3, where $90 \leq w_c \leq 155$ lb/ft^3 (AISC, 2022c). Once these four deformations are determined, they may be substituted into Equations C-4 and C-5 to calculate the base rotational stiffness.

3. If $e < e_{crit}$, then the moment corresponds to the low-eccentricity condition, and there is no uplift on the base plate. In this situation, the deformations occur only in the footing because the anchor rods are not engaged. To address this situation, the following relationship is proposed by Kanvinde et al. (2012) to estimate the rotation, θ, at the applied moment, M_u. See Kanvinde et al. (2012) for a complete physical explanation of this relationship.

$$\theta = \frac{d_{footing}\left(\varepsilon_{footing}^{toe} - \varepsilon_{footing}^{rod}\right)}{\left(f + \frac{N}{2}\right)} \tag{C-13}$$

In Equation C-13, the strains $\varepsilon_{footing}^{toe}$ and $\varepsilon_{footing}^{rod}$ represent the estimated strains under the toe of the base plate and the anchor rod (on the side where the applied moment produces tension). These may be estimated as follows:

$$\varepsilon_{footing}^{toe} = \frac{f_{footing}^{toe}}{E_{concrete}} \tag{C-14}$$

where $f_{footing}^{toe}$ may be determined based on the low-moment condition outlined in Chapter 4, such that:

$$f_{footing}^{toe} = \frac{P_u^2}{P_u BN - 2M_u B} \tag{C-15}$$

The strain at the location of the rod may be determined as:

$$\varepsilon_{footing}^{rod} = \varepsilon_{footing}^{toe}\left(1 - \frac{M_u}{M_{crit}}\right) \tag{C-16}$$

In Equation C-16, $M_{crit} = P_u e_{crit}$, such that when $M_u = M_{crit}$, the strain at the location of the rod $\varepsilon_{footing}^{rod} = 0$. On the other hand, when no moment is applied, $\varepsilon_{footing}^{rod} = \varepsilon_{footing}^{toe}$—that is, a flat strain profile under the plate. Once θ has been determined, it may be used with Equation C-4 to determine the rotational stiffness.

A tool for convenient calculation of the rotational stiffness for exposed base plate connections, $\beta_{connection}$, is available on the AISC website at www.aisc.org/dg1.

It is relevant to note here that the rotational stiffness is sensitive to the level of axial compression present in the column. The axial compression itself may be unknown because the analysis to determine it may require representation of base rotational stiffness. In this context, two observations are presented here:

- For simulations that represent seismic loads, the axial force under the applicable gravity loading (without seismic effects) may be determined by running the analysis assuming a fixed base condition or through tributary load analysis. Then, this may be used within the approach outlined in the preceding to determine $\beta_{connection}$. If desired, iterations may be performed to identify a set of mutually consistent $\beta_{connection}$ and axial force values, recognizing that this may become highly cumbersome for frames with multiple columns.
- In cases where the estimate of base flexibility is required to estimate column effective length and compressive capacity (e.g., $P_n = F_{cr}A_g$), iterative analysis may be performed wherein the determined critical load is consistent with the rotational stiffness $\beta_{connection}$, which is used to determine the effective length.

C.2.2 Estimation of Rotational Stiffness for Shallowly Embedded or Blockout Base Connections

As discussed in Section 3.2.1, blockout connections are used when a slab on grade is provided on top of the exposed base plate connection. This results in a shallow embedment that adds to the stiffness of the connection. In contrast to embedded base connections (discussed in Section C.2.3), shallowly embedded connections have a complete exposed base plate connection beneath the blockout, independently designed to carry loads. The embedment provided by the slab is incidental. Figure C-3 shows such a connection, identifying the key parameters used in stiffness determination.

Richards et al. (2018) provides a detailed analysis of these connections and a method to characterize their stiffness. In this approach, multiple variants of the stiffness-characterization approach are provided, with varying levels of mathematical complexity, with a tradeoff between complexity and accuracy. In this Guide, the most general (and simplified) version of the model is presented; for the more detailed models, the reader is referred to Richards et al. (2018) and Tryon (2016).

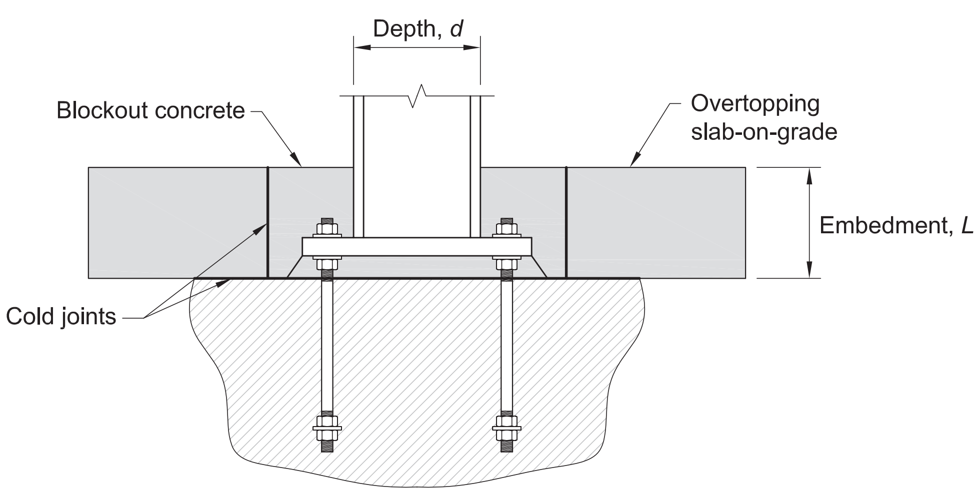


Fig. C-3. Blockout connection with L *and* D *dimensions used for stiffness estimation.*

The base connection stiffness, $\beta_{connection}$, may be determined using the following equations. Different equations are provided for major- and minor-axis bending of the columns.

For major-axis bending of the column:

When $L/D \leq 0.5$:

$$\frac{\beta_{connection}\lambda^{2.85}}{b_f} = 174 \tag{C-17a}$$

When $0.5 < L/D \leq 2.0$:

$$\frac{\beta_{connection}\lambda^{2.85}}{b_f} = 84\frac{L}{D} + 132 \tag{C-17b}$$

When $L/D > 2.0$:

$$\frac{\beta_{connection}\lambda^{2.85}}{b_f} = 300 \tag{C-17c}$$

For minor-axis bending of the column:

When $L/D \leq 0.5$:

$$\frac{\beta_{connection}\lambda^{2.85}}{d} = 129 \tag{C-18a}$$

When $0.5 < L/D \leq 2.0$:

$$\frac{\beta_{connection}\lambda^{2.85}}{d} = 14\frac{L}{D} + 122 \tag{C-18b}$$

When $L/D > 2.0$:

$$\frac{\beta_{connection}\lambda^{2.85}}{d} = 150 \tag{C-18c}$$

In these equations, the term L/D generically represents the ratio of the embedment, L, to the column depth denoted generically as D in the direction of bending. Specifically, D may be taken as d, the column depth, for major-axis bending as shown in Figure C-3 and b_f, the flange width, for minor-axis bending. The term λ incorporates the properties of the concrete and the column, and may be calculated as:

$$\lambda = \sqrt[4]{\frac{k}{4EI}} \tag{C-19}$$

in Equation C-19, the term EI represents the flexural stiffness in the direction of bending, whereas the term k represents the bearing stiffness per unit length of the embedment. More specifically, the term k may be determined as:

$$k = k_0 d_{bearing} \tag{C-20}$$

in which the modulus of subgrade reaction for normal strength concrete may be taken as k_0 (which is in the range of 300–600 kips/in.^{3}) with a recommended value of 500 kips/in.^{3} for normal strength concrete. The bearing width may be taken as $d_{bearing} = 2b_f - t_w$ for major-axis bending, and $d_{bearing} = d$ (i.e., the depth of the column) for minor-axis bending. The former accounts for bearing of both the flanges (see Richards et al., 2018).

A tool for convenient calculation of the rotational stiffness $\beta_{connection}$ for blockout connections is available on the AISC website at www.aisc.org/dg1.

