Best Practice

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Anchor Bolt
Design and Installation

Document Responsibility: Onshore Structures

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Design and Installation

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1 Scope

This Saudi Aramco Best Practice is based on the PIP Design Guide STE05121 and is intended to provide the engineer and designer with guidelines for anchor bolt design and installation. This Best Practice also covers anchor bolt materials, details, strength design, ductile design, reinforcing, shear lugs and pretensioning. This document is intended to be used in conjunction with ACI 318-08, Appendix D, therefore many of the design requirements contained in ACI 318-08 are not repeated here.

2 Use of Best Practice

2.1 Disclaimer

The material in this Best Practices document provides the most correct and accurate design guidelines available to Saudi Aramco which comply with international industry practices. This material is being provided for the general guidance and benefit of the Designer. Use of the Best Practices in designing projects for Saudi Aramco, however, does not relieve the Designer from his responsibility to verify the accuracy of any information presented or from his contractual liability to provide safe and sound designs that conform to Mandatory Saudi Aramco Engineering Requirements. Use of the information or material contained herein is no guarantee that the resulting product will satisfy the applicable requirements of any project. Saudi Aramco assumes no responsibility or liability whatsoever for any reliance on the information presented herein or for designs prepared by Designers in accordance with the Best Practices. Use of the Best Practices by Designers is intended solely for, and shall be strictly limited to, Saudi Aramco projects.

2.2 Conflicts with Mandatory Standards

In the event of a conflict between this Best Practice and other Mandatory Saudi Aramco Engineering Requirement, the Mandatory Saudi Aramco Engineering Requirement shall govern.

3 References

This Best Practice is based on the latest edition of the references below, unless otherwise noted.

3.1 Saudi Aramco References

Saudi Aramco Engineering Standards

SAES-Q-005 *Concrete Foundations*

SAES-Q-007 *Foundations for Heavy Machinery*

Saudi Aramco Materials System Specification

12-SAMSS-007 *Fabrication of Structural and Miscellaneous Steel*

Saudi Aramco Standard Drawing

AA-036322-001 *Anchor Bolt Details – Inch and Metric Sizes*

3.2 Industry Codes and Standards

American Concrete Institute (ACI)

ACI 318-08 *Building Code Requirements for Reinforced Concrete and Commentary*

ACI 349 *Code Requirements for Nuclear Safety Related Concrete Structures, Appendix B*

ACI 355.1R *State-of-the-Art Report on Anchorage to Concrete*

ACI 355.2 *Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete*

American Institute of Steel Construction (AISC)

AISC Manual of Steel Construction - *Thirteenth Edition*. Short title used herein is "AISC Manual".

Steel Design Guide Series 1- *Column Base Plates, Some Practical Aspects of Column Base Selection*, David T. Ricker

American National Standards Institute

ANSI A10.13 *Construction and Demolition Operations – Steel Erection – Safety Requirements*

American Society of Civil Engineers (ASCE)

ASCE Report *Design of Anchor Bolts for Petrochemical Facilities*

ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*

ASTM International

ASTM A36/A36M *Specification for Carbon Structural Steel*

ASTM A143/A143M *Standard Practice of Safeguarding Against Embrittlement of Hot-Dip Galvanized*

	Structural Steel Products and Procedure for Detecting Embrittlement
ASTM A193/A193M	*Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*
ASTM A307	*Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength*
ASTM A563	*Specification for Carbon Steel and Alloyed Steel Nuts*
ASTM F1554	*Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

American Welding Society

AWS D1.1	*Structural Welding Code - Steel*

International Code Council

IBC	*International Building Code*

Occupational Safety and Health Administration (OSHA)

OSHA 29 CFR 1910	*Industrial Safety and Regulatory Compliance*

Process Industry Practices

REIE 686	*Recommended Practice for Machinery Installation and Installation Design*
STE05121	*Anchor Bolt Design Guide*

4 Definitions and Notation

4.1 Use of "Shall's" and "Should's"

Throughout this Practice the word "shall" is used if the item is required by mandatory standard or code and the word "should" is used if the item is just recommended or a good practice.

4.2 Definitions

Cast-in Anchor: A headed bolt, anchor rod or hooked bolt that is installed before the concrete is places.

Post-Installed Anchor: An anchor installed in hardened concrete. Expansion, undercut and adhesive anchors are examples of post-installed anchors.

Pretensioned Anchor Bolt: An anchor bolt that is designed to be tensioned to a predefined force to prevent premature failure of the anchor bolt due to fatigue.

Shear Lug: A pipe or plate section welded to the bottom of a base plate that is used to resist base shear forces.

