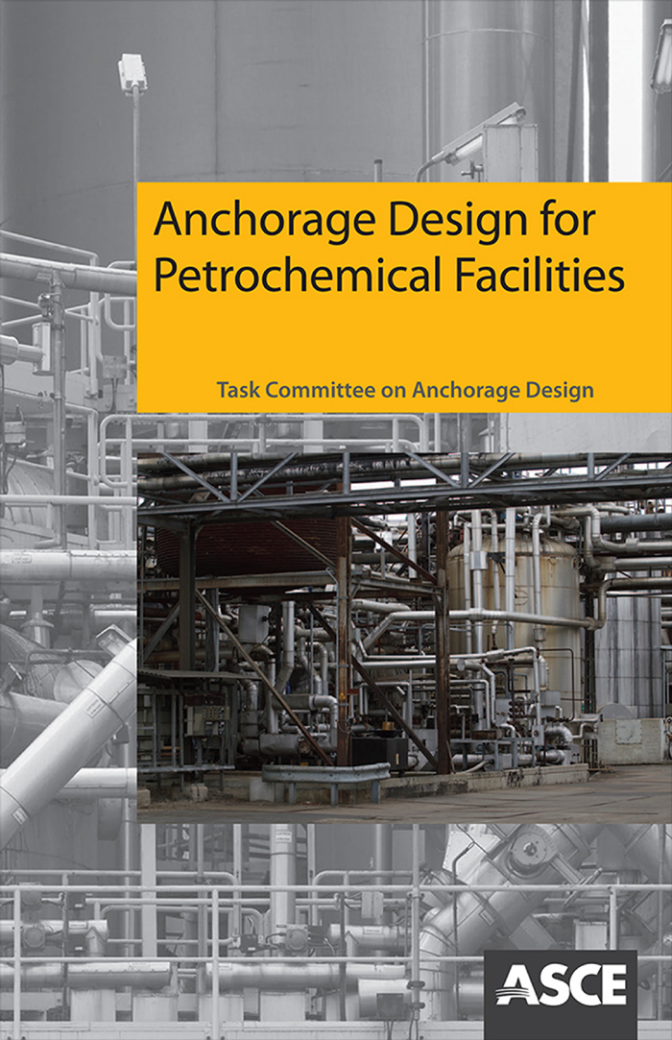




Anchorage Design for Petrochemical Facilities

Task Committee on Anchorage Design

ASCE

ANCHORAGE DESIGN FOR PETROCHEMICAL FACILITIES

PREPARED BY
Task Committee on Anchorage of the
Petrochemical Committee of the
Energy Division of the
American Society of Civil Engineers

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ASCE Petrochemical Energy Committee

This document is one of five state-of-the-practice engineering reports produced, to date, by the ASCE Petrochemical Energy Committee. These engineering reports are intended to be a summary of current engineering knowledge and design practice, and present guidelines for the design of petrochemical facilities. They represent a consensus opinion of task committee members active in their development. These five ASCE engineering reports are:

1. *Design of Blast-Resistant Buildings in Petrochemical Facilities*
2. *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities*
3. *Wind Loads for Petrochemical and Other Industrial Facilities*
4. *Anchorage Design for Petrochemical Facilities*
5. *Design of Secondary Containment in Petrochemical Facilities*

The ASCE Petrochemical Energy Committee was organized by A. K. Gupta in 1991 and initially chaired by Curley Turner. Under their leadership the five task committees were formed. More recently, the Committee has been chaired by Joseph A. Bohinsky and Frank J. Hsiu. The five reports were initially published in 1997.

Building codes and standards have changed significantly since the publication of these five reports, specifically in the calculation of wind and seismic loads and analysis procedures for anchorage design. Additionally, new research in these areas and in blast resistant design has provided opportunities for improvement of the recommended guidelines. The ASCE has determined the need to update four of the original reports and publish new editions based on the latest research and for consistency with current building codes and standards.

The ASCE Petrochemical Energy Committee was reorganized by Magdy H. Hanna in 2005, and the following four task committees were formed to update their respective reports:

- Task Committee on Anchorage for Petrochemical Facilities
- Task Committee on Blast Design for Petrochemical Facilities
- Task Committee on Seismic Evaluation and Design for Petrochemical Facilities
- Task Committee for Wind Load Design for Petrochemical Facilities

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James R. (Bob) Bailey, PhD, PE	Exponent, Inc.—Wind Committee Chairman
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The ASCE Task Committee on Anchorage Design

This updated document was prepared to evaluate the impacts of published reference data, research development and code changes that have occurred since creation of the 1997 report; and provide an updated report that will continue to serve as a source for uniformity in the design, fabrication and installation of anchorage in the petrochemical industry.

Although the makeup of the committee and the writing of this report are directed at petrochemical facility design, these guidelines are applicable to similar design situations in other industries. This report should interest engineers with responsibility for designing anchorage for equipment and structures, and operating company personnel responsible for establishing internal design, fabrication and construction practices.

This report is intended to be a State-of-the-Practice set of guidelines. The guidelines are based on published information and actual design practices. A review of current practices, internal company standards, and published documents was conducted. Also included is a list of references used by the Committee during creation of this report. The Committee acknowledges the work of Process Industry Practices (PIP) (<http://www.pip.org>) for providing much of the information used in this report.

In helping to create this consensus set of guidelines, the following individuals provided valuable assistance:

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Preface

The provisions of this document are written in permissive language and, as such, offer to the user a series of options or instructions, but do not prescribe a specific course of action. Significant judgment is left to the user of this document.

This document was initially prepared to provide guidance in the design, fabrication and installation of anchorage for petrochemical facilities and was issued in 1997 as *Design of Anchor Bolts in Petrochemical Facilities*. The task committee was reestablished in 2005 to update that document.

This document has been prepared in accordance with recognized engineering principles and should not be used without the user's competent knowledge for a given application. The publication of this document by ASCE is not intended to warrant that the information contained therein is suitable for any general or specific use, and ASCE takes no position respecting the validity of patent rights. The user is advised that the determination of patent rights or risk of infringement is entirely their own responsibility.

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Design of anchorages by most petrochemical engineering firms and owner companies uses an extrapolation, variation, or interpretation of the American Concrete Institute (ACI), the American Institute of Steel Construction (AISC), ASCE, and other technical documents as the basis for the design of anchorage systems for the petrochemical industry. This committee's work has been influenced by the continuing need to update the development of a uniform anchorage design methodology that is acceptable throughout the petrochemical industry.

1.2 OBJECTIVES AND SCOPE

The objective of this committee was to update the previous report, summarizing the State-of-the-Practice for the design of cast-in-place anchor rods, welded anchors, and post-installed anchors as used in petrochemical facilities.

The specific objectives were to:

- a. present petrochemical industry anchorage design methods for tension and shear transfer with reinforcement and other embedments;
- b. summarize anchorage materials and properties;
- c. present current practices for fabrication and installation of anchorage;
- d. present recommendations for post-installed anchors;
- e. make comprehensive recommendations for cast-in-place anchor design which are appropriate for use by the petrochemical industry;
- f. present recommended fabrication, constructability, and repair practices.

The committee recognized that while several different types of anchorage systems are used in petrochemical facilities, the most common types are cast-in-place anchors, welded anchors, post-installed anchors, and shear lugs. Therefore, for this report, the committee limited its investigation and recommendations to these common types. This self-imposed limit should not be construed as an attempt to limit the importance of other types of anchorage systems. Instead, this limit allowed the committee to focus attention on the most commonly used devices.

1.3 UPDATES AND ADDITIONS TO PREVIOUS REPORT

Chapter 2 includes a reorganization of Table 2.1, defining ASTM material specifications used for bolts and rods, with expanded notes relating to material welding and galvanizing. New sections have been added for washers and nuts, sleeves, fabrication – threading, headed studs, post-installed anchors, shear lugs, and performance of anchors exposed to extreme temperatures. The ASTM A307 Grade C anchor rod material is deleted and replaced with reference to ASTM F1554 Grade 36.

Chapter 3 has been rewritten for the state-of-the-art Concrete Capacity Design (CCD) Method based on ACI 318 and ACI 349 as applied to the current state of design practices in the petrochemical industry. New and revised sections have been created for anchor configuration and dimensions, strength and ductile design, anchor reinforcement design, frictional resistance, shear lug design, tensioning of anchors, design of welded anchors for embedded plates, and considerations for vibratory and seismic loads. Detailed examples are provided for a column pedestal with supplemental tension and shear reinforcement design, vertical vessel foundation anchorage design, and shear lug design.

Chapter 4 has been revised to include present design information for post-installed mechanical and bonded anchors, including typical installations; static, seismic, and fatigue design considerations; and post-installed qualifications. Anchor types addressed are those that would typically be considered for structural as well as safety-related nonstructural applications. Other light duty fastener types such as powder-actuated fasteners and small screws are not included in this discussion. For information regarding the correct design and installation of such fastener types, the user should refer to the appropriate evaluation reports provided by ICC-ES or other evaluation bodies. It is also advised that these types of light-duty fasteners not be used as single-point fastenings, but rather only in applications where the failure of one or more fasteners will not lead to progressive collapse.

Chapter 5 has been added to present installation and repair information, focusing on post-installed anchors, constructability, and repair procedures.

1.4 CODES AND DESIGN PROCEDURES

Changes in design methodology documented in the publications discussed below have resulted in changes to the formulas and methodologies presented in the original report, which was based on the 45-degree cone method. This report is based on the CCD Method, which assumes a critical spacing of three times the effective embedment depth. This assumption corresponds to a cone angle of approximately 35 degrees. In addition, the equation for basic concrete breakout strength accounts for the size effect associated with relatively high bearing stresses (and strain gradients) in the concrete. The following is a brief summary of the ACI Committee work relating to anchorage design.

ACI Committee 355 published the *State-of-the-Art Report on Anchorage to Concrete* in 1991. This was the first of a two-volume set which emphasized behavior and did not include design methods and procedures. In 2000, ACI Committee 355 published the ACI Provisional Standard, *Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-00) and Commentary (ACI 355.2R-00)*. This document prescribed testing programs and evaluation requirements for post-installed mechanical anchors intended for use in concrete under the design provisions of ACI 318/318R-02. It was designated an ACI Standard in 2001 and has since been updated twice, most recently in 2007.

ACI Committee 318 first approved the inclusion of Appendix D – Anchoring to Concrete in ACI 318/318R-02. It provided strength design requirements for anchorage to concrete that consider several potential failure modes such as steel strength, concrete breakout, anchor pullout, side-face blowout, and anchor pryout (shear) in accordance with the CCD Method. ACI 318-08 includes the following important enhancements to Appendix D:

- a. The requirements for the use of reinforcement to preclude concrete breakout are more clearly defined
- b. A non-ductile anchor option is included in the seismic design provisions
- c. A modification factor for concrete breakout strength is introduced to reduce the conservatism of the provisions for anchorages loaded in shear where the edge distance is large relative to the member thickness

ACI Committee 349 Appendix B introduced provisions for anchor design in 1976. In 1980, revisions to Appendix B based on the 45-degree cone method were proposed; they were incorporated in 1982. (Reference Cannon et al Preface [1981].) This approach involved the assumption of a conical failure surface originating from the outer edge of the bearing head and projecting at an angle of 45 degrees to the concrete surface. This assumption, combined with a calculation for equilibrium based on a uniform stress distribution of $4\sqrt{f'_c}$ over the failure surface, results in an equation for breakout that is proportional to the square of the embedment depth. In 2001, ACI Committee 349 adopted the CCD Method as Appendix B of ACI 349-01. In contrast to ACI 318/318R-02 Appendix D, however, Appendix B of ACI 349-01 included provisions for non-ductile anchors as well as the use of friction to resist shear, and design provisions for shear lugs.

In 2007, ACI Committee 349 published the *Guide to the Concrete Capacity Design (CCD) Method—Embedment Design Examples*. This report presents design examples of single and multiple embedded elements in concrete members based on Appendix D (formerly Appendix B) of ACI 349-06, which is based on the CCD Method. The 2007 edition of the Guide replaced the 1997 edition, which was based on ACI 349-97 and the 45-degree cone method for establishing concrete breakout resistance.

Note: This document was developed using codes that were in force in 2010. After the document was completed, and during the peer review process, ACI 318-11 was issued. This revision had changes in both the adhesion of adhesive anchors and the seismic provisions. The committee elected not to try to implement these changes into this document. Thus, engineers involved with adhesive anchors or the seismic design of anchors should review ACI 318-11 to ensure that they are complying with that code.

1.5 STATE OF RESEARCH

In 1995, Fuchs et al. published a code background paper in the ACI Structural Journal, *Concrete Capacity Design (CCD) Approach for Fastening to Concrete*. As described earlier, the CCD Method is the basis for the design of anchorages embodied in the current ACI 318 and ACI 349 codes and is based on the cone method developed at the University of Stuttgart. This method provides visual explanation for the factors used to account for geometry and loading effects in the prediction of concrete breakout strength. It combines the transparency of the 45-degree cone failure model with the improved accuracy of the cone method, especially for groups and near-edge anchorages, and includes a simple rectangular projected failure surface calculation procedure.

Until recently, test results were limited for anchors in the upper range of sizes and embedment depths commonly used in industrial facilities. The majority of embedment depths included in the international database used to verify the CCD Method are less than 7.87 in. (200 mm) with very few, if any, greater than 21.7 in. (550 mm). Most anchor sizes that had been tested were less than 2 in. (50.8 mm) in diameter, with a majority of the tests having been performed on anchors 1 in. (25 mm) or less in diameter. Klingner and Mendonca (1982a, b) present a literature review of tensile capacity and shear capacity of short anchors and welded studs. Eligehausen et al. (2006) provides a good overview of research in the field of fastening technique from around the world. An ACI technical paper, *Tensile-Headed Anchors with Large Diameter and Deep Embedment in Concrete*, published in 2007, presents test results for larger anchor rods with diameters ranging from 2.75 in. (70 mm) to 4.75 in. (120.7 mm) and embedment lengths ranging from 25 in. (635 mm) to 45 in. (1143 mm), with and without supplemental reinforcing. A companion paper, *Shear Behavior of Headed Anchors with Large Diameters and Deep Embedments*, appeared in the ACI Structural Journal in 2010. In addition, deeper embedments have been modeled using finite elements with advanced concrete modeling (microplane model) by Özbolt et al. (2007). From these studies it has become clear that:

- a. The current expression for concrete breakout in tension in ACI 318 is also applicable to larger embedments. It may also be the case that, for embedments beyond 25 in. (635 mm), the use of expressions for concrete breakout that do

not include size effect may be justified provided that the bearing stresses at failure are kept sufficiently low.

- b. The current expression for concrete edge breakout in shear in ACI 318 becomes unconservative for anchor diameters larger than 2 in. (50.8 mm), and that for such cases the use of appropriately proportioned and detailed hairpin reinforcement is warranted in lieu of dependence on the concrete breakout strength.

Lotze et al. (2001) and Gross et al. (2001) present results of a research program that was conducted to study the dynamic behavior of anchors in concrete under tension and shear, respectively.

The interaction of reinforcing in concrete members with anchors in tension and shear is highly dependent on the specific geometry and loading. For this reason, very little actual testing has been performed to establish the effect of reinforcing on anchor capacity in either shear or tension or both. Lee et al. (2010) included testing with hairpins and other reinforcing configurations in their investigation of large diameter anchors subjected to shear loading.

Extensive testing has been performed to identify edge distance and anchor spacing influences. Lee and Breen (1966) reported on results for 26 bolts and Hasselwander, Jirsa, Breen, and Lo (1977) published a report based on results for 35 bolts. Bailey and Burdette also published a report in 1977 entitled *Edge Effects on Anchorage to Concrete*. Furche and Eligehausen (1991) performed pullout tests with headed studs placed near a free edge and recommended an empirical equation for calculating the failure load in their paper titled *Lateral Blow-out Failure of Headed Studs Near a Free Edge*. ACI 349-01 includes extensive commentary comparing the 45-degree cone method and the CCD Method in this regard.

1.6 FUTURE RESEARCH

The following items should be considered for future research regarding anchorage for petrochemical facilities:

- a. Verify the Strut and Tie Method (STM) design procedure for anchor reinforcement ties for shear load transfer at or near the tops of pedestals and other foundation element locations
- b. Define the effective clear distance between the anchor head and the anchor for development of anchor reinforcement for tensile load transfer
- c. Confirm the side-face-blow-out failure mechanisms of reinforced concrete elements at the anchor head and provide recommendations for reinforcement details and locations relative to the anchor head

- d. Confirm the high-cycle fatigue effect on post-installed adhesive anchors
- e. Conduct testing for tension and uplift anchorage connectors to resist wind, seismic, other transient, and sustained tensile loads at the embedded interface to the top of a pile and into the concrete pile cap
- f. Confirm the effectiveness of corrugated anchor sleeves for increasing the interface stress or bond stress for the grout pocket and the relative location of the anchor head with respect to the center of the sleeve
- g. Confirm the relationship of the stretch length of the anchors to the corresponding inelastic energy deformation
- h. Perform tension load testing of cast-in place headed anchors with larger diameters and longer concrete embedment lengths than those for which test results are presently available
- i. Confirm the current industry practice and theory used to design anchor reinforcement for tensile load transfer and determine development lengths
- j. Identify the failure modes and capacities for concrete breakout strength in tension of anchors in octagonal pedestals

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CHAPTER 2 MATERIALS

2.1 INTRODUCTION

This chapter provides the basic materials, properties, and corrosion protection recommendations for bolt and rod assemblies, headed studs, post-installed anchors, and shear lugs. The engineer must select the proper material, considering properties such as grade, yield strength, tensile strength and weldability; and provide for corrosion resistance so that the anchorage will perform as required and intended.

2.2 BOLT AND ROD ASSEMBLIES

2.2.1 Bolts and Rods

Tables 2.1a & b list the ASTM specifications, yield strengths, ultimate strengths, and range of available diameters for materials commonly used for anchor bolts and studs, and threaded anchor rods, respectively. Unless the anchors are to be used in a special corrosive environment or are subjected to extreme low or high temperatures or other special conditions, the following specifications should be used:

- a. ASTM A307 grade A bolts, ASTM A36/A36M rods or ASTM F1554 Grade 36 rods for low strength requirements
- b. ASTM F1554 Grade 55 rods for moderate strength requirements. Grade 55 rods should be ordered with the weldability supplement
- c. ASTM F1554 Grade 105 rods for high-strength requirements

Note: ASTM F1554 is an anchor bolt manufacturing specification, not a material specification. Therefore, the anchor supplier may furnish any material which meets the ASTM F1554 specification. If conditions require that anchors meet more stringent requirements the engineer must include the special requirements in the purchase order language. An example would be for ASTM F1554 Gr 105 anchors greater than 2 in. (50.8 mm) having to meet the requirements of ACI 318 Appendix D for a ductile steel element.

Table 2.1a: Common Materials for Anchor Bolts and Studs

ASTM Specification	f _y , min, ksi (MPa)	f _{ut} , min, ksi (MPa)	Diameter Range, in. (mm)	Notes
A307 Grade A	Not specified by ASTM	60 (414)	1/4 (6.4) to 4 (102)	For general applications. Weldable if Supplementary Requirement S1 is specified in the purchase order.
A354 Gr BC	109 (752)	125 (862)	1/4 (6.4) to 2 1/2 (64)	
	99 (683)	115 (793)	over 2 1/2 (64) to 4 (102)	
A354 Gr BD	130 (896)	150 (1,034)	1/4 (6.4) to 2 1/2 (64)	Do not galvanize. Hydrogen-stress cracking or stress cracking corrosion may occur on hot-dip galvanized bolts.
	115(793)	140 (965)	over 2 1/2 (64) to 4 (102)	
A449	92 (634)	120 (827)	1/4 (6.4) to 1 (25)	
	81 (558)	105 (724)	over 1 (25) to 1 1/2 (38)	
	58 (400)	90 (621)	over 1 1/2 (38) to 3 (76)	

Table 2.1b: Common Materials for Threaded Anchor Rods

ASTM Specification	f_u , min, ksi (MPa)	f_{uta} , min, ksi (MPa)	Diameter Range, in. (mm)	Notes
A36/A36M	36 (250)	58 (400)	Not specified. Refer to ASTM F1554 Grade 36.	Weldable. ASTM F1554 Grade 36 is referenced in ASTM A36/A36M for anchor bolts.
A193/A193M Gr B7	105 (720)	125 (860)	to 2 1/2 (M64)	Can be galvanized, but it is normally neither required nor recommended. (Section 3.2 of ASTM A193/A193M prohibits coatings unless specified by the purchaser.)
	95 (655)	115 (795)	over 2 1/2 (M64) to 4 (M100)	
	75 (515)	100 (690)	over 4 (M100) to 7 (M180)	
A320/A320M Gr L7	105 (725)	125 (860)	to 2 1/2 (65)	For low temperature application
F1554 Gr 36	36 (248)	58 (400)	1/4 (6.4) to 4 (102)	Weldable
F1554 Gr 55	55 (380)	75 (517)	1/4 (6.4) to 4 (102)	Weldable with Specification's Supplementary Requirement S1
F1554 Gr 105	105 (724)	125 (862)	1/4 (6.4) to 2 (50)	
F1554 Gr 105	105 (724)	125 (862)	larger than 2 (50) to 3 (76)	See note to 2.2.1 for special order requirements

Notes for Tables 2.1a and 2.1b:

1. All materials meet ACI 318 Appendix D ductility requirements unless otherwise noted.
2. ASTM F1554 allows the substitution of weldable Gr 55 steel when Gr 36 is specified. If the engineer does not want this substitution, it must be specifically stated in the purchase order.
3. There are other rod materials that may be suitable for anchorage (for example stainless steel). The application may have special concerns for environmental exposure conditions. See 2.6 and consult with a material specialist for recommendations.
4. Metric equivalents shown in parentheses are those shown in the ASTM standard where provided, or by conversion where not provided. Metric equivalents designated with "M" for ASTM A193/A193M are those provided in the standard.

2.2.2 Washers

Washers are required for all anchors and should conform to the following requirements:

- a. Washers for all anchors other than ASTM A307 bolts should conform to ASTM F436/F436M, except that washers for ASTM F1554 rods shall conform to the requirements of ASTM F1554 Section 6.7
- b. Washers for ASTM A307 bolts may conform to ASTM F844
- c. Washers for high-strength anchors or anchors that are to be tensioned shall be hardened washers conforming to ASTM F436/F436M
- d. Anchors for base plates with hole diameters greater than 3/8 in. larger than the anchor diameter shall have fabricated ASTM A36/A36M washers in addition to the ASTM F436/F436M or F844 washers (See Table 3.3.)

2.2.3 Nuts

Nuts should conform to the following requirements:

- a. Nuts for all anchor bolts and rods other than ASTM A193/A193M, A320/A320M, and ASTM F1554 should conform to ASTM A563/A563M
- b. Nuts for ASTM A193/A193M and A320/A320M rods should conform to ASTM A194/A194M
- c. Nuts for ASTM F1554 rods should conform to either ASTM A194/A194M or ASTM A563/A563M

Note: It is not necessary to specify that zinc-coated nuts which are fabricated to ASTM A563/A563M be tapped oversize, since this requirement is addressed in the specification.

2.2.4 Sleeves

Sleeves are normally made of either of the following materials:

- a. Thin-walled pipe, which can be smooth for non-structural applications or corrugated, where an interlocking action is desired
- b. Polyethylene, either smooth or corrugated

A detailed discussion of anchor sleeves is presented in 3.2.3.

2.2.5 Fabrication

2.2.5.1 General

Flux, slag, and weld-splatter deposits should be removed before galvanizing because the normal pickling process does not remove slag. Toe cracking at weldments around anchor plates is undetectable prior to galvanizing and is easily detected after galvanizing. A post-galvanizing inspection should be considered to detect these cracks.

Materials which have been quenched and tempered should not be welded or hot-dip galvanized. High-strength materials should not be bent or welded since their strength and performance may be affected.

2.2.5.2 Threads

Threads of a mechanical fastener can be produced by cutting, rolling or grinding. Cutting and rolling are the most common. The differences, advantages, and disadvantages of these two types of threads are described below.

2.2.5.2.1 Cut Threads

Cut threads are produced by cutting away or otherwise physically removing steel from a round bar to form the threads.

a. Advantages of Cut Threads

1. Few limitations with regard to diameter and thread length
2. All specifications can be manufactured with cut threads

b. Disadvantages of Cut Threads

1. Significantly longer labor times to cut mean higher costs
2. Can result in stress concentration points

2.2.5.2.2 Rolled Threads

Rolled threads are produced by extruding steel to form the threaded portion of a fastener instead of removing it as in producing cut threads. In this process, a fastener is manufactured from a reduced diameter round bar. The fastener is “rolled” through a set of threading dies, which displaces the steel and forms the threads. The end result is a fastener with a full diameter threaded portion but a reduced body diameter. Producing rolled threads is an extremely efficient process and often results in significant cost savings.

a. Advantages of Rolled Threads

1. Significantly shorter labor times mean lower costs
2. Because a roll-threaded fastener has a smaller body diameter, it weighs less than its full bodied counterpart. This weight reduction reduces the cost of the steel, galvanizing, heat-treating, plating, freight, and any other costs associated with the fastener that are based on weight.
3. Cold working makes threads more resistant to damage during handling. In fact, cold working compresses the grain and increases the yield and tensile strengths, generally from 10 to 30 percent.
4. Rolled threads are often smoother because of the burnishing effect of the rolling operation.

b. Disadvantages of Rolled Threads

1. The availability of pitch diameter round bar is limited for certain material grades
2. Rolled threads cannot be used for anchors having a minimum tensile strength of 150 ksi (1,030 MPa) or greater

2.2.5.3 Upset Threads

Anchor rods with upset threads have a thread section diameter greater than the rod body diameter (Figure 2.1). Upset threads are provided to assure that yielding will occur outside the threaded portion of the anchor. These rods are normally furnished for shoring waler tie rods, bracing tie rods, rail anchor clips or other applications requiring strain length. The threads can be formed by either cutting or rolling. It is recommended that the specifier consult with the anchor supplier prior to specifying in order to verify availability and proper specification. Anchor rods with upset threads are not commonly used in petrochemical facilities.

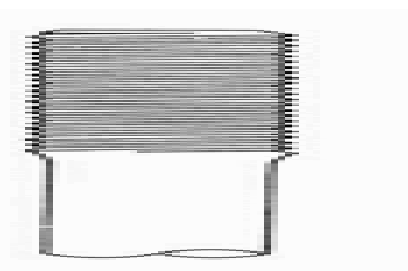


Figure 2.1: Anchor Rod with Upset Threads

2.2.5.4 Shot Peening

Shot peening, defined as shot blasting with small steel balls driven by a blast of air, is a method of removing defects for highly critical anchors, and is specifically recommended for use on anchors subjected to high-cycle fatigue. It is not deemed necessary for other applications.

2.3 HEADED STUDS

Headed studs are manufactured from low carbon steel in accordance with ASTM A108. They have a minimum yield strength of 50 ksi (345 MPa) and a minimum specified tensile strength of 60 ksi (414 MPa).

2.4 POST-INSTALLED ANCHORS

Post-installed anchors are manufactured in a variety of materials. A detailed discussion of post-installed anchors is presented in Chapter 4. The engineer should consult the manufacturer of proprietary systems for materials used, and select the most appropriate material for the intended use of the anchor and the environment in which it will be used.

2.5 SHEAR LUGS

A shear lug is a plate, hollow structural section (HSS), pipe, or wide flange structural shape welded perpendicular to the bottom of a base plate. Plates are manufactured of the same material as the base plate. HSS are normally manufactured in accordance with ASTM A500/A500M, while pipes are normally manufactured from ASTM A53/A53M Grade B material. Wide flange structural shapes are normally manufactured from ASTM A992/A992M material.

2.6 CORROSION

Several forms of corrosion are associated with anchors in concrete, including contact corrosion, crevice corrosion, pitting, and inter-crystalline stress corrosion. As with

reinforcing steel, the embedded portion of an anchor in concrete derives a certain level of protection from the alkalinity of the concrete, resulting in passivation of the steel surface. Over time, loss of alkalinity due to external environmental influences and the intrusion of chlorides will lead to a breakdown in the passivation layer. Typically, the most critical location for contact and crevice corrosion is the point where the anchor protrudes from the concrete and engages the fastened part. Pitting corrosion is particularly problematic for stainless steels, since failure can occur without warning and with little prior external visual evidence of corrosion products.

Corrosion-resistant materials used in the production of anchors include the following:

- a. Hot-dip galvanized carbon steel
- b. Austenitic (chromium-nickel) stainless steels (Type 304, 316)
- c. High-molybdenum stainless steel alloys
- d. Titanium

Corrosion protection may also take the form of a protective coating system or other methods to prevent contact of the anchor with the atmosphere.

For high-sulfur environments, use of galvanizing may be preferable to austenitic stainless steels such as Type 316 or Alloy 20 stainless steel because of the hazard of pitting corrosion. High-molybdenum stainless steels have been shown to be particularly resistant to long-term exposure in road tunnels, exhaust stacks, and similar environments. Titanium offers excellent resistance to corrosion but may be cost prohibitive.

Anchorage service life requires that corrosion protection be an important design consideration. Anchorage material or coating system selection should provide a reliable and high quality service life for an item that is relatively inaccessible for maintenance, repairs, or replacement due to corrosion. There are many factors and environmental exposure conditions that should be considered. The engineer may need to consult with material specialists about corrosion protection during the anchorage material selection process.

2.6.1 Environmental Conditions

It is recommended practice in the petrochemical industry to provide environmental corrosion protection with hot-dip galvanizing for all anchors in exterior applications. Other coating systems may be used, but they are not as common and may be more expensive.

The exposed portion of the anchor at the concrete interface and the embedded portion of the anchor are vulnerable to corrosion from infiltration of moisture, air, and other corrosive elements. Anchorage near waterways and seashores requires additional corrosion protection against wet-dry cycles and excessive salts. Deicing salts in

runoff from areas with snow and ice, sulfates or chlorides that may be present in the concrete or in the site soils can also be particularly corrosive to anchorage.

The hot-dip galvanizing of anchors, sealing of joints and concrete cracks that develop during initial construction, and regular maintenance will provide long-term protection benefits throughout the life of the anchor system.

Galvanized and stainless materials can fail when subjected to corrosive chemicals such as acids or industrial fumes. Such materials used in these applications require additional coating systems.

Anchorage located in controlled environments inside buildings may not require protection from atmospheric corrosion except for exposure to chemicals.

Bare, uncoated, weathering steels should not be used in petrochemical application where premature rusting due to coastal environments and high concentrations of corrosive chemicals or industrial fumes are present.

2.6.2 Codes and Specifications

2.6.2.1 American Concrete Institute (ACI)

Anchorage should be considered as an extension of the concrete, as noted in ACI 318. This requires that exposed reinforcement, inserts, and plates intended for bonding with future extensions be protected from corrosion.

ACI 318 requires that concrete, reinforcing, and anchor rods exposed to injurious amounts of oil, acids, alkalis, salts, organic materials, or other substances that may be deleterious, be protected from those substances.

The amount of soluble chloride ion content in concrete is controlled by ACI 318. See ACI 222R, *Protection of Metals in Concrete Against Corrosion*, for additional information.

When external sources of chlorides are present, anchors should be protected in a manner similar to that required for reinforcing bars, in accordance with ACI 318.

2.6.2.2 American Institute of Steel Construction (AISC)

Anchorage corrosion protection and material selection is outside the scope of AISC specifications. AISC Steel Design Guide 1, *Base Plate and Anchor Rod Design*, and Steel Design Guide 7, *Industrial Buildings – Roofs to Anchor Rods*, include information to assist in some of the practical aspects of design and application of anchor rods.

AISC recommends that anchor rods subjected to corrosive conditions be galvanized. If anchor rods are galvanized, it is best to specify ASTM A307, A36/A36M or F1554 grade 36 materials to avoid the embrittlement that sometimes results when high-strength steels are galvanized. See Table 2.1 for other materials and related notes.

2.6.2.3 American Petroleum Institute (API)

API Std 620 recommends using stainless steel anchorage materials or providing a corrosion allowance when using carbon steels.

API Std 650 states that if corrosion is a possibility, an increase in material thickness should be considered for anchorage. It is recommended that the nominal anchor diameter not be less than 1 in. (25 mm) and that a corrosion allowance of at least 1/4 in. (6 mm) increase in diameter be provided.

2.6.3 Corrosion Rates

There are substantial variations in corrosion rates even under relatively similar conditions. Corrosion rates in actual service can vary from those that are cited or determined by technical sources. During the design of material protection systems, materials and process engineers should be consulted to define the corrosive exposure conditions and what material or coating system is most suitable for providing protection to the anchorage.

If coating is not appropriate for corrosion protection, a corrosion allowance may be required when sizing the anchor. Minimum corrosion protection without galvanizing or other coating system would be a minimum 1/4 in. (6 mm) increase in the required design diameter for corrosion protection for the anchorage. However, the design engineer should understand that this is only a minimum and should evaluate the sufficiency of this corrosion allowance for the specific application.

2.6.4 Coatings

If anchor rods are in an area where the environment is particularly corrosive or abrasive, special coatings to exposed threads and nuts are required. Protective coatings may be preferable to increasing the anchor rod diameter and possibly the length of embedment needed to develop the larger diameter anchor rod. Polyamide epoxies and urethanes for carbon steel anchor rods provide protection against alternating wet-dry environments. Phenolic epoxy coatings provide protection for chemical and acid vapors or fumes which exist in some industrial atmospheres or environments. An epoxy coating system can be field applied to the exposed threads and nuts after the anchor nut is secured in order to provide additional protection to galvanized anchors and nuts. A shop applied coating should not be used prior to anchor and nut installation.

2.6.4.1 Hot-Dip and Mechanical Galvanizing

Coating with a hot-dip or mechanical galvanizing process provides a cost-effective and maintenance-free corrosion protection system for most general applications. Hot-dip galvanizing should conform to ASTM A153/A153M or ASTM F2329 as appropriate.

The designer, the fabricator, and the galvanizer should take precautions against embrittlement in accordance with recommended practice in ASTM A143/A143M. A coating weight of 1 to 2.5 oz/ft² (0.3 to 0.75 kg/m²) is normal for the hot-dip process. A recommended coating weight of 2.3 oz/ft² (0.7 kg/m²) is an average application requirement. A corrosion allowance should not be required or added to galvanized anchor rods.

Carbon steel materials with ultimate tensile strengths less than 150 ksi (1,100 MPa) can be hot-dip galvanized. Alloy steel materials with greater ultimate tensile strengths should not be hot-dip galvanized because, as the tensile strength increases, the possibility of hydrogen embrittlement, where hydrogen is absorbed into the steel during the pickling process, increases. Blast cleaning rather than pickling should be used for alloy materials when considering galvanizing. ASTM A143/A 143M procedures should be used to safeguard against hydrogen embrittlement of hot-dip galvanized alloy steel products.

Galvanizing temperature and the effects of heat on quenched and tempered materials should be reviewed with the anchor manufacturer and galvanizer to confirm that the galvanizing process is below the minimum material stress relief or tempering temperature. Refer to Portland Bolt website FAQ “Galvanizing High Strength Bolts”.

As an alternative to hot-dip zinc coating, mechanical galvanizing (electro-deposited zinc, an inorganic zinc-rich paint, or other coating system specifically selected for corrosion protection), can be used. Mechanical galvanizing should conform to ASTM B695.

2.6.4.2 Cold-Applied Zinc

A cold-applied, organic, zinc rich compound primer or coating should be used for field touch-up of galvanized bolts or rods that have areas damaged during shipment or erection. Commercial zinc products for touch-up are zinc rich paint, zinc spraying, or brushed molten zinc. Touch-up paint should have 94% zinc dust in the dry film and should be applied to a minimum dry film thickness of 8 mils (0.20 mm). Refer to ASTM A780/A780M for additional information.

2.6.4.3 Insulation and Fireproofing

Anchors encased in insulation or fireproofing required for equipment within enclosed facilities may not require corrosion protection depending on service location or if in

coastal environments. In petrochemical facilities, conditions exist for equipment and structural steel columns such that moisture can collect under the insulation or fireproofing. The anchors should be either hot-dip galvanized, coated with a zinc based primer or other coating similar to that to be used for the equipment, or both. Two coats of primer, for a total dry film thickness of 3 to 4 mils (0.08 to 0.10 mm), should provide the necessary corrosion protection for this service. Anchor threads and nuts may need additional protection with an asphaltic mastic coating to allow for future retightening or removal of nuts.

2.6.4.4 Recommendations

A corrosion allowance is not required for anchors that are galvanized or coated. Anchors that are not galvanized or coated should have a minimum corrosion allowance of 1/4 in. (6 mm) added to their diameter, although it is preferable that they be galvanized or coated.

All types of protective coatings should be periodically inspected and maintained to prevent corrosion from reducing the design capacity of the anchorage assembly.

Anchors should be kept free of accumulations of excess materials or debris that may contain or trap moisture around anchors. Concrete and grout surfaces should be sloped to drain water. Avoid details which will create pockets, crevices, and faying surfaces that can collect and accumulate water, debris, and other damp materials around the anchorage.

Foundations located in areas with a high groundwater table are highly susceptible to corrosion. The diameter of anchors exposed to surface drainage or ground water should be increased for corrosion protection as noted above unless a protective coating is provided.

The surfaces between base plates and the concrete or grout supporting critical equipment or structures may require sealing to prevent the infiltration of corrosive elements. Dry pack grout pads formed with cement and sand should be coated or sealed in areas with cyclic wet-dry environments, since this type of grout pad tends to break down with age in a cyclic wet-dry environment.

The service life of a combined system of paint over galvanizing is substantially greater than the sum of the lives of the individual coatings. Precautions must be taken to ensure adherence of the paint to the galvanized surface, which is smooth and does not permit mechanical locking of the coating film.