C.2.3 Estimation of Rotational Stiffness for Embedded Base Connections

Embedded base connections are often assumed to be infinitely stiff to provide a fixed base condition. However, research indicates that there are numerous modes of deformation within the footing that contribute to the flexibility of these connections, such that they cannot be assumed as rigid. Torres-Rodas et al. (2017) analyzed the deformations of embedded base connections and determined that embedded base connections exhibit rotational flexibility due to deformation of the concrete, which results in rigid body motion of the embedded portion of the column in addition to deformations of the embedded portion of the column itself; the latter include both flexural and shear deformations. A simplified method for the estimation of the rotational stiffness of embedded column base connections is not currently available. Consequently, the reader is directed to the following resources (an online spreadsheet-based tool) for convenient calculation of the rotational stiffness of embedded base connections. Theoretical background for this tool is provided in Torres-Rodas et al. (2017).

A tool for convenient calculation of the rotational stiffness $\beta_{connection}$ for embedded base plate connections is available on the AISC website at www.aisc.org/dg1.

C.3 COMMENTARY REGARDING HYSTERETIC PROPERTIES OF BASE CONNECTIONS

Base connections are expected to yield only in a small minority of situations when they are designed as weak bases for seismic loading. Even in these situations, simulating their response may be necessary only in the context of performance assessment [e.g., within a FEMA P-695 framework (2009)], rather than in the context of design. It is expected that given the highly focused nature of such applications (rather than routine design or assessment), significant effort will be made by the user in selecting appropriate software and modeling constructs, calibrating, and then verifying the models before use. Consequently, only a high-level commentary is provided here. The commentary focuses on exposed base plate connections because it is anticipated that embedded base connections will not typically be designed as weak bases.

C.3.1 Physics of Connection Response

Figure C-4 shows the typical hysteretic response of an exposed column base connection, and Figure C-5 illustrates the various phenomena responsible for this hysteretic response.

The response shown in Figure C-5 is representative of a large number of base connections tested in the various studies mentioned in the introduction. The response of the exposed base connection may be deconstructed into six distinct phases. These phases are demarcated by discrete, visually observable events (and correspond to sudden changes in the load deformation response), rather than processes such as concrete spalling, which result in more gradual nonlinearity between these events. The events correspond to one half cycle of loading, and they repeat on subsequent half cycles, accompanied by deterioration due to concrete spalling and residual deformations of the plate and anchor rods. The phases are summarized in the following paragraphs.

Phase I: Shown in Figure C-5(a), this corresponds to the initial loading response, where the load is carried by stresses in the compression bearing block and tension in the anchor rods. The plate itself is subjected to bending on both the tension and compression sides of the connection. The response within this phase is slightly nonlinear due to the response of the concrete/grout. This continues until either the base plate or the anchor rods begin to yield. The plate may yield on the tension or compression side of the connection, depending upon its design (see Chapter 4). For the purposes of illustration, here it is assumed that yielding is on the compression (bearing) side of the base plate.

Phase II: During this phase [Figure C-5(b)], the element (e.g., base plate) that yielded at the end of Phase I continues to yield with increasing deformations. The next event is the yielding of another region of the connection. For example, if the first event is yielding of the base plate on the compression side (as assumed in Phase I), the second event may correspond to yielding of the anchor rods or yielding of the base plate on the tension side.

Phase III: This second yielding event [shown in Figure C-5(c) as yielding of the anchor rods] creates a mechanism in the base connection, resulting in a yield plateau (or an ultimate strength) of the base connection. In general, this phase may continue until one of the following two scenarios occurs: (1) sudden loss of strength due to fracture of an anchor rod or the base plate weld or (2) unloading and loading in the reverse direction.

Phase IV: Shown in Figure C-5(d), this corresponds to elastic unloading as the base plate and anchor rods are relaxed. This continues until the top surface of the base plate loses contact with the bottom surface of the nut-washer arrangement. At this point, a gap is formed between the base plate and the grout due to the inelastic deformations accrued during Phases I and II. This occurs when the entire moment applied by the base plate may be carried in the base without the anchor rods—that is, only due to the prestress effect of the axial compressive force (because the plate-grout interface cannot carry tensile stress, which is necessary for carrying any base moment). If no axial compression is present, then the loss of contact occurs at zero moment.

Phase V: Shown in Figure C-5(e), once the base plate loses contact with the nut-washer (i.e., the anchor rod) it continues to move freely downward, as the connection maintains a roughly constant moment; an intermediate loading plateau that demonstrates a "pinching" behavior. Phase IV ends when the base plate makes contact with the grout.

Phase VI: As shown in Figure C-5(f), once the plate contacts the grout, the unloading becomes much more rapid because the incremental unloading (negative) moment has a much stiffer load path. In combination, Phases IV and V

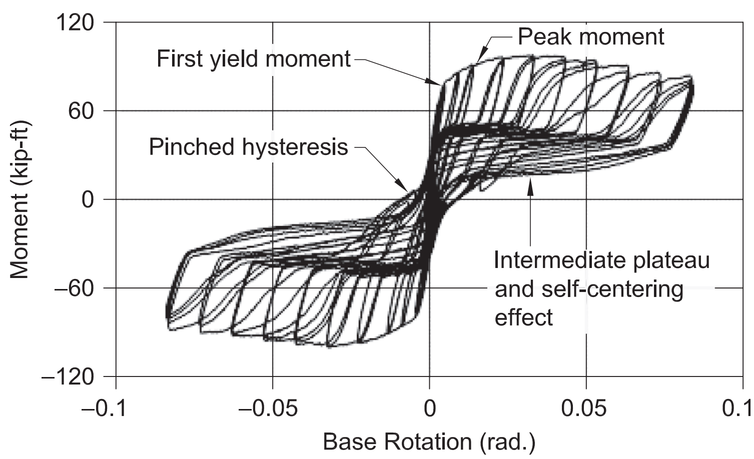


Fig. C-4. Moment rotation response of exposed base plate connection (from Gomez et al., 2010).