Stretch Length: The un-bonded length of a pretensioned anchor between the bottom of the nut and top of the anchor plate.

4.3 Notation

***Note*:** Force and stress units are lb and psi respectively. At times, it is more convenient to show these units in the text, tables, and examples as kips and ksi, respectively. Where this is done, the units will always be shown.

Ad = Nominal bolt area, inches^2
ANc = Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, inches^2
Ase = Effective cross-sectional area of anchor, inches^2
Ar = Reinforcing bar area, inches^2
Arb = Required total area of reinforcing bars, inches^2
Areq = Required bearing area of shear lug, inches^2
AVc = Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, inches^2
AVco = Projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, inches^2
AC = Anchor circle diameter (Figures B-1 and B-2), inches^2
C = Clear distance from top of reinforcing bar to finished surface (concrete cover), inches
ca = Distance from center of an anchor shaft to the edge of concrete, inches
ca,max = Maximum distance from center of an anchor shaft to the edge of concrete, inches
Ca,min = Minimum distance from center of an anchor shaft to the edge of concrete, inches
ca1 = Distance from the center of an anchor shaft to the edge of concrete in one direction, inches. If shear is applied to anchor, **ca1** is taken in the direction of the applied shear. If the tension is applied to the anchor, **ca1** is the minimum edge distance.
ca2 = Distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to **ca1**, inches
cb = Smaller of (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, inches

D = Octagonal pedestal .diameter. (flat to flat), inches
D = Outside diameter of shear lug pipe section, inches
db = Nominal diameter of bar, wire, or prestressing strand, inches
do = Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, inches
ds = Anchor sleeve diameter, inches
E = Elastic modulus of bolt, psi
fc' = Specified compressive strength of concrete (shall not be taken as greater than 10,000 psi), psi
ft = Desired tensile stress, psi
futa = Specified tensile strength of anchor steel, psi
fy = Specified yield strength of reinforcement, psi
fya = Specified yield strength of anchor steel, psi
G = Grout thickness, inches
H = Height of shear lug plate or pipe, inches
Hb = Overall length of anchor under the head or above the base nut (Figure A), inches
he' = Length of anchor below the sleeve (Figure A), inches
hef = Effective embedment depth of anchor (Figure A), inches
hs = Length of anchor sleeve (Figure A), inches
Ktr = Transverse reinforcement index
L = Length of shear lug plate or pipe, inches
l = Bolt stretch length (the distance between the top and bottom nuts on the bolt), inches
la, lb = Portions of standard hook development length (Table 3), inches
ld = Development length of reinforcing bar, inches
ldh = Actual development length of standard hook in tension, inches
lhb = Basic development length of standard hook in tension, inches
Mu = Ultimate moment on shear lug plate or pipe, k-inches or k-inches/inches
Mn = Nominal flexural strength of shear lug pipe, k-inches
n = Number of anchors
Ncb = Nominal concrete breakout strength in tension of a single anchor, lb
Ncbg = Nominal concrete breakout strength in tension of a group of anchors, lb
Npn = Nominal pullout strength in tension of a single anchor, lb
Nsa = Nominal strength of a single anchor in tension as governed by the steel strength, lb
Nsb = Side-face blowout strength of a single anchor, lb
Nsbg = Side-face blowout strength of a group of anchors, lb
P = Normal compression force beneficial to resisting friction force, lb
P = Anchor projection from top of concrete (Figure A), inches
P1 = Anchor projection below bottom nut for Type 2 anchors (Figure A), inches
s = Anchor spacing, center to center, inches

S = Section modulus of shear lug pipe, inches
t = Thickness of the shear lug plate or pipe wall, inches
T = Tensile rebar capacity, lb
Tlc = Bolt threads per unit length
Vapp = Applied shear load on shear lug, kip
Vcb = Nominal concrete breakout strength in shear of a single anchor or shear lug, lb
Vcbg = Nominal concrete breakout strength in shear of a group of anchors, lb
Vcp = Nominal concrete pryout strength of a single anchor, lb
Vf = Resisting friction force at base plate, lb
Vn = Nominal shear strength, lb
Vsa = Nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, lb
Vua = Factored shear force applied to a single anchor or groups of anchors, lb
W = Width of shear lug plate perpendicular to shear force, inches
Wh = Width of anchor head or nut, inches
X = Clear distance between anchor nut and reinforcing bar, inches
Z = Plastic modulus of shear lug pipe, inches3
λ = Modification factor related to unit weight of concrete
φ = Strength reduction factor
φ**b** = Steel resistance factor for flexure
φ**v** = Steel resistance factor for shear
ψ**t =** Factor used to modify development length based on reinforcement location
ψ**e =** Factor used to modify development length based on reinforcement coating
ψ**s =** Factor used to modify development length based on reinforcement size
ψ**c,V** = Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement for anchors in shear (see *ACI 318-05 D.6.2.7*)
μ = Coefficient of friction