2.6.5 Weathering Steel

Steel manufactured in accordance with ASTM A588/A588M, commonly referred to as weathering steel, develops a tight oxide coating that protects against corrosion of the substrate. In certain environments it will provide a relatively maintenance-free

application. The material will form a protective surface with loss of metal thickness of about 2 mils (0.05 mm). Weathering steel will provide atmospheric corrosion resistance that is 4 to 6 times the corrosion resistance of ordinary carbon steel.

Bare weathering steel should not be submerged in water because it will not provide corrosion resistance greater than black carbon steel in the same service. Bare weathering steel should not be exposed to recurrent wetting by salt water, spray, or fogs because the salt residue will cause accelerated corrosion.

Weathering steel may be painted or galvanized as readily as carbon steel, although its appearance may not be uniform because of the higher silicon content. Urethane foam and other fire retardants can be very corrosive when wet with water. Foam suppliers can recommend paint systems that are compatible with their foams.

Weathering steel may be used when anchors are exposed to corrosive atmospheres, but it should be understood that it will rust and stain the foundation concrete if so exposed, and is generally not recommended for petrochemical facilities.

2.7 ANCHORAGE EXPOSED TO EXTREME TEMPERATURES

2.7.1 Exposure to Low Temperatures

When an anchorage is exposed to extreme low temperatures, the main design concern is that the anchor material will become brittle and fail, either prematurely or at a strength level that is less than its design load. In order to mitigate this concern, a sample of the anchor material should be tested at low temperature to measure impact properties. This is typically accomplished using a Charpy V-Notch Test. Testing requirements can be found in ASTM A370. For extreme low temperature exposure, ASTM A320/A320M L7 material is recommended. Tables 2.2 and 2.3 provide recommended testing for different grades and diameters of anchor materials.

The minimum design metal temperature at which anchors are exempt from impact testing requirements depends upon the anchor material specifications. ASME Boiler and Pressure Vessel Code, Section VIII, Division 1, Figure USC-66 provides guidance for impact test exemption for bolting and nuts based on material type and design metal temperature. ASTM A307 anchors should be exempt from impact testing to -20° F (-29° C). Anchors fabricated of ASTM A193/A193M, grade B7 material should be exempt from impact testing to -55° F (-48° C). Anchors fabricated of ASTM A320/A320M, grade L7 material inherently satisfy impact test requirements at low temperatures and no further impact test requirements are necessary.

Table 2.2: Cold Temperature Anchor Material Testing Recommendations

Yield Type	Specified Minimum Yield Strength	Anchor Material Diameter (d_a)		
		$d_a < 0.50"$	$0.50" \leq d_a \leq 2.0"$	$d_a > 2.0"$
I	< 50 ksi (345 MPa)	NT	CV1	CV2
II	50 ksi (345 MPa)	CV1	CV1	CV2

Toughness Class Notes:

NT - No impact testing required to demonstrate toughness

CV1 - Charpy V-Notch Toughness Class 1 as defined in Table 2.3

CV2 - Charpy V-Notch Toughness Class 2 as defined in Table 2.3

Charpy V-Notch: Specimens are V-Notched per ASTM A673/A673M and tested in accordance with ASTM A370

Table 2.3: Charpy V-Notch Test Performance Requirements – Anchor Material

Toughness Class	Test Temperature	Test Values	
CV1	14° F (-10° C)	Minimum Average	20 ft-lbf (27 J)
		Minimum Individual	16 ft-lbf (22 J)
CV2	-4° F (-20° F)	Minimum Average	20 ft-lbf (27 J)
		Minimum Individual	16 ft-lbf (22 J)

2.7.2 Exposure to Elevated Temperatures

When an anchorage is exposed to extreme high temperatures, the main design concerns are with the coating, grouting, and reduction in strength of the anchorage materials (steel and concrete). High temperature concerns for anchors should be addressed at the design stage of the project and carried through to construction, including inspection and testing.

In order for hot-dip galvanized coating to remain effective for long term use the maximum service temperature of the anchor should be less than 390° F (199° C). At a temperature of 390° F (199° C) peeling of the free zinc layer begins to occur. At higher temperatures, the resistance to peeling deteriorates at a higher rate. This does not mean that there is not corrosion protection. When peeling occurs, only the outer free zinc layer has become detached, leaving the zinc-iron alloy layers to provide corrosion protection to the steel. This peeling action, however, is undesirable. If these high temperatures are anticipated on the anchor, then an alternative means of corrosion protection should be employed.

Most epoxy grouts experience excessive creep and loss of strength when exposed to high temperature. Hydrocarbon-based bonding materials such as epoxies, carbonize at approximately 572° F (300° C), leading to permanent loss of mechanical properties. In the case of adhesive anchors, the relationship between temperature rise and bond resistance of the adhesive in situ will determine the load capacity of the anchorage when exposed to high temperature.

Concrete starts to experience a loss of strength at 200° F (93° C). (Refer to AISC 360-05 Table A-4.2.2.)

Carbon steel experiences a loss of elasticity at 200° F (93° C) and a loss of strength at 750° F (399° C) or higher. (Refer to AISC 360-05 Table A-4.2.1.) As the temperature becomes higher than 700° F (371° C) the loss of strength to carbon steel becomes larger. Stainless steels offer greater resistance to extreme high temperature than carbon steels and generally possess a lower thermal coefficient of transmissibility. ASTM A193/A193M Grade B7 material is recommended for use in high temperature service.

2.7.3 Exposure to Fire

Anchors exposed to fire conditions are subject to strength loss primarily on the basis of softening of the exposed steel components. Where threaded parts are exposed directly to flame, failure is often precipitated by softening of the threads.

Anchors may be tested for fire exposure using standardized time-temperature curves as described in ASTM E119 or ISO 834-8.

Adhesive anchors may present special challenges for assessment of fire resistance, since they may also be compromised as a result of either resin softening or carbonization or both, and loss of strength in the concrete in which the anchors are embedded.

Protective measures include increasing embedment depth and ensuring that side cover is sufficient to maintain concrete temperatures well below the carbonization temperature for organic materials [approximately 500 to 575° F (260 to 302° C)] for the design fire exposure duration.

Where anchors are used to suspend mechanical and architectural systems, protection of the anchors without corresponding measures to protect the suspended rods or other elements will probably be ineffective in prolonging fire resistance.

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ASTM F436M-10, *Standard Specification for Hardened Steel Washers [Metric]*, ASTM International: West Conshohocken, PA.

ASTM F844-07a, *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*, ASTM International: West Conshohocken, PA.

ASTM F1554-07a, *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*, ASTM International: West Conshohocken, PA.

ASTM F2329-05, *Standard Specification for Zinc Coating, Hot-Dip, Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts, and Special Threaded Fasteners*, ASTM International: West Conshohocken, PA.

ISO 834-8:2009, *Fire-Resistance Tests -- Elements of Building Construction -- Part 8: Specific Requirements for Non-loadbearing Vertical Separating Elements*, International Organization for Standardization (ISO): Geneva, Switzerland

PORTLAND BOLT, *Galvanizing High Strength Bolts*, (FAQ), www.portland.com/faq/galvanizing-high-strength-bolts

CHAPTER 3

CAST-IN-PLACE ANCHOR DESIGN

3.1 INTRODUCTION

In the past, there has been a lack of guidance in building codes for the design of anchorage to concrete. As a result, engineers have used experience, knowledge of concrete behavior, and guidance from other design recommendations (such as ACI 349 Appendix D) for help in designing these anchorages. In 2002 ACI 318 introduced Appendix D, addressing this important area of design. This appendix and the latest revision of ACI 349 Appendix D are currently considered the state-of-the-art in anchorage design in CCD. When this method was first introduced there was no mention of using anchor reinforcement to prevent concrete breakout of the anchor. Thus, because of small concrete sections, large forces, and correspondingly large anchor sizes, the petrochemical industry could not use the method without modification. The modification that was used was to add reinforcement to transfer anchorage forces to the concrete. ACI 318-08 Appendix D added properly developed anchor reinforcement to resist anchor breakout to the code. Thus, ACI 318 Appendix D can now be used by the petrochemical industry without modification.

CCD uses a pyramid failure surface with a slope of 35 degrees for both tensile and shear loading. The method uses formulas for tension and shear which are proportional to the depth of embedment and edge distance respectively, to an exponent of 1.5. For more information on the basis for ACI 318 Appendix D and ACI 349 Appendix D, the reader is referred to the paper by Fuchs et al. (1995). This paper details how testing has revealed that the CCD Method is a more accurate predictor of concrete capacity for various anchorages than methods using bond strength of anchors to concrete. This has also been verified through probabilistic studies by Klingner et al. (1982a). The reader is cautioned however, that the amount of testing done on anchor arrangements, sizes, and depths of embedment typically found in the petrochemical industry is extremely limited. Therefore, it is difficult to draw definite conclusions about the accuracy of using this method for large anchors and deep embedments without further experimentation.

All of these factors (depth of embedment, arrangement, and anchor sizes) point to the fact that design by the CCD Method will generally produce more conservative designs for the anchor sizes and embedments typically found in the petrochemical industry. However, the paper by Fuchs et al. also notes that the method was primarily developed for anchors in unreinforced concrete and that the use of reinforcement designed to engage failure cones could substantially increase the load capacity of the anchorage. Early evaluation of the CCD Method for typical examples in petrochemical design supports the observation of more conservative results and found that, without the use of reinforcing, this method would lead to unacceptably conservative concrete member sizes.

Based on the above observations, this report will identify the critical steps in anchorage design and make recommendations for providing reinforcing details for safe and economical designs.

Design of foundations in petrochemical facilities often requires the anchorage of tall vessels and structures subject to large wind and seismic forces, which in turn results in large diameter anchors. Transferring the loads from these anchors to the foundation and developing anchor reinforcement often requires large embedment lengths. Thus the embedment length may sometimes control the depth of the foundation.

Since the size of the concrete members in which anchors are embedded is often limited by the available space left after piping, electrical conduit, other foundations, and access requirements are met, design decisions often involve choices not required in other industries.

3.2 ANCHOR CONFIGURATION AND DIMENSIONS

3.2.1 Configuration of Cast-in-Place Anchors

In the past, anchorage to concrete consisted of J-bolts, L-bolts, steel rods with nuts, or steel rods with plate washers. Since J- and L-bolts are no longer recommended for anchorage to concrete because of the potential for slip at service loads (Lee et al., 1966 & Cannon et al., 1975), the primary method of anchorage has become the steel rod threaded at both ends, with a nut at the bottom. Typically, the nut is tack welded if the anchor is fabricated from weldable material. If the anchor is not fabricated from weldable material, two nuts may be provided and jammed together. If a single nut is not adequate to meet the requirements of ACI 318 Section D.5.3.4 to prevent crushing of the concrete, the nut can be replaced with a larger diameter round plate of appropriate thickness. (Square plates should be avoided because of the concrete cracking potential due to the embedment of sharp corners in the concrete.)

Additionally, in 1997, using funding from the Southern Gas Association and their subsidiary The Gas Machinery Research Council, the Southwest Research Institute published research on determining by Finite Analysis, the horizontal forces from gas compressors that need to be restrained and the required capacity for main frame anchorage. (See <http://www.gmrc.org/>- TR-97-2 and TR 97-6.) Subsequent field testing of various anchor configurations verified the research and confirmed that “J” or “L” bolt configurations can lead to anchor bolt pull out under dynamic loading.

See Figure 3.1 for recommended anchor rod terminations.

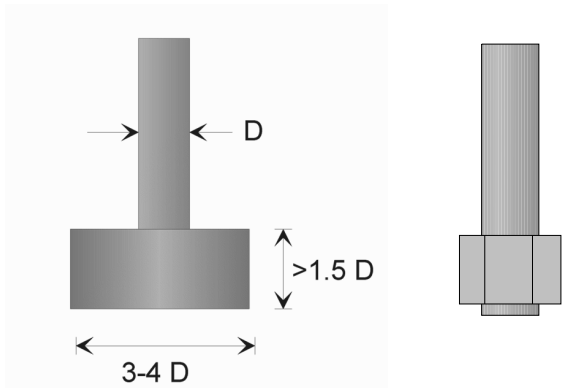


Figure 3.1: Recommended Anchor Rod Terminations
(Reproduced with permission from Gas Machinery Research Council [GMRC])

Recently it has become possible to obtain headed bolts of long lengths without the need to thread and attach a nut. Engineers should check the availability of long headed bolts for use in their area.

3.2.2 Minimum Dimensions

The following minimum anchorage dimensions are suggested for cast-in-place anchors. They are typical dimensions used in the petrochemical industry; they are not intended to provide for developing the full anchor capacity and may require reinforcement.

Embedment in Concrete	$12d_a$
Anchor Projection	two threads above fully engaged nut(s)
Concrete Edge Distance from Centerline of Anchor	
Mild Steel (36 ksi [248 MPa])	larger of $4d_a$ or 4.5 in. (114.3 mm)
High-Strength Steel	larger of $6d_a$ or 4.5 in. (114.3 mm)
Anchor Spacing	$4d_a$

Where:

d_a = anchor diameter

When sleeves are used, the minimum dimensions listed above should be increased as shown below. Additionally, when partial sleeves are used, a minimum clearance between the top of the bottom nut or bearing plate and the bottom of the sleeve shell should be provided to prevent the unexpected pullout of the anchor that may occur as a result of the interruption of the failure plane by the sleeve. For full length sleeves

Table 3.1: Recommended Sleeve Sizes

Anchor Diameter in. (mm)	Recommended Sleeve Size	
	Diameter in. (mm)	Length in. (mm)
1/2 (13)	2 (51)	5 (127)
5/8 (16)	2 (51)	7 (178)
3/4 (19)	2 (51)	7 (178)
7/8 (22)	2 (51)	7 (178)
1 (25)	3 (76)	10 (254)

Full-length sleeves are typically provided for anchors that are to be tensioned or if the engineer determines that a greater allowance is required for alignment. If the anchor is to be tensioned, the full length sleeve should be sealed on top or filled with an approved elastomeric material to prevent grout or liquid from filling the sleeve. For full-length sleeves, the minimum A_{brg} shall be calculated using ACI 318, equation D-15. The required area of the bearing plate should then be determined using the following equation:

Bearing plate area = $A_{brg} + d_s^{2*} \pi/4$

Where:

- A_{brg} = net bearing area of the head, bearing nut or bearing plate of the stud or anchor, in² (mm²)
- d_s = the diameter of the sleeve, in (mm)

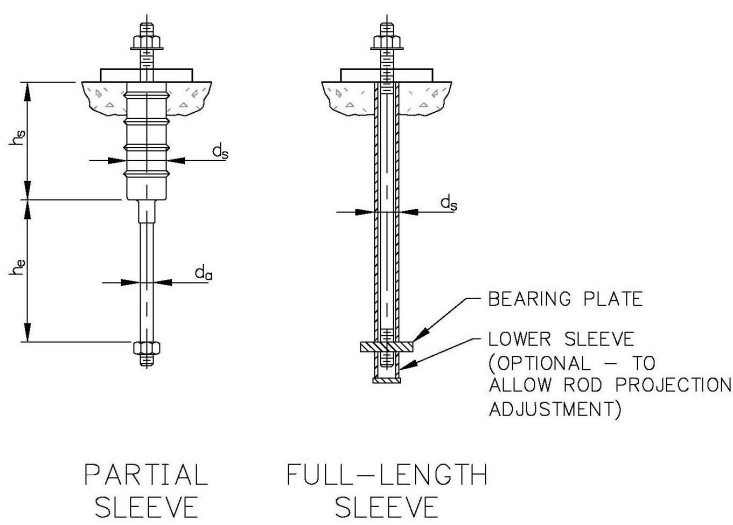


Figure 3.2: Typical Anchor Sleeves

3.2.3.2 Design Considerations When Using Anchor Sleeves

Sleeves will not affect the design of headed anchors subjected to tensile loads, because the tension in the anchor is transferred to the concrete via the anchor head and not by the bond between the anchor shaft and the concrete. Care should be taken when using partial sleeves to provide at least a minimum dimension between the bottom of the sleeve shell and the top of the anchor bearing surface as specified in 3.2.2.

When using partial sleeves, shear should not be transmitted via anchor bearing unless the sleeves are filled with grout to assure a proper bearing surface at the top-of-concrete elevation. See 3.3.3 through 3.3.5 for a detailed discussion of the design considerations associated with the transfer of shear from the attachment to the foundation.

Consideration should be given to the stiffness of the anchors before partial sleeves are specified for anchor alignment during equipment installation. As the anchor size (and thus the stiffness) increases, the ability to move the anchor horizontally in the field decreases. Using partial sleeves for this purpose is not recommended for anchors greater than 1 in. (25 mm) in diameter for this reason, and alternate methods of alignment such as templates should be investigated.

The design of the bearing plate used in full-length sleeves is critical to the functionality of the anchor. Research has shown that if the bearing plate is not sized properly the strength of the anchorage may decrease because of a weakened failure plane in the concrete (Cannon et al. 1981, and Cannon et al. 1975). Thus, when designing the bearing plate, the stiffness of the plate should be taken into consideration along with the strength. See 3.2.1 for general guidance on bearing plate thickness recommendations.

3.3 STRENGTH DESIGN

3.3.1 General

Depending on the loads and details used for anchorage design, the anchor connections are classified as either ductile or non-ductile. For ductile connections, the embedment is proportioned using the ultimate capacity of the ductile element; for non-ductile connections, the embedment is proportioned using the factored design method.

A ductile connection is defined as one that is controlled by the yielding of steel elements (anchor or reinforcement) with large deflections, redistribution of loads, and absorption of energy prior to any sudden loss of capacity of the anchorage resulting from a brittle failure of the concrete. ACI 318 and other building codes favor ductile design for seismic and blast-resistant design. Ductile design may also be required by the client or project standards.

Anchorage design should be approached as a global structural design issue, focusing more on the development of ductile load-resisting paths than on the ductility of a single element. Once these load paths are developed, the engineer can then correctly assess the effect of a ductile connection and decide what requirements should be imposed on an individual anchor.

The engineer should base the decision on whether to use ductile or non-ductile anchorage design on client specifications, building code requirements, the nature of the applied loads, the consequence of failure, and the ability of the overall structural system to take advantage of the ductility of the anchorage. Overconservatism is frequently induced in the anchor design as a result of conservative anchor sizing by equipment manufacturers, corrosion allowances, and inherent conservatism that results from the process of sizing anchors using allowable stress methods, combined with the design of concrete anchorage using ultimate strength methods. As a result, it is not uncommon for the ultimate capacity of an anchor to result in design forces that are more than twice the factored service loads.

As a minimum, anchor design loads should be factored service loads, as required by ACI 318. However, there are valid reasons why the engineer may choose the design load to be the ultimate tensile capacity of the anchor. These may include easier detection and repair of damage from overload, since the anchor elongation can be easily detected.

When peak loads are applied in a short term or impulsive manner, properly designed and detailed connections can allow a structural support to continue to carry loads until the short term peak has passed. Likewise, anchorage design should allow for the redistribution of loads and absorption of energy, as required in seismic or blast resistant design. When the characteristics and magnitude of the load are unpredictable, the anchorage design should be based on the ultimate tensile capacity of the anchor.

In some cases, the consequence of the failure of a single anchorage may be particularly undesirable. If, for instance, the failure of a single anchorage would lead to the collapse of a vessel or piping which contains highly flammable, toxic, or explosive materials, the engineer may want to base the anchorage design on the ultimate tensile capacity of the anchor. Additionally, if yielding of a ductile connection produces a hinge in a structure which leads to or causes a brittle failure elsewhere in the structure, the benefits of this ductility are, at best, underutilized, and the engineer should evaluate alternative methods of introducing ductility into the system. However, these decisions depend on the characteristics of the structure.

If the anchor load is to be transferred to the supporting foundation without accounting for assistance from reinforcement, the concrete strength design, which uses factored loads, should be in accordance with ACI 318 Appendix D.

The current state of practice in the petrochemical industry is to place reinforcement, which is used for the transfer of anchor forces to the concrete, in all pedestals; thus many provisions of ACI 318 Appendix D are not applicable, since they are based on designs that rely on the concrete strength, with minor strength gains resulting from improvement in ductility due to the presence of reinforcing.

3.3.2 Loading

Anchors should be designed for the factored load combinations in accordance with the selected code as discussed above. Care should be taken to ensure that the proper strength reduction factor, ϕ , is used. There are two distinct sets of strength reduction factors; one set applies to using the load combinations from ACI 318 Section 9.2 and a second set for use when load combinations from ACI 318 Appendix C are used.

3.3.3 Anchor Design Considerations

To accommodate reasonable misalignment in setting the anchors, base plates should be provided with extra large sized holes.

Shear force should preferably be transferred to the concrete by frictional resistance (see 3.6), but if the factored shear loads exceed the frictional resistance, another method must be provided to transfer the shear from the base plate to the foundation. This can be accomplished by either of the following methods:

- a. Use a shear lug
- b. Use a mechanism to rigidly connect the base plate to the anchors (such as by field welding bearing washers in place or filling the annular space with grout)

For non-ductile design, if no tensile force is applied to the anchors, the anchors need not be designed for tension. Where the tensile force is adequately transferred to properly designed reinforcement, there is no requirement to check for the concrete breakout strength of the anchor (N_{cb} or N_{cbg}), but the pullout strength (N_p) and side-face blowout strength (N_{sb} or N_{sbg}) should still be evaluated. While reinforcement for side-face blowout can be designed, unless proper installation is assured, this failure mode may still be of concern. See 3.5 for the design of anchor reinforcement.

3.3.4 Concrete Breakout Strength of a Group of Anchors in a Rectangular Pattern in Shear

In accordance with ACI 318 Appendix D, the concrete breakout strength of a group of anchors in a rectangular pattern in shear should be taken as the controlling value of the following:

- a. The concrete breakout strength of the row of anchors closest to the front edge perpendicular to the direction of force on the anchors

- b. The concrete breakout strength of the row of anchors farthest from the front edge perpendicular to the direction of force, if a mechanism is provided to rigidly connect the base plate to this row of anchors (such as by field welding bearing washers in place)

See ACI 318 Appendix D Figure RD.6.2.1 (b) for an illustration of this concept.

This committee proposes the following, although is not specifically addressed in ACI 318 Appendix D:

The concrete breakout strength of the row of anchors farthest from the front edge perpendicular to the direction of force, if closed shear ties or other mechanisms are used to transfer the load from the row of anchors closest to the front edge to the row of anchors farthest from the front edge. (See Figure 3.12.)

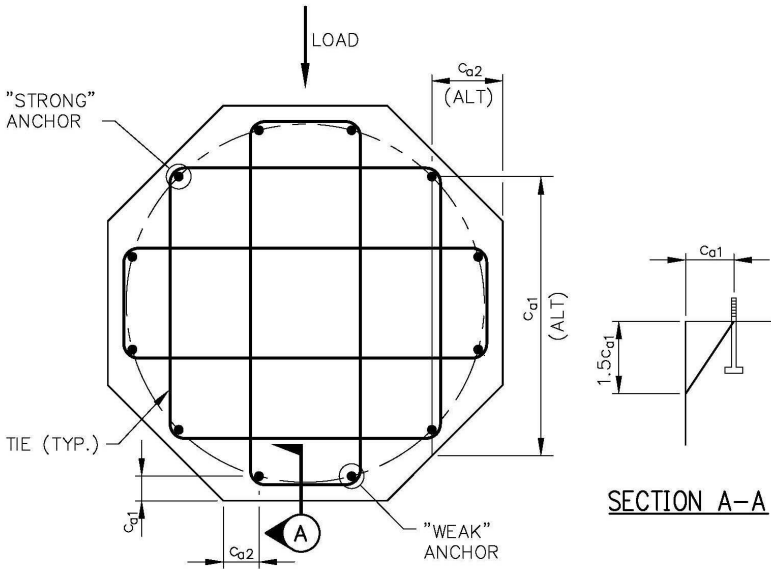
3.3.5 Concrete Breakout Strength in Shear of a Group of Anchors in a Circular Pattern

Anchors for tall vertical vessels are frequently not required to resist shear, since the shear is resisted by friction created by the large compressive force attributable to overturning. See 3.6 for the evaluation of frictional resistance. However, there are cases where the anchor may be required to transfer the shear load, such as for shorter vertical vessels or those subject to seismic design. See 3.11 for seismic design of anchors. Following are two alternative methods for designing the anchors to resist shear:

- a. Determine the concrete breakout strength of the anchor group in shear by multiplying the strength of the “weakest” anchor by the total number of anchors in the circle
- b. Where closed shear ties or other mechanisms transfer the load from the “weak” to the “strong” anchors, determine the concrete breakout strength of the anchor group in shear by calculating the shear strength of the “strong” anchors. (See Figure 3.3.)

3.4 DUCTILE DESIGN

Ductility is the ability of a structure, its components, or the materials used therein, to maintain resistance in the inelastic domain of response. It includes the ability to sustain large deformations and the capacity to absorb energy by hysteretic behavior. Displacement ductility is defined as the ultimate strain of the material divided by its yield strain. For an anchor in tension, it may be taken as the elongation of the anchor at maximum tension load divided by the elongation at yield.



1. FOR ALTERNATIVE a THE RESISTING SHEAR IS CALCULATED FOR THE TOTAL NUMBER OF ANCHORS (IN THIS CASE 12 ANCHORS) TIMES THE STRENGTH OF THE "WEAK" ANCHORS (USING c_{a1} AND c_{a2} FOR THE "WEAK" ANCHORS TO CALCULATE THE BREAKOUT STRENGTH OF ANCHORS IN SHEAR) REFERENCE ACI 318 SECTION D.6.2.
2. FOR ALTERNATIVE b THE RESISTING SHEAR IS CALCULATED FOR THE "STRONG" ANCHORS ONLY (IN THIS CASE 4 ANCHORS) TIMES THE STRENGTH OF THE "STRONG" ANCHORS (USING c_{a1} (ALT) AND c_{a2} (ALT)). BECAUSE c_{a1} (ALT) AND c_{a2} (ALT) ARE MUCH GREATER THAN c_{a1} AND c_{a2} THIS ALTERNATIVE WILL USUALLY PROVIDE GREATER RESISTING SHEAR. HOWEVER TIES TO TRANSFER LOAD FROM THE "WEAK" ANCHORS TO THE "STRONG" ANCHORS MUST BE PROVIDED.

Figure 3.3: Concrete Breakout Strength of a Group of Anchors in a Circular Pattern in Shear

(Adapted and reproduced with permission from PIP)

Anchor ductility is desirable for preventing brittle failure in the connection for two reasons: 1) It provides greater margin against failure because it permits redistribution of load to adjacent anchors and 2) It reduces the maximum dynamic loads by energy absorption and reduction in stiffness. (Refer to ACI 349.) An anchor that is to be characterized as a ductile element should be shown by calculation to have adequate stretch length that is compatible with the ductility required. (See 3.11.5 for an example of how to do this.)

Note: ACI 318 requires that the anchor material in a ductile anchor have an elongation of at least 14 percent and a reduction of area of at least 30 percent during a tensile elongation test. See Table 2.1 for guidance on ductile material selection. For a discussion of ductility of post-installed anchors, see 4.5.3.

An embedment is considered ductile when the failure mechanism of the element, anchor, or reinforcement is controlled by yielding of the element prior to a brittle concrete failure. Where anchor ductility cannot be assured, brittle failure can be prevented by designing the attachment connecting the anchor to the structure to undergo ductile yielding at a load level not greater than 75 percent of the minimum anchor design strength. Where geometric or material strength limitations prohibit such an approach, it may be appropriate to apply an overstrength factor to the load case. It is the opinion of this committee that if the anchor reinforcement is properly designed and developed to prevent failure of the concrete, the resulting connection may be considered ductile. This philosophy is consistent with reinforced concrete design principles.

The anchorage capacity provided by the concrete or properly developed reinforcement need only be ductile for the controlling design strength. For instance, if it can be shown by analysis that increasing tension loading will cause failure of the ductile steel element before the shear strength of the anchorage is reached, then the anchorage need only be shown to be ductile for tension loads. Conversely, if it can be shown by analysis that shear loading will always cause failure of the ductile steel element before tensile loading, then the anchorage need only be designed to be ductile for shear. However, achieving ductility in shear loaded anchorages can be more difficult, especially from the standpoint of achieving a meaningful degree of displacement ductility. A suggested method to provide ductility in such cases is the use of shear lugs with properly designed concrete reinforcement. Alternatively, the use of strength reduction factors similar to that discussed in ACI 318 Section D.3.3.6 is permitted.

3.5 ANCHOR REINFORCEMENT DESIGN

When the concrete has insufficient strength to resist tension and shear loads, reinforcing steel must be designed to transfer the loads into the base concrete.

3.5.1 Background

The load transfer method outlined in this section is based on the requirements listed in ACI 318 Appendix D. In the petrochemical industry, the unreinforced concrete breakout strength in tension and shear is rarely sufficient to exceed the ultimate anchor strength.

It is a construction preference to keep the anchors inside the pedestal and not extend them into the mat or footing. However, in some cases this may not be practical. If the anchor extends into the mat, the concrete breakout strength in the mat must be checked with the effective embedment depth measured from the top of the mat (Figure 3.4) assuming reinforcement is not adequately lapped to transfer the tension.

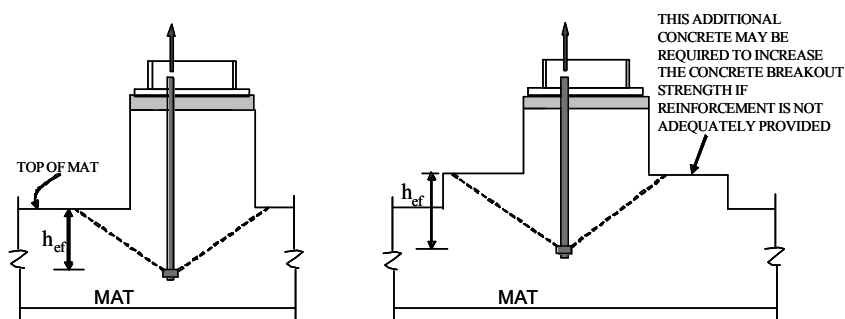


Figure 3.4: Anchor Extended into Mat

Previous editions of ACI 318 recognized the beneficial effects of supplementary reinforcement across the potential concrete breakout cone when evaluating the strength of an anchor. In order to reduce some confusion about this reinforcement, ACI 318 now defines two types of reinforcement that can be used across a potential breakout cone: supplementary reinforcement and anchor reinforcement.

3.5.1.1 Supplementary Reinforcement

Supplementary reinforcement is that which acts to restrain the potential concrete breakout but is not designed to transfer the full design load from the anchors into the structural member. An explicit design and full development of supplementary reinforcement is not required. However, it is recommended that the supplementary reinforcement be arranged to tie the potential failure prism to the structural member (oriented in the direction of the load so that it will be under tension load), similar to the arrangement of anchor reinforcement. Since supplementary reinforcement can improve the deformation capacity for the breakout mode, ACI 318 Sections D.4.4(c) and D.4.5(c) permit the use of a higher strength reduction factor ϕ for concrete failure modes (except for pullout and pryout strengths) if supplementary reinforcement is provided to tie the potential failure prism to the structural member. That is, Condition A applies. Condition B (no supplementary reinforcement) always applies for pullout and pryout strengths.

3.5.1.2 Anchor Reinforcement

Anchor reinforcement is designed and detailed specifically to transfer the full design load from the anchors into the structural member. An explicit design and full

development of anchor reinforcement is required. Where anchor reinforcement is developed on both sides of the breakout surface in accordance with ACI 318 Chapter 12, the design strength of the anchor reinforcement is permitted to be used instead of the concrete breakout strength to determine ϕN_n and ϕV_n . (See ACI 318 Sections D.5.2.9 and D.6.2.9.) For practical reasons, the use of anchor reinforcement is generally limited to cast-in-place anchors since there is insufficient lap length between post-installed anchors and reinforcing steel.

3.5.2 Reinforcement Methods

The concrete reinforcement needed to develop anchor loads shall be designed in accordance with ACI 318 and the following:

- a. The anchor force is assumed to be resisted only by the reinforcement. That is, there should be no load sharing between the concrete section and the reinforcement. Reinforcement is fully activated only after a concrete breakout cone has developed.
- b. The anchor reinforcement may be provided by a single layer or multiple layers of reinforcement, inverted hairpin reinforcement, edge angles attached with anchored reinforcement, spiral reinforcement, or horizontal ties to resist tension, shear, and lateral bursting. Although these alternatives provide valid options from an engineering point of view, their use may cause construction difficulties due to congestion of reinforcement. Engineering judgment is required to determine which alternative is the most appropriate for a given installation.
- c. The anchor tension force is transferred to reinforcement parallel to the anchor. This reinforcement most commonly consists of straight dowels in the pedestal with 90 degree hooks extending into the footing or mat. If the anchor length is too long because of l_d requirements, and additional dowels are not practical, 90-degree or 180-degree hooked bars at the top of the pedestal may be used to reduce the anchor embedment to l_{dh} . This option should be used as the last resort because of constructability considerations. (See Figure 3.5.)
- d. When anchors are used to transfer shear, shear reinforcement is typically required, since minimal edge distances and anchor spacing make it difficult to develop the anchor loads in the concrete member without the use of reinforcement.
- e. The arrangement of reinforcement should consider the clearance requirements for placing and vibrating concrete and the minimum bar spacing requirements of ACI 318.

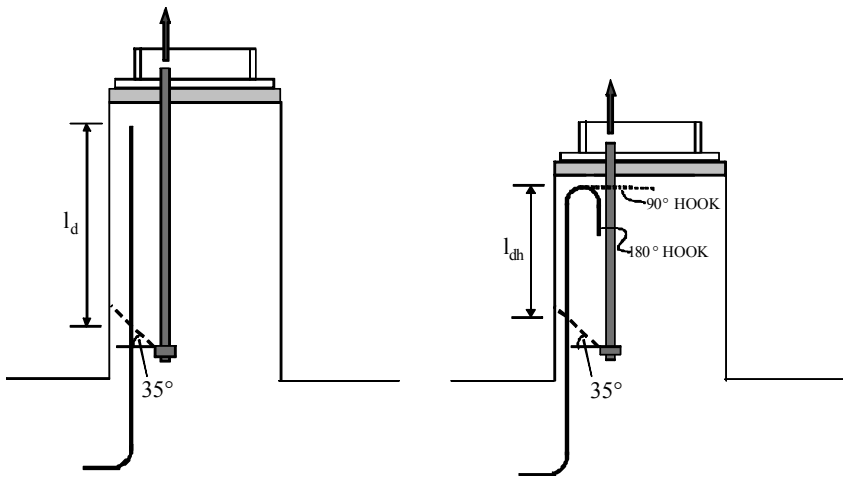


Figure 3.5: Tension Transferred to Reinforcement Parallel to Anchor

- f. If the side-face blowout resistance is less than the required strength, either the edge distance, the bearing area of the anchor head, or both should be increased. Reinforcement near the embedded anchor head or nut may be provided to improve the ductility related to concrete side-face blowout. Cannon et al. (1981) recommended spiral reinforcement be installed around the head to provide concrete confinement. DeVries et al. (1998) found that transverse reinforcement (ties) did not increase the side-face blowout capacity and a large amount of transverse reinforcement installed near the anchor head only increased the magnitude of load that was maintained after the side-face blowout failure occurred (that is, ductility was increased). Therefore, where there is a realistic possibility of side-face blowout, the engineer should make all efforts to change the bolt layout, concrete configuration, or the anchor head bearing area to preclude blowout before committing to a solution that relies on supplemental reinforcing steel. See 3.5.3.1.3 for more details on side-face blowout reinforcement.
- g. Rebar development length should be adequate to fully develop the required load on both sides of the failure surface in accordance with ACI 318.
- h. The failure surface resulting from the applied tension load should be in accordance with ACI 318 Figure RD.5.2.1 for single anchors and group anchors.
- i. The failure surface resulting from the applied shear load should be in accordance with ACI 318 Figure RD.6.2.1(a). For multiple anchors closer together than three times the edge distance, c_{a1} , the failure surface is from the outermost anchors per ACI 318 Figure RD.6.2.1(b).

3.5.3 Reinforcement Design to Transfer Anchor Forces

ACI 318 Sections RD.5.2.9 and RD.6.2.9 state that in sizing the anchor reinforcement, the use of strength reduction factor $\phi = 0.75$ is recommended as is used for the strut-and-tie models (ACI 318 Section 9.3.2.6), implying that the use of the STM design approach in designing anchor reinforcement is an acceptable design approach.

The STM is an ultimate strength design method based on the formation of a truss that transmits forces from loading points to supports. The STM uses concrete struts to resist compression and reinforcing ties to carry tension. Design using the STM involves calculating the required amount of reinforcement to serve as the tension ties and then checking that the compressive struts and nodal zones (joints) are sufficiently large to support the forces. A key advantage of design using the STM is that the designer can visualize the flow of stresses in the member. While the STM is a conceptually simple design tool, it requires assumptions for the following items:

- a. Capacity of struts and nodes
- b. Geometry of struts and nodal zones
- c. Anchorage of tie reinforcement

3.5.3.1 Tension Force

Tension force in anchors induces tensile stress in concrete due to bearing at the embedded anchor head or nut, which in turn induces lateral bursting forces. A recommended arrangement of reinforcement for resisting concrete tensile stress in pedestals of square, rectangular, and octagonal cross-section is shown in Figures 3.6 and 3.7.

3.5.3.1.1 Recommended Location of Anchor Reinforcement for Tension

There is currently no available test data that can be used to strongly recommend the location of anchor reinforcement in typical pedestals. Without such data, it is prudent and good practice to place anchor reinforcement as close as practicable to the anchor in order to prevent any unknown failure mechanism. The following discussion provides guidance on acceptable spacing limits.