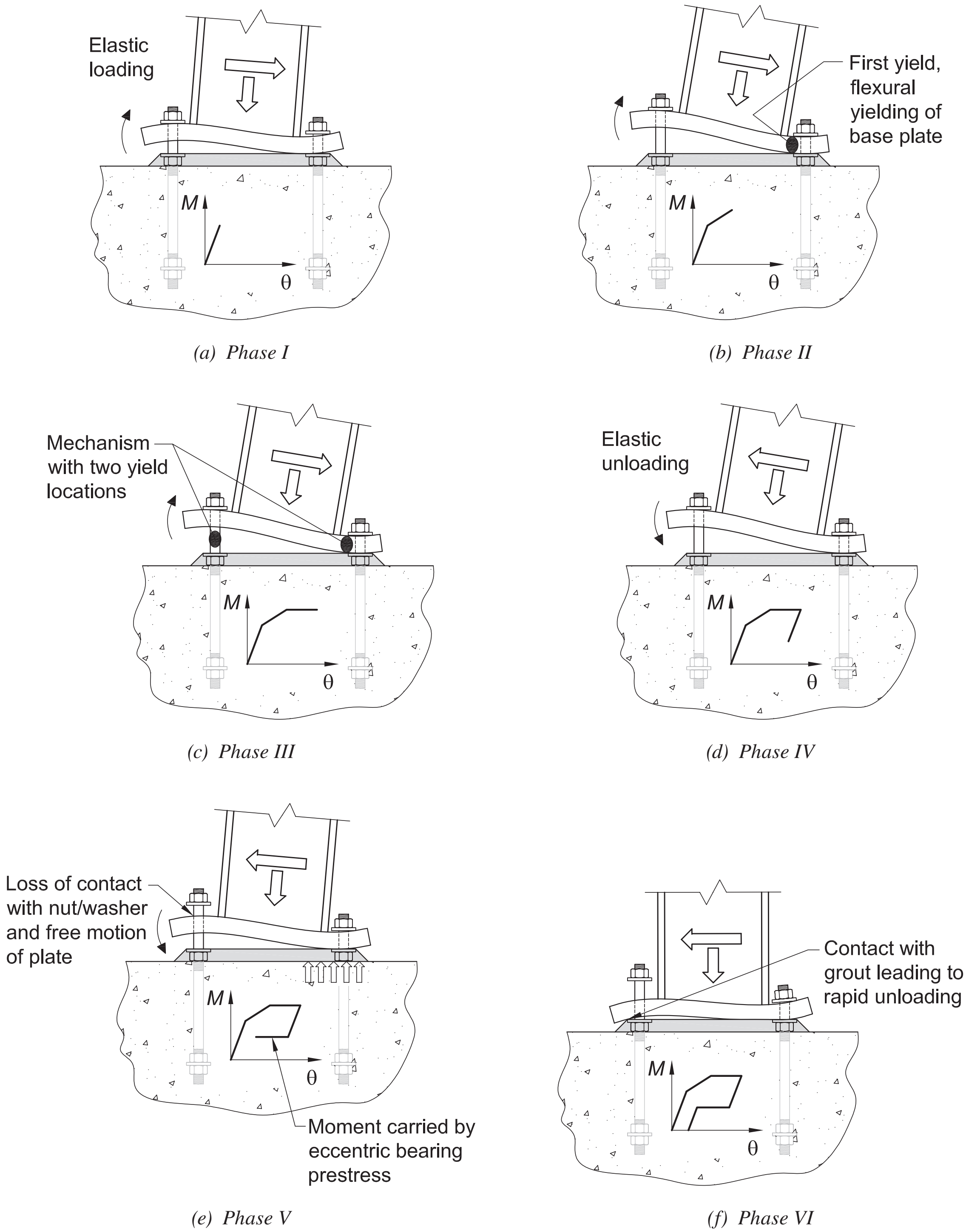


Fig. C-5. Physical phenomena controlling response.

have a recentering effect, such that the residual (or permanent) base rotation at zero moment is fairly low compared to the peak rotation.

Similar phases are observed in the reverse loading direction (and subsequent loading excursions), generating the hysteretic response shown in Figure C-4, which also illustrates that the strength, stiffness, and other aspects of response (such as the intermediate plateau corresponding to Phase IV) show deterioration.

C.3.2 Simulating Base Connection Hysteretic Response

Based on foregoing discussion, the most convenient way to simulate base connection hysteretic response is through representation as a uniaxial rotational spring with appropriately calibrated properties. It is noted that different software programs parametrize the models in dissimilar ways, so the user should exercise discretion in these procedures. Examples of such calibrations for one specific software program (OpenSees; Mazzoni et al., 2007), and a particular model (known as the Ibarra Medina Krawinkler model; Ibarra et al., 2005) are provided in Torres-Rodas et al. (2016), and Falborski et al. (2020b) provide guidance on parameter selection.

Appendix D
Guidance for the Use of Finite Element Analysis for Base Plate Analysis and Design, Focused on Exposed Column Base Connection Details

D.1 CONTEXT AND MOTIVATION

This appendix, focused on exposed base plate connections, is motivated by the increasing accessibility of finite element (FE) simulation software and computational capabilities in professional practice. Availability of these resources provides the opportunity to address situations that are either outside the scope of the approaches presented in this Design Guide or for which the approaches presented in the Design Guide may be challenging to generalize, because they are developed with the objective of simplified analysis and design. In such cases, using FE simulations may improve the accuracy in estimation of internal force distributions in the connection, possibly reducing conservatisms implied by the approaches presented in this Guide. It is important to distinguish here the scope of Appendix D, which focuses on determining forces within the connection with the objective of supporting connection design, as compared to Appendix C, which focuses on appropriate representation of the connection in frame simulations, with the objective of supporting frame design.

The base connection design approaches presented in Chapter 4 are simplified ones and do not address some effects and configurations. Some examples of these are (1) the effect of base plate flexibility on internal force distribution; (2) response under high-eccentricity situations, but with low magnitude moments, in which response of the footing is primarily elastic—whereas the Chapter 4 approach assumes the footing to reach its capacity in these cases; (3) base plates with circular or nonconventional column shapes; (4) base plates subjected to biaxial bending and axial force; and (5) base plates with multiple rows of anchors (greater than 2) in the direction of bending.

It is expected that the user of this appendix will have familiarity with FE simulation concepts, as well as proficiency with FE simulation software. With this assumption, this appendix provides basic, general guidelines and best practices for development of FE models. These guidelines are software and platform independent.

D.2 PROBLEM SCOPE AND STATEMENT

The problem statement involves an exposed-type base connection subjected to arbitrary loads, specifically the applied axial, moment, and shear forces. The objective is to determine one or more of the following: (1) internal force distribution that will support design of components within the base connection—specifically anchor rod forces, base plate internal moments, and concrete stresses—and (2) load deformation response, either to determine the ultimate strength of the entire connection or to represent the nonlinear load-deformation response in frame simulations. The emphasis in this appendix is on the first. This recognizes that the latter involves nonlinear simulation and representation of material and nonlinear effects and usually is used within specialized contexts (e.g., seismic design with weak bases), requiring significant resources for modeling as well as interpretation of results. For examples of such analysis, the reader is referred to Kanvinde et al. (2013).

Figure D-1 shows an example problem, wherein an exposed base plate connection is subjected to axial compression, a uniaxial moment, and shear. Once defined in this way, the aim of the FE simulations is to determine the following:

1. Anchor rod forces in each of the anchors.
2. Bending moments in the base plate. It is important to note here that given the three-dimensional nature of the problem and plate flexibility (e.g., due to out-of-plane bending of the base plate), the bending moments in the plate occur about both axes of bending and vary spatially. For example, it is possible to define only the bending moment per unit width at a particular location and in a particular direction.
3. Bearing stress distribution under the footing to check the footing capacity.

D.3 MODEL CONSTRUCTS

In the context of column base connection simulation, two variants of finite element simulation have been used and successfully validated against test data:

1. Conventional continuum finite element (CFE) simulation, in which each component of the base connection is simulated with its geometry and material constitutive response. This may be implemented in common commercial software. The theoretical and numerical basis for CFE simulation is well-established and is the subject of textbooks.

2. Component based finite element method (CBFEM), in which some components (e.g., the base plate and columns) are represented as continua (similar to the conventional CFE), but some other parts of the connection (e.g., anchors and the foundation) are represented through equivalent springs whose properties are calibrated based on various configurational parameters. This approach has been developed by the Wald research group and is implemented in selected commercial software (Sabatka et al., 2014).

This appendix focuses on the former—that is, conventional CFE—given that the latter requires specialized software and pre-calibrated models for the subcomponents.

D.4 GEOMETRY, BOUNDARY CONDITIONS, AND CONTACT/INTERACTIONS

Geometrical representation of the various components is important to appropriately represent their physical properties while mitigating edge or boundary effects. This section addresses basic considerations in simulating the geometry of components within base connections. Figures D-2(a) and (b) show a full and exploded view of the basic geometrical characteristics of a base connection CFE simulation model. Referring to this figure, the following suggestions are made regarding representation of geometry, boundary conditions, and contact/interactions.

D.4.1 Representation of Geometry of Components

The primary components of the base connection that should be modeled include (1) the column, (2) the base plate, (3) the grout pad, (4) the concrete footing, and (5) the anchor assemblages. If present, the shear lug should also be represented. Representation of the actual weld between the column and the base plate (or between the base plate and shear key) as a physical entity is not essential if the main aim is to size the base plate and anchor rods. Specific notes about the representation of each component are now provided:

1. *Column*. The column cross section should be simulated with the nominal dimensions. It is particularly critical to appropriately represent the overall depth of the column section, d, the flange width, b_f, and the thickness of the web and flange, t_f and t_w. However, the fillet radii and curved transitions between the web and flange are less critical. The primary consideration in simulating the length of the column is that the loading applied at the end of the column does not create boundary/edge effects that influence the stress distribution at the connection. It is recommended that the length of the column (above the surface of the base plate) be at least five times the depth of the column. The static equivalents of the applied loads at the connection (i.e. P, M, and V), may be applied at this location.