5 Materials and Coatings

5.1 Anchor Bolts, Nuts and Washers

5.1.1 Anchor Bolts and Rods

For most ordinary structures ASTM A307 headed bolts, ASTM A36/A36M rods, or ASTM F1554 grade 36 rods should be specified. Applications requiring high-strength materials should use ASTM A193/A193M grade B7 or ASTM F1554 grade 105 anchor bolts and rods. The table below provides properties for anchor bolt materials specified in 12-SAMSS-007. The ASCE Anchor Bolt Report, Chapter 2 contains a more comprehensive list of common materials for used for anchors bolts & rods.

Anchor Bolts Materials and Properties

<table>
<tr><td colspan="2">Anchor Material Type</td><td>f_{ya}
ksi (Mpa)</td><td>f_{uta}
ksi (Mpa)</td><td>Galvanize?</td><td>Ductile?</td></tr>
<tr><td colspan="2">A 307</td><td>Not Clearly Defined</td><td>60 (410)</td><td>Yes</td><td>Yes</td></tr>
<tr><td colspan="2">A 36 or F1554 Grade 36</td><td>36 (250)</td><td>58 (400)</td><td>Yes</td><td>Yes</td></tr>
<tr><td colspan="2">F1554 Grade 105</td><td>105 (720)</td><td>125 (860)</td><td>Yes*</td><td>Yes</td></tr>
<tr><td rowspan="3">A 193 Grade B7
Based on bolt diameter (d_b)</td><td>$d_o \leq$ 2.5" (64 mm)</td><td>105 (720)</td><td>125 (860)</td><td>Yes*</td><td>Yes</td></tr>
<tr><td>2.5" (64 mm) < d_o $\leq$ 4" (102 mm)</td><td>95 (660)</td><td>115 (790)</td><td>Yes*</td><td>Yes</td></tr>
<tr><td>4" (102 mm) < d_o $\leq$ 7" (180 mm)</td><td>75 (515)</td><td>100 (690)</td><td>Yes*</td><td>Yes</td></tr>
</table>

* See section 5.3.1 for safeguarding against embrittlement.

Note: Bolts made from ASTM F1554 grade 105, ASTM A193 grade B7, and ASTM A354 materials should not be welded as part of the bolt fabrication process. Therefore, tack welding of anchor nut as shown in for Type 2 anchor bolts on Standard Drawing AA-036322.001 is not permitted for these bolt materials. Alternatively, two anchor nuts jammed together or a plate jammed between two nuts could be provided in place of the tack-welded nut as shown for the Type 3 standard anchor bolts.

5.1.2 Suitable nuts are shown in the table below.

Anchor Bolt Nut Specifications

Anchor Material Specification	Nut Material Specification
ASTM A307, Grade A	ASTM A563, Gr. A
ASTM A36/A36M	ASTM A563, Gr. A
ASTM F1554, Gr. 36	ASTM A563, Gr. A
ASTM A193/A193M, Gr. B7	ASTM A563, DH Heavy-Hex or ASTM A194/A194M
ASTM F1554 Gr. 105	ASTM A563, DH Heavy-Hex or ASTM A194/A194M

5.1.3 Washers

Because base plates typically have holes oversized holes to allow for tolerances on the location of the anchor rod, washers are usually furnished from ASTM A36 steel plate. They may be round, square, or rectangular, and generally have holes 1/16-in. larger than the anchor rod diameter. The thickness must be suitable for the forces to be transferred. Minimum washer sizes are given in Table 14.2 of the AISC Manual of Steel Construction 13th Edition.

5.2 Sleeves

Anchors should be installed with sleeves when small movement of the bolt is desired after the bolt is set in concrete. There are two types of sleeves commonly used with anchors. A partial length sleeve is primarily used for alignment requirements, while the full sleeve is used for alignment as well as pretensioning. Partial length sleeves do not affect the tensile capacity of a headed anchor or plate bolt because the tension in the anchor is transferred to the concrete through the head and does not rely on the bond between the anchor and surrounding concrete. Sleeved anchors can only resist shear forces when the sleeve is filled with grout. The two most common examples follow:

5.2.1 A partial length sleeve is used where precise alignment of anchors is required during installation equipment or where bolt groups or patterns involve six or more anchor bolts with interdependent dimensional requirements. In this situation, the sleeve should be filled with grout after installation is complete. Partial length sleeves should not be used for base plates on structural steel columns which have oversized holes.