- a. Cannon et al. (1981) indicated that for hairpin reinforcement to effectively intercept the potential failure planes, each leg should be located within $h_{ef}/3$ from the edge of an anchor head, where h_{ef} is the effective embedment depth of the anchor. However, no test data was referenced as the basis for the recommendation.

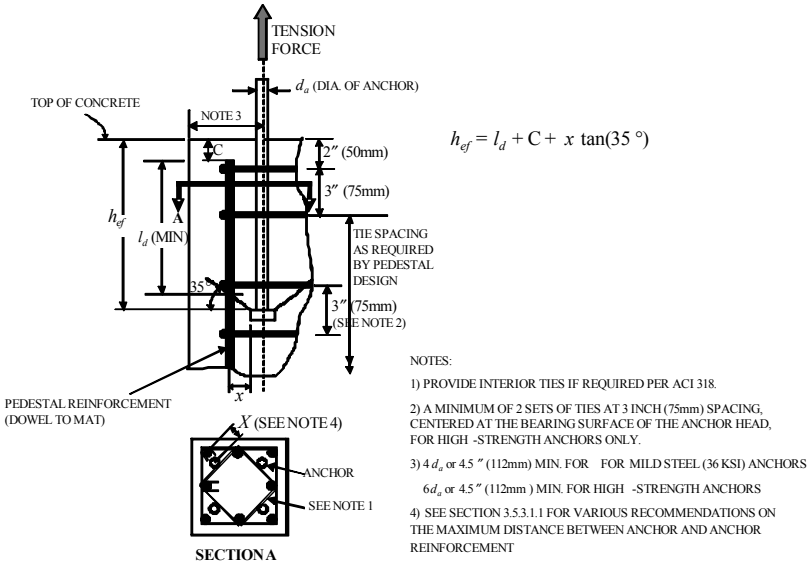


Figure 3.6: Reinforcement for Resisting Anchor Tension in Square and Rectangular Pedestals

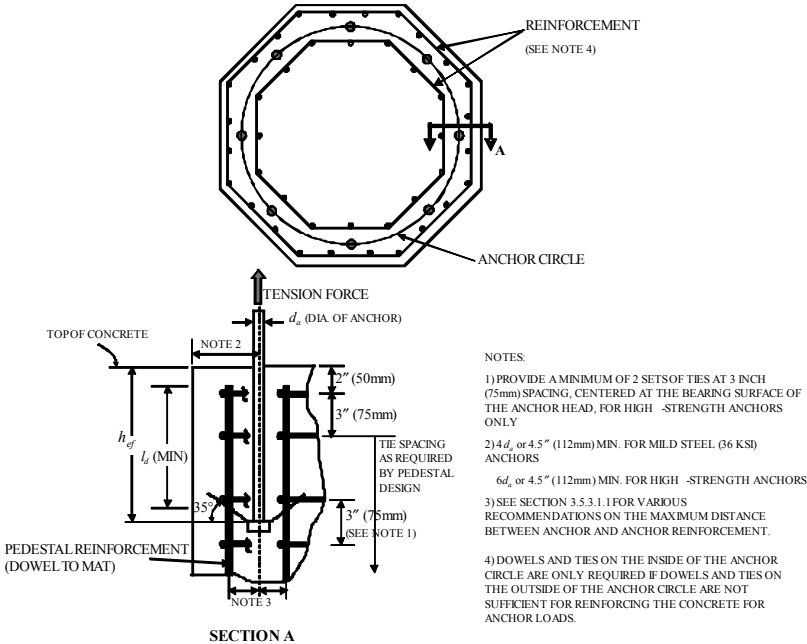


Figure 3.7: Reinforcement for Resisting Anchor Tension in Octagonal Pedestals

- b. In the previous edition of this report, it is stated that to be considered effective, the distance of the reinforcement from the edge of the anchor head should not exceed the lesser of one-fifth h_{ef} or 6 in. (152 mm).
- c. Comité Euro-International Du Béton (1997) recommended that the reinforcement be placed as close as possible to the headed anchors (and preferably be tied to the anchors).
- d. Lee et al. (2007) stated that supplementary hairpin reinforcement may be used to increase the concrete breakout strength if arranged in a manner similar to that tested (≤ 4 in. [102 mm], or $\leq 0.15h_{ef}$ from the anchor). However, they also indicated that their test results cannot be used to develop a general design model for anchors with supplementary reinforcement because of limited test data.
- e. ACI 318 Section RD.5.2.9 indicates that reinforcement should be placed as close as practicable but not more than $0.5h_{ef}$ from the anchor centerline. This recommendation is based on research of embedded studs with hairpin reinforcement using a maximum diameter similar to that of a #5 bar.

3.5.3.1.2 Concrete Breakout

Vertical reinforcement intersects potential crack planes adjacent to the anchor head, thus transferring the tension load from the anchor to the reinforcement. Proper reinforcement development length is required to develop the required strength both above and below the failure plane-reinforcement intersection.

The minimum required area of reinforcement per anchor, A_{st} , where anchor ductility is not required, is as follows:

$$A_{st} \geq \frac{N_{ua}}{\phi f_y}$$

Design for anchor ductility requires that the necessary conditions for elongation over a reasonable gage length are fulfilled; that is, that strain localization will not limit the yield strain. This may involve the use of upset threads (see 2.2.5.3) or other detailing methods to avoid strain localization. Furthermore, if it is desired that yielding of the anchor provide the required ductility in the connection, the reinforcement should be proportioned to develop the strength of the anchor as follows:

$$A_{st} \geq A_{se,N} \frac{f_{uta}}{f_y}$$

Where:

$$\begin{aligned} A_{st} &= \text{minimum required area of reinforcement, in}^2 \text{ (mm}^2\text{)} \\ A_{se,N} &= \text{effective cross-sectional area of anchor in tension, in}^2 \text{ (mm}^2\text{)} \end{aligned}$$

N_{ua}	=	factored tension design load per anchor, lb (N)
ϕ	=	strength reduction factor = 0.75
f_y	=	specified minimum yield strength of reinforcement, psi (MPa)
f_{uta}	=	specified minimum tensile strength of anchor steel, psi (MPa)

Where ductility cannot be achieved and the anchor is sized for factored tension design loads T_u^E , the reinforcement should be designed according to the equation below, thus satisfying IBC and ASCE/SEI 7 requirements for Seismic Design Categories C, D, and E.

$$A_{st} \geq \frac{T_u^E}{\phi f_y}$$

Where:

T_u^E	=	factored tension design load from load combinations that include an overstrength factor of 2.5 applied to the seismic loads (per anchor), lb
---------	---	--

When considering placement of reinforcing bars relative to the anchor, it may be necessary to explicitly consider the effect of the secondary moment caused by the couple between the anchor and the rebar. Alternatively, the reinforcing can be placed symmetrically around the anchor as shown in Figure 3.6. For typical cases where anchors are embedded in the tops of structural column pedestals it is generally not required that the reinforcing be placed symmetrically around the bolt, because the secondary moments can be accommodated within the section depth.

In order to limit the embedment length of an anchor, a larger number of smaller size reinforcing bars is preferred over fewer, larger size bars. In larger foundations, such as an octagonal pedestal for a vertical vessel, two concentric layers of vertical reinforcement may be provided as shown in Figure 3.7 if required to transfer the anchor tension load. Tensile loads can be transferred effectively by using hairpin reinforcement or vertical dowels with proper development lengths. (See Figure 3.5.) The area of vertical reinforcement calculated above is not to be considered additive to the reinforcement required strictly for resisting the moment and tension in sections of the pedestal. The calculated area of steel required for resisting the external loads applied to the pedestal should be compared to the area of steel required for resisting the tension in the anchor to determine the appropriate amount of reinforcement needed. The area of vertical reinforcement provided should equal or exceed the area of steel required for resisting the anchor tension or ultimate capacity ($A_{se} \times f_{uta}$) of the anchor if ductility is required.

The development length (l_d or l_{dh}) of reinforcing bars resisting the load should be calculated in accordance with ACI 318. The development length may be reduced when excess reinforcement is provided per ACI 318 Section 12.2.5. Reduction in the development length cannot be applied in Seismic Design Categories C, D, and E.

3.5.3.1.3 Side-face Blowout

Local concrete side-face blowout (lateral bursting) failure is caused by the quasi-hydrostatic pressure in the region of the anchor head (Eligehausen et al., 2006).

Test results for unreinforced concrete have shown that the side-face blowout failure load is independent of the embedment depth, and that the critical edge distance for the failure changes from concrete breakout to side-face blowout is equal to 0.4 times the anchor embedment (Furche and Eligehausen 1991); it is this research that forms the basis for ACI 318 Section D.5.4.1. However, where anchor reinforcement is provided to prevent the concrete breakout failure mode, the correlation between the critical edge distance and the embedment depth is no longer valid. Therefore, because of the lack of test data on side-face blowout strength in reinforced concrete, side-face blowout strength should be checked using ACI 318 Section D.5.4.1 regardless of the ratio of embedment depth to edge distance. Where there is a realistic possibility of side-face blowout, the engineer should try to increase the edge distance, bearing area, or concrete strength before committing to a solution that relies on supplemental reinforcing steel.

When reinforcement is used to restrain side-face blowout and improve ductility related to side-face blowout, it should have sufficient strength and stiffness in the direction of the lateral force causing the side-face blowout.

From a strength perspective, Figure 3.8 shows a recommended model for designing anchor reinforcement to resist side-face blowout force F . In Figure 3.8, the value of α indicates the ratio of side-face blowout force F to the tension force N_{ua} . Research indicates that the magnitude of F depends on the concrete bearing pressure on the anchor head, since the lateral strain in the concrete increases as the bearing pressure increases. Furche and Eligehausen (1991) suggested:

$$\alpha = 0.11 \sqrt{\frac{p}{f_{cc,200}}} \cong 0.11 \sqrt{\frac{N_{ua}/A_{brg}}{f'_c/0.83}} \quad (1)$$

Where:

α	=	ratio of F to N_{ua}
F	=	side-face blowout force
N_{ua}	=	factored tension force
A_{brg}	=	net bearing area of the head of stud or anchor bolt
$f_{cc,200}$	=	concrete compressive strength based on a 200 mm cube
f'_c	=	specified compressive strength of concrete
$f'_c/0.83$	=	an approximation of $f_{cc,200}$
p	=	the bearing pressure, which is equal to the tension force N_{ua} divided by the net bearing area of the anchor head A_{brg}

Note: See ACI 355.3R-11 , Appendix A-Tables A2 for cast-in-place anchors, threaded rods with nuts, threaded rods with nuts and washers, and the dimensional properties of bolts, studs and nuts for determining bearing area(A_{brg}).

Based on extensive non-linear numerical investigations, Hofmann and Eligehausen (2002) proposed:

$$\alpha = 0.045 \frac{P}{f_{cc,200}} \cong 0.045 \frac{N_{ua} / A_{brg}}{f'_c / 0.83} \leq 0.5 \quad (2)$$

Because of the lack of comprehensive test results, this committee recommends that the maximum α from the following be used:

- $\alpha = 0.25$
- α from Eq. (1)
- α from Eq. (2)

When the resulting force F exceeds the concrete strength computed by ACI 318 Section D.5.4, anchor reinforcement in the form of regular transverse ties, hairpins or spiral reinforcement should be designed to carry the side-face blowout force F . For ductile design, F is determined based on the ultimate capacity of the anchor. Since the majority of action occurs at the bearing surface of the anchor head, the anchor reinforcement should be placed as close as possible to the bearing surface of the anchor head. When there is not enough space near the bearing surface of the anchor head for all anchor reinforcement, all anchor reinforcement should be placed within the region shown in Figure 3.8 or Figure 3.9. This recommendation is based on a fracture mechanics model for studying crack propagation around a headed stud (Elfgren et al., 1982) and is based on the observed size of failure surface shown in Figure 3.9 (Furche and Eligehausen 1991).

It is believed that the effectiveness of anchor reinforcement in resisting the side-face blowout force depends on its location and stiffness. The effectiveness decreases as the distance from the bearing surface of the anchor head increases. It also decreases when the stiffness of anchor reinforcement in the direction of F decreases. For smaller rectangular pedestals, the anchor reinforcement could be in the form of regular transverse ties. For larger rectangular and octagonal pedestals, the anchor reinforcement could be spirals or U-shaped bars (hairpins), where the open legs extend away from the free edge.

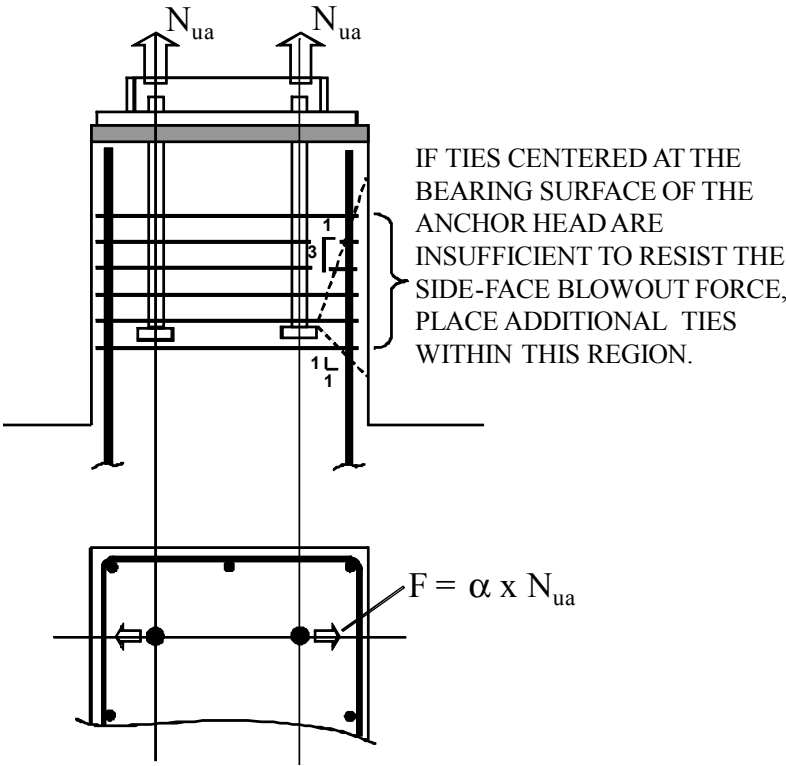


Figure 3.8: Model for Designing Anchor Reinforcement to Resist Side-face Blowout Force

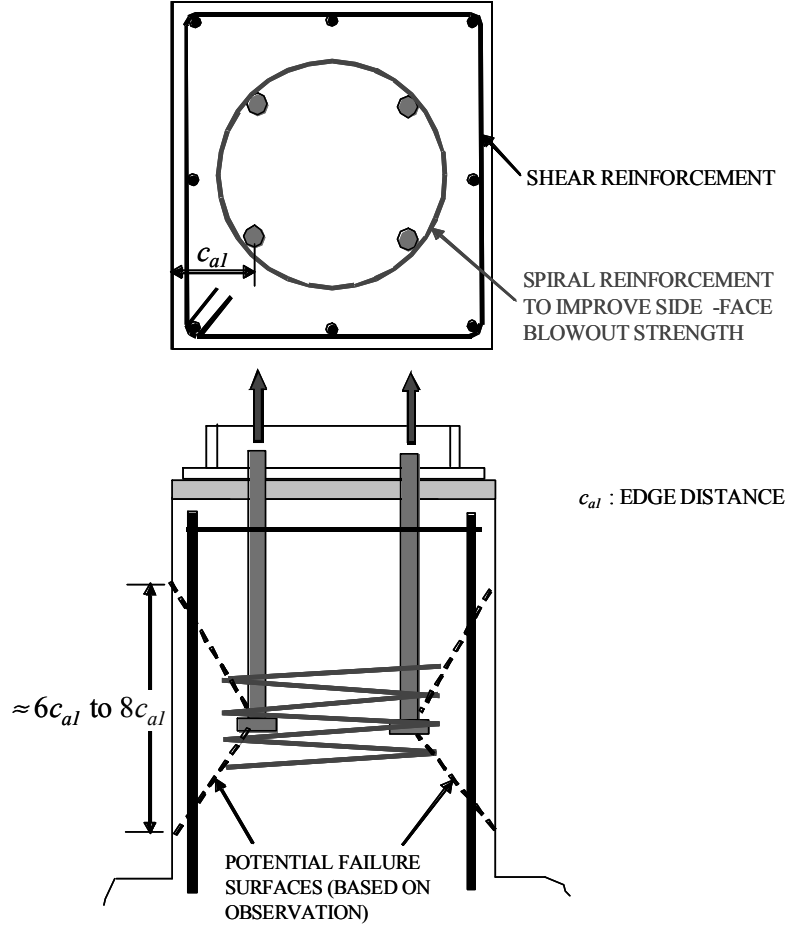


Figure 3.9: Spiral Reinforcement at Anchor Head to Improve Side-face Blowout Strength

3.5.3.1.4 Alternate Model for Tension Loading Using the Strut-and-Tie Model

One possible STM for tension loading is shown in Figure 3.10. Using STM, it is assumed that the diagonal concrete struts propagate radially from the anchor head. As a result of the diagonal concrete struts, there are radially horizontal force components propagating from the anchor head. For clarity, only the horizontal force in one direction is shown in Figure 3.10. This horizontal force component is the force that can cause:

- Side-face blowout (lateral bursting) failure when concrete cover is insufficient
- Splitting cracks when concrete cover is insufficient

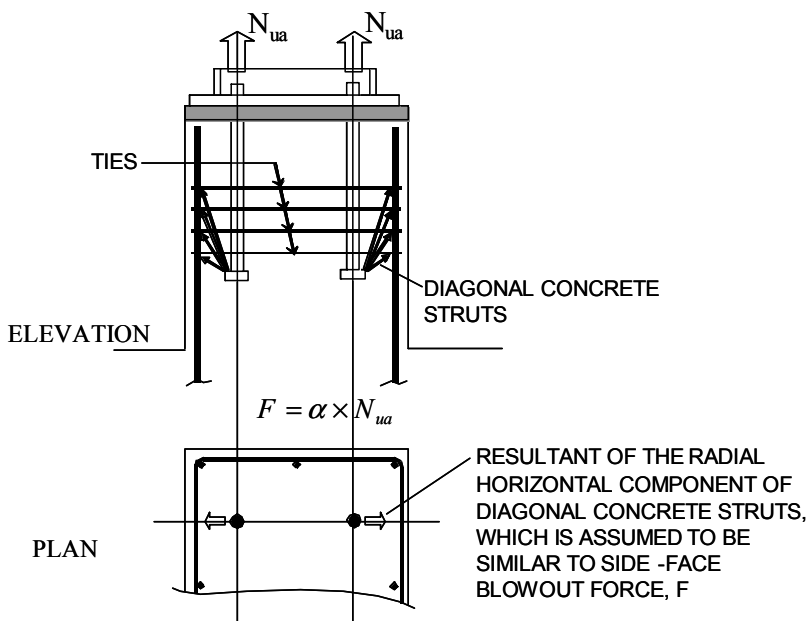


Figure 3.10: Possible STM for tension loading

To be consistent with the notation shown in Figure 3.7, the resultant of the radial horizontal component propagating from the anchor head is denoted F . As discussed in 3.5.3.1.3, the magnitude of F depends on the concrete bearing pressure on the anchor head. Therefore, the angle of diagonal concrete struts also depends on the concrete bearing pressure on (and thus the area of) the anchor head. Instead of one single diagonal strut, most likely there are several diagonal (fan shaped) struts propagating from the anchor head. As a result, the available area of the nodal zone (where the

diagonal struts meet with the vertical reinforcing bars) and the strut area are relatively large. Thus it is assumed that there is sufficient strength for such nodes and struts in typical concrete pedestals. Since the area of the nodal zone is relatively large, it is also reasonable to assume the available length is measured from the intersection between the 35-degree breakout cone angle and the vertical reinforcing bars when checking the available development length of vertical reinforcing bars. This assumption is consistent with the Thompson et al. (2006) recommendation for the mechanism of force transfer between opposing lapped headed bars: “the angle of concrete struts between opposing lapped headed bars is 35 degrees.”

Since the areas of the nodal zone and struts are assumed to be sufficiently large, the STM for tension loading is only used to proportion ties (that is, vertical reinforcing bars and horizontal ties) based on the overall equilibrium of the system. Based on vertical force equilibrium, the vertical reinforcing bars should be proportioned to carry the total tension force N_{ua} in the pedestal. Based on horizontal force equilibrium, the ties should be proportioned to carry the total horizontal force F and distributed within the recommended region shown in Figure 3.8.

3.5.3.2 Shear Force

Shear may be transferred by frictional resistance between the base plate and the concrete, with the anchors used for transfer of tension only. For large shear forces, where frictional resistance (see 3.6) is insufficient, shear lugs or anchors may be used to transfer the load. The shear forces then must be carried by the concrete or reinforcement.

Where anchors are used to transfer shear, reinforcement is typically required, since it is generally difficult to develop the anchor loads in the concrete member only. This is because of limited concrete breakout strength due to small edge distances and anchor spacing. Shear reinforcement should be designed to carry the entire shear load, excluding any contribution from concrete. Strut-and-tie models can be used to analyze shear transfer to closed ties.

Several shear reinforcement configurations can be considered to prevent failure of the concrete (such as hairpins, anchored reinforcement, closed ties, and shear angles). (See Figures 3.11 through 3.14 adapted from PIP STE05121, *Anchor Bolt Design Guide*.) For ductile design, the shear reinforcement should be designed to develop the ultimate shear capacity of the anchors. Alternatively, for cases involving seismic loading, the shear reinforcement can be designed for load combinations that include an overstrength factor of 2.5 applied to the seismic loads.

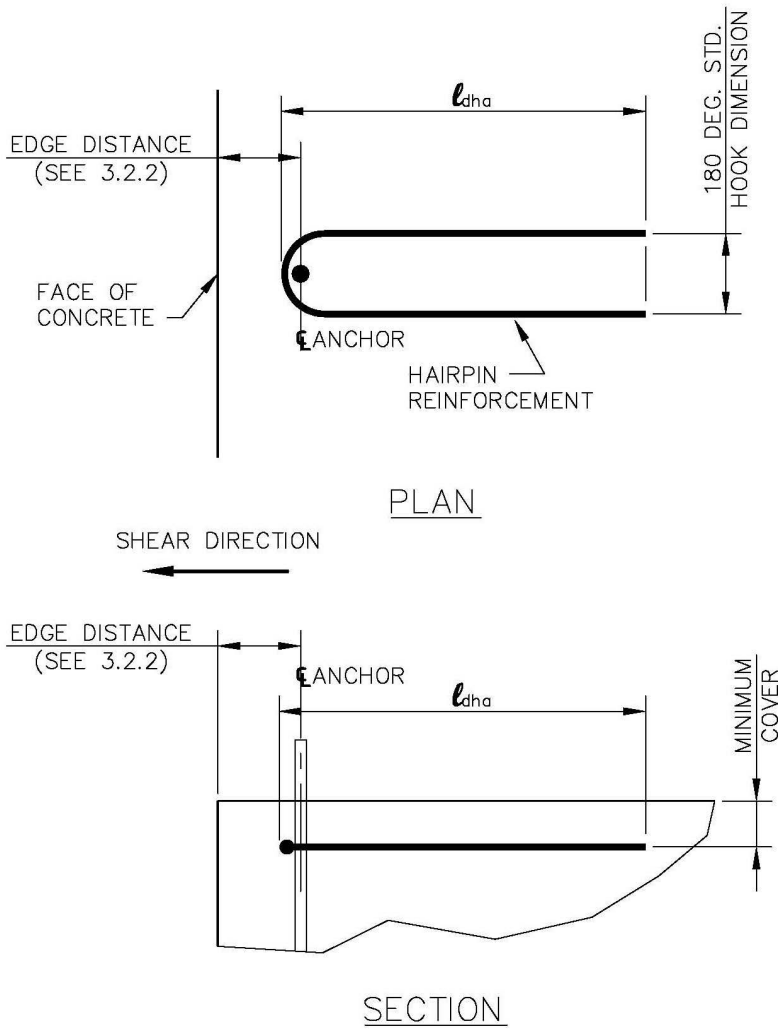


Figure 3.11 Horizontal Hairpin
(Adapted and reproduced with permission from PIP)

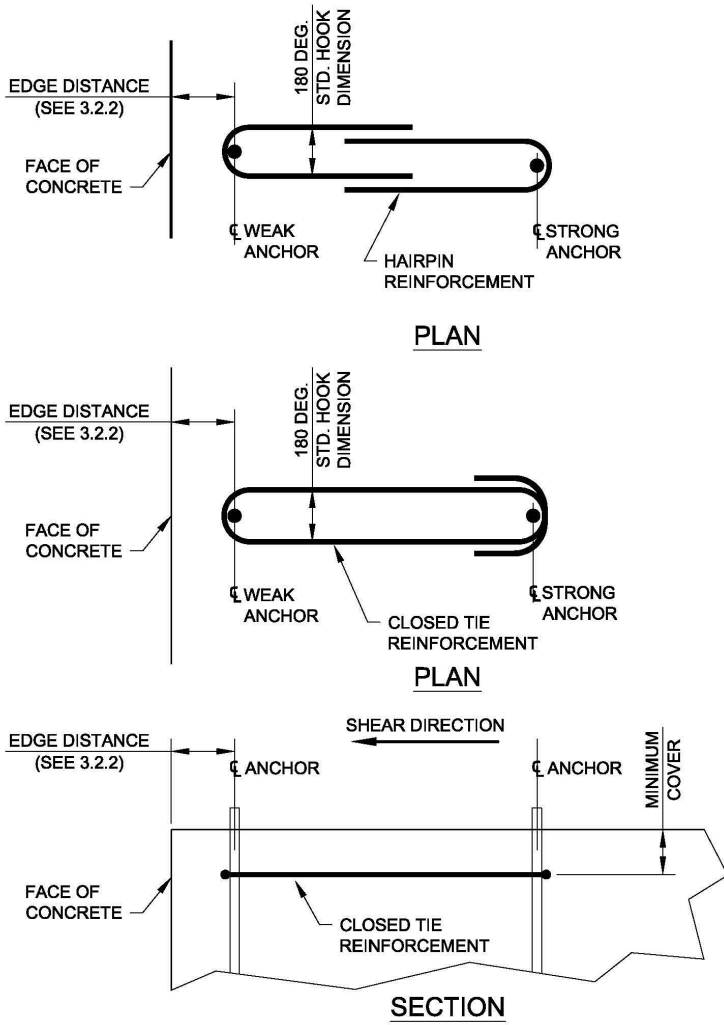


Figure 3.12 Closed Ties
(Adapted and reproduced with permission from PIP)

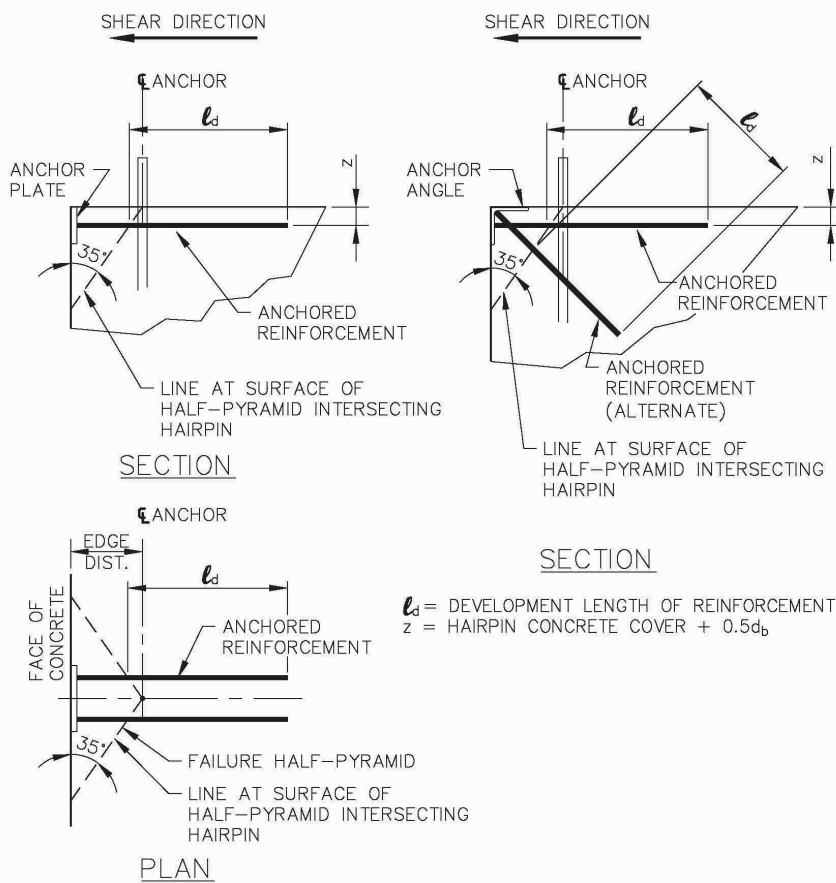
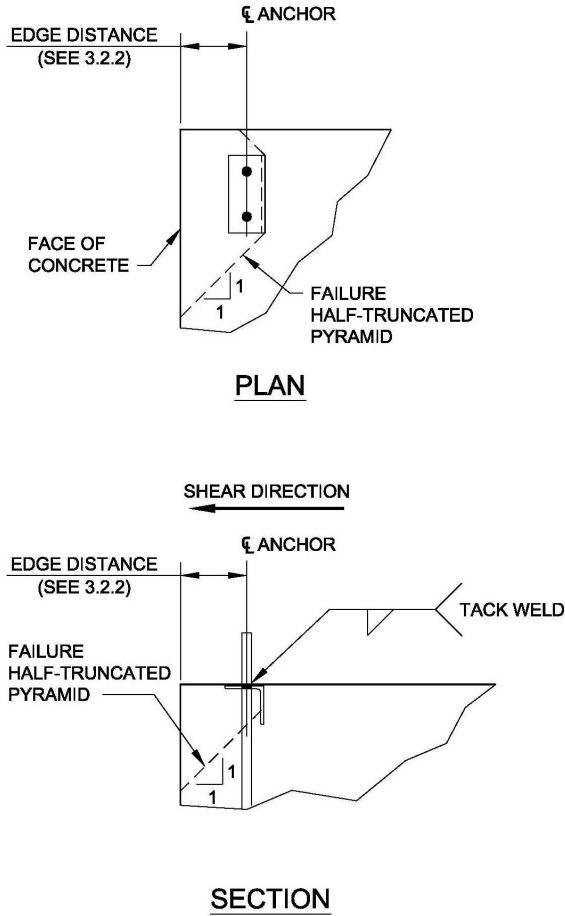


Figure 3.13: Anchored Reinforcement
(Adapted and reproduced with permission from PIP)



NOTE: DEDUCT AREA OF BEARING SURFACE OF SHEAR ANGLE IN CALCULATING A_{Vc} (THE PROJECTION OF THE FAILURE HALF-TRUNCATED PYRAMID).

Note:

The concrete failure plane for the shear angle is shown at a 1:1 slope (45°), rather than the 35° angle for anchors specified in ACI 318 Appendix D. This is because the committee considers a shear angle to be similar to a shear lug. (See 3.7.3, item 6.)

Figure 3.14: Shear Angles
(Adapted and reproduced with permission from PIP)

3.5.3.2.1 Recommended Location of Anchor Reinforcement for Shear

For shear loading, ACI 318 Section D.6.2.9 indicates that anchor reinforcement should either be developed in accordance with ACI 318 Chapter 12 on both sides of the breakout surface, or should enclose the anchor and be developed beyond the breakout surface. In order to ensure yielding of the anchor reinforcement, the enclosing anchor reinforcement should be in contact with the anchor and placed as close as practicable to the concrete surface. (ACI 318 is based on research with the maximum diameter of the anchor reinforcement similar to that of a #5 bar.) When a grid of surface reinforcement is used as anchor reinforcement (Figure 3.15), only reinforcement spaced less than the lesser of $0.5c_{a1}$ and $0.3c_{a2}$ from the anchor centerline should be included as anchor reinforcement, as shown by research with the maximum diameter of the anchor reinforcement similar to that of a #6 bar. In order to satisfy the equilibrium, edge reinforcement must be provided.

3.5.3.2.2 Alternate Model for Shear Loading Using the Strut-and-Tie Model (STM)

The advantage of using the STM for analyzing shear transfer and designing shear reinforcement for pedestal anchorages is the elimination of questionable assumptions related to the size and shape of the concrete breakout cone, the crack location (whether the shear cracks propagate from the middle of pedestals, front-row anchors, or back-row anchors), and the amount of shear reinforcement that is effective to restrain the concrete breakout cone.

One possible STM for shear loading on a rectangular pedestal is shown in Figure 3.16.

The following assumptions are suggested in order to proceed with the use of the STM for shear transfer analysis on pedestal anchorage and for designing the anchor shear reinforcement:

1. Concrete strength for struts and bearings f_{ce} is $0.85f'_c$ based on ACI 318 Appendix A. This assumption is conservative considering the significant amount of confinement in pedestals.
2. The concrete struts from anchors to vertical rebar are shown in Figure 3.17. ACI 318 Section D.6.2.2 indicates that the maximum load bearing length of the anchor for shear is $8d_a$. Therefore, the bearing area of the anchor is assumed to be $(8d_a)d_a = 8d_a^2$. The compressive force from the anchor to rebar is assumed to spread with a slope of 1.5 to 1. When the internal ties are not required (the case where axial force in the pedestal is so small that ACI 318 Section 7.10.5.3 does not apply), the STM shown in Figure 3.16 can be used. For a given anchor shear, V_{ua} , the tension tie force T in Figure 3.16 is larger than T_f in Figure 3.17.

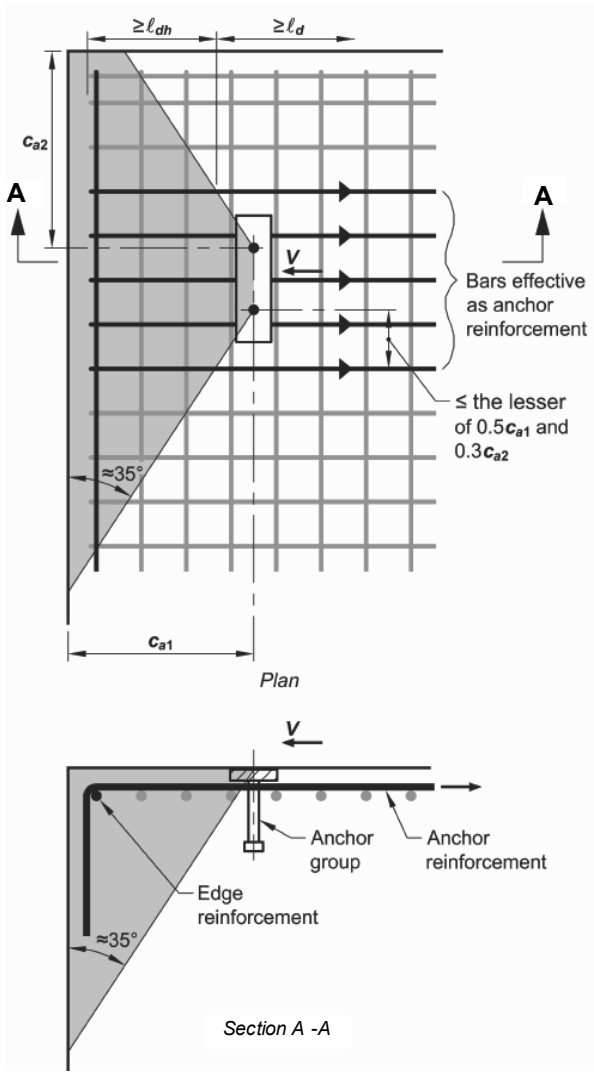


Figure 3.15: Grid of Surface Reinforcement as Anchor Reinforcement for Shear Loading (ACI 318)

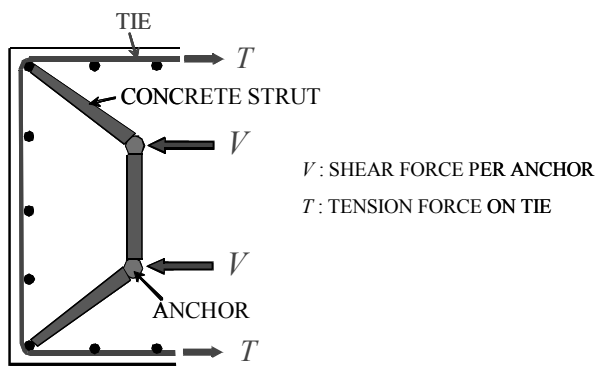
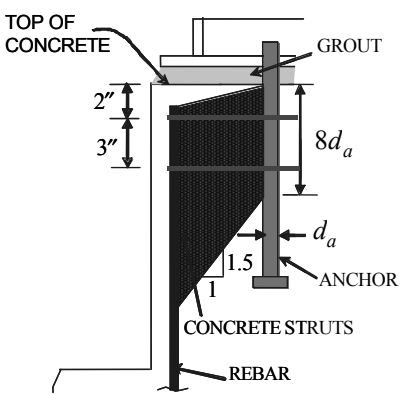
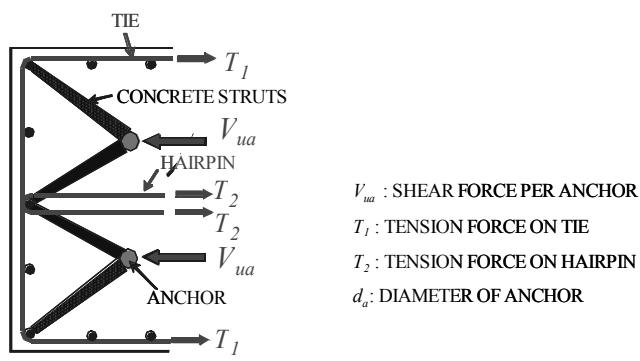


Figure 3.16: STM without Internal Ties



NOTE:
 SECTION 7.10.5.6 OF ACI 318 -08 INDICATES THAT THE LATERAL REINFORCEMENT SHALL SURROUND AT LEAST FOUR VERTICAL BARS, SHALL BE DISTRIBUTED WITHIN 5 INCHES OF THE TOP OF CONCRETE OF THE PEDESTAL, AND SHALL CONSIST OF AT LEAST TWO #4 OR THREE #3 BARS.