2. *Base plate*. The nominal dimensions (width, length, and thickness) of the base plate should be modeled. If oversized

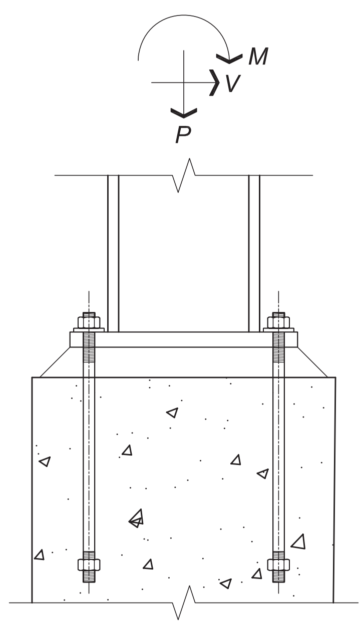


Fig. D-1. Basic geometry and loading.

holes with plate washers are used, then the holes should be modeled as oversized because plate washer flexibility may influence overall response.

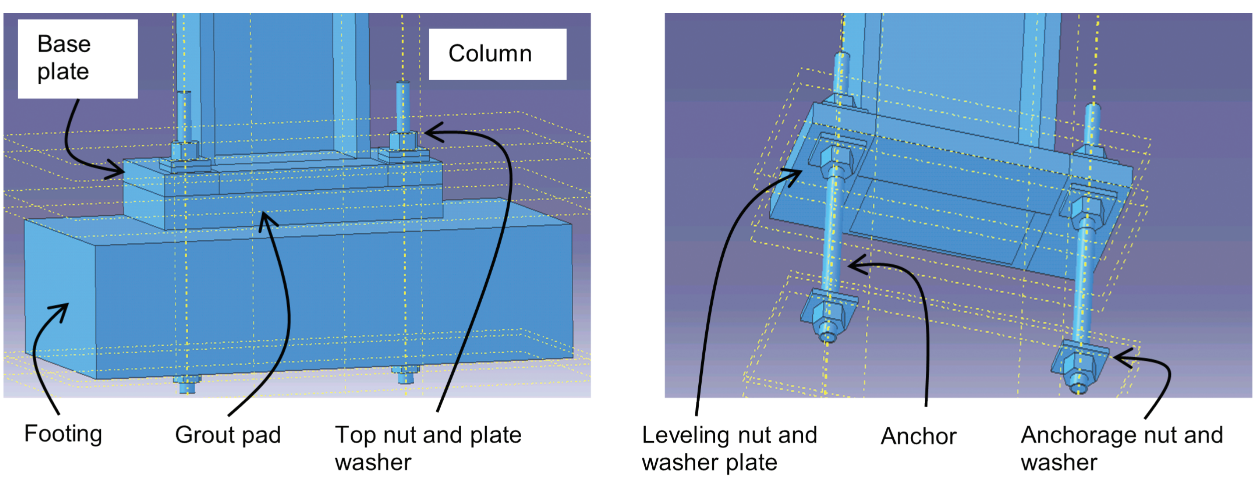
3. *Footing dimensions*. It is important to use the actual dimensions of the footing to represent the effects of elastic confinement. If a large mat foundation is present, then the footing should be modeled to a distance L extending out from each edge of the base plate, where L is at least $0.25 \times B$ and N (the plan dimensions of the base plate). The depth of the footing should be represented as per nominal dimensions.
4. *Grout pad*. The grout pad (if present) must be represented appropriately with nominal dimensions because it has a significant effect on the effective bearing stiffness of the footing under the base plate and influences the internal stress distribution.
5. *Anchor assemblies*. The anchor rods may be represented as cylinders, with the diameter equal to the pitch diameter, and the full length of the anchors. It is important to represent the full length of the anchor rod to appropriately represent their axial stiffness, which has an impact on the internal stress distribution. At the bottom, the anchor may either be attached to the footing through a constraint, or plate washers may be attached to the bottom of the anchor, which may then be attached to the footing. At the top of the anchor, it is important to simulate the plate washer assembly, especially if using oversized holes.

Computational cost may be greatly reduced by utilizing symmetry along the web-plane, especially if only uniaxial flexure of the connection is being studied; this is shown in Figure D-2.

D.4.2 Application of Boundary Conditions and Loads

All surfaces of the footing, except the top surface, should be restrained against motion in all directions. If symmetry is utilized, then appropriate symmetry boundary conditions should be applied over the symmetry plane. For example, in the model shown in Figure D-2 (where web symmetry is utilized), all out-of-plane (i.e., normal to the web) displacements should be restrained for all components intersecting the symmetry plane, while all in-plane displacements should be free. The applied loads (i.e., axial force, moment, and shear) should be applied at the top of the column segment, which is a minimum of 5 times the column depth as discussed previously. It is ideal to apply these as statically equivalent distributed tractions (or pressures) on the free (i.e., exposed) surface of the column to avoid localized effects due to point load application. However, this may become challenging for the application of moments due to the stress gradient at the cross section. This may be addressed in one of the following ways:

1. Selection of a column length such that the exposed (end) of the column is at the point of inflection so that only a shear force may be applied.
2. Superposition of an elastic (triangular) stress distribution consistent with the moment on the exposed end of the column.
3. The use of section constraints, if permitted by the software, that allow the application of concentrated loads or moments at a node, to which the sectional deformations are constrained.



(a) Overview *(b) Exploded view*

Fig. D-2. Modeled geometry of a base connection.

Table D-1. Suggested Modeling of Interactions among Various Base Connection Components							
Component	**Column**	**Base Plate**	**Grout**	**Footing**	**Anchor Top**	**Anchor Bottom**	**Top Washer**
Column	NA	Tie	NI	NI	NI	NI	NI
Base plate		NA	Note 1	NA	NA	NA	Note 2
Grout			NA	Tie	NI	NI	NI
Footing				NA	NI	Tie	NI
Anchor top					NA	NI	Tie
Anchor bottom						NA	NI
Top washer							NA

Notes:
1. A contact interaction should be specified between the base plate and the grout. The normal (i.e., perpendicular to the interface) properties of the contact should be specified as a hard contact, whereas the tangential properties should be specified as frictional with friction coefficient, μ = 0.45.
2. If welded plate washers are provided (e.g., to carry shear), then the faying surfaces of the top plate washer and the base plate should be provided with a tie constraint. If the plate washers are not welded, then contact properties should be assigned, with hard contact in the normal direction, and frictional contact in the tangential direction, with appropriate frictional coefficient based on the condition of the steel surfaces.
3. NI denotes no interaction between components.
4. NA denotes not applicable.

D.4.3 Interactions between Various Components

Different components may interact with each other in various ways at the surfaces or interfaces they share (e.g., full deformation compatibility or "tie" constraints), or contact with tangential and normal properties (e.g., friction or hard contact). Table D-1 and the associated footnotes provides some recommendations for how to simulate these interactions within base connections, noting that alternate ways of representing these interactions may also be possible. The table assumes that (1) a grout pad is present; if this is not the case, then the top surface of the footing should be treated as the grout pad, and (2) a shear key is not present.

As a further note, the interactions between the footing and anchor (the outer surface of the anchor) or the grout and the anchor are less important and have only a mild effect on internal stress distribution. On the other hand, modeling them accurately—for example, with bond-slip, frictional, or contact properties—is significantly time consuming and requires the calibration of additional properties. However, it is shown in experiments that in most cases (with headed anchors), the tension anchors separate from the footing due to Poisson contraction and then effectively respond as axial members in tension and show minimal interaction with the surrounding footing, especially for deformations consistent with design loads.

D.5 FINITE ELEMENT TYPES AND MATERIAL PROPERTIES

The following considerations are important from the standpoint of meshing and element selection:

1. *Convenience and ease of mesh generation*. This is particularly important for oddly shaped objects and components, including anchors, end assemblies, or plate washers with holes. Tetrahedral elements usually are the most facile from this standpoint, whereas hexahedral (brick) elements are more challenging. However, the former needs to be used with care, due to possible inaccuracy in simulation results.
2. *Element formulation*. Elements that are geometrically similar (e.g., tetrahedral) may use different formulations in terms of interpolation functions as well as special characteristics such as reduced integration to mitigate various forms of element locking.
3. *Mesh size*. This is important from the standpoint of mesh convergence—that is, accuracy of solution.