5.2.2 Full length anchor bolt sleeves are used when anchor bolts will be pretensioned in order to maintain the bolt under continuous tensile stresses during load reversal. Pretensioning of anchor bolts requires the bolt to be un-bonded over a well defined "stretch length." When sleeves are used for pretensioned bolts, the top of these sleeves should be sealed and the sleeve should be filled with elastomeric material to prevent grout or water from filling the sleeve. (See Figure G)

5.3 Coatings and Corrosion

Corrosion of an anchor can be a serious situation affecting the strength and design life of the anchor. When deciding which anchor material to use or what precaution to take against corrosion, consider the following.

a) Is the anchor encased in concrete or exposed to the elements?

b) What elements will the anchor contact?

Chemical compounds
Salt water

Raw water or salt water used to wash down plant areas

Ground water

Caustic gases

c) What limitations are present, affecting anchor size, length, and material, fabrication options, availability, and cost?

5.3.1 Galvanizing is the preferred coating for ASTM A307 bolts and ASTM A36/A36M and ASTM F1554 grade 36 threaded rods. ASTM F1554 grades 55 and 105, ASTM A193/A193M grade B7 bolts may also be galvanized if appropriate safeguards are in place. Where loss of ductility is an issue, ASTM A143/A143M provided guidance concerning safeguarding hot-dip galvanized steel against embrittlement.

5.3.2 A 3 mm (1/8 in) corrosion allowance is required on anchor bolts subjected to highly corrosive environments, (such coastal areas, anchors subjected to frequent wash down water or frequent deluge testing, etc.). This corrosion allowance is accomplished by sizing the anchor bolt based on the design loads and then adding 3 mm (1/8 in) to the diameter required by design. (Refer to SAES-Q-005)

5.3.3 Pedestal design and anchor arrangement should consider water collection and anchor environment in order to reduce the amount of contact with corrosive substances or plant wash down water. Standard SAES-Q-005 requires the top of pedestals to be at least 150 mm above the finished paving surface.

5.3.4 If the engineer determines prolonged contact with a corrosive substance is unavoidable, a metallurgist should be consulted to determine alternate anchor materials or protective options.

6 Details and Layout

6.1 Bolt Types

6.1.1 Cast-in-place anchor bolts can come in several configurations. The following is a list of some of the most common types:

- Headed bolts or threaded rods with heavy hex nuts. This type of anchor is the type used on the Standard Drawing AA-036322-001.
- J & L shaped anchor rods. J & L shaped rods were common in the past but are less common now. Current design practices do not rely on the concrete bond to develop the tension capacity of smooth rods bent into J and L shapes. While these rods still appear in ACI 318-08, they

have no advantage over headed bolts and can be harder to fit into small foundation pedestals. Saudi Aramco no longer allows J and L shaped anchor rods.

- Anchor rods with anchor plates. Plate bolts are not normally required because research has shown that a headed bolt or an anchor rod with a heavy hex nut attached will develop the full strength of the anchor rod in tension for the steel materials used in the best practice. The main purpose for an anchor plate is when bolts will be pretensioned and it is advantageous to reduce the concrete bearing stresses in order to maintain a large percentage of the initial pretension force.
- Sleeved pretensioned bolts with anchor plates. Anchor plates must be used for postensioned bolts with full length sleeves to provide the necessary bearing area outside of the sleeve.
- Anchor bolts with sleeve couplers. Sleeve nuts are sometimes used to eliminate the projecting portion of the anchor bolt during equipment installation. These would be used when the equipment cannot be lifted onto the anchor bolts and must be rolled into position. Sleeve nut embedded in the concrete can pose a potential corrosion problem and should be avoided when possible.

6.1.2 Post-Installed Anchor Bolts

This Best Practice was written primarily to cover cast-in anchors; however there are occasions where a post-installed anchor bolt must be used. Post-installed bolts include; expansion bolts, undercut bolts, adhesive anchors, etc. Design procedures for some types of post- installed bolts are covered by ACI 318-08, Appendix D. Other bolt types, like adhesive bolts, are normally covered by the manufacturer's technical data. All post installed bolts should be qualified per ACI 355.2.

6.2 Anchor Bolt Layout

Determining the number, type, projection and diameter of anchor bolts should be done as follows:

6.2.1 Structural steel base plates require a minimum of four anchor bolts for stability during construction per the latest OSHA safety requirements. The layout of the anchor bolts and required foundation pedestal size should be established based on the design forces and minimum dimensions in Section 6.3 below. Doubly-symmetrical anchor bolt patterns are preferred because if the bolt pattern is inadvertently rotated 90 degrees the base plate will still fit on the anchor bolts.