Figure 3.17: Concrete Struts and Tension Ties for Carrying Anchor Shear Force

3. For tie reinforcement, and with reference to Figure 3.18, the following assumptions are suggested:

- a. Only the uppermost two layers of ties (assume two #4 ties within 5 in. (127 mm) of the top of the pedestal as required by ACI 318 Section 7.10.5.6) are effective
- b. Tie reinforcement should consist of ties with seismic hooks. If internal ties are required, hairpins could be used. As an alternative, diamond-shaped ties can also be used.

Note: Moderate to high seismic design requires 135-degree hooks.

- c. The location of hooks and the direction of hairpins should be alternated as shown
- d. If the available development length of hairpin, l_{dha} , is shorter than the required straight development length for a fully developed hairpin, l_{dh} , the maximum yield strength that can be developed in a hairpin is:

$$f_y \times \frac{l_{dha}}{l_{dh}}$$

where f_y is the yield strength of the hairpin. If l_{dha} is shorter than 12 in. (304.8 mm), (that is, the minimum development length based on ACI 318 Section 12.2.1), then a hairpin should not be used.

- e. Away from the hook, the tie is assumed to be fully developed. For example, under the shear force V_{ua} , the tie on layer A can develop f_y at nodes 1 and 6
- f. At the node where the hook is located, the tie cannot develop f_y . For example, under the shear force V_{ua} , while the tie on layer A can develop f_y at node 6, the tie on layer B cannot, because the hook of the tie on layer B is located at node 6. In order to calculate the contribution of the tie on layer B to the tension tie at node 6, and with reference to Figure 3.19, the stiffness of a hooked bar bearing on concrete (Case 1 - smooth rebar with 180° hook bearing in concrete [Fabbrocino et al., 2005]) is compared to the stiffness of a hooked bar bearing on rebar (Case 2 - the conventional single-leg stirrup with reinforcing bars inside the bends [Leonhardt and Walther, 1965 as cited in Ghali and Youakim, 2005]).

Even though the capacity of Case 2 may be higher than that of Case 1 because of bearing on rebar of a larger size than the stirrup, contact may not always be present because of common imprecise workmanship. When the contact is not present, Case 2 is assumed to behave as Case 1. Leonhardt and Walther (1965) found that in order to develop f_y on the

bends of 90°, 135°, and 180° hooks when engaging bars located inside the bends (Case 2), there was a slip of about 0.2 mm (0.0079 in.). Based on the test results of Fabbrocino et al. (2005), the stress that was developed at the hook of the smooth rebar with a 180° hook bearing in concrete when it slipped 0.2 mm was about 20 ksi (138 MPa). Therefore, it is assumed that the tie can only develop 20 ksi (138 MPa) at the node where the hook is located.

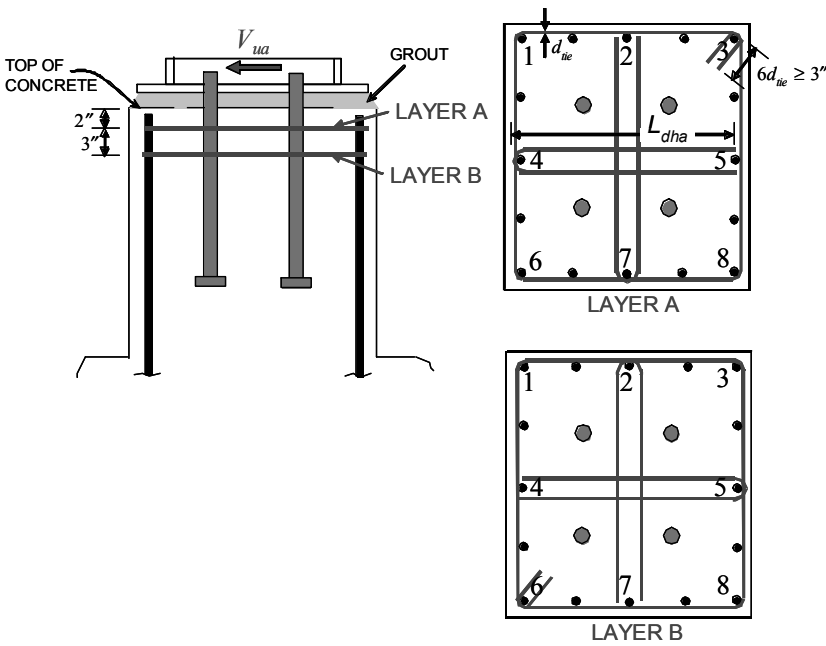


Figure 3.18: Alternated Direction of Hooks and Hairpins for the Upper Two Layers of Ties



Figure 3.19: Bearing of J-shape Bar on Concrete and Bearing of Conventional Stirrup on Rebar

3.6 FRICTIONAL RESISTANCE AND TRANSMITTING OF SHEAR FORCE INTO ANCHORS

3.6.1 General

1. Anchors need not be designed for shear if it can be shown that the factored shear loads are transmitted through frictional resistance developed between the bottom of the base plate and grout at the top of the concrete foundation. If there is moment on a base plate, the moment may produce a downward load that will develop frictional resistance even if the column or vertical vessel is in uplift, and this downward load can be considered in calculating frictional resistance. Care should be taken to assure that the downward load that produces frictional resistance occurs simultaneously with the shear load.
2. Shear has traditionally been assumed to be directly transferred from the base plate to the anchors by bearing of the base plate against the anchors if frictional resistance is exceeded and other means of shear transfer are not utilized. This assumption implies that some slippage will occur until the base plate bears against one or more anchors. It is common in the petrochemical industry to assume that in a typical 4-anchor arrangement, 2 of the anchors are engaged in transferring the shear and, conservatively, the two anchors with the smaller concrete edge distance in the direction of the shear are engaged. For this assumption to be reasonable, anchor hole diameters in the base plate should be as small as possible to accommodate specified construction tolerance of the anchors and minimize the amount of slippage to no more than say 1/4" (6 mm). For industrial structures, slippage of base plates on the order of 1/4"(6 mm) is considered acceptable, whereas in a commercial building that amount of slippage may not be acceptable. The responsible engineer should assure himself or herself that for the structure being designed, this slippage and anchor hole diameter requirement is reasonable.
3. If the assumption of paragraph 2 does not yield sufficient shear capacity to transfer the shear from the base plate through bearing on the anchors, then the anchor diameters, material, or edge distances could be increased to achieve sufficient shear capacity. If it is not practical or economically feasible to increase anchor diameters, material, or edge distances sufficiently, then the use of a shear lug could be considered. Shear lugs are recommended only where a more cost-effective solution is not possible or practical.
4. The frictional resistance can be used in combination with shear lugs to resist the factored shear load, but should not be used in combination with the shear resistance of anchors unless a mechanism exists to keep the base plate from slipping before the anchors can resist the load. One mechanism to prevent base plate slippage is to install plate washers between the base plate and the anchor nut. Plate washers with holes 1/16 in. (1.6 mm) larger than the anchor rods can be field welded to the base plate to assure minimal slip between the

base plate and the anchor. Hardened washers should not be used in this application because of the poor weldability of the material.

Note: Field welding is something that is typically avoided due to time, cost and problems associated with field welding galvanized steel (that is, prep for welding and increased corrosion potential). Hot-dip galvanizing is by far the most common and effective form of corrosion protection for structural steel and anchors used in the petrochemical industry, unlike the building industry which more typically uses black, primed, or painted steel. Secondly, plate washers can result in larger base plates and anchor spacing to avoid interferences, especially if larger anchor hole diameters are used.

Another option to prevent slippage is to fill the annular spaces between the anchors and the holes with grout.

5. Adding a tension load to high strength anchors (with adequate stretch lengths provided) can increase the frictional resistance. (See 3.8.) This load has the effect of increasing the normal factored compression force, P_u , in the equation shown in 3.6.2. The use of pre-tensioning should be limited to high strength bolts, as the use of pre-tensioning of mild steel such as ASTM A307 and ASTM A36 are often ineffective, Kulak, et al (1987).

3.6.2 Calculating Resisting Friction Force (Reference AISC Steel Design Guide 1)

The resisting friction force, V_f , may be computed as follows:

$$V_f = \mu P_u$$

Where:

$$\begin{aligned} P_u &= \text{normal factored compression force} \\ \mu &= \text{coefficient of friction} \end{aligned}$$

The coefficient of friction, μ , is governed by the placement of the base plate and grout pad as described below and shown in Figure 3.20. These factors are for limit state conditions (LRFD); if these factors are used with ASD they should be used with a safety factor of 2 (AISC LRFD Manual, First Edition, 1986).

$$\begin{aligned} \mu &= 0.90 \text{ for concrete placed against as-rolled steel with the contact plane a full plate thickness below the concrete surface.} \\ \mu &= 0.70 \text{ for concrete or grout placed against as-rolled steel with the contact plate coincidental with the concrete surface.} \end{aligned}$$

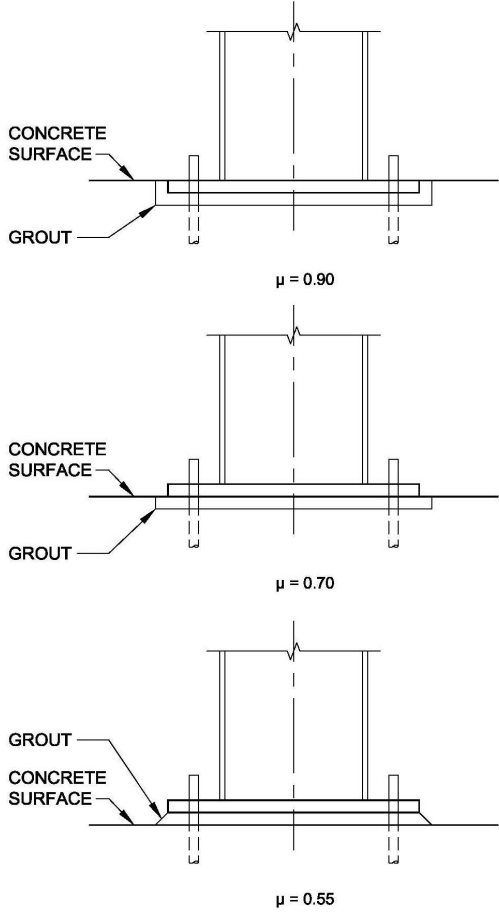


Figure 3.20: Coefficients of Friction

$\mu = 0.55$ for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface. (This is the normal placement of the base plate and grout pad.)

The compressive force, P_u , is the factored axial load between the base plate and pedestal, which acts concurrently with the lateral force, V_{ud} . This axial load is a result of load combinations, calculated in accordance with the governing code load combination equations, due to dead, live, wind, and seismic loads. If the anchor(s) are tensioned, the design tension load should be included as part of the dead load in the load combinations listed above. In addition, if there is fixity at the base plate and a moment occurs, the compressive force (between the base plate and the pedestal)

resulting from the couple between tension in the anchors and compression on the concrete should also be included in the load combinations. Of course if uplift occurs at the base plate due to wind or seismic loads, this should be combined as a negative force when calculating P_u .

3.7 SHEAR LUG DESIGN

3.7.1 General

Normally, frictional resistance and the shear capacity of the anchors used in a foundation adequately resist column base shear forces. In some cases, however, the engineer may find the shear force too great and may be required to transfer the excess shear force to the foundation by another means such as shear lugs. If the total factored shear loads are transmitted through friction plus shear lugs, the anchors need not be designed for shear, but the eccentricity induced by the couple of the applied shear and the shear lug resultant force should be taken into account when designing the anchor for tension.

A shear lug allows for complete transfer of the shear force, thus removing shear force from the anchors. The only portion of the shear lug that should be considered effective in resisting shear is that which bears against the grout in the grout pocket that is surrounded by concrete; the portion of the lug that bears against the grout that is above the top of concrete should be disregarded. Although the actual bearing load against the shear lug is probably higher near the top of concrete and reduces towards the bottom of the lug, the bearing load is normally assumed to be uniform from the top of concrete to the bottom of the shear lug (AISC Steel Design Guide 1).

The shear lug should be designed for the portion of applied shear not resisted by friction between the base plate and the concrete foundation (AISC Steel Design Guide 1). Grout must completely surround the lug plate or section and must entirely fill the slot created in the concrete. When using a rectangular or square hollow structural section or pipe section as a shear lug, a hole approximately 2 in. (50 mm) in diameter should be drilled through the base plate into the inside of the rectangular or square hollow structural section or pipe section to allow for grout placement and inspection to assure that grout is filling the entire section.

3.7.2 Shear Load Applied to Shear Lug

The applied shear load used to design the shear lug should be computed as follows:

$$V_{app} = V_{ua} - V_f$$

Where:

$$\begin{array}{ll} V_{app} & = \text{applied shear load} \\ V_{ua} & = \text{factored lateral load} \end{array}$$

V_f = resisting friction load carried by other means (that is, frictional resistance or anchor shear)

3.7.3 Design Procedure for Plate Shear Lug

The procedure for designing a plate shear lug is as follows:

1. Calculate the required bearing area for the plate
2. Determine the plate dimensions, assuming that bearing occurs only on the portion of the plate below the top of concrete
3. Calculate the factored cantilever end moment acting on a unit length of the plate assuming a uniform bearing load
4. Determine the plate thickness based on the value of the moment calculated in Step 3. The plate shear lug should not be thicker than the base plate
5. Design the weld between the plate shear lug and the base plate considering the shear and moment calculated in step 3.
6. Calculate the concrete breakout strength of the plate shear lug in shear. (The method shown in Example 3 is from ACI 349-06 Section D.11.

Note: The stress area is calculated using 45 degrees as opposed to the approximate 35 degrees used for the concrete breakout strength of anchors.)

See Appendix A, Example 3 for an example of the design of a pipe shear lug.

3.8 TENSIONING

3.8.1 General

Tensioning is inducing a tension in the anchors by elongating the shaft after the anchors have been placed, the base plate installed, and the concrete and grout have reached their design strength.

Tensioning induces preset tensile stresses into the anchors before actual loads are applied. When properly performed, tensioning can reduce deflection of the anchored item, avoid stress reversal, add to frictional resistance, and minimize the vibration amplitude of dynamic machinery. Tensioning should be considered for the following situations:

- a. Anchoring vertical vessels (towers) that are sensitive to wind. Sensitivity to wind can be determined using the following method by Freese (1959), which is illustrated in *The Pressure Vessel Design Manual* by Dennis R. Moss.

Note: The table in step 3 below was developed by converting information from the graph (Moss Figure 3-9) into the table.

1. Calculate the natural period of vibration of the vessel. For cylindrical steel shells this can be determined by the following equation:

$$T = 7.65x[10]^{-6} \left(\frac{H}{D} \right)^2 \sqrt{wD/t} \text{ (U.S. Customary Units)}$$

$$T = 2.00x[10]^{-6} \left(\frac{H}{D} \right)^2 \sqrt{wD/t} \text{ (SI Units)}$$

2. Calculate wD/t , lb/ft (N/m)

Where:

T = natural period of vibration, sec (sec)

H = height of vessel, ft (m)

D = diameter of vessel, ft (m)

w = weight per unit height, lb/ft (N/m)

t = thickness of vessel shell, ft (m)

3. Determine the critical natural period of vibration using the following table.

wD/t		Critical Natural Period of Vibration
lb/ft	N/m	sec
1,000 to 3,000	68.5 to 206	0.40 to 0.45
3,000 to 10,000	206 to 685	0.45 to 0.50
10,000 to 30,000	685 to 2,060	0.50 to 0.57
30,000 to 100,000	2,060 to 6,850	0.57 to 0.64
100,000 to 300,000	6,850 to 20,600	0.64 to 0.70
300,000 to 1,000,000	20,600 to 68,500	0.70 to 0.80
1,000,000 to 3,000,000	68,500 to 206,000	0.80 to 0.90
3,000,000 to 10,000,000	206,000 to 685,000	0.90 to 1.00

4. If the natural period of vibration, T , is greater than the critical natural period of vibration, then the vessel is considered sensitive to wind.
- b. To prevent fluctuation of the tensile stress in the anchors and therefore, eliminate fatigue concerns

- c. Where load reversal might result in the progressive loosening of the nuts on the anchors
- d. Anchoring dynamic machinery such as compressors (See API Recommended Practice 686/PIP REIE 686 and ACI 351.3R-04.)

In practical applications, the engineer should decide whether to tension the anchor by considering the advantages and disadvantages listed in 3.8.2 and 3.8.3.

3.8.2 Advantages

The advantages of tensioning are as follows:

- a. Can prevent stress reversals on anchors susceptible to fatigue weakening or the loosening of the nuts during the reversals
- b. May increase dampening for pulsating or vibrating equipment
- c. Will decrease, to some extent, the drift for tall slender structures and equipment under wind or seismic load
- d. Will increase the downward force and thus the frictional resistance for process towers, other equipment, and structural base plates

3.8.3 Disadvantages

The disadvantages of tensioning are as follows:

- a. Can be costly to install accurately
- b. No recognized code authority gives guidance on the design and installation of tensioned anchors. (There is little research in this area.)
- c. The long-term load on the anchor is questionable because of the reduction in tension due to creep of the concrete under the tension load
- d. The pre-stretch during anchor tensioning reduces the amount of inelastic stretch that may be considered effective for energy dissipation under seismic loads
- e. Typically, there is no bearing resistance to shear on the anchor because, during tensioning, the sleeve around the anchor is not filled with grout
- f. There is little assurance that the anchor will be properly installed and tensioned in the field

- g. Direct damage from tensioning is possible. That is, the tensioning itself can damage the concrete if not properly designed or if the tension load is not properly regulated
- h. It is difficult to ensure that there is consistency between the design of the anchor and the design of the vessel anchor chair; that is, to ensure that the vessel anchor chair has been designed to carry the anchor tension load.
- i. The stress level is difficult to maintain because of concrete shrinkage and creep, and relaxation of the anchor material
- j. Only alloy anchor bolts can be effectively pre-tensioned. ASTM A307 and ASTM A36 bolts do not hold their pre-tensioning values and are thus ineffective in this regard (Kulak, et al (1987))

3.8.4 Tension Load

ACI 351.3R Section 4.4.2.1 requires that a sufficient clamping force be available to maintain the critical alignment of the machine, stating that, “The clamping force should allow smooth transmission of unbalanced machine forces into the foundation so that the machine and foundation can act as an integrated structure. Generally, higher clamping forces are preferred because high clamping forces result in less vibration being reflected back into the machine. In the presence of unbalanced forces, a machine that has a low clamping force (400 psi [2.8 MPa]) at the machine support points can vibrate more than the same machine with high clamping forces (1,000 psi [7 MPa]). In the absence of more refined data, designing for a clamping force that is 150% of the anticipated normal operating anchor force is good practice. A minimum anchor clamping force of 15% of the anchor material yield strength is often used if specific values are not provided by the equipment manufacturer. Higher values are appropriate for more aggressive machines.”

With regard to anchor preload, ACI 351.3R Section 4.4.2.3 states, “To avoid slippage under dynamic loads at any interface between the frame and chock and soleplate, or chock and foundation top surface, the normal force at the interface multiplied by the effective coefficient of friction must exceed the maximum horizontal dynamic force applied by the frame at the location of the tie-down.

In general, this requires

$$F_r = \mu (T_{min} + W_a) \text{ or}$$

$$T_{min} = \frac{F_r}{\mu} - W_a$$

Where:

$$\begin{array}{ll} F_r & = \text{maximum horizontal dynamic force} \\ \mu & = \text{coefficient of friction} \end{array}$$

T_{min}	=	minimum required anchor tension (clamping load)
W_a	=	equipment weight at anchor location

An anchor bolt and concrete anchorage system that has long-term tensile strength in excess of T_{min} and maintains preload at or above this tension, coupled with a chock interface whose coefficient of friction equals or exceeds μ , will withstand the force, F_r , to be resisted. A conservative approach neglects W_a (assumes it to be zero) because distortion of the frame or block may reduce the effective force due to weight at any one anchor location.”

It is important to establish an appropriate coefficient of friction to be used. ACI 351.3R Section 4.4.2.3 reports on a research program by the Gas Machinery Research Council (GMRC) that states that the “breakaway” friction coefficients include a range from 0.22 to 0.41 for dry interface between cast iron and various epoxy products. The presence of oil in the sliding interfaces reduces the friction coefficient for cast iron on epoxy to a range from 0.09 to 0.15. Thus, maintaining an oil free interface greatly enhances frictional holding capacity. In an example, ACI 351.3R uses a coefficient of friction of 0.12 and sets the contribution of the compressor weight at zero.

3.8.5 Concrete Failure

In certain situations, high-strength anchors embedded in concrete and subjected to high tension forces may cause the ultimate capacity of the concrete to be exceeded by prematurely breaking out the concrete in the typical failure pyramid. Whether this situation can occur depends on the depth of the anchor, edge conditions, the arrangement of the base plate, and other factors. To ensure that premature concrete failure does not occur, tensioned anchors should be designed so that the concrete breakout strength of the anchor in tension is greater than the maximum tension force applied to the anchor. In the case of a stiff base plate covering the concrete failure pyramid, the stresses induced by external uplift on the concrete are offset by the clamping force and the gravity loads. For this case, the concrete breakout strength needs only to be designed for the amount that the external uplift exceeds the gravity load.

3.8.6 Vessel Anchor Chair Failure

Failure of a vessel anchor chair may occur because of failure to design it for the induced tension load of the anchor. This can be avoided by proper communication between the anchor designer and the chair designer. (See 3.8.3h.)

3.8.7 Stretching Length

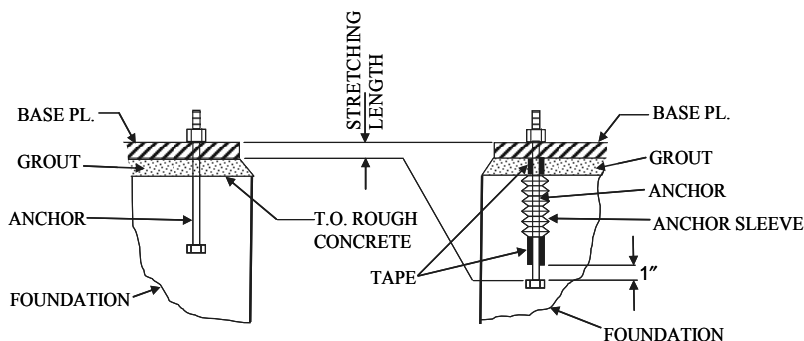
Tensioning should be implemented only when the stretching (spring) length of the anchor extends down to or near the embedded anchor head. On a typical anchor embedment, where there is no provision for a stretching length, if a tensile load is applied to the anchor, the anchor starts to shed its load to the concrete through bond.

At that time, a high bond stress exists in the first few inches of embedment. This bond will relieve itself over time and thereby reduce the load on the anchor. Therefore it is important to prevent bonding between the anchor and concrete for tensioned anchors.

Debonding of concrete to the anchor shaft may be achieved by wrapping the shaft with industrial tape to within 1 in. (25 mm) of the embedded anchor head before placing concrete (Figure 3.21). Care must be taken not to allow tape to come into contact with the head of the nut. This is the reason for stopping the tape one inch from the head. Likewise, grout must not be allowed to bond to the anchor using a similar method. Sleeved anchors that are to be tensioned should be installed using the methods mentioned above for debonding the shaft below the sleeve.

Anchor corrosion may be caused by chloride leaching from PVC pipe sleeves or tape used for debonding purposes in high temperature applications. This can be avoided by specifying polyethylene or polypropylene sleeves or tape.

The use of chairs that extend above the base plate for anchors also contributes to the available stretching length of the anchors. Chairs can be used on structural columns, process vessels, and other types of equipment



NOTE: Stretching Length = That portion of anchor allowed to freely stretch

Figure 3.21: Anchor Stretching Length

3.8.8 Tensioning Methods

Methods used to apply preload are as follows:

- Hydraulic Jacking:** Hydraulic jacking is the most accurate method and is recommended if the tension load is essential to the integrity of the design. The anchor design should accommodate any physical clearance and anchor projections required for the hydraulic equipment.

- b. **Mechanical Jacking:** Mechanical jacking is an alternative to hydraulic jacking that is used to achieve the same stretch. Typical mechanical devices called multi-jackbolt tensioners (MJTs) do this by incorporating a ring of small jack screws in the nut body that bear against a hardened steel washer, thus stretching the anchor as the jack screws are sequentially tightened. See Figure 3.22 for an example of a multi-jackbolt tensioner.



Figure 3.22: Multi-Jackbolt Tensioner

Note: Hydraulic and mechanical tensioners have to translate hydraulic pressure or torque on the small jack screws, respectively, into preload. If more accuracy for measuring the tension in the bolt is required, and hydraulic or mechanical jacking has been specified, a device such as the RotaBolt™ or the equivalent can be incorporated into the anchor design.

- c. **Proprietary Alternatives to Hydraulic or Mechanical Jacking:** Where accurate preload must be set and maintained throughout the life of the anchored item, as may be the case with some dynamic equipment, proprietary alternatives to hydraulic or mechanical jacking, such as the RotaBolt™ Load Monitor or the equivalent, should be considered. These alternatives provide a very accurate way to measure the actual stretch in the anchor and show if the preload is correct or if the load has been reduced because of relaxation or other

mechanisms such as thermal relaxation. These devices can be used with hydraulic jacking or mechanical stretching using multi-jackbolt tensioners.

- d. Torque Wrench: Because it is the rod stretch rather than the torque on the nut that matters, torque wrench tensioning provides only a rough measure of actual tension load. However, it can be the method of choice if the amount of tension load is not critical. When re-torquing the anchor, the static bond between the nut and the base plate needs to be broken to get a true measurement of the torque within the anchor. Torque values for use with oiled threads are given in API Recommended Practice 686/PIP REIE 686.
- e. Turn-of-nut: This method is a direct measure of the elongation of the anchor, which is used to calculate the tension in the anchor. However, there are questions as to the accuracy of the tension load. The tension load from stretching the anchor can be closely determined, but accounting for the compression of the concrete between the base plate and the nut at the bottom of the anchor is difficult. The required amount of nut rotation from the “snug tight” condition, as defined by AISC (AISC “Specification for Structural Joints Using ASTM A325 or A490 Bolts”, Section 8.1), required to produce a desired tensile stress in the anchor, f_t , can be determined using the following equation:

$$\text{Nut rotation (in degrees)} = 360 L_{stretch} A_{se,N} f_t n_t / (E A_d)$$

Where:

$L_{stretch}$	=	anchor stretching length (See note below)
$A_{se,N}$	=	effective cross-sectional area of anchor in tension
f_t	=	desired tensile stress
n_t	=	threads per unit length (See Table 3.2)
E	=	elastic modulus of anchor material
A_d	=	nominal area of anchor

Note: $L_{stretch}$, the anchor stretching length, is the distance between the top and bottom nuts on the anchor if the anchor is debonded from the concrete in that distance. $L_{stretch}$ may be less if it is not debonded along the full distance between the nuts.

If the anchor is to be retightened to compensate for any loss of pre-load, this method requires that nuts be loosened, brought to a “snug tight” condition, and then turned the number of degrees originally specified.

Table 3.2: Anchor Threads per Inch (n)

Nominal Anchor Diameter, in.	Threads per inch (UNC series unless noted otherwise)			
	ASTM F1554 (All Grades) Standard Order	ASTM A193/193M Standard Order, A354 & F1554 Special Order	ASTM A307	ASTM A354 Standard Order
5/8	11	11	11	11
3/4	10	10	10	10
7/8	9	9	9	9
1	8	8UN	8	8
1 1/8	7	8UN	7	7
1 1/4	7	8UN	7	7
1 1/2	6	8UN	6	6
1 3/4	5	8UN	5	5
2	4 1/2	8UN	4 1/2	4 1/2
2 1/4	4 1/2	8UN	4 1/2	4 1/2
2 1/2	4	8UN	4	4
2 3/4	4	8UN	4	4
3	4	8UN	4	4
3 1/4	4	8UN	4	4
3 1/2	4	8UN	4	4
3 3/4	4	8UN	4	4
4	4	8UN	4	4

3.8.9 Relaxation

This committee has done some theoretical analysis of the effect of concrete creep and shrinkage on the tension load on anchors. The amount of creep and shrinkage depends on mix design, physical characteristics of the aggregate, concrete age when exposed to drying, concrete age when exposed to the tension load, size and shape of member, amount of steel reinforcement, environmental exposure conditions (such as relative humidity, temperature, and carbon dioxide content of the air), and curing conditions. The following coefficients are rough averages for the maximum creep that can be expected:

$$E_{cr} = 1.0 \times 10^{-6} \text{ in/in/psi } (145 \times 10^{-3} \text{ mm/mm/kPa})$$
$$E_{sh} = 600 \times 10^{-6} \text{ in/in } (600 \times 10^{-6} \text{ mm/mm})$$

Where:

- E_{cr} = coefficient for creep
- E_{sh} = coefficient for shrinkage

As mentioned, the age of the concrete at the time the anchor is tensioned affects the amount of creep and shrinkage; thus if the anchor is tensioned after the concrete has cured, there will be less creep and shrinkage, which will result in less loss of tension in the anchor. The reductions in expected creep and shrinkage due to age at loading are shown in the Handbook of Concrete Engineering, Figures 6-38 and 6-40 (Fintel, 1974).

Using these coefficients along with the reductions from the Handbook of Concrete Engineering Figures 6-38 and 6-40 and assumptions below, Table 3.4 was developed.

Assumptions:

- a. Only the average compression load was considered (the high compression over the anchor or nut head was not considered). The area of concrete that was considered for the compression load was arbitrarily taken as 250 square in. (161,290 square mm).
- b. No effect of reinforcing steel was considered

Table 3.4: Loss in Tension for Various Scenarios

Tightening Scenario	Anchor Diameter & Material	Tension Load	Tension after 10 years	Loss in tension
Anchor tightened 28 days after concrete placement	2 in. (50.8 mm) diameter Mild Steel (ASTM F1554 Gr 36)	50 kips (222.4 kN)	24.55 kips (109.2 kN)	50.9 %
Anchor tightened 90 days after concrete placement	2 in. (50.8 mm) diameter Mild Steel (ASTM F1554 Gr 36)	50 kips (222.4 kN)	30.53 kips (135.8 kN)	38.9 %
Anchor tightened 90 days after concrete placement	1 3/8 in. (34.9 mm) diameter high-strength steel (ASTM F1554 Gr 50)	50 kips (222.4 kN)	39.48 kips (175.6 kN)	21.0 %
Anchor tightened 90 days after concrete placement then retightened 90 days later	1 3/8 in. (34.9 mm) diameter high-strength steel (ASTM F1554 Gr 50)	50 kips (222.4 kN)	41.77 kips (185.8 kN)	16.5 %
Anchor tightened 90 days after concrete placement then retightened 1 year later	1 3/8 in. (34.9 mm) diameter high-strength steel (ASTM F1554 Gr 50)	50 kips (222.4 kN)	44.97 kips (200 kN)	10.1 %
Anchor tightened 90 days after concrete placement then retightened 1 year later	1 3/8 in. (34.9 mm) diameter high-strength steel (ASTM F1554 Gr 50) Anchor length reduced by 20 in. (508 mm)	50 kips (222.4 kN)	45.96 kips (204.4 kN)	8.1 %

- c. Except for the last case, the anchor length embedded in the concrete was assumed to be 40 in. (1,016 mm) and the overall grip length of the anchor was taken as 54 in. (1,371.6 mm). For the last case, the anchor length embedded in the concrete was assumed to be 20 in. (508 mm) and the overall grip length was assumed to be 34 in. (863.6 mm).

The following information can be deduced from Table 3.4:

- a. The longer one waits to do the tensioning after the concrete is placed the smaller the tension loss
- b. High-strength anchors will reduce the amount of tensioning loss. However, in order to take advantage of this, one has to reduce the diameter of the anchor for the same tension load while keeping the bolt lengths the same. This has the effect of increasing the stretch length on the anchor, so that the same reduction in length will result in less tension loss.
- c. Retensioning the anchor 90 days after the initial tightening will further reduce the tension loss and retensioning 1-year after the initial tightening will reduce the tension loss even further
- d. Using shorter rather than longer anchors will reduce the amount of tensioning loss

3.8.10 Tightening Sequence

Anchors should be tightened to the design tension load in three equal stages (Bickford 1995). Tensioning of anchors is to be performed in a criss-cross pattern. See Figure 3.23 for a circular anchor pattern sequence.

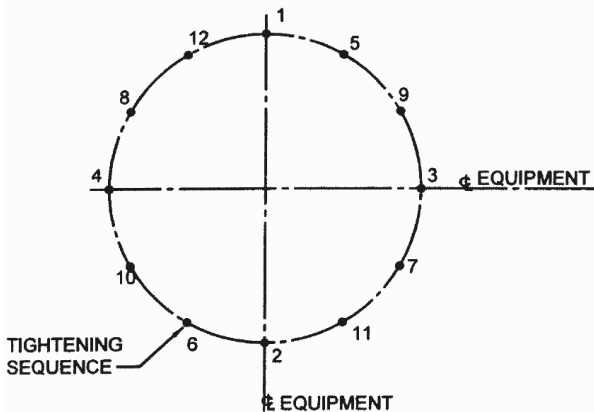


Figure 3.23: Anchor Tightening Sequence

Refer to API Recommended Practice 686/PIP REIE 686, Part 5, Annex 6, for information on the sequence for tightening and leveling anchors for machinery.

3.8.11 Monitoring Tension

When deemed necessary, the tensioning can be measured by a built in load monitor such as the “RotaBolt™ Load Monitor or equivalent. A load monitor gives the owner a means of measuring and correcting any loss of tensioning over time. A good practice is to correct any loss of tension at least annually.

3.9 WELDED ANCHORS FOR EMBEDDED PLATES

3.9.1 General

Steel embedded plates are often used to transfer loads from structural members to concrete structures or foundations. Such plates are often cast-in-place for constructability and to provide a smooth surface for attachment. These plates are attached to the concrete with welded anchors, which typically consist of headed studs, headed anchors, weldable rebar, or shear lugs; they can be designed to resist applied tension, shear, and moment. Welding should be compatible with the anchor type.

Embedded plates may be designed using one type of anchor or a combination of different types. A combination may be desirable when large one-directional moments are encountered. The embedded plate thickness should be designed to carry the tension, compression, shear, and moment to the anchors in a manner similar to that used to design the thickness of a column base plate.

The guidelines for designing welded anchors for embedded plates presented herein are based on ACI 318 Appendix D, and the user should refer to that document for details not included herein. Shear lugs on embedded plates are similar to those discussed in 3.7 except without grout, and will not be discussed further.

3.9.2 Headed Stud Anchors

AWS D1.1/D1.1M requires studs to be Type B made from cold drawn bar stock conforming to ASTM A108. Since headed studs are relatively short, it is not practical to consider reinforcing steel in design as might be the case with longer anchors.

3.9.3 Headed Anchor Rods

Headed anchor rods may be used in lieu of headed studs to increase the embedment and assure ductile design, or if studs are not available. Design is similar to that for headed studs. The user must ensure that the anchor rod is made of a weldable material.

If, as in the case of a column pedestal, there is insufficient concrete to resist the tension or shear, reinforcing steel can be designed as anchor reinforcement. (See 3.5.) This can be considered a ductile design since both the anchor and the reinforcing steel are ductile.

3.9.4 Rebar Anchors

Welding rebar to the embedded plate is another alternative to headed studs. Rebar welding may require special electrodes or specification of a proper rebar material, such as ASTM A706/A706M. Rebar length can be established using l_d or l_{dh} in order to develop the bar strength if a ductile design is required.

Both rebar and headed anchor rods welded to an embedded plate can be cumbersome to handle since the lengths can be long. If this is undesirable for shipping or coating, a threaded coupler can be welded to the embedded plate and the rebar or anchor rods threaded to match. In this case the coupler weld would have to be sufficient to transfer the tension and shear loads.

3.9.5 Tension Considerations

The minimum of the following strengths should be taken as the tensile design strength, ϕN_n , of the anchorage.

- Stud Steel Strength: Type B welded studs are ductile steel elements. ϕ should be selected in accordance with ACI 318.
- Concrete Breakout Strength: The strength reduction factor, ϕ , should be in accordance with ACI 318 for tension.
- Concrete Side-face Blowout Strength: The nominal side-face blowout strength, N_{sb} or N_{sbg} , for a single or multiple headed anchors with deep embedment close to an edge ($c_{al} < 0.4h_{ef}$), should be checked.
- Concrete Pull-out Strength: Concrete pull-out strength should be checked in order to prevent local crushing of concrete at the head. Such crushing will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

3.9.6 Shear Considerations

The minimum of the following strengths should be taken as the shear design strength, ϕV_n of the anchorage.

- Steel Strength: Steel strength of welded anchors should comply with the design requirements of ACI 318 Appendix D. ACI 318 Appendix D provides two equations for the calculation of shear strength, (D-19) and (D-20).

Equation (D-19) is for cast-in headed stud anchors. It is based on the fixity of the anchor to the embedment and is appropriate for use in designing anchors welded to embedded plates.

b. Concrete Breakout Strength:

For shear force perpendicular to an edge, the capacity of the anchor group is allowed to be checked with an edge distance based on the anchor farthest from the edge as stated in ACI 318 for anchors welded to a plate.

For shear force parallel to an edge, the capacity of the anchor group is allowed to be twice the value of the shear capacity calculated perpendicular to an edge.