In terms of general element selection considerations, it is recommended to use hexahedral brick elements with quadratic or linear interpolation to the extent possible. In some cases, this may become unfeasible, either due to meshing difficulty or computational expenses. In these cases, alternate elements may be used—for example, tetrahedral elements for solid components and shell or plate elements for the base plate or web and flanges of the column. However, if these alternate elements are used, the following considerations are important:

- In the case of tetrahedral elements, quadratic interpolation is greatly preferred. These are commonly available as 10-node tetrahedral in commercial software.
- In the case of using shell or plate elements, it is important to note that the element thickness (which is zero) does not represent the true thickness of the component (e.g., plate) being represented. This may create issues, especially in the context of contact.

Note that these considerations are applicable in the context of linear elastic analysis; additional issues (e.g., volumetric locking) may become important when inelastic analysis is conducted. This usually raises a range of other considerations that are outside the scope of this appendix.

The mesh size in each component should be refined to achieve convergence to a desired degree—in other words, further refinement of the mesh should not result in an unacceptable change in outputs of interest (e.g., anchor forces). Some guidelines for initial mesh selection include the following (Figure D-3 shows a sample mesh):

1. *Base plate*. A minimum of 10 elements along the width and the length of the base plate (i.e., the plan dimensions) and at least 2 elements through the thickness (if using solid elements).
2. *Column and beam flanges.* A minimum of 10 elements along the width of the flange or the height of the web should be provided. In the longitudinal direction of the column, the mesh should be refined in the vicinity of the base plate; providing roughly square elements—but it may be coarsened away from the base plate.
3. *Anchor rods*. A minimum of 4 elements should be provided in the cross section of the rod. The length of each element along the rod should be on the order of the rod diameter.
4. *Grout and footing*. Element sizes in the grout and footing should be on the order of element sizes in the base plate.

An initial mesh may be selected based on the preceding guidelines. Then, convergence studies should be conducted with a reduced mesh size (e.g., with element sizes smaller than those selected in the preceding) to ensure that the results are within a convergence tolerance acceptable to the user.

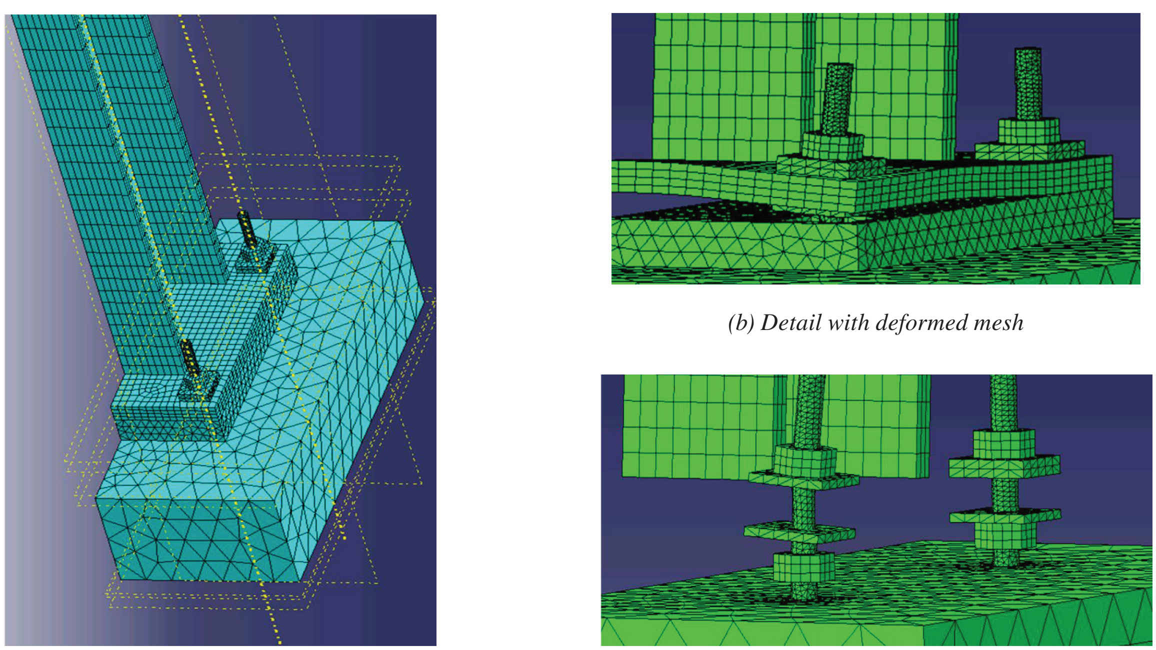
Elastic material properties should be specified for all the materials. These may be selected as follows:

1. For all steel elements (column, base plate, anchor rods, and washers):
   - Modulus of elasticity, E_{steel} = 29,000 ksi
   - Poisson's ratio, $\nu = 0.3$
2. For the footing concrete and grout:
   - $E_{concrete} = 57{,}000\sqrt{f_c'}$, where both f_c' and $E_{concrete}$ are in psi units.

D.6 VERIFICATION OF RESULTS

Once the FE simulations have been conducted, it is important to verify this process against benchmark experimental data. Two types of checks are recommended:

1. Agreement between the load deformation curve in the elastic regime of loading. If response over the inelastic



(a) Overall model with undeformed mesh

(b) Detail with deformed mesh

(c) Detail showing anchor rods and mesh

Fig. D-3. Finite element model illustrating mesh.

regime is intended to be simulated, then agreement over the full load-deformation curve is important.

2. Agreement between the relationship between the applied load and response quantity of interest (e.g., anchor rod force) in the elastic phase of loading.

Detailed data and metadata for a set of benchmark experiments is provided on the AISC website at www.aisc.org/dg1.

Additional resources and case studies (including comparison between simulated and tested specimens) may be found in Kanvinde et al. (2013) and Hassan et al. (2022).

D.7 INTERPRETATION OF RESULTS

The finite element simulations may be interpreted as general surrogates for the rectangular or triangular stress block methods—that is, to determine internal force distributions within the connections, given the applied loads. The finite element simulations usually return or compute point-wise stresses (i.e., stresses at each continuum location). Interpretation of these stresses needs additional consideration. This is now briefly discussed:

1. For anchor rods, the stresses in the longitudinal direction are of interest. However, the anchor rods may show a slight degree of bending deformations and, consequently, nonuniform stresses through the cross section. It is recommended to use the average longitudinal stress through the cross section of the anchor to compute a rod force and then compare to the capacity.
2. For the base plates, the following considerations are important.
   a. If the simulations report only stresses, then the in-plane stresses (in the plane of the base plate) should be noted. These stresses may be used to determine the moment (through integration) at any cross section of the plate, which may then be compared to the moment capacity of the plate at that cross section. It is recommended to not simply compare the stresses to the yield stress because this may result in conservatism, given that the plate flexural capacity corresponds to the plastic moment, not the yield moment.
   b. The critical orientation of plate flexure may not be aligned with the major and minor axis direction. Consequently, it is useful to examine the principal in-plane stresses to determine the critical orientation.
   c. If the simulations report a cross-sectional bending moment (less common in the case of continuum elements but the default in the case of shell elements), then the cross-sectional moment may be directly compared to flexural capacity.
3. There may be locations, especially in the concrete near the corners of the base plate, or even within the base plate, where the stresses are extremely high due to the presence of reentrant corners or other sharp discontinuities in the geometry. It is important to note that in reality, these stresses will be reduced through a combination of (a) local crushing or yielding and/or (b) finite radii present at these features. As a result, it may be acceptable to cap these stresses (for the purposes of comparison to capacities) at the crushing strength of the footing or ultimate strength of the steel, as applicable.

References

ACI (2008), *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08, American Concrete Institute, Farmington Hills, Mich.

ACI (2010), *Specification for Tolerances for Concrete Construction and Materials and Commentary*, ACI 117-10, American Concrete Institute, Farmington Hills, Mich.