6.2.2 The minimum anchor bolt size is 20 mm (¾") for most items per SAES-Q-005, however there is an exception for very small equipment when suggested by the manufacturer or for small miscellaneous steel items such as ladders, stair stringers, small base pipe supports, etc.

6.2.3 For most structures and equipment, ordinary strength anchor bolts can be used (ASTM A307, ASTM A36/A36M or F1554 gr. 36). If the anchor bolt loads require anchor bolts in excess of 50 mm (2"), high-strength anchor bolt material should be considered.

6.2.4 Anchor Bolt Projection

Anchor bolt projection above the top of concrete is computed as follows:

Single Nuts

Thickness of grout + thickness of base plate (or height of anchor bolt chair above the top of grout) + 2.0 x diameter of anchor bolt

Double Nuts

Thickness of grout + thickness of base plate (or height of anchor bolt chair above the top of grout) + 3.0 x diameter of anchor bolt

6.2.5 The thread length required at the top of the anchor bolt must be sufficient to accommodate two nuts and about ½ of the anchor bolt diameter projecting above the top nut. Normally a thread length of about 4 bolt diameters of will be sufficient to provide some tolerance for errors in the elevation of the anchor bolt placement in the field. The Standard Drawing AA-036322-001 calls for 4 bolt diameters of thread length for Types 2 & 3 anchor bolts. Type 1 bolts have a standard ASTM A307 thread length which is less than 4 bolt diameters.

6.3 Minimum Dimensions

Minimum edge distance shall be in accordance with the ACI 318-08 Code and should be in accordance with ASCE recommendations. Minimum embedment and anchor spacing should be in accordance with the recommendations of the ASCE Anchor Bolt Report. Refer to Table 1 and Figure A.

6.3.1 Edge distance

(a) ACI 318-08 requires that cast-in headed fasteners, which will be torqued, have minimum edge distances of $6d_o$.

(b) Standard SAES-Q-005 requires the clear distance for anchor bolts or anchor bolt sleeves to the edge of the concrete shall be a minimum of 100 mm. This clear distance is intended to prevent corrosion and to make sure that the anchor bolts are not in contact with the rebar cage. Otherwise, the only design requirement for edge distance is that there is enough cover to prevent side-face blowout or shear failure.

(c) For constructability reasons the ASCE Anchor Bolt Report recommends a minimum edge distance of $4d_o$ for ASTM A307 or ASTM A36 or their equivalent and $6d_o$ for high strength bolts.

6.3.2 Embedment depth

There is no minimum embedment depth specified in ACI 318-08 as long as there is enough effective embedment depth to resist uplift forces. If ductility is required, greater embedment may be necessary. Anchor bolts should not be so long that they extend through the pedestal and into the footing. The ASCE Anchor Bolt Report recommends a minimum embedment depth of 12 diameters.

$$h_{ef} = 12\ d_o$$

6.3.3 Spacing between anchors

There is no minimum spacing specified in ACI 318-08 as long as the minimum spacing between reinforcing is maintained. Saudi Aramco requires $\mathbf{s} \geq 6d_o$.

6.3.4 Where **anchor sleeves** are used, the above minimum dimensions should be modified as follows:

(a) **Edge distance** should be increased by an amount equal to one half the sleeve diameter minus one half the anchor diameter, $0.5(d_s - d_o)$.

(b) **Embedment length** for anchor bolt diameters to or greater than 1 inch (25 mm) should not be less than the larger of twelve anchor diameters ($12d_o$) or the partial sleeve length plus six anchor diameters (sleeve length + $6d_o$). For anchors less than 1 inch (25 mm) in diameter, the embedment length should not be less than the partial sleeve length plus 6 inches (150 mm).

(c) **Spacing** between anchors should be increased by an amount equal to the difference between the sleeve diameter and the anchor diameter. $\mathbf{s} \geq 6d_o + (d_s - d_o)$ for A307/A36 anchors or their equivalent.

6.3.5 When a plate is used at the bottom of the anchor, similar to that shown in Figure G, the edge distance should be increased by ½ of the plate width or diameter minus ½ W_h and the spacing should be increased by the plate width or diameter minus W_h.

7 Strength Design

Strength Design, which utilizes factored loads, shall be in accordance with ACI 318-08 Appendix D. In this design guide, strength design will apply to headed bolts and headed stud anchors, solidly cast in concrete. Per ACI 318-08, D4.2.2, the exclusion for bolts over 2 inches (50 mm) in diameter or embedded over 25 inches (608 mm) may be ignored, however only equation D-7 (not equation D-8) shall be used for checking the breakout strength in cracked concrete.