For anchors located at a corner, the minimum capacity calculated above, for parallel and perpendicular loads, should be taken as the design capacity, ϕV_{cbg} .

c. Concrete Pryout Strength: Concrete pryout strength should be calculated in accordance with ACI 318.

3.9.7 Interaction of Tensile and Shear Forces

Interaction between tensile and shear forces should be in accordance with ACI 318.

3.9.8 Seismic Considerations

This section is applicable for Seismic Design Category C, D, E, or F. When anchor design includes seismic forces, the anchor design strength associated with concrete failure modes should be reduced in accordance with ACI 318 requirements. The philosophy for the design of steel embedments subject to seismic loads is that the system should have adequate ductility. Anchor strength should be governed by ductile yielding of a steel element. If the anchor cannot meet these ductility requirements (which is the case for most embedded plates with welded studs because of relatively short embedment depth and close spacing), then either the attachment is designed to yield (ACI 318 Section D.3.3.5) or the calculated anchor strength is substantially reduced to minimize the possibility of a brittle failure (ACI 318 Section D.3.3.6). Alternatively, longer welded rebar may be used as opposed to welded studs. (See 3.9.4.)

3.9.9 Examples of Design of Welded Anchors for Embedded Plates

There are several examples of single and multiple studs welded to embedded plates under tension, shear, moment, and combinations of these loads in ACI 349.2R. Engineers are encouraged to use this reference when the need arises.

3.10 CONSIDERATIONS FOR VIBRATORY LOADS

3.10.1 General

Vibratory loads are only a consideration in the design of anchorage in petrochemical facilities if they are high-cycle, that is, more than 2×10^6 cycles. Neither ACI 318 nor ACI 349 addresses the design of anchors for high-cycle fatigue. Fatigue testing of adhesive anchors indicates that fatigue of the bonding materials is not critical.

Fatigue behavior is the most critical for anchor groups having anchors installed through holes in a steel plate or other fixture, since there is significant potential for unequal shear load distribution. Where fatigue due to shear is determined to be important, it is advisable to eliminate movement in the connection via welded thickened washers or supplemental grouting of the annular gap.

Fatigue due to tension loading can be reduced through tensioning of the anchor. (See 3.8.) Tensioning requires sufficient anchor length to develop strains that are large compared to the strain associated with concrete relaxation and creep. The residual tension in the anchor should exceed the peak cycling load. The resistance to fatigue is directly related to the ratio between the minimum and the maximum cyclic stress. Figure 3.24 illustrates this point.

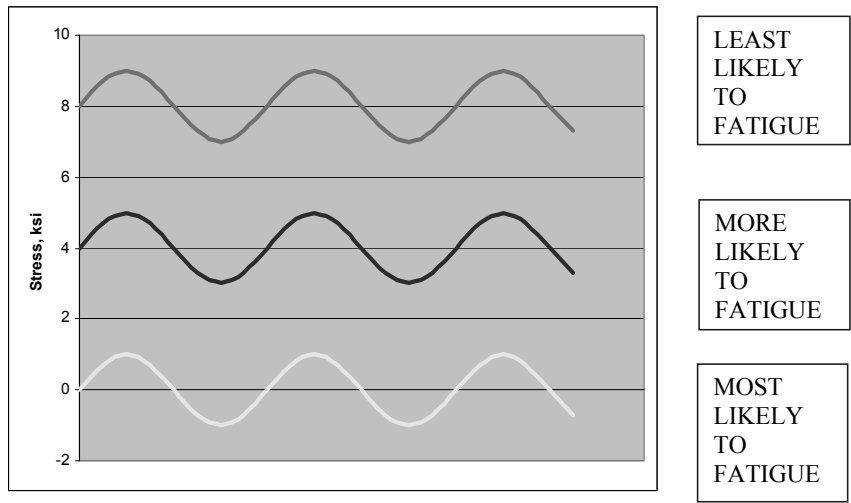


Figure 3.24: Effect of Preloading Anchors on Fatigue

The lower curve is for no static preload, the middle curve is for a static preload of 4 ksi (27.6 MPa) and the upper curve is for a static preload of 8 ksi (55.2 MPa). The cyclic load amplitude of 2 ksi (13.8 MPa) is the same in all cases. The ratio of the

minimum to the maximum cyclic load for the lower curve is $-1/1 = -1$, the ratio for the middle curve is $3/5 = 0.6$, and the ratio for the upper curve is $7/9 = 0.778$.

For these load cases, the load case illustrated by the upper curve is the least likely to fatigue, the case illustrated by the middle curve is more likely to fatigue, and the case illustrated by the lower curve is the most likely to fatigue, since it has complete load reversal.

3.10.2 Rules for Avoiding Fatigue Failure

Fabrication processes (forming, cutting, welding, heat treatment, and galvanizing) and the thread production method and configuration are critical for the behavior of threaded connections subjected to high-cycle fatigue loading. This is particularly important in order to eliminate crack initiation, particularly at the first thread inside the nut, where tension fatigue failures typically occur due to the increased stress at this location. Thus, the following rules should be observed in order to avoid fatigue failure.

- a. Use the proper grade of nut with the bolt and ensure full thread engagement in the nut
- b. Use rolled threads to avoid stress risers in the threads and shot peening to induce residual compressive stresses in the bolt
- c. Use spherical washers beneath the nut to avoid inducing bending loads in the bolt when it is tensioned due to lack of parallelism between the bottom of the nut and the bolted parts
- d. Use the fewest possible elastic materials in the joint (gaskets, chocks, etc.) in order to maintain anchor preload and avoid long term relaxation
- e. Avoid bending and shear loads on the anchors. Anchors loaded in pure tension are the least likely to fatigue.
- f. Use the longest bolt possible to get the greatest strain (stretch) for the applied preload. Bolted joints are held together by the elastic energy stored in the bolt. The amount of energy stored goes up as the square of the stretch length, which in turn increases linearly with length. For example, a 4-in. (101.6 mm) bolt stretched to 70 percent yield will stretch twice as far as a 2-in. (50.8 mm) bolt stretched to 70 percent yield, but the longer bolt contains four times the elastic energy as the shorter one.
- g. Put the maximum possible preload on the anchors.

Note: Many practitioners use 80-90% of the yield stress for 40 ksi steel anchors and 50-70% of the yield stress for ASTM A193 Grade B7 steel anchors because of the potential for stress corrosion cracking at higher stresses.

When maintaining the prestress tension is important, a load monitor such as the RotaBolt® Load Monitor or equivalent can provide an easy method of checking that there has been no loss of tension that would allow a load reversal.

3.11 CONSIDERATIONS FOR SEISMIC LOADS

3.11.1 General

The flow chart shown in Figure 3.25 provides clarity to the procedure of designing anchorages for earthquake considerations. This flow chart gives a logical procedure for considering the requirements of ACI 318 Appendix D, AISC 341, and ASCE/SEI 7 regarding earthquake design.

Although ductile anchorage is recommended for all anchorages, seismic detailing is required by code only for structures assigned to Seismic Design Categories C, D, E, and F, regardless of the governing load combination.

Unless otherwise required, anchorages should be designed to resist seismic loads from all load combinations that include non-amplified seismic loads in accordance with the applicable building code. An example where an anchorage should be designed for member strength or amplified loads is a column base connection designed in accordance with AISC 341, *Seismic Provisions for Structural Steel Buildings*. Amplified seismic loads are loads that result from load combinations that include the overstrength factor Ω . An example of member strength design is designing a connection for the tensile strength of the brace for a Special Concentrically Braced Frame (SCBF) in accordance with AISC 341. When a connection with anchorage is not required to be designed for member strength or amplified seismic loads the nominal capacities of anchors for structures that have been assigned to Seismic Design Categories C, D, E, or F should be subject to the following additional requirements:

- a. To reflect the uncertainty associated with anchorage resistance in a concrete structure or foundation that is undergoing inelastic deformations, anchorage design strength capacity in tension and shear associated with concrete failure modes should be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where N_n and V_n are the nominal strengths associated with the controlling concrete failure modes in tension and shear, respectively, as determined in accordance with ACI 318 Appendix D. If rebar is used to develop anchor forces it should also be designed in accordance with the above guideline.
- b. In order to assure a ductile anchorage, the concrete strength as determined in paragraph (a) (that is, concrete breakout, pullout, and side-face blowout) should be greater than the strength of the ductile steel embedment element.
- c. Where ductility in the anchor cannot be achieved, it is acceptable to force ductile yielding in the attachment, for instance the base plate, by designing the

attached component to yield at forces no greater than the design strength of the anchors as described in paragraph (a).

- d. Where yielding in the attached component or in the anchor cannot be achieved, it is acceptable to design the anchorage for 2.5 times the seismic loads transmitted by the attachment. ACI 318 Section D.3.3.6 strength reduction factors should not be used in conjunction with the 2.5 amplification factor.

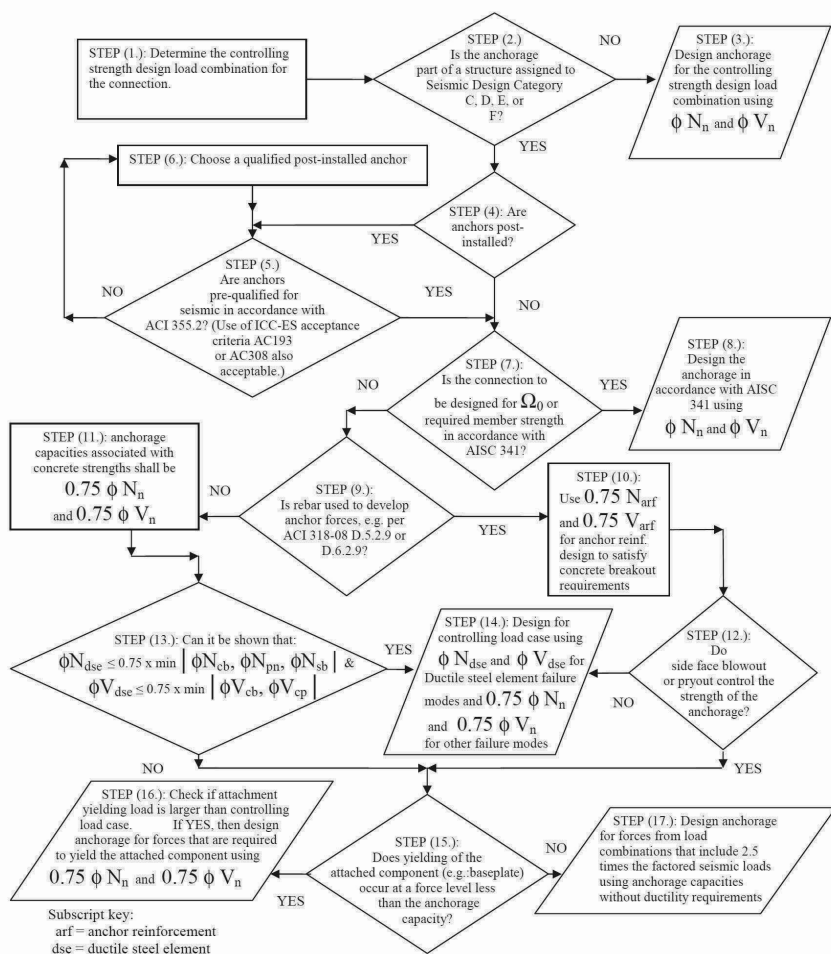


Figure 3.25: Flow Chart for Seismic Design of Anchorage

3.11.2 Connections Designed in Accordance with AISC 341

Seismic detailing of structural steel is specified in AISC 341. Steel structures assigned to Seismic Design Categories D, E, and F should be detailed in accordance with AISC 341 unless covered by exceptions provided in ASCE/SEI 7 Chapter 15. Steel structures in Seismic Design Categories B and C designed in accordance with ASCE/SEI 7 Table 12.2-1 Part H., "Steel Systems Not Specifically Detailed for Seismic Resistance, Excluding Cantilever Column Systems" are exempt from AISC 341 detailing requirements, as are all structures in Seismic Design Category A.

Column bases, including the anchorage, of structures conforming to AISC 341, are designed in accordance with Chapter 8 of that document. AISC 341 Section 8.5a requires that the axial capacity of the column base be taken as the sum of the forces and member capacities of all elements framing into the base. AISC 341 Sections 8.5b and 8.5c require column bases to be designed for the column expected shear strength and column expected flexural strength, respectively. Typically, the anchorage strength demands determined in accordance with AISC 341 (based on member strengths or overstrength factors) will govern the design, as opposed to the strength demands determined in accordance with the non-amplified seismic load combinations of ASCE/SEI 7. Anchorage for components is typically required to be designed for a higher seismic load than anchorage for items that are not components. This is due to the nature of seismic demands on components during earthquakes. The purpose of the additional requirements for component anchorage is to provide a continuous load path of sufficient strength and stiffness between the component and the supporting structure.

3.11.3 Nonstructural Components

Nonstructural components are subject to special requirements for anchorage that are not specifically addressed in this report. ASCE/SEI 7 Chapter 13 provides specific anchorage requirements for components and defines detailing and design parameters for components such as piping, conduit, cable tray, and small equipment. With the exception of storage racks, the dividing line between nonstructural components as addressed in ASCE/SEI 7 Chapter 13 and nonbuilding structures as addressed in ASCE/SEI 7 Chapter 15 is made on the basis of the weight of the component as a percentage of the overall structure weight.

3.11.4 Pedestal Anchorage

Reinforced concrete pedestals designed to receive loads from supported steel structures, tanks, and vessels are typically required to transfer large concentrated forces at the anchorage interface, typically at the top of the pedestal. The design of such anchorages is complicated by the reduced edge distances and anchor spacing as well as the need for large tension and shear capacity to accommodate the calculated lateral and overturning forces in the attachment. For typical cases, additional ties as shown in Figure 3.26 may be adequate to facilitate shear transfer. Special cases may

require other solutions such as shear lugs or side plates. As previously discussed, the transfer of tension forces to the vertical pedestal reinforcing will likely be governed by the large splitting stresses generated around the anchorage, and as such, a design of the anchor embedment corresponding to development/splice length in accordance with the provisions of ACI 318 Chapter 12 should be considered. It is also recommended that additional ties be provided at and directly above the level of the head of headed anchor bolts to take up the bursting forces generated around the anchor head.

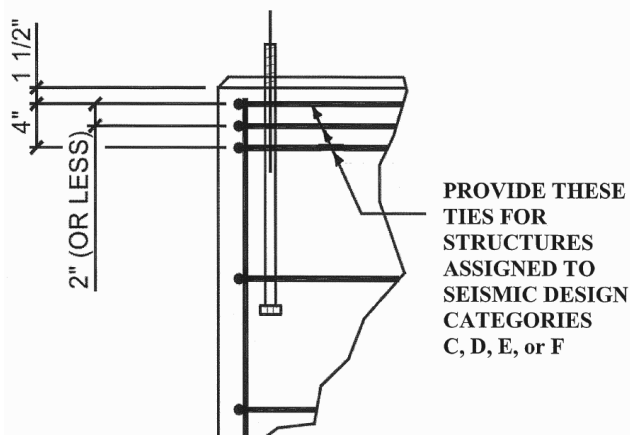


Figure 3.26: Seismic Pedestal Ties for Anchorage.

As noted previously, anchor reinforcement properly designed in accordance with ACI 318 Appendix D precludes the need to calculate concrete breakout strength. Proper detailing is critical to assure load transfer from the anchorage to the reinforcement. In Appendix D this is accomplished by requiring that the anchor reinforcement be developed on both sides of the theoretical crack plane corresponding to concrete breakout. Note that for tension-loaded anchors where splitting of the concrete will likely govern the anchor strength (that is, anchors in the top of a column or pedestal with limited edge distance), it may be advisable to treat the load transfer from anchor to reinforcement as a non-contact lap splice and to refer to the development length provisions of ACI 318 Chapter 12. It is also recommended that those provisions of ACI 318 (for example, 12.2.5) that permit the reduction of development length based on the provision of more than the required reinforcing area ($A_{s, \text{required}}/A_{s, \text{provided}}$) should not be used when developing anchor reinforcement to resist anchorage-induced seismic loads.

3.11.5 Seismic Design of Vertical Vessel Anchors

Historically, the foundation anchors for tall vertical vessels and stacks have tended to stretch beyond yield when subjected to strong ground motion, which probably prevented collapse of these vessels. Based on this experience, it is recommended that

these anchors be designed with ductile embedment into the foundation. (Special care should be taken not to significantly oversize the anchors.) Oversizing could cause the anchors to not yield during a seismic event, thus increasing the load on the foundation and creating overturning moments in the foundation beyond those assumed in the design.

In specific instances where anchor elongation is required for inelastic displacement of the supported equipment or structure, a minimum stretch length of anchors should be calculated and detailed. These provisions are particularly important for facilities that rely primarily on the foundation and anchors for ductility, such as fixed base cantilever stacks and skirt supported vertical vessels. It is industry practice to use a minimum stretch length of 12 anchor diameters in these situations. Some examples of detailing provisions that provide anchor stretch are: using extended anchors with high chairs on vessel skirts, providing full length sleeves filled with elastomeric material, and using industrial tape or grease to break the concrete bond on the anchor shaft.

A procedure for determining the minimum stretch length of vertical vessel anchors is shown in Figure 3.27. In order to use this procedure the static displacement at the top of the vertical vessel due to the Equivalent Lateral Force Procedure seismic loads, Δ_s , should first be calculated. The amplified displacement at the top of the vessel, Δ_t , equals Δ_s plus Δ_{ie} . The inelastic portion of the vessel amplified displacement, Δ_{ie} , is assumed to be caused by anchor bolt stretch because inelasticity should not occur in the vessel or skirt and foundation rocking can lead to instability. The elongation length of the anchor bolts, Δ_a , required to cause the inelastic portion of vessel amplified displacement can be found from the geometry shown in Figure 3.27. The required anchor bolt stretch length, $L_{stretch}$, can be determined by assuming a reasonable amount of anchor bolt elongation strain, e_a .

When the anchors extend only into the pedestal, the pedestal dowels should be designed to transfer the overturning moment into the footing (minus the resisting moment developed by the pedestal self weight). The dowels should be able to develop an overturning moment equivalent to the overturning moment based on anchor strength. If the anchor bolts extend into the footing, which is often the case for very tall vessels, pedestal dowels do not transfer overturning moment to the footing, and in this case it is only necessary to provide a nominal number of dowels to minimize concrete cracking.

The anchors should be designed to resist the entire seismic shear load at the base if the overturning moment from the seismic forces, acting alone, cannot develop the required frictional resistance between the vessel base and the top of pedestal. In most cases, this frictional resistance is adequate to resist seismic shear forces; therefore, there is no shear force transferred through the anchors.

V_f = frictional resistance force

In order to avoid shear loading on the anchor bolts:

$$V_u \leq \phi V_f$$

Where:

V_u = factored shear load at base of vessel, calculated using load factors in load combinations for uplift cases (see loading combinations and load factors – Strength Design)

ϕ = strength reduction factor = 0.75

In order to minimize the need for excessive bolt edge distance or shear reinforcement when the anchors are designed for seismic shear, the bolts on a 90-degree arc in the direction of the horizontal force are ignored, and the horizontal seismic force is then carried only by the bolts on the remaining 270-degree arc (that is, three-fourths the total number of bolts). (See Figure 3.28.) If this force transfer methodology is followed, special detailing will be required to transfer the lateral load from the vessel to the anchors and foundation.

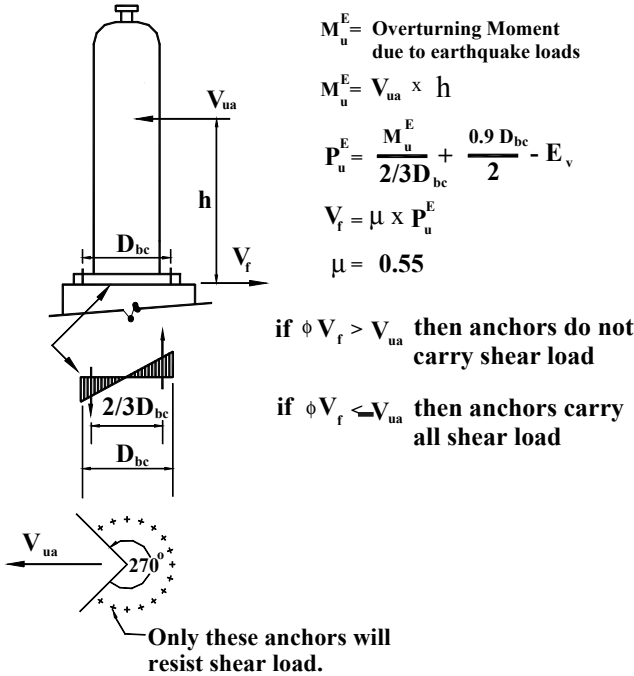


Figure 3.28: Shear Transfer Methodology for Vertical Vessel Anchors

3.11.6 Other Anchorage Seismic Design Considerations

Although double nuts for anchors are sometimes recommended for vibratory equipment or tower vessels they are not necessary for anchors resisting seismic loads.

Anchors with upset threads (see 2.2.5.3) provide the advantage of assuring that yielding will occur outside of the threaded portion of the anchor. Upset threads however, are not necessary for anchors resisting seismic loads.

In regions of frequent high seismic events it is recommended that anchors be provided with full-length sleeves (Figure 3.2) or other proprietary canister anchors. Benefits of anchors of this nature are a full length anchor stretch and many of the proprietary anchors allow for the rod replacement after a seismic event where the rod has been inelastically stretched or damaged.

3.12 CONSTRUCTABILITY CONSIDERATIONS

The following design practices should be implemented to facilitate constructability – including minimizing the need for future anchor repair or replacement.

- a. Specify the use of anchor installation tolerances provided in PIP STS03001, *Plain and Reinforced Concrete Specification*, Section 4.3.5.3.
- b. If coated anchors (galvanized or other coating) are not used, investigate long-term anchor corrosion issues for uncoated anchors in the design phase to determine whether a corrosion allowance size increase is warranted. In corrosive environments, such oversizing will minimize the need for future anchor repair or replacement (API Std 620 Section 5.11.2.3). Coated anchors (galvanized or other coating) are preferred.
- c. Use the structural base plate hole diameters shown in Table 3.3 to minimize impacts of misalignment. While the hole diameters listed in this table are not consistent with the current AISC recommendations they are consistent with industry practice and have been successfully used for years. Larger holes may be used if the annular spaces are grouted or specially designed thickened washers are specified (Fisher and Kloiber, 2004).
- d. Use doubly symmetric anchor layout patterns wherever possible to minimize the potential for orientation layout errors in the field (Fisher and Kloiber, 2004)
- e. Use conservative anchor projection and thread lengths to minimize the impact of anchors being installed “too short” in the field (Fisher and Kloiber, 2004)
- f. Minimize the number of setting patterns, anchor lengths, and diameters when designing anchor layouts for column base plates. Although the resulting

designs may be conservative in some cases, the detailing and installation process will be simplified, and the potential for confusion and installation errors in the field are minimized (Swiatek, Whitbeck, and Shneur, 2004).

- g. Specify slightly oversized column base plates – thereby allowing room for drilling of oversized holes should anchor misalignment occur in the field (Swiatek, Whitbeck, and Shneur, 2004)
- h. Provide at least 1 in. (25 mm) design clearance between the outside edge of the anchor (or associated bottom plate or washer) and the nearest vertical or tie bar when installing anchors within tied, vertical bar arrangements. Fabrication and installation variances could result in a reinforcement installation that is slightly “tighter” than specified on the design drawings and could result in interference issues with the anchors if not accounted for by providing the 1 in. (25 mm) design clearance.
- i. Specify the construction sequence on construction drawings if improper sequencing could impact the anchor installation. For example, early installation of an adjacent wall could hinder the ability to install anchors for a column base plate. Engineering drawings should specify that the column is to be installed prior to placement of the wall (Swiatek, Whitbeck, and Shneur, 2004).
- j. If practical, design anchors so that they do not extend into the footing but remain in the pedestal. This is very desirable for construction.
- k. Where dense rebar is located in foundations, clearances for anchors or embedded items should be checked (PIP STE01100)
- l. If projecting anchors can interfere with construction or maintenance activities, use of coupled type anchors should be considered (PIP STE01100)

Table 3.3: Recommended Maximum Sizes for Anchor Holes in Base Plates and Minimum Fabricated Washer Sizes

Anchor Diameter, in. (mm)	PIP and this Committee's Recommended Base Plate Hole Diameter, in. (mm) (See note 3.)	Minimum Washer Size, in. (mm) (See note 3.)	Minimum Washer Thickness, in. (mm) (See note 3.)
1/2 (13)	13/16 (21)	See note 4	See note 4
5/8 (16)	15/16 (24)	See note 4	See note 4
3/4 (19)	1 1/16 (27)	See note 4	See note 4
7/8 (22)	1 3/16 (30)	See note 4	See note 4
1 (25)	1 1/2 (38)	2 5/8 (67)	5/16 (8)
1 1/4 (32)	1 3/4 (44)	2 7/8 (73)	3/8 (10)
1 1/2 (38)	2 (51)	3 1/8 (79)	1/2 (13)
1 3/4 (44)	2 1/4 (57)	3 3/4 (95)	1/2 (13)
2 (51)	2 3/4 (70)	4 1/2 (114)	3/4 (19)
2 1/4 (57)	3 (76)	4 3/4 (121)	3/4 (19)
2 1/2 (64)	3 1/2 (89)	5 (127)	7/8 (22)
2 3/4 (70)	3 3/4 (95)	5 1/4 (133)	See note 5
3 (76)	4 (102)	5 1/2 (140)	See note 5

Notes:

1. Base plate hole size recommendations are based on the AISC ASD Manual, ninth edition, adjusted such that standard ASTM F436/ASTM F436M washers will cover the base plate holes. They are also recommended in PIP STE05121 – Anchor Bolt Design Guide and by this committee. AISC hole size recommendations in the current AISC Manual, thirteenth edition, have been revised and are larger.
2. Washers for the oversized holes should be fabricated from ASTM A36/A36M steel plate. They may be round, square, or rectangular, and generally have holes that are 1/16-in. (1.6 mm) larger than the anchor. The thickness must be suitable for the forces to be transferred. Minimum washer sizes and thicknesses are shown in the table. (AISC Manual, 13th Edition, Part 14 and Table 14-2). Washers which will be welded to the base plate in order to transmit shear must be thickened to avoid overstressing in bearing and to be sufficiently thick for fillet welding.

Note: Hardened washers recommended in 2.2.2 are in addition to the fabricated ASTM A36/A36M washers.

3. If the responsible engineer believes that the contractor can place anchors to a tight enough tolerance to allow base plate holes only 3/8 in. (10 mm) larger in diameter than the anchor, then the base plate hole diameters can be reduced to

3/8 in. (10 mm) larger than the anchor and the ASTM A36/A36M fabricated washers can be eliminated.

4. Fabricated ASTM A36/A36M washers are not required for anchors 7/8 in. (22 mm) and smaller if hardened washers are used and the recommended hole diameter is used.
5. Fabricated plate washer thickness for 2 3/4 in. (70 mm) and 3 in. (76 mm) diameter anchors should be specifically designed for the application.

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CHAPTER 4

POST-INSTALLED ANCHOR DESIGN

4.1 INTRODUCTION

The term post-installed anchor is used to describe devices installed in holes drilled in hardened concrete for the purpose of transferring loads. Post-installed anchor types include expansion, undercut, screw, grouted, and adhesive anchors.

The use of post-installed anchors in petrochemical facilities ranges from pipe hangers to vessel anchorage. Design considerations associated with anchoring trapeze hangers, emergency lighting, and guardrails are quite different from those associated with large-scale foundation anchors. Inasmuch as the full range of anchoring challenges is typically present in a petrochemical facility, an understanding of the functional characteristics of the various post-installed anchor types is provided here.

Classification of anchor types is generally based on the mechanism of action for transfer of tension loads and the manner of setting the anchor. Figure 4.1 outlines one such classification system.

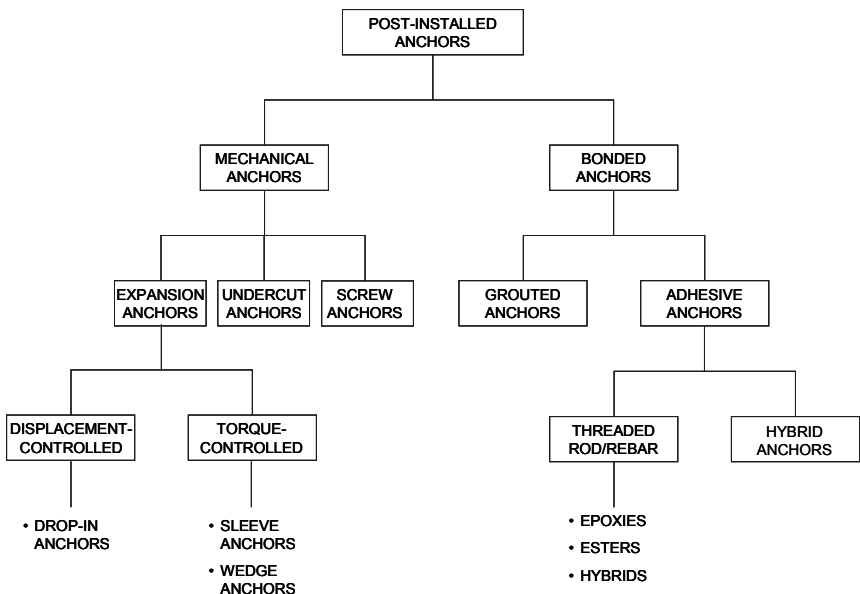


Figure 4.1: Post-Installed Anchor Classification

4.2 POST-INSTALLED MECHANICAL ANCHORS

4.2.1 Expansion Anchors

Expansion anchors transfer tension loads to the base material via friction between the expansion elements of the anchor and the wall of the hole. The magnitude of the friction resistance is directly proportional to the degree of expansion force developed by the anchor. Expansion forces are produced in response to the relative movement of sloping surfaces within the anchor mechanism. The manner in which this relative movement is produced is important for distinguishing the anchor function in response to tension loads. The two most common mechanisms for producing expansion forces are represented by displacement-controlled and torque-controlled anchors:

- a. Drop-in anchors are the most common representative of displacement-controlled anchors. They are set by driving a conical plug into the body of the anchor (Figure 4.2a). The interior of the anchor body is sloped, and slits in the anchor body permit outward expansion of the shell against the hole wall in response to the position of the plug within the anchor body. Full set of the anchor is determined by the relative position of the top of the plug with respect to the upper lip of the anchor shell. The level of expansion force developed by the anchor, and thus its ability to resist external tension loads, is at a maximum immediately after setting and decreases thereafter as a function of creep and relaxation.
- b. Torque-controlled expansion anchors, which include wedge anchors and sleeve type anchors, are set by the application of torque to the anchor, resulting in vertical movement of a conical element and outward expansion of the sleeve element(s) surrounding the cone (Figure 4.2b). Critical to the function of these anchors is the relationship between the friction developed at the hole wall and the friction between the inclined surfaces of the anchor (internal friction). Reexpansion of the anchor in response to external tension loads is called follow-up expansion. It is this behavior which differentiates torque-controlled expansion anchors from displacement-controlled expansion anchors.

4.2.2 Undercut Anchors

Undercut anchors transfer tension loads to the base material via bearing rather than friction, and as such offer a generally more reliable mechanism for resisting applied loads. This is achieved by producing a hole geometry (that is, an undercut) that permits the anchor to key into the base material.

Undercut anchors represent a superior class of post-installed mechanical anchor. By relying on bearing to transfer tension loads, they offer several advantages over expansion anchors:

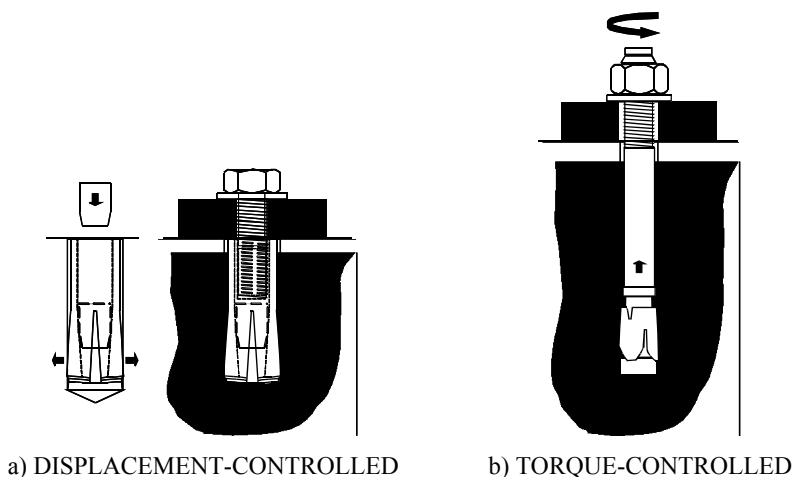


Figure 4.2: Expansion Anchor Types

- a. They do not require large expansion forces to set properly. This in turn allows them to be set closer to free edges or to other anchors without precipitating splitting failures.
- b. They are much more tolerant of variations in the base material, such as cracking or other localized defects

Many undercut anchors are developed around the concept of using a specialized tool to prepare the undercut in a previously drilled hole. These systems are capable of producing excellent anchorages at a variety of embedment depths. They are particularly suited to retrofit applications, although they may be costly and difficult to install because of the complexity of the undercutting tools and the time required to prepare the undercut and set the anchors properly.

Self-undercutting designs produce the undercut in the process of setting the anchor through a combination of drilling and hammering action, thus reducing the time and cost associated with installation and ensuring good compliance between the undercut geometry and anchor bearing surfaces.

Note: The anchor embedment is usually fixed for a given anchor diameter (Figure 4.3).

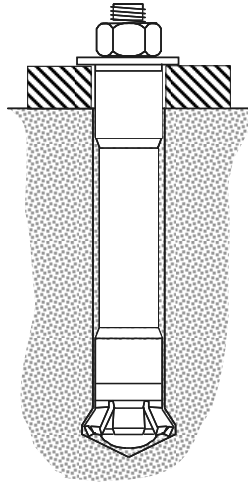


Figure 4.3: Undercut Anchor

4.2.3 Screw Anchors

Screw anchors (Figure 4.4) transfer tension loads via the interlock of the screw threads with matching female threads cut into the concrete by the hardened forward threads. They are often used for light and medium-duty applications where speed and ease of installation are a factor. The high hardness required for cutting the threads into the concrete makes screw anchors susceptible to hydrogen embrittlement and stress corrosion, particularly under the head, and caution should be exercised where they are used in unprotected environments. Depending on the depth of the threads, screw anchors may have superior tension resistance relative to other expansion anchor types in cracked concrete conditions.

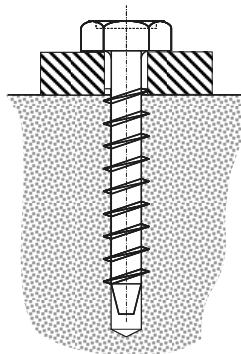


Figure 4.4: Screw Anchor

4.3 POST-INSTALLED BONDED ANCHORS

4.3.1 Grouted Anchors

Grouted anchors are distinguished from adhesive anchors in that they typically consist of a smooth-shanked anchor (headed or un-headed) embedded in bonding material (cementitious, polymer, or hybrid grout) in an oversized hole (hole diameter typically greater than one and a half times the anchor diameter). In terms of design, the distinction is made based upon the failure mode, whereby for grouted anchors both the bond at the concrete to grout interface as well as the bond at the bolt to grout interface are relevant for determining the tension strength corresponding to concrete failure modes. (Zamora et al.) This distinction does not typically apply to adhesive anchors.

A sleeve may be used to provide an unbonded length for tensioning (Figure 4.5). Depending on the embedment depth and diameter of the anchor, various techniques may be used to facilitate installation of the anchor using the various grout types.

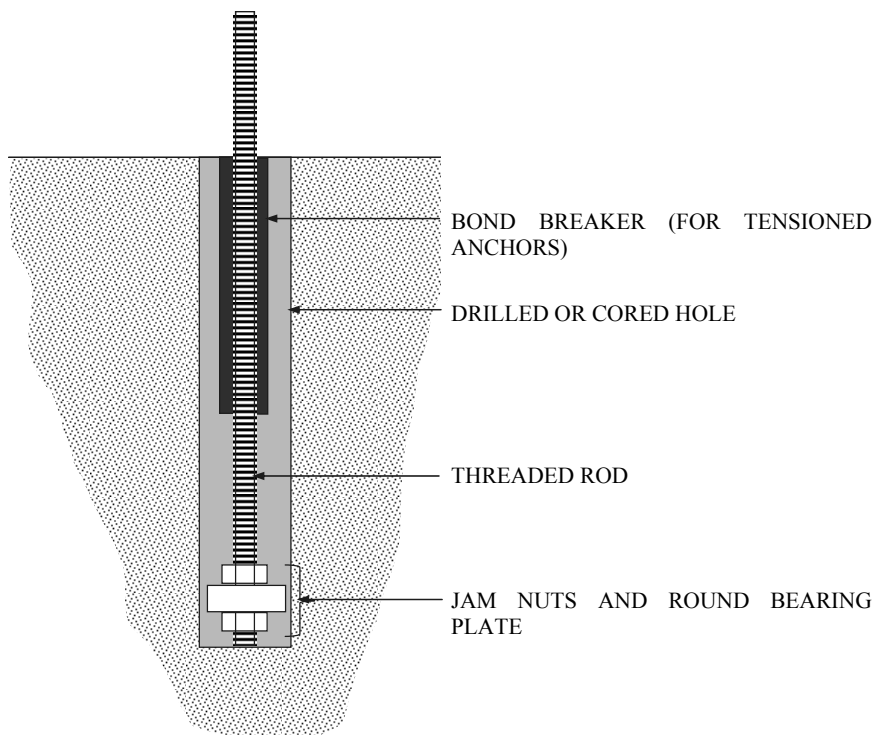


Figure 4.5: Headed Grouted Anchor

4.3.2 Adhesive Anchors

The term adhesive anchor is generally understood to refer to threaded rod installed in a drilled hole with a polymer adhesive (Figure 4.6a). Anchor rods for deep embedments may be equipped with a sleeve or wrapped with de-bonding paper to facilitate tensioning (Figure 4.6b).