ACI (2013), *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, ACI 349-13, American Concrete Institute, Farmington Hills, Mich.

ACI (2020), *Specifications for Structural Concrete*, ACI 301-20, American Concrete Institute, Farmington Hills, Mich.

ACI (2022), *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-19(22), American Concrete Institute, Farmington Hills, Mich.

AISC (1997), *Hollow Structural Sections Connections Manual*, American Institute of Steel Construction, Chicago, Ill.

AISC (2005), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-05, American Institute of Steel Construction, Chicago, Ill.

AISC (2016), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16, American Institute of Steel Construction, Chicago, Ill.

AISC (2018), *Seismic Design Manual*, 3rd Ed., American Institute of Steel Construction, Chicago, Ill.

AISC (2021), *Specification for Structural Stainless Steel Buildings*, ANSI/AISC 370-21, American Institute of Steel Construction, Chicago, Ill.

AISC (2022a), *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2022b), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2022c), *Specification for Structural Steel Buildings*, ANSI/AISC 360-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2022d), *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, ANSI/AISC 358-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2023), *Steel Construction Manual*, 16th Ed., American Institute of Steel Construction, Chicago, Ill.

ASCE (2013), *Anchorage Design for Petrochemical Facilities*, Task Committee on Anchorage Design, American Society of Civil Engineers, Reston, Va.

ASCE (2022), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures,* ASCE/SEI 7-22, American Society of Civil Engineers, Reston, Va.

ASME (2020), *Unified In. Screw Threads (UN, UNR, and UNJ Thread Forms*), B1.1-2019, American Society of Mechanical Engineers, New York, N.Y.

ASME (2022), *Nuts for General Applications: Machine Screw Nuts; and Hex, Square, Hex Flange, and Coupling Nuts (In. Series)*, B18.2.2-2022, American Society of Mechanical Engineers, New York, N.Y.

Astaneh-Asl, A. (2008), *Seismic Behavior and Design of Base Plates in Braced Frames*, SteelTIPS, Structural Steel Educational Council.

ASTM (2015), *Standard Specification for Zinc Coating, Hot-Dip, Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts, and Special Threaded Fasteners*, ASTM F2329/F2329M-15, ASTM International, West Conshohocken, Pa.

ASTM (2017), *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*, ASTM A354-17e2, ASTM International, West Conshohocken, Pa.

ASTM (2019a), *Standard Specification for Carbon Structural Steel*, ASTM A36/A36M-19, ASTM International, West Conshohocken, Pa.

ASTM (2019b), *Standard Specification for Hardened Steel Washers In. and Metric Dimensions*, ASTM F436/F436M-19, ASTM International, West Conshohocken, Pa.

ASTM (2019c), *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*, ASTM A588/A588M-19, ASTM International, West Conshohocken, Pa.

ASTM (2019d), *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*, ASTM F844-19, ASTM International, West Conshohocken, Pa.

ASTM (2020a), *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*, ASTM F1554-20, ASTM International, West Conshohocken, Pa.

ASTM (2020b), *Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*, ASTM A449-14(2020), ASTM International, West Conshohocken, Pa.

ASTM (2021a), *Standard Specification for Carbon and Alloy Steel Nuts (In. and Metric)*, ASTM A563/A563M-21ae1, ASTM International, West Conshohocken, Pa.

ASTM (2021b), *Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60 000 PSI Tensile Strength*, ASTM A307-21, ASTM International, West Conshohocken, Pa.

ASTM (2021c), *Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel*, ASTM B695-21, ASTM International, West Conshohocken, Pa.

ASTM (2021d), *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*, ASTM A572/A572M-21e1, ASTM International, West Conshohocken, Pa.

ASTM (2022a), *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*, ASTM A615/A615M-22, ASTM International, West Conshohocken, Pa.

ASTM (2022b), *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*, ASTM A706/A706M-22a, ASTM International, West Conshohocken, Pa.

ASTM (2022c), *Standard Specification for Structural Steel Shapes*, ASTM A992/A992M-22, ASTM International, West Conshohocken, Pa.

Aviram, A., Stojadinovic, B., and Kiureghian, A. Der. (2010), "Performance and Reliability of Exposed Column Base Plate Connections for Steel Moment-Resisting Frames," PEER Report 2010/107, Pacific Earthquake Engineering Research Center, Berkeley, Calif.

AWS (2020), *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2020, American Welding Society, Miami, Fla.

AWS (2021), *Structural Welding Code—Seismic Supplement*, AWS D1.8/D1.8M:2021, American Welding Society, Miami, Fla.

Barnwell, N. (2015), "Experimental Testing of Shallow Embedded Connections between Steel Columns and Concrete Footings," M.S. Thesis, Brigham Young University, Provo, Utah.

Choi, J-H. and Ohi, K. (2005), "Evaluation on Interaction Surface of Plastic Resistance for Exposed Type Steel Column Based under Biaxial Bending," *Journal of Mechanical Science and Technology*, Vol. 19, No. 3, pp. 826–835.

Choi, J-H. and Choi, Y. (2013), "An Experimental Study on Inelastic Behavior for Exposed-Type Steel Column Bases under Three-Dimensional Loadings," *Journal of Mechanical Science and Technology*, Vol. 27, No. 3, pp. 747–759.

Christensen, H. (2010), "Determining the Validity of Design Provisions for HSS to Base Plate Connections with Corner Anchor Rods Subjected to Axial Tension," M.S. Thesis, Milwaukee School of Engineering, Milwaukee, Wisc.

Cozzens, P.A., Rassati, G.A., and Swanson, J.A. (2021), "Pull-Through Testing of Plate Washers for Column Anchor Rod Applications," Technical Report, AISC, Chicago, Ill.

CSI (2020), *MasterFormat 2020 Edition*, Division 3, The Construction Specifications Institute, Alexandria, Va.

Cui, Y., Nagae, T., and Nakashima, M. (2009), "Hysteretic Behavior and Strength Capacity of Shallowly Embedded Steel Column Bases," *Journal of Structural Engineering*, ASCE, Vol. 135, No. 10, pp. 1,231–1,238.

Da Silva Seco, L. (2019), "Column Base Plates under 3D Loading," Ph.D. Dissertation, Universite Bretagne Loire, Rennes, France.

Denavit, M. (2022), Private communications.

DeWolf, J.T. and Ricker, D.T. (1990), *Column Base Plates*, Design Guide 1, 1st Ed., AISC, Chicago, Ill.

Dexter, R.J. and Ricker, M.J. (2002), "Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports," NCHRP Report 469, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.

Di Sarno, L., Pecce, M.R., and Fabbrocino, G. (2007), "Inelastic Response of Composite Steel and Concrete Base Column Connections," *Journal of Constructional Steel Research*, Elsevier, Vol. 63, No. 6, pp. 819–832. http://dx.doi.org/10.1016/j.jcsr.2006.08.007.

Doyle, J.M. and Fisher, J.M. (2005), "Discussion of Beam-Column Base Plate Design—LRFD Method," *Engineering Journal*, AISC, Vol. 42, No. 1, pp. 273–274.

Drake, R.M. and Elkin, S.J. (1999), "Beam-Column Base Plate Design—LRFD Method," *Engineering Journal*, AISC, Vol. 36, No. 1, pp. 29–38.

Dumas M., Beaulieu, D., and Picard, A. (2006), "Characterization Equations for Steel Column Base Connections," *Canadian Journal of Civil Engineering*, Vol. 33, No. 4, pp. 409–420.

Eatherton, M.R. and Murray, T.M. (2023), *End-Plate Moment Connections*, Design Guide 39, AISC, Chicago, Ill.

Falborski, T., Hassan, A.S., and Kanvinde, A.M. (2020a), "Column Base Fixity in Steel Moment Frames: Observations from Instrumented Buildings," *Journal of Constructional Steel Research*, Elsevier, Vol. 168, pp. 105993–1–105993-13. https://doi.org/10.1016/j.jcsr.2020.105993.