ACI 318-08 D.6.2.7 states that anchors located in a region of a concrete member where analysis indicates no cracking at service loads will occur, then the factor $\psi_{c,v} = 1.4$. Stated differently, if a pedestal is in compression throughout its section at service load (no load factors) then the factor $\psi_{c,v} = 1.4$.

7.1 Loading

Anchors shall be designed for the factored load combinations in accordance with the ACI 318-08 section 9.2 or Appendix C. Care shall be taken to ensure that the proper strength reduction factor, ϕ, is used. That is if the load combinations in section 9.2 are used, use the ϕ's from Section 9.2. If the load combinations from Appendix C are used, use the ϕ's from Appendix C.

7.2 Anchor Design Considerations

Anchors need not be designed for shear if it can be shown that the factored shear loads are transmitted through friction or that the load is taken through a shear lug. Refer to Sections 10 and 11.

If the base plate is designed with oversized bolt holes and there is shear load in excess of the amount that can be transmitted through friction, it is recommended that

(a) either shear lugs be used or

(b) a mechanism to transfer load from the base plate to the bolt without slippage is incorporated (such as welding washers in place)

If no tensile force is effectively applied to the anchors, the anchors need not be designed for tension. Where the tensile force is adequately transferred to properly designed rebar, there is no requirement to check for concrete breakout strength of the anchor or anchors in tension (N_{cb} or N_{cbg}). Refer to Section 9.3.

7.3 Shear Strength of Anchors in a Rectangular Pattern

Per ACI 318-08, the concrete design shear strength of a group of anchors shall be taken as the greater of:

7.3.1 The design strength of the row of anchors closest to the edge perpendicular to the direction of force on the anchors.

7.3.2 If the anchors are welded to the attachment so as to distribute the force to all fasteners, the strength may be based on the strength of the row of anchors furthest from the edge.

7.3.3 Although not specifically accepted in ACI 318-08, the furthest row may also be used if there are closed shear ties or other mechanisms to transfer the load to the row of anchors furthest from the edge. Refer to Figure D-2.

7.4 Shear Strength of Anchors in a Circular Pattern

7.4.1 For anchors on a "circular" pattern, the design shear strength of the anchor group may be determined by multiplying the strength of the weakest anchor, times the total number of anchors in the circle. Refer to Figure B-1.

7.4.2 Alternatively, the design shear strength of a group of anchors on a "circular" pattern, where closed shear ties or other mechanisms transfer the load from the weak to the strong anchors, can be determined by calculating the shear capacity of the strong anchors. Refer to Figure B-2.

7.5 Anchor Bolt Design Spreadsheet (Available to PIP Members Only)

A spreadsheet has been developed utilizing ACI 318-08 Appendix D and this design guide. This spreadsheet will give shear and tensile capacities of an anchor or anchor group and the concrete around it. The spreadsheet will also let the user know if the anchor configuration is ductile (refer to Section 6). The user needs to use this spreadsheet in combination with ACI 318-08 Appendix D and this design guide. This spreadsheet merely saves the user time in laborious calculations but is no substitute for the engineer's knowledge and expertise. This spreadsheet is available to PIP member Companies only. This can be accessed via www.PIP.org / PIP Member Area / resources.

8 Ductile Design

8.1 Ductile Design Philosophy

A ductile anchorage design can be defined as one where the yielding of the anchor (or the reinforcement or the attachment that the anchor attaches to) controls the failure of the anchorage system. This will result in large deflections, redistribution of loads, and absorption of energy prior to any sudden loss of capacity of the system resulting from a brittle failure of the concrete. (ASCE Anchor Bolt Report).

Anchors embedded in concrete and pulled to failure, fail either by pullout of the concrete cone or by tensile failure of the anchor itself. The former is a brittle failure and the latter is a ductile failure. A brittle failure is sudden and without warning possibly causing catastrophic results. In contrast, a ductile failure will cause the steel to yield, elongate gradually, and absorb a significant amount of energy, often preventing structures from collapsing. Consequently, when the design of a structure is based upon ductility or energy absorption, one of the following mechanisms for ductility shall be used.

8.1.1 The anchors shall be designed to be governed by tensile or shear strength of the steel and the steel shall be a ductile material (see Section 5.1.1).

8.1.2 In lieu of 8.1.1, the attachment that the fastener is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level no greater than 75% of the minimum fastener design strength.

This ductile design philosophy is consistent with that of ACI 318-08.