Typically, optimal performance of adhesive anchors is achieved with a relatively thin bond line (that is, an annular gap of 1/16-1/8 in. [1.6-3.2 mm]). The hole diameter may be increased in order to facilitate installation of rebar and large diameter or deep anchors; however, the use of larger hole diameters requires larger volumes of adhesive with attendant potential for excessive heat generation during the curing process and resultant shrinkage. Rebar is often substituted for threaded rod for concrete-to-concrete applications.

Bonding materials for adhesive anchors fall into three basic categories:

- a. Bucket-mixed epoxy grouts
- b. Capsule anchors
- c. Cartridge injection systems

Bucket-mixed epoxy grouts, often mixed with sand, are employed for downhole anchors as well. Alternatively, bulk mixers may be used to automate the mixing and delivery process.

Capsule anchors were developed as a means of controlling the relative quantities of the resin, hardener and aggregate components in the adhesive matrix by placing them together in a sealed glass ampoule. More recently, capsules fabricated from foilized polyester film have been introduced to reduce the hazard of accidental capsule breakage. Capsule anchor systems contain a resin component, aggregate/sand and benzoyl peroxide as an accelerator or hardener. The capsule is fragmented and integrated into the resin matrix during installation.

Cartridge injection systems are the most prevalent option for the delivery of two-component epoxies and other polymer-based grouts used for anchoring. A and B component cartridges are typically joined by a plastic manifold that controls metering. A clear plastic nozzle equipped with an internal mixing helix attaches to the manifold and may be extended as necessary to enable delivery of the mixed adhesive to the back of the hole. The cartridges are placed in manually- or pneumatically-operated dispensers, similar in operation to a caulking gun.

Hybrid adhesives, comprised of a polymer adhesive and a synthetic cement, are also used in cartridge injection systems. These are generally fast-cure adhesives and may exhibit superior resistance to high temperatures.

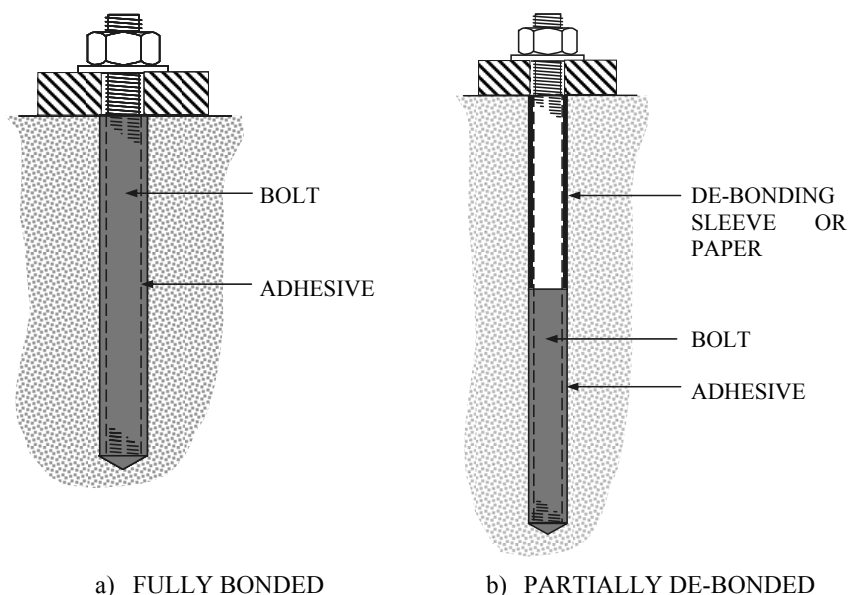


Figure 4.6: Adhesive anchors

4.3.3 Hybrid Systems

Hybrid anchors (not to be confused with hybrid adhesives as discussed in 4.3.2) combine the working principles of adhesive anchors with expansion or undercut mechanisms. Torque-controlled adhesive anchors (Figure 4.7) transfer tension loads via friction. Because of their ability to re-expand upon the application of tension loads, they are particularly suited for use in concrete that may crack over the anchor life, and may be used in a variety of applications where the flexibility of an adhesive anchor system is required. They are also less sensitive to hole cleaning procedures than ordinary adhesive anchors. Grouted undercut anchors, like standard undercut anchors, transfer tension loads via bearing. The grout improves the form-fit between the anchor and the concrete thereby reducing initial anchor movement under load.

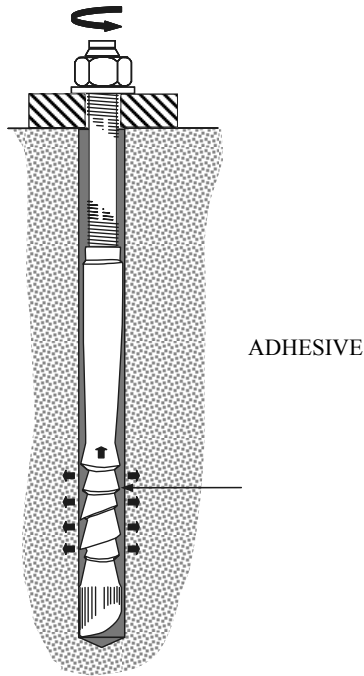


Figure 4.7: Torque-controlled Adhesive Anchor

4.4 CONSIDERATIONS IN POST-INSTALLED ANCHOR DESIGN

The following factors should be considered when designing connections using post-installed anchors:

- a. Loading type and direction (4.4.1)
- b. Required edge distances, anchor spacing, embedment depth, and anchor length (4.4.2)
- c. Concrete quality and condition (4.4.3)
- d. Installation conditions (4.4.4)
- e. Exposure to weather, temperature fluctuations, chemicals, and fire (4.4.5)
- f. Importance of the connection and consequences of failure (4.4.6)

4.4.1 Loading Type and Direction

Direct tension applications are particularly important where a substantial portion of the load is sustained over time. For these cases, undercut systems are preferable. Where seismic loads dominate, adhesive anchors may be more suitable to engage a larger volume of the structure in resisting possible overloads. High-cycle fatigue loading requires that special attention be paid to the anchor and nut assembly and may also call for tensioning of the connection to avoid stress fluctuation in the bolt, in which case an anchor detail with sufficient stretch length to ensure acceptable preload retention should be used. In addition, detailing to prevent nut unwinding, especially in the case of alternating shear, should be employed. This may include the use of double nuts or lock nuts. The use of wedge-type expansion anchors to resist vibration loading, for example, in connection with compressors and pumps, has been associated with failure/loosening of the expansion mechanism over time. This may be addressed either by ensuring that sufficient preload is maintained in the bolt to prevent load fluctuation at the wedges, or by the use of undercut or adhesive anchors. The use of Bellville (coned disc spring) washers may be appropriate to maintain tension.

4.4.2 Required Edge Distances, Anchor Spacing, Embedment Depth, and Anchor Length

Anchors that rely on friction produced through expansion forces typically require larger edge distances to avoid splitting failures during anchor installation or under working loads. Where the connection geometry requires that anchors be installed near free edges or close to one another, use of anchor types that do not generate expansion forces on installation may be preferable. These include adhesive anchors (but not torque-controlled adhesive anchors) and undercut anchors. For cases where the member depth is limited relative to the anchor embedment, the anchor selection should consider whether the required distance from the bottom of the drilled hole to the opposite concrete surface is adequate to prevent blow-through during drilling or splitting during anchor installation.

The anchor selection process should also include a check for the necessary anchor projection to accommodate the attachment requirements, including the length of thread available.

Information regarding the anchor length, minimum edge distance, anchor spacing, and member depth are contained in evaluation reports issued by ICC-ES, or other evaluation services and in product literature. Design for reduced anchor spacing and edge distance is addressed in the provisions of ACI 318 Appendix D.

4.4.3 Concrete Quality and Condition

Where it is suspected that the concrete contains significant voids, use of expansion, screw and undercut anchors should be avoided. Likewise, capsule anchors, which provide a finite amount of adhesive, may be inappropriate. Where voids are suspected

to be large, use of an adhesive anchor system with a screen tube as developed for use in hollow masonry may be advisable. Concrete that has suffered extensive chloride infiltration or that no longer provides corrosion protection for embedded steel items (for example, reinforcement) through passivation of the steel surface may also dictate the use of highly corrosion resistant anchor solutions. (See also 4.4.5.)

4.4.4 Installation Conditions

It should be verified that sufficient clearance exists to effectively perform the steps necessary for installation. For expansion, screw and undercut anchors this includes ensuring that adequate room is available for hammer drills, torque wrenches and other setting equipment. For adhesive anchors, in addition to clearances, consideration should be given to the jobsite conditions during anchor installation, such as air temperature, possible exposure to rain, and, where deep holes are necessary, access to the hole for cleaning prior to adhesive injection.

In general, where anchors are to be installed overhead to carry sustained tension loads, special attention should be paid to selecting systems that have been thoroughly tested (for example, in accordance with AC193 or AC308 as discussed in 4.8) for their ability to resist sustained tension. Mechanical anchor systems (expansion, undercut, screw) are generally easier to install overhead and are less susceptible to installation error than their adhesive counterparts. Where adhesive anchors must be installed overhead, specific attention should be paid to the selection of a system that has been prequalified for this orientation and that includes specific measures to avoid inclusion of air in the bond line during injection of the adhesive. Capsule anchors (4.3.2) using "soft" foilized polyester film may offer specific advantages in this regard. Hybrid torque-controlled adhesive anchor systems (4.3.3) have also been used for overhead installations.

For additional discussion of constructability considerations see 5.3.

4.4.5 Exposure to Weather, Temperature Fluctuations, Chemicals, and Fire

The embedded portion of anchors is generally protected from corrosion by passivation of the steel surface in contact with the concrete. At the surface of the concrete, however, the anchor is particularly susceptible to corrosion. Where anchors are subjected to moisture or other possible corrosion-inducing agents, consideration must be given to a number of factors, including, but not limited to, compatibility of the anchor steel with that of the attached component or base plate (for example, in terms of separation on the galvanic scale), access to the connection for visual inspection and the potential for non-visual corrosion forms such as pitting or crevice corrosion. The use of adhesive anchors in concrete where temperatures may change considerably over time should be constrained to the concrete temperature limits for which the anchor systems have been prequalified. For additional discussion of temperature effects, including fire, see 2.7.

Qualification provisions for adhesive anchors currently contain tests that measure the effect of alkalinity, water, and sulfur on bond strength. Where anchors are to be exposed to particularly aggressive environments, it is advisable to consult with the anchor manufacturer regarding specific testing to address the condition in question.

4.4.6 Importance of the Connection and Consequences of Failure

Post-installed anchor connections may be deserving of special attention if they: a) transfer loads as part of the structural load path of a building or other structure, b) are used in a building or structure that has been assigned a high importance classification (for example, emergency response facilities), or c) should be designed with post-installed anchor systems that are more robust. Post-installed anchor bolts that tend to be more robust include most undercut anchor systems, some heavy-duty expansion anchors, and many adhesive anchor systems.

4.5 POST-INSTALLED ANCHOR DESIGN

The design of proprietary post-installed anchors generally depends on information developed via prequalification testing. ACI 355.2 is a prequalification standard for post-installed mechanical anchors that provides data for design in accordance with ACI 318 Appendix D. It has been incorporated into ICC-ES acceptance criterion AC193 for the purpose of issuing evaluation reports for these products.

The design of adhesive anchors is not directly addressed in ACI 318, and only peripherally in ACI 349. ICC-ES AC308 provides the necessary modifications to ACI 318 for the design of adhesive anchors in the form of additional equations to address the bond capacity of single anchors and anchor groups. In tension, the lesser of the bond and concrete breakout capacities is taken as the controlling strength for concrete failure. AC308 is a separate acceptance criterion which includes additional design requirements for adhesive anchors.

4.5.1 Allowable Stress Design

Traditionally, post-installed anchor design has been based on mean ultimate test data divided by a global safety factor of four (4). This approach has been replaced by a system involving more rigorous qualification testing and strength design concepts.

4.5.2 Strength Design

The use of the CCD Method contained in ACI 318 Appendix D and ACI 349 Appendix D requires at a minimum the following information for the specific post-installed anchor in question:

- a. Anchor category (1, 2 or 3) for determination of appropriate strength reduction factors

Note: For specific anchor category designation consult the post-installed anchor manufacturer or associated evaluation report from ICC-ES or other evaluation service.

- b. K_c -factor(s) for determination of concrete breakout capacity in uncracked and cracked concrete, as applicable
- c. The characteristic bond strength in uncracked and cracked concrete for adhesive anchors, as applicable
- d. Steel strength and critical cross-sectional area
- e. Effective embedment depth, h_{ef}
- f. Effective length, ℓ_e , and diameter, d_a , for determination of shear capacity
- g. Bolt elongation and cross-section reduction at break for determination of ductility status
- h. Minimum member thickness, critical edge distance (for expansion anchors) and minimum edge and spacing dimensions
- i. Presence of supplementary reinforcement
- j. Pullout values as applicable for static and seismic tension
- k. Seismic shear capacity as applicable

This information should be documented in accordance with ACI 355.2 and/or in accordance with ICC-ES acceptance criteria AC193 for mechanical anchors or AC308 for adhesive anchors.

4.5.3 Ductility of Post-installed Anchors

ACI 318 Appendix D contains the following definition of a ductile steel element:

Ductile steel element – An element with a tensile test elongation of at least 14 percent and reduction in area of at least 30 percent. A steel element meeting the requirements of ASTM A307 shall be considered ductile.

The elongation and cross-section reduction requirements were originally selected to correspond to those of ASTM A193 B7/A193M B7, a common anchoring material. ASTM A193 B7/A193M B7 now exceeds these requirements. The commentary notes that the measurement of elongation should be taken over the requisite gauge length specified in the appropriate ASTM standard for the specimen in question. In most

cases, ASTM F606/F606M is taken as the applicable standard, and the gauge length is typically five diameters.

Note: ASTM F606/F606M also allows for the turning of dog-bone tension specimens from threaded parts. (For cold-worked specimens, this may remove the hardened portions of the bolt.) Most steels used for the production of post-installed anchors will meet these requirements, although establishing this via test can prove challenging.

It should also be noted here that typical reinforcing bars do not meet this requirement since their elongation is measured over a length corresponding to one full repeat of the deformation pattern. Thus, the definition of what is "ductile" and what is not becomes somewhat arbitrary in practice.

Ductile steel elements are necessary for better load distribution to anchors in groups, and as such, ACI 318 Appendix D provides a higher strength reduction factor for the steel resistance to anchors that qualify as ductile. In addition, ductile steel elements are a prerequisite to satisfying the requirements for ductile anchor design in ACI 318 Section D.3.3. (See 4.6.)

Practically speaking, most post-installed mechanical (expansion, undercut, screw) anchors will not satisfy the ductile design criteria of ACI 318 Appendix D. That is, for the embedment depth to diameter ratio and steel grade typically found in common mechanical anchors, it is not possible to demonstrate by calculation that steel failure will control the tension or shear strength, even for higher strength concretes. Some undercut anchor systems are adaptable to deeper embedments, and in these cases a ductile anchor design may be possible. It is also possible to embed adhesive or grouted anchors at sufficient depth to ensure steel failure; in such cases use of an unbonded length or projection of the anchor element out of the concrete a sufficient distance (as with a vessel anchor chair) is required to achieve meaningful stretch.

4.6 SEISMIC LOADING

Post-installed anchors must satisfy certain qualification requirements in order to be used to resist seismic loads in a structure assigned to Seismic Design Categories C, D, E, or F. These involve the performance of specific tests and application of acceptance criteria for qualification and determination of relevant design parameters. ACI 318 Appendix D references ACI 355.2, *Qualification of Post-installed Mechanical Anchors in Concrete*, for the qualification of expansion and undercut anchors. This document has been incorporated into acceptance criteria used by the ICC Evaluation Service for issuance of Evaluation Service Reports on anchors to demonstrate conformance with IBC Section 104.11, *Alternative materials, design and methods of construction and equipment*. AC193 provides acceptance criteria for mechanical anchors. AC308 provides acceptance criteria as well as supplementary design provisions for adhesive anchors.

For further information on seismic design of anchors see 3.11.

4.7 DESIGN FOR HIGH-CYCLE FATIGUE

High-cycle fatigue is handled in a manner similar to that for cast-in-place anchors. (See 3.10.1) Additionally, as discussed in 4.4.1, detailing to prevent nut unwinding should be employed.

4.8 POST-INSTALLED ANCHOR QUALIFICATION

Requirements for qualification testing and assessment of post-installed anchors are defined by ACI 355.2 and the relevant ICC-ES acceptance criteria (AC193 and AC308). Three types of tests are included:

- a. Reference tests – Reference tests establish a baseline for anchor evaluation
- b. Reliability tests – Reliability tests are designed to test the anchor function under less than ideal installation and use conditions in order to determine whether there exists a unique susceptibility to foreseeable variations from manufacturers' installation mandates. Reliability tests are not intended to anticipate gross errors in installation or to sanction the incorrect installation of the tested products.
- c. Service condition tests – Service condition tests establish the anchor conformance to design models for service conditions (edge distance, spacing, member thickness) shear and seismic loading

Because of the relative complexity and sensitivity of the testing involved, it is important that the testing and evaluation agency be accredited for the relevant standards under the guidelines provided in ISO 17025, *General Requirements for the Competence of Testing and Calibration Laboratories*, (formerly known as ISO Guide 25) and have demonstrated experience and competence in performing the required tests. Evaluation Service Reports issued by ICC-ES provide a means of verifying compliance with these standards.

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ACI 318-08, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute: Farmington Hills, MI.

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ASTM A307-10, *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*, ASTM International: West Conshohocken, PA.

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CHAPTER 5 INSTALLATION AND REPAIR

5.1 INTRODUCTION

This chapter provides basic information regarding installation of anchors, with the initial focus being on key factors and practices affecting post-installed anchor installations. Anchor constructability considerations which address and detail quality control, inspection, design, and construction practices that will help ensure constructible and structurally effective anchor installations are then outlined. Finally, anchor repair procedures are provided – with specific recommendations made for the repair of common anchor installation problems such as misalignment and erroneous projections.

5.2 POST-INSTALLED ANCHOR INSTALLATION

Successful installation of post-installed anchors depends largely on the experience of the installer with the product in question, the ease of access to the anchor location, field conditions and the degree to which the anchor installation is verified and inspected. Training and, where appropriate, certification of installers for the installation of specific anchor types, is advisable, particularly for adhesive anchors that are to be used to carry substantial loads or sustained loads. Training may be accomplished through the manufacturer or through third-party certification organizations, but in all cases it should focus on three essential aspects: 1) evaluation of site conditions; 2) thorough understanding of and adherence to the manufacturer's installation instructions; and 3) adherence to all worker safety requirements.

5.2.1 Mechanical Anchors

Mechanical anchors are typically less sensitive to hole cleaning, provided sufficient hole depth is furnished to permit installation of the anchor to the specified embedment depth. Drop-in anchors should be checked for under-setting with the specified installation tool. Torque-controlled anchors that do not develop the specified torque within a reasonable number of turns (typically less than 5) should be abandoned or removed. Most undercut anchors provide a system of visual verification of proper set.

5.2.2 Grouted Anchors

Grouted anchors are always installed downhand (gravity assisted). Pre-filling the hole and installing the anchor, which is the typical procedure for adhesive anchors, may not be practical for anchors with large bearing plate diameters. Care must be taken, however, to prevent the formation of air bubbles in the grout matrix during installation.

5.2.3 Adhesive Anchors

Adhesive anchors are most sensitive to hole preparation. The presence of dust, drilling slurry, or water can significantly disrupt the bond capacity of polymer grouts. It is critical that adhesive anchor hole cleaning steps as specified by the manufacturer for the drilling method and concrete condition (dry, wet) be followed in order to achieve the bond strength assumed for design. Hole roughness also plays an important role in bond development. For this reason, cored holes are typically less ideal for good adhesive anchor performance than those produced with rotary-impact hammers or rock drills, and cleaning directions for cored holes are often different from those used for hammer-drilled holes. Where deep holes are required, special provisions should be made to ensure that the holes are properly cleaned and dry. In particular, extensions on cleaning brushes and compressed air wands may be required. Where there is doubt about the competency of the concrete, it may also be necessary to inspect the holes prior to injection with a borescope or similar device.

Capsule anchors are typically installed by driving a chisel-pointed anchor rod chunked into a rotary impact drill through the capsule using a drilling and hammering action. The drilling and hammering action serves both to fragment the capsule and to mix and activate the components (resin and accelerator). Care must be taken to use a rotary-impact tool suitable for the size anchor being installed, and to not overdrive the anchor (that is, allow the drill to rotate longer than the specified period).

Capsule anchors set rapidly; however, attainment of full strength is dependent on use of the correct size rotary impact drill and protection of the anchor from loading or disturbance during the gel period. Because the quantity of resin provided by the capsule is limited, it is important that the hole diameter and depth be closely controlled. Multiple capsules may be used for larger hole diameters/depths. For these installations, use of an appropriately sized installation tool is critical.

Cartridge anchor systems offer the advantage of controlled resin metering and delivery and reduce the risks associated with the handling of volatile resin components. Nevertheless, care must be exercised to ensure that properly mixed resin is injected without substantial voids in the drilled hole. The following steps are generally common to all cartridge systems:

1. After installing a cartridge in the dispenser, an initial quantity of dispensed resin remains unmixed and must be discarded. This step must be repeated for each new cartridge. Where extensions on the injection nozzle are used, the extension must be removed to prevent the initial quantity of adhesive from each new cartridge from ending up in the hole.
2. The resin must be injected from the back of the hole to the front. For deep holes, it may be necessary to employ special methods to ensure that air will not be entrained in the injected adhesive. Air that is trapped in the hole by

resin results in voids which not only reduce the bond area but may also negatively affect resin cure and promote corrosion of the embedded rod.

3. Unlike bucket-mixed or cementitious grouts, cartridge injection anchor systems are generally suitable for horizontal and, in some cases, overhead installation because of the use of thixotropic resin formulations. When installing adhesive anchors overhead, special measures must be taken to ensure that the resin remains in the hole and that the anchor rod does not displace downward during resin cure. Care must be taken to avoid skin and eye exposure to uncured resin.

5.2.4 Large Adhesive Anchors

Typical proprietary adhesive anchor systems provide engineering data for embedments up to 1-1/4 in. (31.8 mm) in diameter and 15 in. (381 mm) deep (12 diameters). For larger diameters and embedments, special provisions for installation and design are typically required.

Hole cleaning methods suitable for shallower embedments may not be effective for deep holes. Compressed air, vacuums, and internal side-action wire brushes should be employed in repetitive sequences as required to produce a hole of the correct depth and with a relatively dust-free surface, particularly at the bottom of the hole.

Holes for larger anchors are often drilled with diamond core rigs. This typically results in smoother holes and reduced bond resistance. The reduction in bond resistance is aggravated if the drilling slurry is allowed to remain on the surface of the hole wall. Flushing of cored holes with water is the most common method of cleaning. Subsequent scouring with a wire brush and removal of dust and residual moisture with compressed air is recommended. Unlike cementitious grouts, which require that the hole be soaked with water prior to grout placement, adhesive grouts require a dry, clean hole for optimum performance.

Rock drills typically produce rougher hole surfaces. Cleaning methods appropriate for holes drilled with carbide bits are generally suitable for holes drilled with rock drills.

Installation of large diameter adhesive anchors involves providing for

- a. ensuring proper cleaning of the hole surface and removal of free water prior to injection;
- b. avoiding trapped air in the cured resin matrix;
- c. facilitating injection of large volumes of correctly metered adhesive within the adhesive pot life; and

- d. ensuring the correct placement of the anchor element after injection of the adhesive.

Pneumatically-driven injection systems designed to accommodate large-volume cartridges may be coupled through a manifold to facilitate rapid adhesive delivery. Extensions attached to the static mixing nozzle and fitted with a donut-shaped stopper matched to the hole diameter may be used to prevent the introduction of air bubbles into the grout.

5.3 CONSTRUCTABILITY CONSIDERATIONS

Successful application of quality control, inspection, design, and construction practices will help ensure a constructible, structurally effective anchor installation. Discussions and recommendations for each of these processes are provided below.

5.3.1 Implementation of Quality Plan

Quality control is a key factor in assuring effective, constructible anchor installations. Experience has shown that the secondary costs of compensating for anchors being misaligned or installed “out of plumb”, having material properties noncompliant with construction specifications, etc. justify taking great care in the creation and implementation of an effective quality control plan. Such a plan should address the following issues:

- a. Engineering specifications and drawings should indicate clearly the intent of the design, including individual anchor and hardware details, required material properties, location of the anchor, projection and embedment dimensions with respect to the finished concrete grade, tapping requirements, location and plumb installation tolerances, coating, length, diameter, length of threaded portion, diameter and thickness of washers, number of nuts (single, double, single or double plus leveling, etc.), sleeve details, and tensioning requirements - if any (Swiatek, Whitbeck, and Shneur).
- b. Material certification submittal requirements for the fabricator should be clearly stated in the material requisition documents. These submittals should include listing all information required by ASTM specified material certifications including options that are applicable. Certifications are recommended for high-strength anchors and for critical applications.

For anchors that fall within the seismic force-resisting system categories defined in ASCE/SEI 7, Appendix 11A, the specified minimum quality assurance requirements are to be applied.

5.3.2 Cast-in-place Anchor Inspection Plan

An inspection program should be established that verifies proper installation of the anchor prior to placement of concrete (ACI 349 Section D.9). Such a plan should ensure that:

- a. the size and location of anchors are in accordance with the design drawings and specifications
- b. the anchors are securely held in place to prevent movement during anchor placement
- c. bolts are coated correctly
- d. bolts are lubricated with the correct materials prior to installation
- e. bolts are taped if required

5.3.3 Post-installed Anchor Inspection Plan

Establishment of a comprehensive inspection regime for anchor installation can be a strong motivating factor in ensuring contractor compliance.

- a. Pre-installation Inspection – Inspection typically includes a review of the means and methods to be used for the installation prior to start of the work, verification of the use of the specified product, and detection of existing concrete embedments and reinforcing prior to commencement of drilling.

Pre-installation inspection of mechanical anchors may include verification of the use of drill bits of the correct type and diameter, methods for removing drilling debris from the hole, and the use of properly calibrated torque wrenches as required.

Pre-installation inspection of adhesive anchors may include review of the following:

- methods for hole drilling and preparation, and grout injection
 - anchor setting procedures
 - procedures to ensure protection from disturbance during the required cure period
 - drilled hole depth, diameter and anchor lengths
 - proper storage and use of the adhesive components, including any pre-conditioning methods for cold or hot environments (should be checked against manufacturer's requirements)
- b. Ongoing Inspection and Proof Loading – Inspection during anchor installation is intended to verify compliance with the specifications as well as successful

anchor set. Proof loading is recommended as both a means of detecting unsuccessful anchor installations and as a motivational tool. Proof loading of torque-controlled anchors can be accomplished through the re-application of the setting torque. Adhesive anchors may also be proof loaded through the application of torque; however, it is preferable to use direct tension testing for these cases. The percentage of installed anchors to be proof loaded typically ranges from 10-50%, depending on the criticality of the installation. Anchor proof loads are generally taken as the lesser of 50% of the anchor ultimate capacity as governed by bond or concrete failure, or 80% of the anchor yield capacity. Criteria for acceptance are usually characterized in terms of little or no perceptible movement of the anchor at proof load.

5.3.4 Specific Construction Practices

The following construction practices should be documented in the construction specifications and implemented by construction to help minimize anchor constructability problems/issues in the field – including the need for future anchor replacement or repair.

- a. Thoroughly clean anchors of rust, thread cutting oil, or any other substance that could reduce bond to concrete. Common cleaning methods include wire brushing and/or applying a degreasing solvent.
- b. Have a registered surveyor be responsible for laying out anchors – as opposed to common practice whereby the general contractor's carpenter foreman handles the task (Fisher and Kloiber). When possible, it is recommended that Total Station technology be used for the layout effort, as opposed to the more traditional string line and tape measure method (Nasvik).
- c. Ensure anchors maintain proper alignment and plumbness by rigidly wiring them to reinforcement prior to the placement of concrete. Use wood or steel templates firmly fastened to the footing or pedestal forms, or engineering approved, vendor-supplied anchor stabilization products - for example, a template (Figure 5.1).

Note: One method for avoiding template and anchor dislocation is to pour a mud mat beneath the proposed foundation. A rigid support frame is constructed and bolted to the mud mat – simultaneously supporting the template and ensuring no movement of the template or anchors occurs during concrete placement [Nasvik].

- d. Protect anchor threads against concrete spillage, rusting and any other damage.

- e. Roughen the interface of a formed grout pocket to eliminate the possibility of bond failure between the grout and concrete. Alternatively, pockets can be formed with corrugated steel tubes to provide a structural interlock mechanism. Debonding tape is beneficial when an anchor that is to be pre-tensioned is installed inside the grout pocket.



Figure 5.1: Manufacturer-supplied Template

5.4 REPAIR PROCEDURES

When an anchor is installed outside specified construction tolerances, the structural adequacy of the installation should be verified by the Engineer-of-Record and repair procedures implemented as necessary.

During the repair process, it is necessary to provide quality control in the form of inspections – possibly including nondestructive testing – or other reviews to verify the adequacy of the repair process and materials (ACI 349.3R Chapter 8).

The following sections discuss installation problems often encountered and address recommended methods for their remediation.

5.4.1 Misalignment Issues

Misalignment issues pertain to anchors that have been installed “out-of-plumb” or outside construction specification location tolerances. Most often these misalignments occur as a result of survey error or anchor shifting during placement of concrete. In some cases, vendor drawings with incorrectly detailed locations of the anchor bolts are the source of anchor bolt locations not matching equipment base

plates. The impact is a base plate or equipment base that will no longer fit the installed anchor layout. The following measures can be used when investigating anchors that have been installed outside acceptable installation tolerances. Each must be evaluated by the engineer to determine applicability, economic impact, and structural adequacy of the proposed fix.

- a. Evaluate the need for the nonconforming anchor. Perhaps not all are required for a particular installation. If not, the misaligned anchor can be cut flush with the surface and abandoned in place (Fisher and Kloiber).

Note: Per OSHA, if less than four anchors are secured for a column, the erector must be made aware of the situation and take the necessary precautions when erecting the member – holding the column with a crane, guying the column, etc. (OSHA 29 CFR 1926.755).

- b. Bend a misaligned (out-of-plumb) anchor into position. This may require removal of the concrete around the anchor to soften the bend angle. Engineering assessment of the bend on anchor strength and/or anchor fatigue properties may be required. This repair method is not recommended for high-strength anchors (Fisher and Kloiber).
- c. Remove a misaligned anchor by core drilling and replacing with post-installed anchors
- d. Drill an oversized hole in base plate as required to fit a misaligned anchor. Install an A36 thickened plate washer over the anchor and weld the washer directly to the base plate. Size washer to ensure adequate transfer of design loads to the anchor. Spherical or beveled washers may be required to provide uniform bearing at the washer – base plate interface. Some misaligned anchors may also require modification and reinforcement of the column web or flange (Fisher and Kloiber). As an alternative to installing the plate washer, adequate shear transfer can be accomplished by filling the annular space between the anchor rod and anchor hole with grout.
- e. Fabricate and weld base plate extension if misaligned anchor falls outside area of existing base plate
- f. Fabricate new base plate to fit the misaligned anchors
- g. Demolish and re-construct concrete element that contains non-conforming anchors

5.4.2 Inadequate Anchor Projection

The following measures can be utilized when anchors are installed with inadequate projections – that is, if threads are not projecting fully enough to completely engage the nut(s). Many of the solutions provided below call for welding of the anchor; therefore, it is recommended that the engineer review the weldability of the anchor material prior to implementing any of these proposed methods.

- a. Evaluate the structural effectiveness of the threads engaged to determine whether the installation will be acceptable with a partially installed nut. This can be done based on a linear interpolation of full threads engaged versus the number of threads installed within the nut (Fisher and Kloiber).
- b. If the structural effectiveness of the engaged threads is not adequate, weld the nut to the anchor to achieve the required anchorage capacity (Figure 5.2). The engineer should confirm that the weld acting alone will develop the strength of the anchor, since the capacity of the welds and the engaged threads are not additive. Alternatively, weld the anchor directly to the base plate, if the hole diameter is not excessive (Figure 5.3).
- c. Extend the short projection anchors by welding on a threaded extension. See Figures 5.4 and 5.5 for weld details that could be used to properly extend anchors (Fisher and Kloiber). Before welding, confirm that the anchorage material is weldable to the strength required.
- d. Use a coupling nut to extend the anchor. The AISC Manual shows coupling nuts that are capable of developing the full strength of the anchor. To accomplish this, the concrete will have to be chipped away enough to cut off the old anchor and thread the embedded portion as required to attach the coupling nut (Fisher and Kloiber).
- e. In cases where two nuts are called for, evaluate whether adequate bolt length is provided to install one nut and whether the installation will be acceptable with only one nut provided.

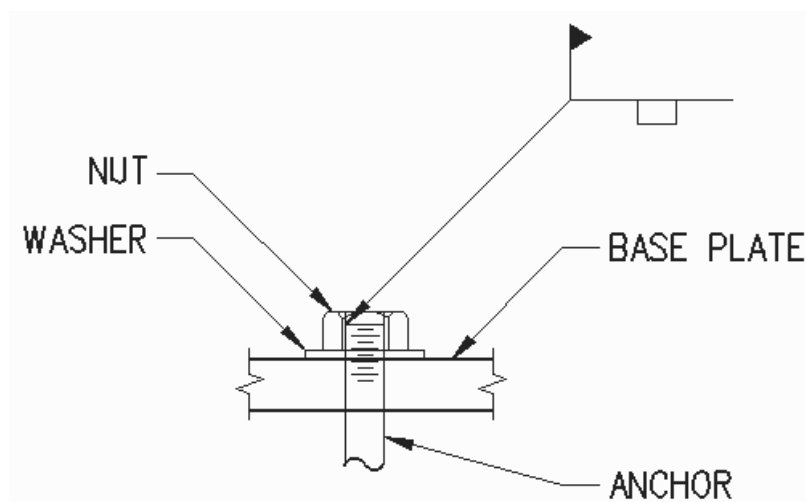


Figure 5.2: Welding of Nut to Anchor

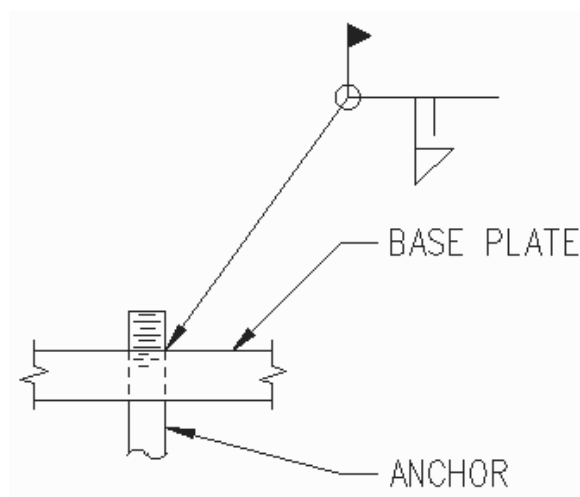
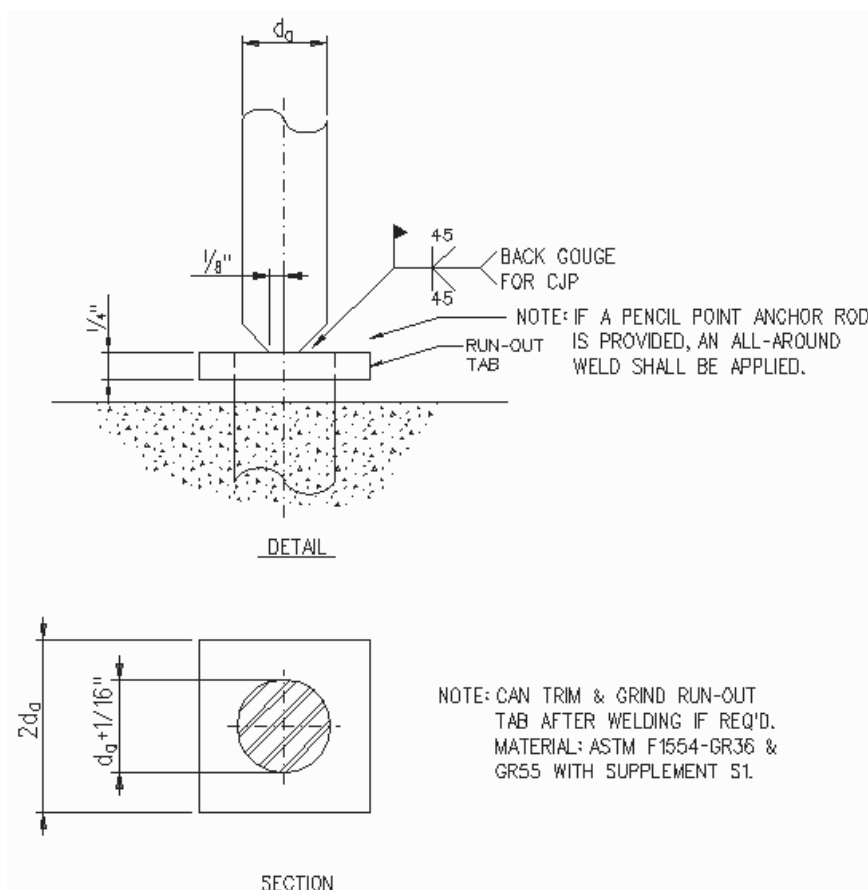
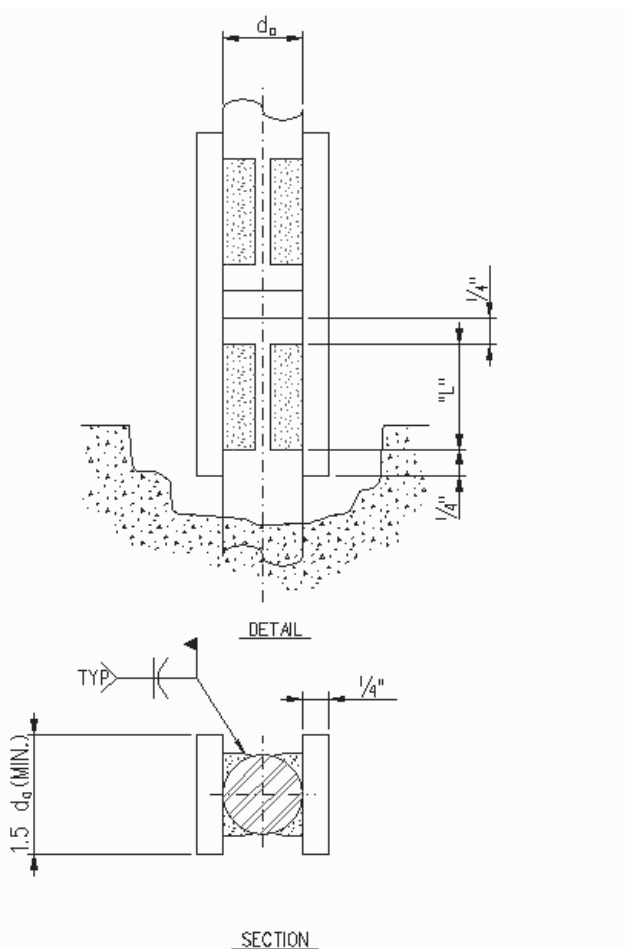


Figure 5.3: Welding of Anchor to Base Plate



IN ORDER TO PROPERLY GROOVE-WELD A THREADED STUB TO EXTEND AN ANCHOR ROD, IT IS NECESSARY TO FIRST PREPARE THE JOINT BY BEVELING THE STUB TO A CHISEL POINT. NEXT, A RUN-OUT TAB MUST BE PROVIDED. THIS CAN BE DONE BY TAKING A $1/4$ " THICK PLATE WITH A HOLE $1/16$ " LARGER THAN THE ROD. THEN TACK-WELD THE TAB FLUSH WITH THE TOP OF THE EMBEDDED ROD. THE WELD THEN CAN BE PLACED IN THE GROOVE AND EXTENDED OUT ONTO THE PLATE. IF A CJP WELD IS REQUIRED THE WELD SHOULD BE GOUGED TO SOUND METAL BEFORE WELDING THE SECOND SIDE. THIS CAN BE DONE WITH A GRINDER. IF NECESSARY FOR CLEARANCES, THE RUN-OUT AND EXCESS WELD CAN BE CUT OFF AFTER THE WELD IS COMPLETE.