Falborski, T. and Kanvinde, A.M. (2022), "Estimation of Nonstructural Stiffness in Instrumented Steel Frames," *Engineering Structures*, Elsevier, Vol. 255, pp. 113947–1–113947-11. 10.1016/j.engstruct.2022.113947.

Falborski, T., Torres-Rodas, P., Zareian, F., and Kanvinde, A.M. (2020b), "The Effect of Base Connection Strength and Ductility on the Seismic Performance of Steel Moment Resisting Frames," *Journal of Structural Engineering*, ASCE, Vol. 146, No. 5, pp. 04020054–1–04020054-15.

Fasaee, M.A.K., Banan, M.R., and Ghazizadeh, S. (2018), "Capacity of Exposed Column Base Connections Subjected to Uniaxial and Biaxial Bending Moments," *Journal of Constructional Steel Research*, Elsevier, Vol. 148, pp. 361–370.

FEMA (2009), *Quantification of Building Seismic Performance Factors*, FEMA P-695, Federal Emergency Management Agency, Washington, D.C.

Fisher, J.M. (1981), "Structural Details in Industrial Buildings," *Engineering Journal*, AISC, Vol. 18, No. 3, pp. 83–89.

Fisher, J.M. (2019), *Industrial Building Design*, Design Guide 7, 3rd Ed., AISC, Chicago, Ill.

Fisher, J.M. and Kloiber, L.A. (2006), *Base Plate and Anchor Rod Design*, Design Guide 1, 2nd Ed., AISC, Chicago, Ill.

Frank, K.H. (1980), "Fatigue Strength of Anchor Bolts," *Journal of the Structural Division*, ASCE, Vol. 106, No. ST6, pp. 1,279–1,293.

Fuchs, W., Eligehausen, R., and Breen, J. (1995), "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," *ACI Structural Journal*, ACI, Vol. 92, No. 1, pp. 73–94.

Gomez, I., Deierlein G., and Kanvinde, A. (2010), "Exposed Column Base Connections Subjected to Axial Compression and Flexure," Final Report, AISC, Chicago, Ill.

Gomez, I.R., Kanvinde, A.M., and Deierlein, G.G. (2011), "Experimental Investigation of Shear Transfer in Exposed Column Base Connections," *Engineering Journal*, AISC, Vol. 48, No. 4, pp. 245–264.

Grauvilardell, J.E., Lee, D., Ajar, J.F., and Dexter, R.J. (2005), "Synthesis of Design, Testing and Analysis Research on Steel Column Base Plate Connections in High Seismic Zones," Structural Engineering Report No. ST-04-02, Department of Civil Engineering, University of Minnesota, Minneapolis, Minn.

Gresnigt, N., Romejin, A., Wald, F., and Steenhuis, M. (2008), "Base Plate in Bending and Anchor Bolts in Tension," *HERON*, Vol. 53, No. 1/2.

Grilli, D.A., Jones, R., and Kanvinde, A.M. (2017), "Seismic Performance of Embedded Column Base Connections Subjected to Axial and Lateral Loads," *Journal of Structural Engineering*, ASCE, Vol. 143, No. 5, pp. 04017010–1–04017010-10.

Grilli, D.A. and Kanvinde, A.M. (2016), "Tensile Strength of Embedded Anchor Groups: Tests and Strength Models," *Engineering Journal*, AISC, Vol. 53, No. 2, pp. 87–97.

Grilli, D.A. and Kanvinde, A.M. (2017), "Embedded Column Base Connections Subjected to Seismic Loads: Strength Model," *Journal of Constructional Steel Research*, Elsevier, Vol. 129, pp. 240–249.

Haninger, E.R. and Tong, B.M. (2014), "Two-Way Bending of Base Plates under Uniaxial Moment Loading—Alternative Approach," *Engineering Journal*, AISC, Vol. 51, No. 4, pp. 229–236.

Hassan, A.S. and Kanvinde, A.M. (2023), "Seismic Performance of Embedded Column Base Connections with Attached Reinforcement: Tests and Strength Models," *Engineering Journal*, AISC, Vol. 60, No. 4.

Hassan, A.S., Song, B., Galasso, C., and Kanvinde, A.M. (2022), "Seismic Performance of Exposed Column-Base Plate Connections with Ductile Anchor Rods," *Journal of Structural Engineering*, ASCE, Vol. 148, No. 5, pp. 04022028–1–04022028-19. 10.1061/(ASCE)ST.1943-541X.0003298.

Hassan, A.S., Torres-Rodas, P., Giulietti, L., and Kanvinde, A.M. (2021), "Strength Characterization of Exposed Column Base Plates Subjected to Axial Force and Biaxial Bending," *Engineering Structures*, Elsevier, Vol. 237, pp. 112165–1–112165-13. https://doi.org/10.1016/j.engstruct.2021.112165.

Horn, D. (2011), *Design of Monopole Bases,* Technical Manual 1, Tower Numerics, Inc., Lexington, Mass.

Horová, K., Wald, F., and Sokol., Z. (2011), "Design of Circular Hollow Section Base Plates," EUROSTEEL 2011, Budapest, Hungary.

Ibarra, L.F., Medina, R.A., and Krawinkler, H. (2005), "Hysteretic Models That Incorporate Strength and Stiffness Deterioration," *Earthquake Engineering and Structural Dynamics*, Vol. 34, No. 12, pp. 1,489–1,511.

ICC (2009), *International Building Code*, International Code Council, Falls Church, Va.

ICC (2021), *International Building Code*, International Code Council, Falls Church, Va.

Idris, A. and Umar, A.I. (2007), "Reliability Analysis of Steel Column Base Plates," *Journal of Applied Sciences Research*, Vol. 3, No. 3, pp. 189–194.

Inamasu, H., de Castro e Sousa, A., and Lignos, D.G. (2022), "Development and Experimental Validation of Dissipative Embedded Column Base Connections for Enhanced Seismic Performance of Steel Moment-Resisting Frames," *Journal of Structural Engineering*, ASCE, Vol. 148, No. 3, pp. 04021280–1–04021280-18. https://doi.org/10.1061/(ASCE)ST.1943-541X.0003259.

Inamasu, H., Kanvinde, A.M., and Lignos, D.G. (2019), "Seismic Stability of Wide-Flange Steel Columns Interacting with Embedded Column Base Connections," *Journal of Structural Engineering*, ASCE, Vol. 145, No. 12, pp. 0401915–1–0401915-15.

Inamasu, H., Kanvinde, A.M., and Lignos, D.G. (2020), "Seismic Design of Non-Dissipative Embedded Column Base Connections," *Journal of Constructional Steel Research*, Elsevier, Vol. 177, pp. 106417–1–106417-16. https://doi.org/10.1016/j.jcsr.2020.106417.

Inamasu, H. and Lignos, D.G. (2022), "Finite Element Modeling and Behavior of Dissipative Embedded Column Base Connections under Cyclic Loading," *Journal of Constructional Steel Research*, Elsevier, Vol. 189, pp. 107063–1–107063-18. doi : 10.1016/j.jcsr.2021.107063.

Kanvinde, A.M., Grilli, D.A., and Zareian, F. (2012), "Rotational Stiffness of Exposed Column Base Connections: Experiments and Analytical Models," *Journal of Structural Engineering*, ASCE, Vol. 138, No. 5, pp. 549–560.

Kanvinde, A.M., Higgins, P., Cooke, R.J., Perez, J., and Higgins, J. (2014), "Column Base Connections for Hollow Steel Sections: Seismic Performance and Strength Models," *Journal of Structural Engineering*, ASCE, Vol. 141, No. 7, pp. 04014171–1–04014171-13. 10.1061/(ASCE)ST.1943-541X.0001136.

Kanvinde, A.M., Jordan, S.J., and Cooke, R.J. (2013), "Exposed Column Baseplate Connections in Moment Frames—Simulations and Behavioral Insights," *Journal of Constructional Steel Research*, Elsevier, Vol. 84, pp. 82–93.

Kaczinksi, M.R., Dexter, R.J., and Van Dien, J.P. (1996), "Fatigue-Resistant Design of Cantilevered Signal, Sign, and Light Supports," NCHRP Report 412, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.