8.2 Critical Areas Requiring Ductile Design

Anchors designed to resist critical loads, where magnitudes can not be precisely quantified, (e.g., design is based upon energy absorption), shall be designed using the requirements for ductile design. Examples are anchors in intermediate or high seismic areas and anchors used for blast load resistance.

8.3 Requirements for Ductile Design

If the mechanism described in 8.1.1 is used, the ductile design is achieved when the anchoring capacity of the concrete is greater than that of the embedded anchor in tension, in shear or in a combination of both. This is a strength requirement and is independent of the magnitudes of the applied loads. If it can be shown that failure will occur due to tensile loads prior to failure due to shear loads, then the anchor need only be ductile for tensile loads. (The reverse would also be true but would not normally be applicable to design.)

The first step is to select the anchor size considering only the steel failure modes. That is by using 0.75ϕNsa and 0.75ϕVsa. The steel material selected should be ductile steel as listed in Section 5.1.1. The loads and size can then be input into the PIP spreadsheet to check the second and third steps (next two paragraphs).

The second step is to ensure that the concrete pullout capacities (concrete breakout strength in tension, pullout strength of fastener in tension, and concrete side-face blowout strength) are greater than the tensile steel capacity of the anchor.

$$\phi N_{cb} \text{ or } \phi N_{cbg}, \phi N_{pn}, \text{ and } \phi N_{sb} \text{ or } \phi N_{sbg} > \phi N_{sa}$$

The third step is to ensure that the concrete shear capacities (concrete breakout strength in shear and concrete pryout strength in shear) are greater than the steel shear capacity of the anchor.

$$\phi V_{cb} \text{ or } \phi V_{cbg} \text{ and } \phi V_{cp} > \phi V_{sa}$$

In lieu of the above requirements, the attachment that the fastener is connecting to the structure may be designed so that the attachment will undergo ductile yielding at a load level no greater than 75% of the minimum fastener design strength.

8.4 Means to Achieve Ductile Design

If conditions as specified in Section 8.3 can not be met, the concrete capacity can be increased to achieve a ductility design using the following:

8.4.1 Increase Concrete Tensile Capacity

Concrete pullout capacity can be increased by:

a) increasing concrete strength (not permitted under Saudi Aramco Standards)
b) increasing embedment depth
c) increasing edge distance (for near edge cases)
d) increasing anchor spacing (for closely spaced anchor group)
e) extending anchor head beyond reinforcing bars

For situations where space is limited, such as anchors embedded in pedestals, the above methods may not be practical. For these cases, reinforcing bars can be placed close to the anchor to transfer the load. Refer to Section 9.3.

8.4.2 Increase Concrete Shear Capacity

Concrete shear capacity can be increased by:

a) increasing concrete strength (not permitted under Saudi Aramco Standards)
b) increasing edge distance (for near edge cases)
c) increasing anchor spacing (for closely spaced anchor group)

If the above methods are impractical due to space limitations, reinforcing hairpins looped around the anchors can be designed to carry the entire shear, neglecting any contribution from concrete. Refer to Section 9.4.

Another alternative is the use of a shear lug. Refer to Section 11. If this alternative is chosen, one of the following must be adhered to:

a) The shear lug needs to be designed to undergo ductile yielding prior to failure of the concrete.

b) The attachment that the shear lug connects to must undergo ductile yielding at a load level no greater that 75% of the minimum shear lug design strength.

9 Reinforcing Design

9.1 General

When anchor embedment or edge distances are not sufficient to prevent concrete failure due to factored loads, or for a "ductile type" connection, if ϕN_{cb} **or** $\phi N_{cbg} < \phi N_{sa}$ or ϕV_{cb} **or** $\phi V_{cbg} < \phi V_{sa}$, then reinforcing steel may be used to prevent concrete failure.

The reinforcing needed to develop the required anchor strength shall be designed in accordance with ACI 318-08.

9.2 Failure Surface

Reinforcement shall be fully developed for the required load on both sides of the possible failure surfaces resulting from tensile or shear forces. Development lengths and reinforcement covers shall be in accordance with ACI 318-08.

9.2.1 The assumed tensile concrete failure surface shall be one of the following:

a) For a single bolt the failure surface is that of a pyramid with the depth equal to the embedded depth of the anchor (h_{ef}) and the base being a square with each side equal to 3 times the embedded depth (3 h_{ef}). (Refer to Figure RD.5.2.1 (a) of ACI 318-08, Appendix D).

b) For a group of bolts where the bolts are closer together that 3 h_{ef} the failure surface is that of a truncated pyramid. This pyramid is formed by a line radiating at a 1.5 to 1 slope from the bearing edge of the anchor group, edge of nuts, toward the surface from which the anchors protrude. (Refer to Fig. RD.5.2.1 (b) of ACI 318-08, Appendix D).