Figure 5.4: Welding of Anchor Extension – Option 1



IT IS POSSIBLE TO SPLICE AN ANCHOR ROD USING TWO PIECES OF FLAT BAR STOCK.

1. THE STRENGTH OF THE FOUR FLARE BEVEL GROOVE WELDS ARE TO BE CALCULATED BY THE ENGINEER.
2. THE MINIMUM WELD LENGTH "L" SHOULD BE AT LEAST 1".
3. THE REQUIRED PLATE AREA SHOULD BE CHECKED FOR TENSION CAPACITY.

IT IS IMPORTANT THAT THE WELDER BE CERTIFIED TO WELD IN THE VERTICAL POSITION. OSHA REQUIRES ALL ANCHOR-ROD MODIFICATIONS TO BE APPROVED BY THE ENGINEER OF RECORD.

Figure 5.5: Welding of Anchor Extension – Option 2

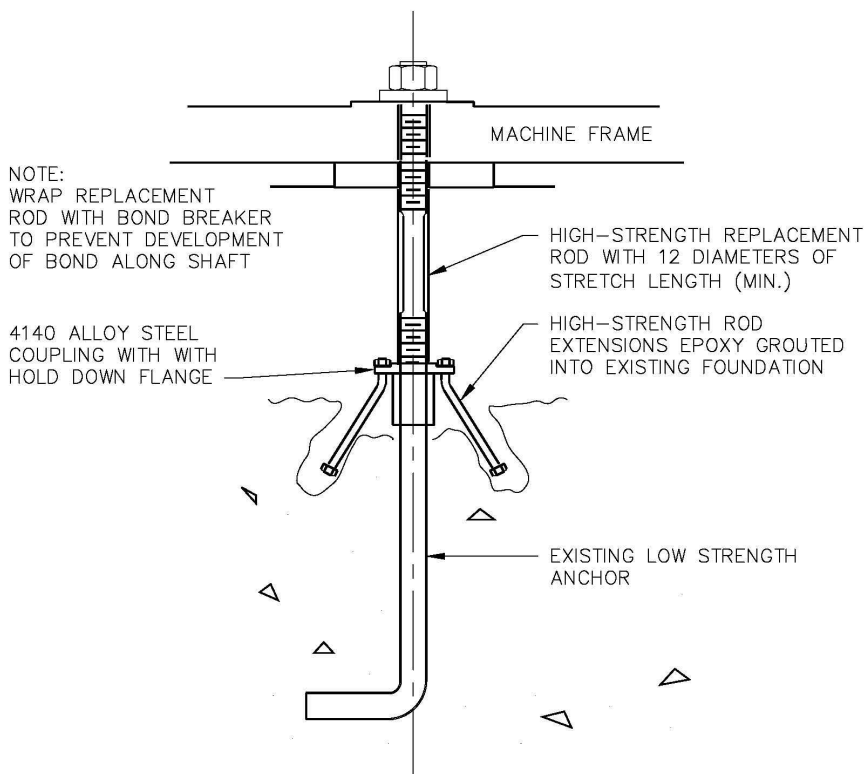
5.4.3 Excessive Anchor Projection

When anchors are installed with excessive projections, a scenario may arise where the threaded portion of the anchor stops somewhere above the top side of the connected element (for example, base plate) – resulting in a situation where the nut cannot be fully tightened to the element. In cases like these a filler plate or washers can be added, so that the nut can be fully tightened against the connected element. This filler plate or washers must be welded directly to the base plate if shear transfer through the anchors is required and the holes in the base plate are oversized to the extent that excessive slippage would occur before the edges of the base plate holes engage with the anchors. Alternatively, shear transfer can be accomplished by filling the annular space between the anchor rod and anchor hole with grout.

5.4.4 Material Property Issues

The following measures can be used when investigating anchors that have already been installed but are later discovered to have inadequate material strength properties – for example, due to fabrication errors or incorrect anchors being installed. These measures can also be used for installations where existing anchors need to be upgraded as a result of design load increases.

- a. Remove the unacceptable anchor by core drilling and replace with an adequately sized post-installed anchor
- b. Use a chip and repair method as illustrated in Figure 5.6 – which reflects an existing compressor anchor installation with tensioned anchors. In this method, concrete is chipped away to expose and cut off the existing anchor. The remaining portion is then threaded and a repair coupling, which includes a flange with holes, is attached. Four (4) threaded rods are put through the flange holes and extend down into the existing foundation (deeper drilling may be required for these four threaded rod “tendons”). Add the top anchor portion that extends upward (Rowan).



1. CHIP OUT EXISTING GROUT AND CONCRETE APPROXIMATELY 18 – 22 INCHES BELOW MACHINE BASE TO ALLOW FOR THE REPLACEMENT ROD, EXTENSIONS AND COUPLING NUT.
2. CUT EXISTING ANCHOR SHAFT, BUT ALLOW ENOUGH PROJECTION FOR THREADING FOR A LENGTH 1-1/2 TIMES THE ANCHOR DIAMETER USING AN 8 PITCH ADJUSTABLE THREAD DIE.
3. AFTER THREADING EXISTING ANCHOR SHAFT, INSTALL COUPLING NUT WITH FLANGE AND HIGH STRENGTH EXTENSIONS INTO DRILLED HOLES AS SHOWN. INSTALL NEW HIGH-STRENGTH ROD AND REPLACE CONCRETE WITH EPOXY GROUT.

Figure 5.6: Chip and Repair Method

(Reprinted with permission from Robt. L. Rowan & Associates, Inc.)

5.4.5 Failure to Tape Pre-tensioned Anchors

As stated earlier, industrial tape is to be applied to the intended “stretching length” of pre-tensioned anchors. On rare occasions, this concrete separation measure is not applied, resulting in an anchor that cannot be effectively pre-tensioned. When this occurs, it is recommended that the affected anchor be reworked using the reinforcement measures discussed earlier in 5.4.3.b. Such measures will result in a high load resistant installation, without having to core drill and completely remove the existing anchor. Alternative measures include 1) extending the anchor (see 5.4.2) and creating a tensionable “High Chair” arrangement and 2) core drilling around the anchor to provide a gap (annular space) to allow stretching.

5.4.6 Interference with Existing Reinforcement

As discussed earlier, interferences with existing reinforcement can result in the inability to install anchors in their desired locations. Interferences will need to be evaluated on a case-by-case basis to determine whether to move the anchor versus the reinforcement. If the anchor needs to be relocated, many of the repair procedures discussed above can be evaluated and applied as deemed appropriate (Rowan).

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APPENDIX A

EXAMPLES

EXAMPLE 1: ANCHOR DESIGN FOR COLUMN PEDESTALS

Design the anchor for the steel column located at the top of concrete pedestals shown in Figure A1-1. Anchors resist tension and shear forces.

Maximum total factored loads (per Section 9.2 of ACI 318-08 and Section 2.3 of ASCE 7-05):

Tension: $N_{ua_total} := 80\text{kip}$

Maximum shear in the X-direction: $V_{ua_total_X} := 20\text{kip}$

Maximum shear in the Y-direction: $V_{ua_total_Y} := 20\text{kip}$

Assumptions:

1. For new pedestal design
2. Untorqued, cast-in anchors
3. No sleeve is used
4. Low seismic risk and ductile design is not considered (Seismic design category A or B)
5. Tension force is distributed equally among all anchors
6. Washers are welded to the base plate to transfer the load to the anchor. However, to be conservative, it is assumed only two anchors resisting shear.
7. Shears in the X-direction and the Y-direction do not act simultaneously.

Note: All equations and code section numbers referred in this example are from ACI 318-08.

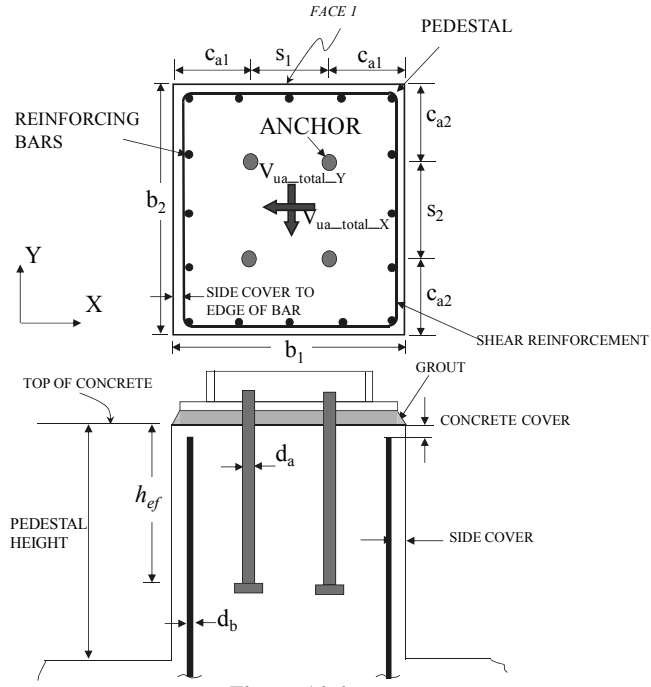


Figure A1-1

Pedestal Data:Specified compressive strength of concrete: $f'_c = 4000\text{psi}$ (Blocks are input data)Height: $\text{Pedestal_height} = 28\text{in}$ Vertical dowel concrete cover: $\text{Concrete_cover} = 1.5\text{in}$ Vertical dowel concrete side cover: $\text{Side_cover} = 2\text{in}$ *Note: In many cases, the height of the pedestal is a design constraint.*Cross-section dimensions: $b_1 = 24\text{in}$ $b_2 = 24\text{in}$ Edge Distance: $c_{a1} = 8\text{in}$ $c_{a2} = 8\text{in}$ Anchor Spacing: $s_1 = 8\text{in}$ $s_2 = 8\text{in}$ **Anchors:**Specification: ASTM F1554, Gr 36 $f_{ya} = 36\text{ksi}$ $f_{uta} = 58\text{ksi}$

ASTM F1554, Gr 36 is a ductile steel (See: Table 2.1 of this ASCE Committee report).

Therefore: $\phi_T = 0.75$ (tension loads) $\phi_V = 0.65$ (shear loads) (D.4.4.a)*Note: Load combinations shall be per Chapter 9 of ACI 318-08 (or Chapter 2 of ASCE 7-05)***Reinforcing bars:**Grade 60 steel: $f_y = 60\text{ksi}$ Vertical (longitudinal rebars): # 6 $d_b = 0.75\text{in}$ $A_s = 0.44\text{in}^2$ Shear reinforcement: # 4 $d_{tie} = 0.5\text{in}$ $A_{s_tie} = 0.2\text{in}^2$ **Design assumptions:**

1. The tension and the shear forces in the anchors are transferred to the longitudinal rebars and shear reinforcement, respectively, which will be designed as anchor reinforcement. Therefore, the concrete breakout strength in tension and shear (D.5.2 and D.6.2) is not checked. The concrete pryout strength in shear (D.6.3) is assumed OK by inspection because it is usually critical for short and stiff anchors.
2. Even though welded washers are used, only the two back anchors are assumed to carry shear (D.6.2.1)

Design steps:**1. Determine the size of anchors**

The size of anchors is determined based on the steel strength of anchor in tension and shear. Since the tension force is assumed to be distributed equally, each anchor carries $80\text{ kip} / 4 = 20\text{ kip}$. There are two anchors in both X and Y directions (i.e. half of the total number of anchors) are effective in resisting shear, the maximum shear force carried by one anchor is $20\text{ kip} / 2 = 10\text{ kip}$. If there is any shear in the X-direction acting simultaneously, it may be added vectorially here.

Try 1.25-in. anchor: $d_a = 1.25\text{in}$ (Threads per inch: $n_t = 8$)

Effective cross-sectional area: $A_{se_N} = \frac{\pi}{4} \left(d_a - \frac{0.9743\text{-in}}{n_t} \right)^2$ (Ref: ASME B1.1, "Unified Inch Screw Threads (UN and UNR Thread Form)," ASME, Fairfield, NJ, 2003)

$$A_{se_N} = 1\text{-in}^2 \quad A_{se_V} = A_{se_N}$$

The steel strength of one anchor in tension: $N_{sa} = A_{se_N} \cdot f_{uta}$ (D-3)

$$\phi_T N_{sa} = (0.75)(1)(58)$$

$$\phi_T N_{sa} = 43.487 \text{ kip} > N_{ua} = 20 \text{ kip (OK !)}$$

The steel strength of one anchor in shear: $V_{sa} = 0.8 \cdot 0.6 A_{se_V} \cdot f_{uta}$ (D-20)

$$\phi_V V_{sa} = (0.65)(0.8)0.6(1)(58)$$

$$\phi_V V_{sa} = 18.091 \text{ kip} > V_{ua} = 10 \text{ kip (OK !)}$$

Note: Shear strength of anchors with base plate supported on grout pads shall be multiplied by 0.8 (D.6.1.3).

Since $N_{ua} > 0.2 \phi_T N_{sa}$ and $V_{ua} > 0.2 \phi_V V_{sa}$, check interaction equation (D-32):

$$\frac{N_{ua}}{\phi_T N_{sa}} + \frac{V_{ua}}{\phi_V V_{sa}} = 1.013 < 1.2, \text{ OK !}$$

The recommended minimum embedment depth of the non-sleeve 1.25-in anchor per Section 3.2.2 of this ASCE Committee report: $h_{ef, \min} = 12d_a = 15 \text{ in}$

Note: Since the pedestal height is 28 inches, try $h_{ef} = 24 \text{ in}$. This effective embedment depth will be checked to determine if it is sufficient for the required development length of the vertical reinforcing bars.

2. Check the pullout resistance of anchor in tension (D.5.3.4)

Section D.5.3.4 indicates the load at which the concrete above the anchor head begins to crush. Since the local crushing above the head will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure, the pullout resistance of the anchor in tension must be ensured to be larger than the factored tension load (N_{ua}).

If the ductile design (which is not considered herein) is performed, the pullout resistance of the anchor in tension should be larger than the tensile capacity of the anchor ($\phi_T N_{sa}$).

Use the heavy hex nut (on the anchor head) with the flat-to-flat dimension of 2 inches.

$$\text{Net bearing area: } A_{brg} = 0.866 \cdot 2^2 - \frac{\pi}{4} \cdot 1.25^2 \quad A_{brg} = 2.237 \text{ in}^2$$

The pullout resistance in tension of a single headed bolt: $N_{pn} = \psi_{c_P} 8 A_{brg} f'_c$ (D-14 and D-15)

Assume concrete cracks: $\psi_{c_P} = 1$ (D.5.3.6) $N_{pn} = (1)8(2.237)(4) = 71.6 \text{ kip}$

Strength reduction factor for anchor governed by pullout is 0.70 ($\phi = 0.70$, Condition B per D.4.4.(c)). Note that even though supplementary reinforcement is provided, Condition B applies for pullout strength.

Therefore: $\phi N_{pn} = (0.70)(71.6) = 50.1 \text{ kip} > N_{ua} = 20 \text{ kip} \quad \text{OK !}$

3. Check the side-face blowout resistance of anchor in tension (D.5.4)

Because of symmetry, only side-face blowout resistance on one face of the pedestal needs to be considered.

Strength reduction factor for anchor governed by side-face blowout, assuming condition A (supplementary reinforcement is provided to tie the failure prism): $\phi = 0.75$ (D.4.4.(c))

Check if the corner effect (for the corner anchor) and close spacing (for the group of anchors) have to be considered:

Corner effect: $c_{a2} < 3 c_{a1} \rightarrow$ Consider corner effect

$$N_{sb} = 160 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot \sqrt{f_c} \cdot \frac{1 + \frac{c_{a2}}{c_{a1}}}{4} \quad (\text{D-17, } \lambda=1)$$

$$N_{sb} = 160 \cdot (8) \cdot \sqrt{2.237} \cdot \frac{\sqrt{4000}}{1000} \cdot \frac{1 + (1)}{4} = 60.5$$

$$N_{sb} = 60.5 \text{ kip} > N_{sa} = 58 \text{ kip} \quad \text{OK!}$$

Even though it is not required when ductile design is not performed, it is a good practice to have $N_{sb} > N_{sa}$ so that the failure is not governed by a non-ductile side-face blowout failure.

$$\phi N_{sb} = (0.75)(60.5) = 45.4 \text{ kip} > N_{ua} = 20 \text{ kip} \quad \text{OK!}$$

Close spacing effect: $s_1 < 6 c_{a1} \rightarrow$ Consider close spacing

$$N_{sbg} = \left(1 + \frac{s_1}{6 \cdot c_{a1}} \right) \cdot N_{sb} \quad (\text{D-18}) \quad N_{sbg} = 70.583 \text{ kip}$$

This N_{sbg} is for the two anchors located on one face of the pedestal.

$$\phi N_{sbg} = (0.75)(70.583) = 52.9 \text{ kip} > 2 * N_{ua} = 2(20) = 40 \text{ kip} \quad \text{OK!}$$

4.2 Development length

The vertical rebar should be developed on either side of the potential failure plane. The part of the rebar above the failure surface is commonly straight and the part of the rebar that goes into the mat is commonly terminated with a 90-degree hook (as shown in Figure A1-2). Therefore, the straight-bar development length applies to the part of the rebar above the failure surface and the 90-degree hooked bar development length applies to the part of the rebar below the failure surface excluding the portion between the failure surface and the construction joint at the base of the pedestal.

Since the development length for the 90-degree hooked bar (below the failure surface) is part of the pedestal/foundation design, it is not considered in this calculation.

Development length for straight bars above the failure surface:

The minimum development length, l_d (≥ 12 in), is determined based on Sections 12.2.2 and 12.2.4 of ACI 318-08 as follows:

Bar location factor: $\psi_t = 1$ (for vertical bar)

Coating factor: $\psi_e = 1$ (for uncoated bar)

Concrete density factor: $\lambda = 1$ (for normal concrete)

For # 6 and smaller bars, use:
$$l_d = \left[\frac{f_y \cdot (\psi_t \cdot \psi_e)}{25 \lambda \sqrt{f'_c}} \right] \cdot d_b \quad l_d = 28.5 \text{ in}$$

Available development length based on the pier height and the embedment depth of the anchor bolt:

$$l_{d_available} = h_{ef} - \text{Concrete_cover} - d_{actual} \cdot \tan(35\text{deg})$$

From Figure A1-2, $d_{actual} = 6.95$ in $l_{d_available} = 24 - 1.5 - 6.95 \tan(35\text{deg}) = 17.63$ in $< l_d$

However, since the provided number of effective rebar is significantly more than the required number of rebars and for low seismic risks, l_d can be reduced using the excess reinforcement factor per 12.2.5 (but cannot be less than 12 in. per 12.2.1).

$$l_{d_reduced} = l_d \cdot \frac{A_{s_required}}{A_{s_provided}} \quad l_{d_reduced} = 28.5 \cdot \frac{1}{3} = 9.5 \text{ in} \quad (12 \text{ in. min.})$$

$$l_{d_reduced} = 12 \text{ in.} < l_{d_available} \quad \text{OK !}$$

4.3 Strut-and-Tie Method for Tension Loading (Section 3.5.3.1.4 of this ASCE Committee report)

Using the STM design approach, it is assumed that the diagonal concrete struts propagate radially from the anchor head. As a result of the diagonal concrete struts, there are radially horizontal force components propagating from the anchor head. In Figure A1-3, the resultant of the radial horizontal component propagating from the anchor head is denoted F , which depends on the concrete bearing pressure on the head. Anchor reinforcement in the form of regular transverse ties or hairpins should be designed to carry F . Since the majority of action occurs at the bearing surface of the anchor head, it is recommended that the anchor reinforcement be placed as closely as possible to the bearing surface of the anchor head.

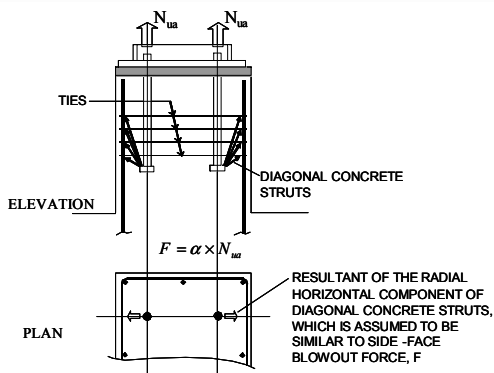


Figure A1-3

Bearing pressure at the anchor head: $p = \frac{20\text{kip}}{A_{\text{brg}}}$ $p = 8.941\text{ ksi}$

Section 3.5.3.1.3 of this ASCE Committee report:

$$\alpha = \max\left(0.25, 0.1 \cdot \sqrt{\frac{p}{f_c}}, 0.037 \cdot \frac{p}{f_c}\right) \quad \alpha = 0.25$$

$$F = \alpha \cdot 20\text{kip}$$

$$F = 5\text{-kip}$$

Assuming the ties will be fully-develop, the required number of layers of # 4 ties to resist F is:

$$n_{\text{layers}} = \frac{F}{\phi_s \cdot (A_{s_{\text{tie}}} \cdot f_y)} \quad n_{\text{layers}} = 0.556$$

Therefore, 1 layer of tie is required to be placed as close to the bearing surface of the anchor head as possible.

Herein, since the available area for nodes and struts are assumed to be sufficiently large, the STM for tension loading is used only to proportion ties (i.e. vertical reinforcing bars and horizontal ties) based on the overall equilibrium of the system. Based on vertical force equilibrium, the vertical reinforcing bars should be proportioned to carry the total tension force N_{ua_total} in the pedestal (i.e., Section 4.1 of this example problem).

5. Design of shear reinforcement

5.a. Use the concept of concrete breakout cone

Calculate the available development length l_{da} :

Layer A: $l_{da_A_L} = 13.1in$

$l_{da_A_R} = 7.9in$

Layer B: $l_{da_B_L} = 11in$

$l_{da_B_R} = 10in$

The required hooked development length to fully-develop the ties:

$$l_{dh} = \left(\frac{0.02 \cdot \psi_e \cdot f_y}{\lambda \cdot \sqrt{f'_c}} \right) \cdot d_{tie} \quad (\text{Section 12.5.2 of ACI 318-08})$$

$$l_{dh} = 9.5in$$

Note: This approach is conservative considering that typical shear reinforcement design for columns assumes that the shear reinforcement can be fully-developed when closed ties are used. In this example, the maximum steel stress is calculated based on the ratio between the available development length and the required hooked development length to fully-develop the ties.

In order to simplify the problem, the minimum l_{da} is used.

The steel stress based on the minimum l_{da} :

$$f_s = \frac{\min(l_{da_A_L}, l_{da_A_R}, l_{da_B_L}, l_{da_B_R})}{l_{dh}} \cdot f_y$$

$$f_s = 49.895\text{-ksi}$$

Required area of ties: $A_{tie_required} = \frac{V_{ua_total} \cdot X}{0.75 \cdot f_s}$

$$A_{tie_required} = 0.534 \cdot in^2$$

Requires 3 legs of # 4 tie: $3 \times 0.2 \text{ in}^2 / \text{tie} = 0.6 \text{ in}^2 > 0.534 \text{ in}^2$

Use two layers of # 4 tie as shown in Figure A1-4.

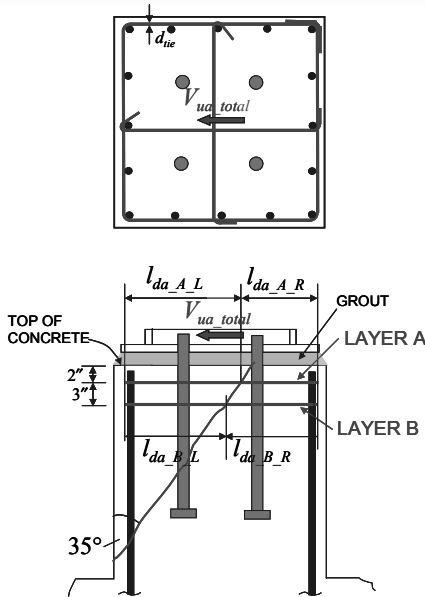
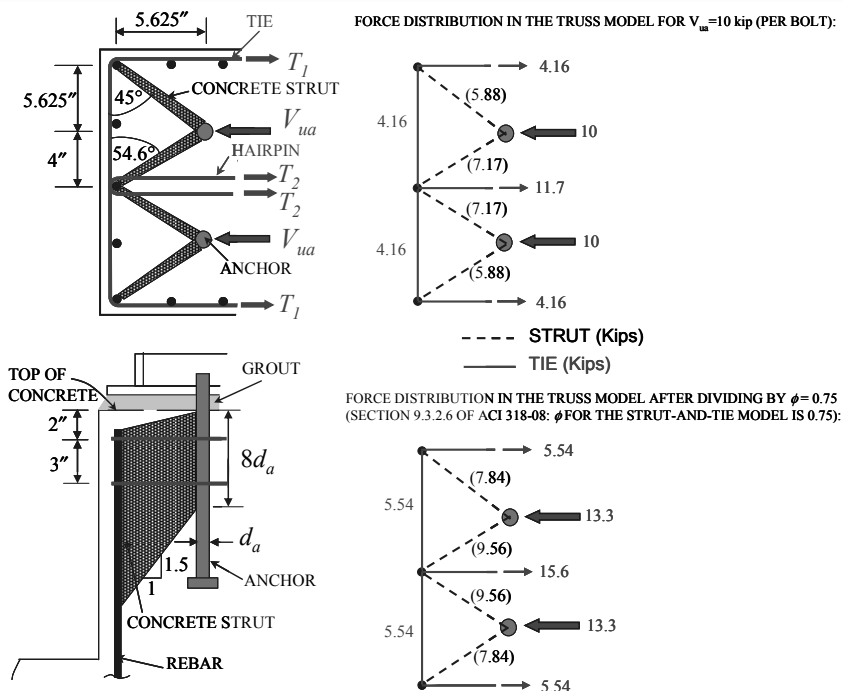


Figure A1-4

5.b. Use the concept of Strut-and-Tie Method approach (Section 3.5.3.2.2 of this ASCE Committee report).

Assumptions:

1. Strut-and-tie modeling (Figure A1-5) is used to analyze shear transfer to concrete pedestal and to design the required amount of shear reinforcement.
2. Since the shear forces in both directions are the same and the total number of anchors resisting the total shear forces in both directions are the same, only the shear in the X-direction is presented in this example problem.



In this example, the Strut-and-Tie Model that requires internal ties (i.e., T_2) is used. As explained in Section 3.5.3.2.2 of this report, the Strut-and-Tie Model that does not require internal ties can also be used. However, when internal ties are not provided, the exterior ties (i.e., T_1) will be heavier.

5.b.1 Check a geometry of the truss model to see if a direct strut can develop

Since the angles between the axes of all struts and ties entering a single node are larger than 25 degrees, direct struts can develop (Section A.2.5 of ACI 318-08).

5.b.2 Develop a truss model and calculate member forces

The truss model and member forces are shown in Figure A1-5.

5.b.3 Check strength of bearing

Assume concrete strength for checking the strength of bearing and compression struts: $f_{ce} = 0.85 f'_c$

5.b.3.a. Bearing of the anchor

$$\text{Bearing area: } A_{\text{bearing_anc}} = 8 \cdot d_a \cdot d_a$$

$$A_{\text{bearing_anc}} = 12.5 \cdot \text{in}^2$$

$$\text{Strength: } f_{ce} = 0.85 \cdot (f'_c)$$

$$f_{ce} = 3400 \cdot \text{psi} > \frac{13.3 \text{ kip}}{A_{\text{bearing_anc}}} = 1064 \cdot \text{psi} \quad \text{OK !}$$

5.b.3.b. Bearing of the reinforcing bars

By inspection, bearing on the rebar at the node D (Figure A1-6) governs (larger force):

$$\begin{aligned} \text{The clear distance between the nodes B and D, } l_{BD}: \quad l_{BD} &= \sqrt{(5.625 \cdot \text{in})^2 + (4 \cdot \text{in})^2} - \frac{d_a}{2} - \frac{d_b}{2} \\ l_{BD} &= 5.902 \cdot \text{in} \end{aligned}$$

$$\text{Bearing area: } A_{\text{bearing_rebar}} = (8 \cdot d_a + 1.5 \cdot l_{BD} - \text{Concrete_cover}) \cdot d_b$$

$$A_{\text{bearing_rebar}} = 13.015 \cdot \text{in}^2$$

Even though the tension ties are only present at the location of the shear reinforcement, the strength of nodal zone anchoring two or more ties per Section A.5.2.3 of ACI 318-08 is conservatively used.

$$\beta_n = 0.6$$

$$\text{Strength: } f_{ce} = 0.85 \cdot \beta_n \cdot (f'_c) \quad (\text{A-8})$$

$$f_{ce} = 2040 \cdot \text{psi} > \frac{9.56 \text{ kip}}{A_{\text{bearing_rebar}}} = 735 \cdot \text{psi} \quad \text{OK !}$$

**FORCE (KIPS) DISTRIBUTION IN THE TRUSS MODEL
AFTER DIVIDING BY $\phi=0.75$:**

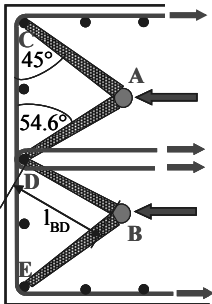
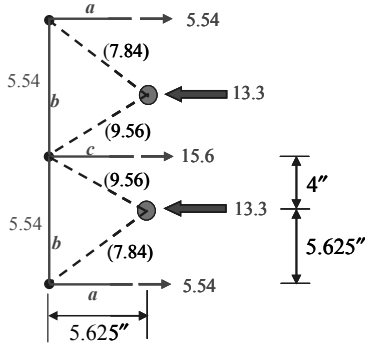


Figure A1-6



5.b.4. Check strength of struts

Since it is assumed that the strength of strut is the same as the bearing strength ($f_{ce} = 0.85 \beta_n f'_c$) and the available area for struts is typically larger than the available area for bearing, the bearing strength governs over the strength of struts. Therefore, if the bearing strengths at the anchor and rebar are OK, the strength of struts does not need to be checked.

5.b.5. Select tie reinforcement

Assumptions (See Section 3.5.3.2.2 of this ASCE Committee report):

1. Only the top most two layers of ties (within 5" of pedestal as required by Section 7.10.5.6 of the ACI 318-08), shown in Figure A1-7, are effective.
2. Tie reinforcement consists of tie with seismic hooks. Hairpins are used as internal ties.
3. The location of hooks and the direction of hairpins are alternated as shown in Figure A1-7.
4. At the nodes away from the hook (e.g., Node 6 on Layer A), the tie is assumed to be fully developed.
5. At the node where the hook is located (e.g., Node 6 on Layer B), the contribution of the hoop to the tension ties T is: $T_1 = A_{s_tie} \cdot (20 \text{ ksi})$

$$T_1 = A_{s_tie} \cdot 20 \text{ ksi}$$

$$T_1 = 4 \text{ kip}$$

$$T_{\text{hook}} = T_1$$

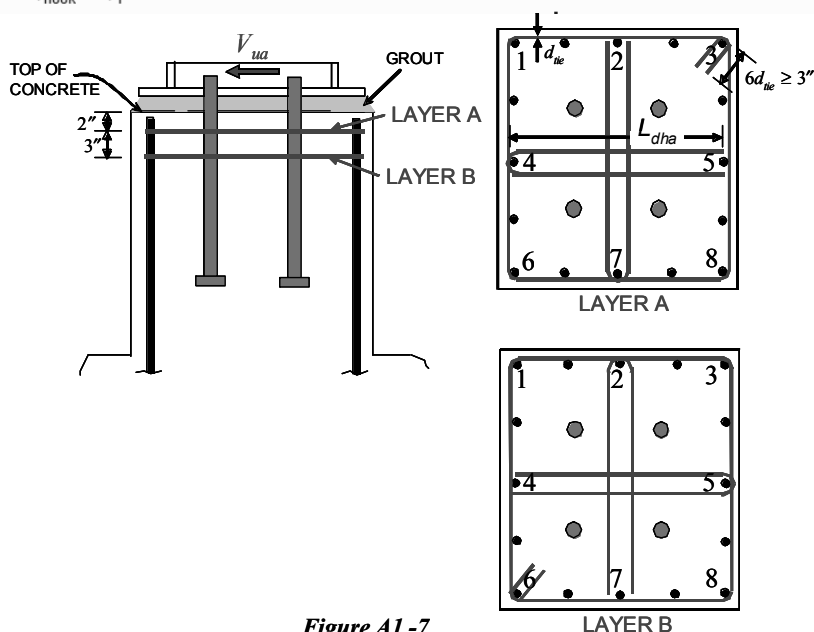


Figure A1-7

Note: The required one layer of tie very close to the bearing surface of the anchor head (from Section 4.3 of this calculation) is not shown in Figure A1-7.

Ties a and b (see Figure A1-6):

Ties a and b are resisted by exterior ties.

Assuming that one layer of the exterior tie can develop f_y and the other layer can provide T_{hook} :

$$\begin{aligned}\text{Total resistance: } R_{tot_ab} &= A_{s_tie} \cdot f_y + T_{hook} \\ R_{tot_ab} &= 16 \cdot \text{kip} > 5.54 \text{ kip} \quad \text{OK !}\end{aligned}$$

Tie c (see Figure A1-6):

Tie c is resisted by a hairpin.

$$\begin{aligned}\text{Diameter of hairpin: } d_{hairpin} &= 0.5 \text{ in} & A_{s_hairpin} &= \frac{\pi}{4} \cdot d_{hairpin}^2 \\ \text{Yield stress of hairpin: } f_{y_hairpin} &= 60 \text{ ksi}\end{aligned}$$

Check the stress that can be developed in the hairpin:

$$\begin{aligned}\text{Check available length of the hairpin: } l_{dha} &= 24 \text{ in} - 2 \cdot \text{Side_cover} - 2 \cdot d_{tie} \\ l_{dha} &= 19 \text{ in}\end{aligned}$$

Required straight development length for a fully developed hairpin:

$$\begin{aligned}\text{Bar location factor: } \psi_t &= 1.3 \\ \text{Coating factor: } \psi_e &= 1 \quad (\text{for uncoated bar}) \\ \text{Concrete density factor: } \lambda &= 1 \quad (\text{for normal concrete})\end{aligned}$$

$$\begin{aligned}\text{For \# 6 and smaller bars, use: } l_{d_hairpin} &= \left[\frac{f_y \cdot (\psi_t \cdot \psi_e)}{25 \lambda \sqrt{f_c}} \right] \cdot d_{hairpin} \\ l_{d_hairpin} &= 24.7 \text{ in} \quad \text{cannot be less than 12 in per Section 12.2.1 of the ACI 318-08.}\end{aligned}$$

The stress that can be developed in the hairpin:

$$f_{s_hairpin} = \frac{l_{dha}}{l_{d_hairpin}} \cdot f_{y_hairpin} \quad f_{s_hairpin} = 46.154 \text{ ksi}$$

Since the direction of hairpin is alternated, only one layer of hairpin can be accounted as tie reinforcement.

$$\begin{aligned}\text{Total resistance: } R_{tot_c} &= 2A_{s_hairpin} \cdot f_{s_hairpin} \quad (\text{Note: 2 legs per hairpin}) \\ R_{tot_c} &= 18.125 \cdot \text{kip} > 15.6 \text{ kip} \quad \text{OK !}\end{aligned}$$

6. Check the minimum distance requirements to preclude splitting failure

The following minimum distances for anchors shall be satisfied unless reinforcement is provided to control splitting.