Koenigs, M.T., Botros, T.A., Freytag, D., and Frank, K.H. (2003), "Fatigue Strength of Signal Mast Arm Connections," Report No. FHWA/TX-04/4178-2, University of Texas, Austin, Tex.

Labelle, J. (2016), "Steelwise: Strength and Engagement," *Modern Steel Construction*, September, AISC, Chicago, Ill.

Mazzoni, S., McKenna, F., Scott, M.H., and Fenves, G.L. (2007), "OpenSees Command Language Manual," Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkely, Calif.

Melchers, R.E. (1992), "Rotational Stiffness of Shallow Footings," *Computers and Geotechnics*, Elsevier, Vol. 13, No. 1, pp. 21–35.

Miller, D.K. (2017), *Welded Connections—A Primer for Engineers*, Design Guide 21, 2nd Ed., AISC, Chicago, Ill.

Muir, L, and Thornton, W.A. (2014), *Vertical Bracing Connections—Analysis and Design*, Design Guide 29, AISC, Chicago, Ill.

Myers A.T., Kanvinde, A.M., Deierlein, G.G., and Fell, B.V. (2009), "Effect of Weld Details on the Ductility of Steel Column Baseplate Connections," *Journal of Constructional Steel Research*, Elsevier, Vol. 65, No. 6, pp. 1,366–1,373.

NSBA (2022), *Uncoated Weathering Steel Reference Guide*, National Steel Bridge Alliance, AISC, Chicago, Ill.

OSHA (2001), *Safety and Health Regulations for Construction*, Standards–29 CFR 1926 Subpart R—Steel Erection, Occupational Safety and Health Administration, Washington, D.C.

OSHA (2020), *Safety and Health Regulations for Construction*, Standards–29 CFR 1926 Subpart R—Steel Erection, Occupational Safety and Health Administration, Washington, D.C.

Paris, P.C. and Erdogan, F. (1963), "A Critical Analysis of Crack Propagation Laws," *Journal of Basic Engineering*, Vol. 85, No. 4, pp. 528–533. doi:10.1115/1.3656900.

PCI (2017), *PCI Design Handbook,* 8th Ed., Precast/Prestressed Concrete Institute, Chicago, Ill.

Richards, P.W., Barnwell, N.V., Tryon, J.E., and Sadler, A.L. (2018), "Flexural Strength and Stiffness of Block-Out Connections for Steel Columns," *Engineering Structures*, Elsevier, Vol. 173, pp. 404–415.

Sabatka, L., Wald, F., Kabelac, J., Godrich, L., and Navratil, J. (2014), "Component Based Finite Element Model of Structural Connections," *12th International Conference on Steel, Space, and Composite Structures*, May 28–30, Prague, Czech Republic.

Seco, L., Couchaux, M., Hijaj, M., and Costa-Neves, L. (2021), "Column Base Plates under Biaxial Bending Moment and Axial Force," *Engineering Structures*, Elsevier, Vol. 249, pp. 112893–1–112893-29.10.1016/j.engstruct.2021.112893.

Song, B., Galasso, C., and Kanvinde, A.M. (2020), "Reliability Analysis and Design Considerations for Exposed Column Base Plate Connections Subjected to Flexure and Axial Compression," *Journal of Structural Engineering*, ASCE, Vol. 147, No. 2, pp. 04020328–1–04020328-14. 10.1061/(ASCE)ST.1943-541X.0002903.

Soules, J.G., Bachman, R.E., and Silva, J.F. (2016), *Chile Earthquake of 2010: Assessment of Industrial Facilities around Concepción,* American Society of Civil Engineers, Reston, Va.

Steenhuis, M., Wald, F., Sokol, Z., and Stark, J. (2008), "Concrete in Compression and Base Plate in Bending," *HERON*, Vol. 53, No. 1/2, pp. 51–68.

Thornton, W.A. (1990), "Design of Small Base Plates for Wide-Flange Columns," *Engineering Journal*, AISC, Vol. 27, No. 3, pp. 108–110.

Till, R.D. and Lefke, N.A. (1994), *The Relationship between Torque, Tension, and Nut Rotation of Large Diameter Anchor Bolts*, Michigan Department of Transportation, Materials and Technology Division, Lansing, Mich.

Torres-Rodas, P., Fayaz, J., and Zareian, F. (2020), "Strength Resistance Factors for Seismic Design of Exposed Base Plate Connections in Special Steel Moment Resisting Frames," *Earthquake Spectra*, Earthquake Engineering Research Institute, Vol. 36, No. 2, pp. 537–553. https://doi.org/10.1177/8755293019891714.

Torres-Rodas, P., Zareian, F., and Kanvinde, A.M. (2016), "Hysteretic Model for Exposed Column Base Connections," *Journal of Structural Engineering*, ASCE, Vol. 142, No. 12, pp. 04016137–1–04016137-14.10.1061/(ASCE)ST.1943-541X.0001602.

Torres-Rodas, P., Zareian, F., and Kanvinde, A.M. (2017), "Rotational Stiffness of Deeply Embedded Column Base Connections," *Journal of Structural Engineering*, ASCE, Vol. 143, No. 8, pp. 04017064–1–04017064-11.

Torres-Rodas, P., Zareian, F., and Kanvinde, A.M. (2018a), "A Hysteretic Model for the Rotational Response of Embedded Column Base Connections," *Soil Dynamics and Earthquake Engineering*, Elsevier, Vol. 115, pp. 55–65.

Torres-Rodas, P., Zareian, F., and Kanvinde, A.M. (2018b), "Seismic Demands in Column Base Connections of Steel Moment Frames," *Earthquake Spectra*, Earthquake Engineering Research Institute, Vol. 34, No. 3, pp. 1,383–1,403. https://doi.org/10.1193/062317EQS127.

Trautner, C., Hutchinson, T., Copellini, M., Grosser, P., Bachman, R., and Silva, J. (2017a), "Developing Ductility Using Concrete Anchorage," *ACI Structural Journal*, ACI, Vol. 114, No. 1, pp. 101–112.

Trautner, C.A., Hutchinson, T., Grosser, P.R., and Silva, J.F. (2017b), "Investigation of Steel Column-Baseplate Connection Details Incorporating Ductile Anchors," *Journal of Structural Engineering,* ASCE, Vol. 143, No. 8, pp. 04017074–1–04017074-10. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001759.

Tryon, J.E. (2016), "Simple Models for Estimating the Rotational Stiffness of Steel Column to Footing Connections," M.S. Thesis, Brigham Young University, Provo, Utah.

Wald, F., Sokol, Z., and Jaspart, J-P. (2008a), "Base Plate in Bending and Anchor Bolts in Tension," *HERON*, Vol. 53, No. 1/2.

Wald, F., Sokol, Z., Steenhuis, M., and Jaspart, J. (2008b), "Component Method for Steel Column Bases," *HERON* Vol. 53, No. 1/2.

West, M.A. and Fisher, J.M. (2020), *Erection Bracing of Low-Rise Structural Steel Buildings*, Design Guide 10, 2nd Ed., AISC, Chicago, Ill.

Wilsmann, J. (2012), "Weld Behavior in a Rectangular HSS Base Plate Connection with Corner Anchor Rods Subjected to an Axial Tensile Force," M.S. Thesis, Milwaukee School of Engineering, Milwaukee, Wisc.

Worsfold, B. and Moehle, J. (2023), "Moment Transfer at Column-Foundation Connections: Analytical Studies," *ACI Structural Journal*, ACI, Vol. 120, No. 2, pp. 205–216.

Worsfold, B., Moehle, J.P., and Silva, J.F. (2022), "Moment Transfer at Column-Foundation Connections: Physical Tests," *ACI Structural Journal*, ACI, Vol. 119, No. 5, pp. 95–110.

Zareian, F. and Kanvinde, A.M. (2013), "Effect of Column Base Flexibility on the Seismic Safety of Steel Moment Resisting Frames," *Earthquake Spectra*, Earthquake Engineering Research Institute, Vol. 29, No. 4, pp. 1–23.

Smarter. Stronger. Steel.
American Institute of Steel Construction
312.670.2400 | www.aisc.org

D801-24