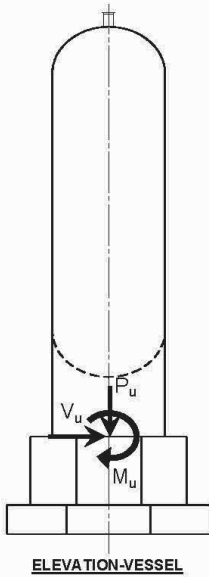
I. Center-to-center spacing (D.8.1): $s_{\min_untorqued} = 4 \cdot d_a = 5 \text{ in} < \min(s_1, s_2)$ OK !

II. Minimum edge distance (D.8.2):

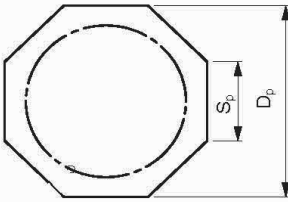
For untorqued cast-in anchors, the minimum edge distances shall be based on minimum cover requirements for reinforcement specified in Section 7.7 of ACI 318-08.

$c_{\min_untorqued} = \text{cover} = 1.5 \text{ in} < \min(c_{a1}, c_{a2})$ OK !

EXAMPLE 2: ANCHOR DESIGN FOR OCTAGONAL PEDESTAL



ELEVATION-VESSEL



PLAN-OCTAGON

1. GIVEN:

Anchor Circle Dia. (D_{bc}): 144 inch
Anchors: 12 Qty 1 1/2" Dia. with Heavy Hex Nuts
Material: ASTM F1554 Grade 36
 f'_c : 4000 psi
Face to Face of Pedestal (D_p): 13.28 ft
No supplementary reinforcement provided
WIND LOADS (Strength Level)

M_u : 1328 k-ft
 V_u : 44 kips

EARTHQUAKE LOADS (Operating-Strength Level)

M_u : 600 k-ft
 V_u : 20 kips
 E_v : 9 kips

Siesmic Design Category: B

VESSEL WEIGHTS (Unfactored)

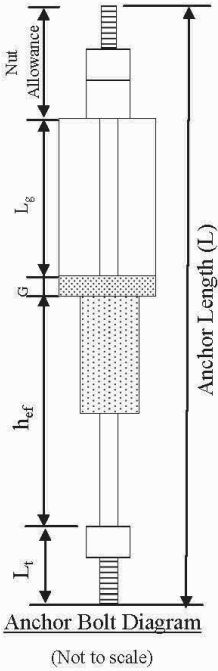
Empty Weight, W_e : 98 kips
Operating Weight W_o : 240 kips

REQUIRED:

Check adequacy of anchorage for given loads

2. Anchor General Information

Anchor Length, L = 2.67 ft
Grout G = 1 in
Grip Length L_g = 10 in
Thread Length at Bottom L_t = 2.5 in
Nut Allowance (2 nuts) = 4 in
Effective Embedment Length, h_{ef} = 14.50 in
Anchor Circle Dia., D_{bc} = 144.00 in
Number of Anchors, n_a = 12 Nos.
Effective C/S Area $A_{se,N}$ = 1.41 in²
Bearing Area of Head/Stud A_{brg} = 3.12 in²
Specified Tensile Strength f_{uta} = 58 ksi
Specified Yield Stress f_{ya} = 36 ksi



Anchor Bolt Diagram

(Not to scale)

3. Determine The Factored Shear Force Applied to Single Anchor (V_{ua})**a) Check For Shear Due To Wind under Empty Condition**

$$\text{Empty Weight, } W_e = 98 \text{ kips}$$

$$\text{Factored Wind Moment, } M_u = 1328 \text{ kip-ft}$$

$$\text{Factored Wind Base Shear, } V_{ua} = 44 \text{ kips}$$

$$\text{Factored Comp. Force } P_u = M_u / (0.67 D_{bc}) + 0.5(0.9 W_e)$$

Due to gravity load and couple force from overturning moment

$$P_u = 1328 \times 12 / (0.67 \times 144.00) + 0.5 \times (0.9 \times 98)$$

$$\text{Thus, } P_u = 209.3 \text{ kips}$$

Determine if shear is taken 100% by shear friction or 100% by the anchorage

$$\text{Friction @ Base } V_f = \mu P_u \quad \text{where } \mu = 0.55$$

$$V_f = 0.55 \times 209.3 = 115 \text{ kips}$$

No supplementary reinforcement provided thus, for shear $f = 0.70$

ACI 318 - D.4.4

$$\phi V_f = 0.70 \times 115$$

$$\text{Thus, } \phi V_f = 80.6 \text{ kips}$$

which is $> V_{ua} = 44$ kips, Thus, no shear on the anchors

b) Check For Shear Due To Earthquake under Operating condition

Given :

$$\text{Unfactored Operating Weight, } W_o = 240 \text{ kips}$$

$$\text{Factored Applied EQ Moment, } M_u = 600 \text{ kip-ft}$$

$$\text{Factored Applied EQ Base Shear, } V_{ua} = 20 \text{ kips}$$

$$E_v = 9 \text{ kips}$$

$$\text{Factored Comp. Force } P_u = M_u / (0.67 D_{bc}) + 0.5(0.9 W_o) - E_v / 2$$

$$P_u = 600 \times 12 / (0.67 \times 144.00)$$

$$+ 0.5 \times (0.9 \times 240) - 0.5 \times 9$$

$$P_u = 178 \text{ kips}$$

Determine if shear is taken 100% by shear friction or 100% by the anchorage

$$\text{Friction @ Base } V_f = \mu P_u \quad \text{where } \mu = 0.55$$

$$V_f = 0.55 \times 178 = 98 \text{ kips}$$

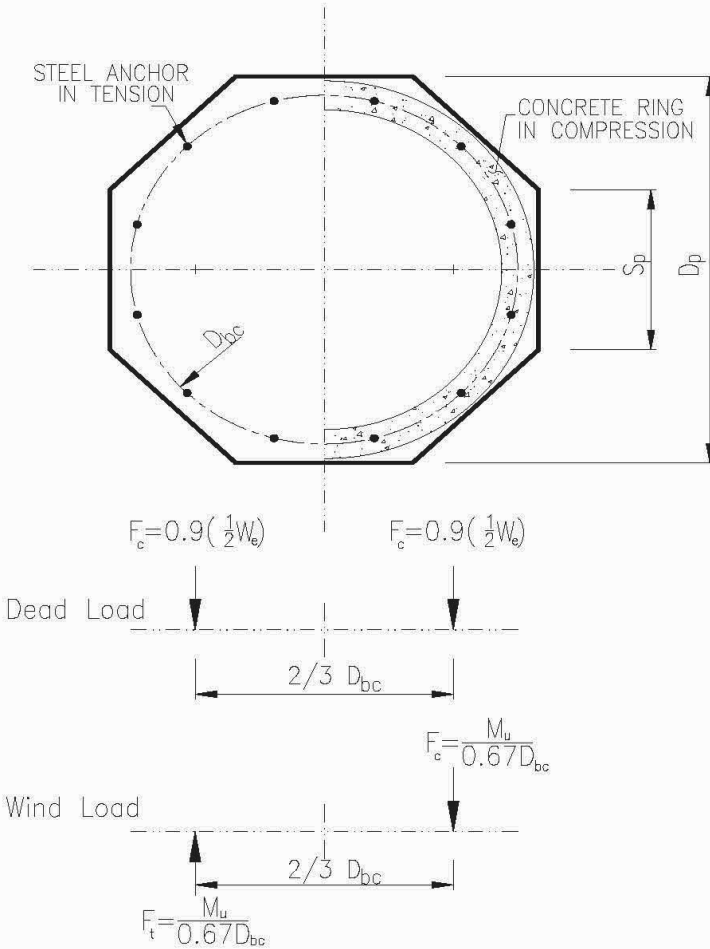
No supplementary reinforcement provided thus, for shear $f = 0.70$

$$\text{Thus, } \phi V_f = 0.70 \times 98 = 68.6 \text{ kips}$$

which is $> V_{ua} = 20$ kips, Thus, no shear on the anchors

Shear is transferred directly from the vessel base plate to the pedestal by shear friction.

4. Determine the Maximum Factored Tensile Load on Single Anchor (N_{ua})



Based on inspection M_u Wind $>$ M_u EQ, thus wind will govern the loads

Unfactored Empty Weight, $W_e = 98$ kips

Factored Wind Moment, $M_u = 1328$ k-ft (Strength Level)

Maximum Factored Tensile Load on the Anchors (N_{ua})

$$N_{ua} = \frac{4M_u}{n_a D_{bc}} - \frac{0.9W_e}{n_a}$$

$$N_{ua} = 4 \times 1328 \times 12 / (12 \times 144.00) - 0.9 \times 98 / 12 = 29.5 \text{ kips}$$

5. Determining the Strength of Anchorage in Tension and Shear

(a) Steel Strength for Anchor in Tension (ϕN_{sa}) ACI 318 - D.5.1

Supplementary reinforcement provided (Condition A)

Tension Resistance Factor $\phi = 0.75$ ACI 318 - D.4.4a

Anchor governed by strength of a ductile steel element

Nominal Tensile Strength $N_{sa} = n_a A_{se,N} f_{uta}$ ACI 318 - Eq. D-3

where, $n_a = 1$ for single anchor

$$\begin{aligned} N_{sa} &= 1 \times 1.41 \times 58 = 81.8 \quad \text{kips} \\ \phi N_{sa} &= 0.75 \times 81.8 = 61.3 \quad \text{kips} \end{aligned}$$

(b) Steel Strength for Anchor in Shear (ϕV_{sa})

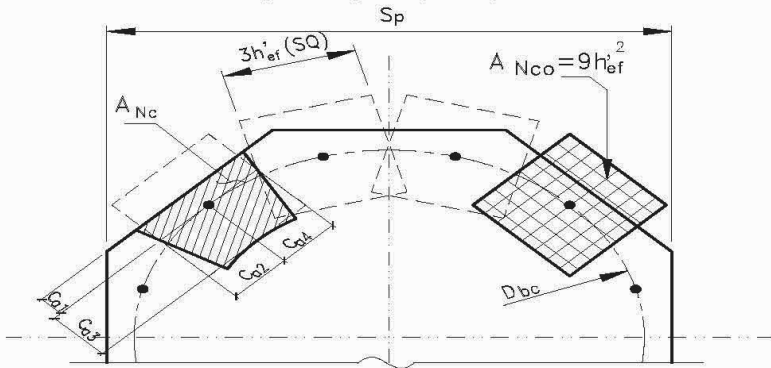
Shear is taken 100% by friction as shown in Part 3 above.

(c) Concrete Breakout Strength of a Single Anchor in Tension (ϕN_{cb})

ACI 318 - D.5.2

Using the Saw-Cut Method for calculating N_{cb}

[Ref : Wey et al (2010-1) Saw Cut Method - Method 2]



$$\text{Pedestal Side, } S_p = D_p / 2.414 = 13.28 / 2.414 = 5.50 \quad \text{ft}$$

$$\text{Pedestal Area, } A_p = (0.8284 \times (13.28 \times 12)^2) = 21038 \quad \text{in}^2$$

Embedment Length, h_{ef}

Edge Distances:

$$c_{a1} = (D_p - D_{bc}) / 2 = (12 \times 13.28 - 144.00) / 2 = 7.68 \quad \text{in}$$

$$c_{a2} = \frac{D_{bc}}{2} \tan\left(\frac{360}{2n_a}\right)$$

$$c_{a2} = 144.00 / 2 \times \tan\left(\frac{360}{(2 \times 12)}\right) = 19.29 \quad \text{in}$$

$$c_{a4} = c_{a2} = 19.29 \quad \text{in}$$

Limited Effective Embedment Length, h'_{ef}

All three edge distances are less than $1.5h'_{ef}$, therefore;

$$c_{a,max} = \text{MAX} (7.68, 19.29, 19.29) = 19.29 \text{ in}$$

$$h'_{ef} = c_{a,max} / 1.5 = \text{MIN} (14.50, 19.29 / 1.5) = 12.86 \text{ in} \quad \text{ACI 318 - D.5.2.3}$$

$$c_{a3} = \text{Min}(1.5 h'_{ef}, 0.5 D_{bc}) = \text{MIN} (1.5 \times 12.86, 0.5 \times 144.00)$$

$$c_{a3} = 19.29 \text{ in}$$

$$c_{a,min} = \text{MIN} (7.68, 19.29, 19.29, 19.29) = 7.68 \text{ in}$$

Projected Concrete Failure Area, A_{Nco}

$$A_{Nco} = 9h'^2_{ef} = 9 \times 12.86^2 = 1489 \text{ in}^2 \quad \text{ACI 318 - Eq. D-6}$$

Effective Concrete Failure Area, A_{Nc}

$$A_{Nc} = \frac{A_p - \frac{\pi}{4} [(D_{bc} - \text{Min}(3h'_{ef}, D_{bc}))^2]}{n_a}$$

$$A_{Nc} = [21038 - 3.14 / 4 \times \{144.00 - \text{MIN}(3 \times 12.86, 144.00)\}^2] / 12$$

$$\text{Calculated } A_{Nc} = 1026 \text{ in}^2$$

For a single anchor $A_{Nc} > A_{Nco}$ thus,

$$A_{Nc} = \text{Min}(\text{Calculated } A_{Nc} \text{ and } n_a A_{Nco}) \quad \text{ACI 318 - D.5.2.1}$$

$$A_{Nc} = \text{MIN} (1026, 1489)$$

$$\text{Thus, } A_{Nc} = 1026 \text{ in}^2$$

$$A_{Nc} / A_{Nco} = 1026 / 1489 = 0.69$$

Basic Concrete Breakout Strength, N_b

$$N_b = k_c \lambda \sqrt{f'_c} h'^{1.5}_{ef} \quad \text{ACI 318 - Eq. D-7}$$

$$k_c = \frac{24}{\text{(cast-in anchors)}} \quad f'_c = \frac{4000}{\text{psi}}$$

$$\lambda = \frac{1.00}{\text{(reduction factor for light weight concrete)}}$$

$$N_b = (24 \times 1 \times 4000^{0.5} \times 12.86^{1.5}) / 1000 = 70.0 \text{ kips}$$

• Modification Factors, ψ

$$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 h'_{ef}} \quad (c_{a,min} < 1.5 h'_{ef}) \quad \text{ACI 318 - Eq. D-11}$$

$$\psi_{ed,N} = 0.7 + (0.3 \times (7.68 / (1.5 \times 12.86))) = 0.819$$

$$\psi_{c,N} = \underline{1.25} \quad \text{(cast-in anchor in uncracked section)} \quad \text{ACI 318 - D.5.2.6}$$

$$\psi_{cp,N} = \underline{1.00} \quad \text{ACI 318 - D.5.2.7}$$

• Concrete Breakout Strength of Anchor in Tension

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318 - D-4}$$

$$N_{cb} = (1026 / 1489) \times 0.819 \times 1.25 \times 1.00 \times 70.0 =$$

$$N_{cb} = 49.4 \text{ kips}$$

$$\phi = 0.7 \quad \text{ACI 318 - D.4.4c (Condition B)}$$

$$\phi N_{cb} = 49.4 \times 0.7 = 34.6 \text{ kips}$$

(d) Concrete Breakout Strength in Shear (ϕV_{cb})

Shear is taken 100% by shear friction as shown in Part 3 above.

(e) Pullout Strength of anchor in tension (ϕN_{pn})

ACI 318 - D.5.3

$$N_{pn} = \psi_{c,p} N_p$$

ACI 318 - Eq. D-14

$$\psi_{c,p} = 1.4 \quad (\text{Analysis indicates no cracking at service loads})$$

$$\text{Pullout strength of a single anchor } N_p = 8 A_{brg} f'_c$$

ACI 318 - Eq. D-15

$$\text{using, } f'_c = 4000 \text{ psi}$$

$$N_p = 8 \times 3.12 \times 4000 / 1000 = 99.8 \text{ kips}$$

$$\phi = 0.7$$

ACI 318 - D.4.4c

$\phi N_{pn} = .7 \times 1.4 \times 99.8 =$	97.8 kips
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(f) Concrete side-face blowout strength of anchor in tension (ϕN_{sb})

ACI 318 - D.5.4

$$N_{sb} = F_{cor} (160 c_{a1} \sqrt{A_{brg}}) \lambda \sqrt{f'_c}$$

ACI 318 - Eq. D-17

$$\text{As calculated in step 5.c, } c_{a1} = 7.68 \text{ in}$$

$$c_{a2} = 19.29 \text{ in}$$

$$\text{Effective Embedment Length, } h_{ef} = 14.50 \text{ in}$$

$$\lambda = 1$$

$$\text{Factor } F_{cor} = (1 + c_{a2}/c_{a1})/4, \text{ if } c_{a2} < 3c_{a1}$$

ACI 318 - D.5.4.1

$$\text{where, } 1 \leq c_{a2}/c_{a1} \leq 3.0$$

$$F_{cor} = 0.878 \text{ as, } c_{a2} < 3c_{a1}$$

$$N_{sb} = 0.878 \times (160 \times 7.68 \times 3.12^{0.5} \times 1 \times 4000^{0.5}) / 1000$$

$$N_{sb} = 121 \text{ kips}$$

$$\phi = 0.7$$

ACI 318 - D.4.4c

$\phi N_{sb} = .7 \times 121 =$	84.4 kips
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(g) Concrete pryout strength of anchor in shear

Shear is taken 100% by shear friction as shown in Part 3 above.

6. Calculate the Lowest Design Tensile Strength of Anchorage

ACI 318 - D.4.1.2

$$\phi N_n = \text{Min}(\phi N_{sa}, \phi N_{cb}, \phi N_{pn}, \phi N_{sb})$$

$$\phi N_n = \text{MIN}(61.3, 34.6, 97.8, 84.4) = 34.6 \text{ kips}$$

ACI 318 - D.4.1.1

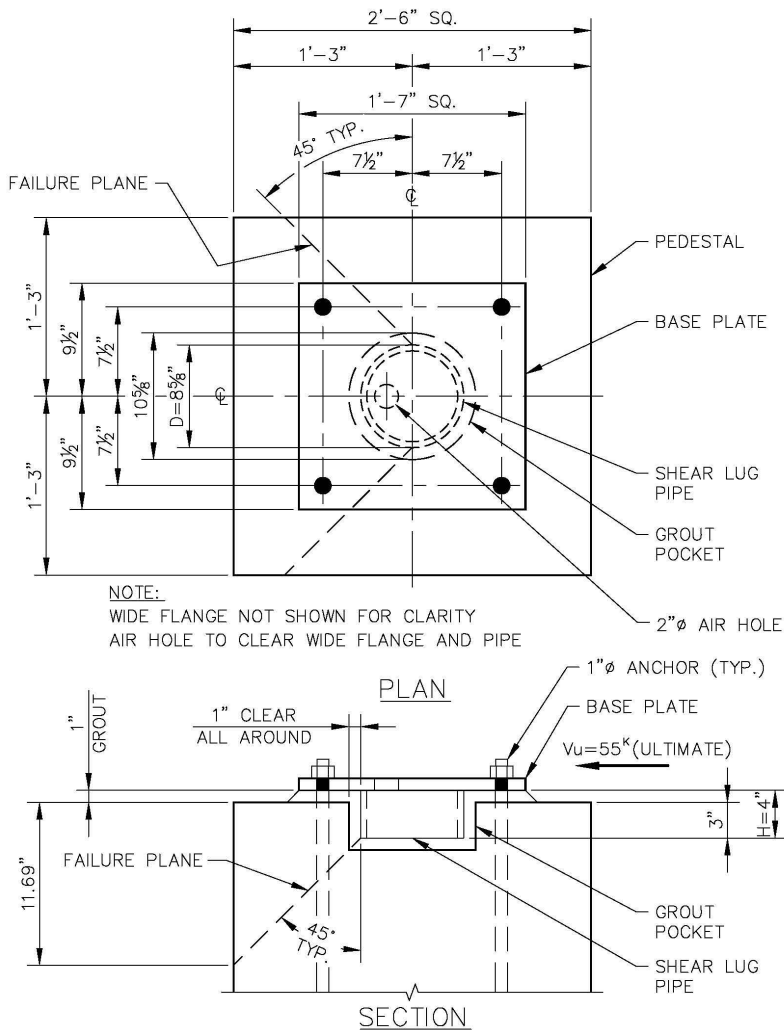
7. Conclusion

$$\phi N_n = 34.6 \text{ kips} > N_{ua} = 29.5 \text{ kips}$$

$$\text{Unity Ratio} = 29.5 / 34.6 = 0.85$$

Thus, OK. Anchorage adequate for Tensile Failure modes

EXAMPLE 3: SHEAR LUG PIPE SECTION DESIGN



Design a shear lug pipe section for a 19-in. square base plate, subject to a factored axial dead load of 25 kips, a factored axial live load of 50 kips, and a factored horizontal shear load of 55 kips. The base plate and shear lug have $F_y = 36$ ksi and the concrete has a strength, $f'_c = 4$ ksi. The contact plane between the grout and base plate

is assumed to be 1 in. above the concrete (coefficient of friction, $\mu = 0.55$). A 2-ft 6-in. square pedestal is assumed. Ductility is not required. $\phi = 0.75$ for the concrete breakout strength of the pipe in shear per ACI 318-08 D.4.4 (Condition A, supplementary reinforcement is present due to the ties at the top of the pedestal). For bearing of the lug against the concrete, $\phi = 0.65$ per ACI 318 Section 9.3.2.4.

Note: References to the AISC Manual in this example are to the 13th Edition.

$$V_{ua} = V_u - V_f = 55 - (0.55)(25) = 41.3 \text{ kips}$$

$$\text{Bearing area} = A_{req} = V_{ua} / (0.85 \phi f'_c) = 41.3 \text{ kips} / (0.85 * 0.65 * 4 \text{ ksi}) = 18.7 \text{ in}^2$$

(AISC Steel Design Guide 1)

Based on base plate size, assume the pipe diameter will be 8-in. nominal std. weight pipe.

$$(D = 8.63 \text{ in}; D/t = 28.8; Z = 20.8 \text{ in}^3; \text{Area} = 7.85 \text{ in}^2)$$

(AISC Manual, Table 1-14)

$$\text{Height of pipe} = H = (A_{req} / D) + G = (18.7 \text{ in}^2 / 8.63 \text{ in}) + 1 \text{ in} = 3.2 \text{ in. Use } 4.0 \text{ in.}$$

$$\begin{aligned} \text{Factored moment} &= M_u = V_{ua} * (G + (H - G)/2) \\ &= 41.3 \text{ kips} * (1 \text{ in.} + (4.0 \text{ in.} - 1 \text{ in.})/2) = 103 \text{ k-in.} \end{aligned}$$

Check Moment:

Check if pipe section is compact per AISC Manual Table B4.1, Case 15:

$$D/t = 28.8$$

$$\lambda_p = 0.07 E/F_y = 0.07 * 29000/36 = 56.4 > 28.8$$

Therefore section is compact and buckling does not apply.

$$M_n = F_y * Z = 36 \text{ ksi} * 20.8 \text{ in}^3 = 749 \text{ k-in}$$

(AISC Specification equation F8-1)

$$\phi_b = 0.9$$

$$\phi_b M_n = (0.9)(749 \text{ k-in.}) = 674 \text{ k-in.} > 103 \text{ k-in.} \quad \text{OK}$$

Check Shear:

$$\begin{aligned} V_n &= 0.6 F_y \text{ Area} \\ &= 0.6 * 36 \text{ ksi} * 7.85 \text{ in}^2 = 169.6 \text{ kips} \end{aligned}$$

$$\phi_v = 0.9$$

$$\phi_v V_n = (0.9)(169.6 \text{ kips}) = 152.6 \text{ kips} > 41.3 \text{ kips} \quad \text{OK}$$

This 4.0-in.-long x 8-in.-diameter nominal std. weight pipe will be sufficient to carry the applied shear load and resulting moment.

Design weld:

Minimum weld size = 3/16 in (AISC Manual Table J2.4)

Try 3/16" fillet weld:

Capacity of 3/16 in. fillet weld - LRFD

For $F_{EXX} = 70$ ksi $\phi = 0.75$

$\phi F_w = \phi 0.60 F_{EXX} = 0.75 * 0.60 * 70 \text{ ksi} = 31.5 \text{ ksi}$
(AISC Manual Table J2.5 Shear)

Load on weld:

$M_u = 103 \text{ k-in}$ $V_u = 41.3 \text{ kips}$ $t = 3/16 \text{ in}$

Area of weld = $A_w = \pi D t = \pi * 8.63 * 3/16 = 5.08 \text{ in}^2$

Section Modulus of weld, $S_x = t \pi r^2$ (Blodgett – Table 2)

$r = \frac{1}{2} D = 4.315$ $S_x = 3/16 * \pi * (4.315)^2 = 10.97 \text{ in}^3$

$f_w = [(M_u/S_x)^2 + (V_u/A_w)^2]^{0.5} = [(103 \text{ k-in} / 10.97 \text{ in}^3)^2 + (41.3 \text{ k} / 5.08 \text{ in}^2)^2]^{0.5}$
 $= 12.4 \text{ ksi} < 31.5 \text{ ksi}$ OK

Check concrete breakout strength of the shear lug in shear.

Distance from edge of pipe to edge of concrete = $(30 - 8.625) / 2 = 10.69 \text{ in}$

Projected breakout area is calculated assuming a 45-degree plane from the bearing edge of the shear lug to the free surface. The bearing area of the shear lug is excluded from the projected area. (ACI 349-06 Section D.11.2)

Projected breakout area = $A_{Vc} = 30 * 13.69 - 8.63 * 3 = 385 \text{ in}^2$

Concrete Breakout Strength = $V_{cb} = A_{Vc} * 4 * \phi * [f'_c]^{0.5}$
(ACI 349-06 Section D11.2)

$V_{cb} = 385 * 4 * 0.75 * [4000]^{0.5} = 73049 \text{ lb} = 73.0 \text{ kips} > 41.3 \text{ kips}$ OK

Note: If the concrete break out strength was not adequate, reinforcing could be designed to transfer the load across the assumed failure plane with adequate rebar embedment on both sides of the failure plane.

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NOTATION

A_{brg}	net bearing area of the head, bearing nut or bearing plate of the stud or anchor, in ² (mm ²)
$A_{bearing_anc}$	assumed bearing area of a compression strut on an anchor in Strut-and-Tie Model for shear, in ² (mm ²)
$A_{bearing_rebar}$	Assumed bearing area of a compression strut on a reinforcing bar in Strut-and-Tie Model for shear, in ² (mm ²)
A_d	nominal area of anchor, in ² (mm ²)
A_{Nc}	projected concrete failure area of a single anchor or group of anchors for calculation of strength in tension, in ² (mm ²)
A_{Nco}	projected concrete failure area of a single anchor for calculation of strength in tension if not limited by edge distance or spacing, in ² (mm ²)
A_p	pedestal area, in ² (mm ²)
A_{req}	bearing area required for shear lug, in ² (mm ²)
A_s	area of nonprestressed longitudinal tension reinforcement, in ² (mm ²)
$A_{se,N}$	effective cross-sectional area of anchor in tension, in ² (mm ²)
A_{se_tie}	area of one leg of tie reinforcement, in ² (mm ²)
$A_{se,V}$	effective cross-sectional area of anchor in shear, in ² (mm ²)
A_{st}	total area of longitudinal nonprestressed reinforcement, in ² (mm ²)
A_{Vc}	projected concrete failure area of a single anchor, group of anchors, or shear lug for calculation of strength in shear, in ² (mm ²)
A_w	area of weld, in ² (mm ²)
b_1	pedestal dimension in one direction, in (mm)
b_2	pedestal dimension in the direction perpendicular to b_1 , in (mm)
C	cover distance to top of rebar, in (mm)
$c_{a,max}$	maximum distance from center of an anchor shaft to the edge of concrete, in (mm)
$c_{a,min}$	minimum distance from center of an anchor shaft to the edge of concrete, in (mm)
c_{a1}	Distance from center of an anchor shaft to the edge of concrete in one direction, in (mm). If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance
c_{a2}	distance from center of an anchor shaft to the edge of concrete in the

	direction perpendicular to c_{a1} , in (mm)
c_{a3}	Distance from center of an anchor shaft to the edge of the effective tensile stress area towards the center of an octagon shaped concrete pedestal, in (mm). See Example 2.
c_{a4}	Distance from center of an anchor shaft to the edge of the effective tensile stress area opposite to c_{a2} of an octagon shaped concrete pedestal, in (mm). See Example 2.
C_d	ratio of deflection of vertical vessel due to deflection from elastic analysis to total deflection
D	vertical dead load, lbs (N); diameter of pipe or weld, in (mm)
d_a	outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, in (mm)
d_{actual}	actual distance between an anchor and reinforcing bars under consideration, in (mm)
d_b	nominal diameter of rebar, in (mm)
D_{bc}	bolt circle diameter, in (mm)
d_{max}	maximum distance between an anchor and reinforcing bars where the reinforcing bars can be considered to be effective for resisting anchor tension, in (mm)
D_p	Face-to-face dimension of pedestal, ft (m)
d_s	diameter of sleeve shell, in (mm)
d_{tie}	nominal diameter of tie reinforcement bar, in (mm)
E_{sh}	coefficient for shrinkage, in/in (mm/mm)
E_v	vertical component of seismic load, kips (kN)
F	side-face blowout force, kips (kN)
f'_c	specified compressive strength of concrete, psi (kPa)
F_c	compression force at anchors, kips (kN)
$f_{cc,200}$	the concrete compressive strength based on a 200 mm cube, psi (kPa)
f_{ce}	effective compressive strength of the concrete in a strut or nodal zone (Strut-and-Tie Model [STM]), psi (kPa)
F_{cor}	factor to modify the side-face blowout near a corner
F_{EXX}	electrode classification number, ksi (MPa)
F_r	maximum horizontal dynamic force, kips (kN)
f_t	desired tensile stress in anchor due to tensioning, psi (kPa)

F_t	tension force at anchors, kips (kN)
f_{uta}	specified tensile strength of anchor steel, psi (kPa)
f_w	weld stress, ksi (MPa)
F_w	nominal strength of weld metal per unit area, ksi (MPa)
f_y	specified yield strength of reinforcement, psi (kPa)
F_y	specified yield strength of structural steel, psi (kPa)
f_{ya}	specified yield strength of anchor steel, psi (kPa)
G	grout thickness, in (mm)
h	distance from center of seismic load on a vertical vessel to the bottom of the vertical vessel base plate, in (mm)
H	height of pipe used for shear lug, in.; height of vertical vessel, in (mm)
h'_e	minimum nut-sleeve clearance, in (mm)
h_{ef}	effective embedment depth of anchor, in (mm)
h'_{ef}	limiting value of h_{ef} when anchors are located less than 1.5 h_{ef} from three or more edges, in (mm)
h_s	height of sleeve, in (mm)
I	importance factor
k_c	coefficient for basic concrete breakout strength in tension
L	length of anchor, in (mm); length of weld, Fig. 5.5, in (mm)
LA	lever arm between centroid of tension loads on anchors and the centroid of the compression load, in (mm)
l_d	development length in tension of reinforcement, in (mm)
l_{da}	available development length of reinforcement, in (mm)
l_{dh}	development length in tension of reinforcement with a standard hook, in (mm)
l_{dha}	available development length of hairpin, in (mm)
l_e	load bearing length of anchor for shear, in (mm)
L_g	grip dimension of anchor bolt, in (mm)
$L_{stretch}$	anchor stretch length (the distance between the top and bottom nuts on the anchor), in (mm)
L_t	thread length at bottom of anchor, in (mm)
M_u^E	factored overturning moment at the vessel base due to seismic effect acting alone, k-ft (kN-m)

M_n	nominal flexural strength, k-in (kN-mm)
M_u	factored overturning moment, k-ft (kN-m)
n or n_a	number of anchors
n_layers	required number of layers of ties to resist the resultant of the radial horizontal component of diagonal concrete struts (assumed to be similar to side-face blowout force)
N_{arf}	tension to be taken by anchor reinforcement, lb (N)
N_b	basic concrete breakout strength in tension of a single anchor in cracked concrete, lb (N)
N_{cb}	nominal concrete breakout strength in tension of a single anchor, lb (N)
N_{cbg}	nominal concrete breakout strength in tension of a group of anchors, lb (N)
N_{dse}	controlling tension for ductile steel element failure, lb (N)
N_n	nominal strength in tension, lb (N)
N_p	pullout strength in tension of a single anchor in cracked concrete, lb (N)
N_{pn}	nominal pullout strength in tension of a single anchor due to crushing of concrete under anchor head, lb (N)
N_{sa}	nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, lb (N)
N_{sb}	side-face blowout strength of a single anchor, lb (N)
N_{sbg}	side-face blowout strength of a group of anchors, lb (N)
n_t	anchor threads per in (mm)
N_{ua}	factored tensile force applied to anchor or group of anchors, lb (N)
p	bearing stress on the head of an anchor, psi (kPa)
P_u^E	factored compression force at top of pedestal due to seismic effect acting alone, (including the vertical component of seismic load acting upward), lb (N)
P_u	normal factored compression force, lb (N)
r	radius, in (mm)
s_1	center-to-center spacing of anchors in one direction, in (mm)
s_2	center-to-center spacing of anchors in the direction perpendicular to s_1 , in (mm)
S_p	face dimension of octagonal pedestal, ft (m)
S_x	section modulus of weld, in ³ (mm ³)

t	wall thickness of pipe; thickness of weld, in (mm)
T, T_t	tensile force on tie, lb (N)
T_2	tensile force on hairpin, lb (N)
T_u^E	factored tension design load from load combinations that include an overstrength factor of 2.5 applied to the seismic loads (per anchor), lb (N)
V_{cb}	nominal concrete breakout strength in shear of a single anchor or shear lug, lb (N)
V_{cbg}	nominal concrete breakout strength in shear of a group of anchors, lb (N)
V_{cp}	nominal concrete pryout strength of a single anchor, lb (N)
V_{dse}	controlling shear for ductile steel element failure, lb (N)
V_f	resisting friction force, lb (N)
V_n	nominal shear strength, lb (N)
V_{sa}	nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb (N)
V_u	factored shear force at section, lb (N)
V_{ua}	factored shear force applied to single anchor, group of anchors, or shear lug, lb (N)
W_a	equipment weight at anchor location, lb (N)
W_e	vessel empty weight, lb (N)
W_o	vessel operating weight, lb (N)
z	vertical hairpin concrete cover + $0.5d_b$, in (mm)
Z	plastic section modulus, in ³ (mm ³)
α	ratio of F to N_{ua}
β_n	factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone
Δ_a	amount of stretch in anchor, in (mm)
Δ_A	amplified displacement at the top of vertical vessel, in (mm)
Δ_{ie}	inelastic portion of displacement at the top of vertical vessel, in (mm)
Δ_s	deflection at the top of vertical vessel from elastic analysis, in (mm)
λ	modification factor related to unit weight of concrete
λ_p	limiting slenderness parameter for compact element

μ	coefficient of friction
ϕ	strength reduction factor
ϕ_b	resistance factor for flexure (structural steel)
ϕ_s	strength reduction factor used for anchor reinforcement design
ϕ_T	strength reduction factor, tension loads
ϕ_V	strength reduction factor, shear loads
ϕ_v	resistance factor for shear (structural steel)
$\Psi_{c,N}$	factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete
Ψ_c	factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete
Ψ_c	factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement
$\Psi_{cp,N}$	factor used to modify tensile strength of post-installed anchors intended for use in uncracked concrete without supplementary reinforcement
Ψ_e	factor used to modify the development length because of reinforcement coating
$\Psi_{ec,N}$	factor used to modify tensile strength of anchors based on eccentricity of applied loads
$\Psi_{ec,V}$	factor used to modify shear strength of anchors based on eccentricity of applied loads
$\Psi_{ed,N}$	factor used to modify tensile strength of anchors based on proximity to edges of concrete member
$\Psi_{ed,V}$	factor used to modify shear strength of anchors based on proximity to edges of concrete member
Ψ_l	factor used to modify development length based on reinforcement location
Ω_o	seismic overstress factor

GLOSSARY

- Anchorage** – A structural assembly designed to transmit all components of the design force from a structure or equipment to the foundation; it consists of a combination of anchors, shear lugs, concrete, and reinforcement.
- Attachment** – An element used to transfer the design force from a structure or equipment to the anchors, shear lug, and foundation; it consists of plates or structural members (such as wide flange shapes or channels).
- Anchor** – A rod element of the anchorage used to transmit components of the design force from a structure or equipment to the foundation. Anchor types include cast-in-place rods, welded studs, and manufactured post-installed elements.
- Embedment** - Portion of the anchorage that is within the concrete foundation. The following anchorage elements could be considered part of the embedment, depending on the anchorage detail: reinforcement, the attachment, the shear lug, or a portion of the anchor.
- Shear Lug** – A short element of the anchorage used to transmit the portion of the shear component of the design force that exceeds the frictional resistance from the structure or equipment to the foundation; it consists of plate(s) or a structural member (such as a wide flange shapes, square structural tubes, or pipes).

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