9.2.2 The assumed shear concrete failure surface is defined as a half pyramid radiating at a 1.5 to 1 slope in all directions originating at the top of the concrete where the anchor protrudes and ending at the free surface in the direction of the shear. (Refer to Fig. RD.6.2.1 (a) of ACI 318-08). For multiple anchors closer together than 3 times the edge distance, c_{a1}, the failure surface is from the outermost anchors. (Refer to Fig. RD.6.2.1 (b) of ACI 318-08).

9.3 Reinforcing Design to Transfer Tensile Forces

(Refer to Figs. C-1 & C-2 and Tables 2 & 3)

9.3.1 The required area of reinforcing bars, $\mathbf{A_{rb}}$, per anchor is:

$$\mathbf{A}_{rb} = \frac{N_{sa}}{f_y}$$

Obtain $\mathbf{h_{ef}}$, the embedment depth of the anchor as follows:
(Refer to Fig. C-1)

$$\mathbf{h_{ef}} = l_d + C + (X + d_b/2)/1.5$$

a) Calculate $\mathbf{l_d}$, the development length of the reinforcing bars resisting the load using ACI 318-08.

b) Add **C**, the concrete cover over the top of reinforcing bars to the finished surface.

c) Add **X**, the clear distance from the anchor nut to the reinforcing bars (maximum $X = h_{ef}/3$).

d) Add $\mathbf{d_{rb}/2}$, half the diameter of the reinforcing bars.

It is noted that the reinforcing bars were probably sized during pedestal design. If more reinforcement is required by the pedestal design than required by the anchor load transfer, the reinforcing bar development length may be reduced by the ratio of the reinforcing bar area required to the reinforcing bar area provided.

$$\mathbf{l_d} \text{ required} = \mathbf{l_d} \times [(\mathbf{A_{rb}}) \text{ required} / (\mathbf{A_{rb}}) \text{ provided}]$$

This reduction is in accordance with ACI 318-08, Section 12.2.5 and can not be applied in high seismic areas.

9.3.2 Direct tensile loads can be transferred effectively by the use of "hairpin" reinforcement or vertical dowels according to the guidelines below.

a) "Hairpin" legs and vertical dowels shall be located within $\mathbf{h_{ef}}/3$ from the edge of the anchor head.

b) "Hairpin" legs and dowels shall extend a minimum of $\mathbf{l_d}$, beyond the potential failure plane, or additional rebar area shall be provided to reduce the required embedment length (see Section 9.3.1).

9.4 Reinforcing Design to Transfer Shear Forces

9.4.1 Several shear reinforcement configurations or assemblies can be considered effective in order to prevent failure of the concrete. Depending of the particular situation, one of the following types of shear reinforcement could be used:

a) "Hairpins" wrapped around the anchors. (Fig. D-1)

b) "Closed Ties" transferring load to the stronger anchors. (Fig. D-2)

c) "Anchored" reinforcing intercepting the failure plane. (Fig. D-3)

d) "Shear angles" welded to anchors. (Fig. D-4)

e) Strut-and-tie model. (Refer to Appendix A of ACI 318-08 and Figure D-5 of this Practice.)

9.4.2 Shear reinforcing shall extend a minimum of $\mathbf{l_d}$, beyond the potential failure plane. Where excess rebar is provided, $\mathbf{l_d}$ may be reduced by the ratio of the reinforcing bar area required divided by the reinforcing bar area provided. See Section 9.3.1.

9.4.3 Where shear reinforcing is designed, it should be designed to carry the entire shear load, neglecting any contribution from the concrete.

9.4.4 For pedestals, a minimum of two 12 mm diameter (No. 4) ties within 150 mm (6”) are required close to the top of each pedestal. Refer to Figure E. Use three ties close to the top of each pedestal, if shear lugs are used or if the pedestals are located in a high seismic area.

9.5 Reinforcing to Prevent Bursting Failure

When sufficient edge distance cannot be provided to prevent a bursting failure one of the following reinforcing methods can be used:

a) "Hairpins" above and below the bearing head or plate that intersect the failure plane.

b) "Spiral Ties" around the anchor bolt and extending below the bearing surface.

10 Frictional Resistance

10.1 General

Where allowed by code, for non-ductile design, anchors need not be designed for shear if it can be shown that the factored shear loads are transmitted through friction developed between the bottom of the base plate and the top of the concrete foundation. If there is moment on a baseplate, the moment may produce a downward load that will develop friction even when the column has an upward load on it. This can also be considered in calculating frictional resistance. Care shall be taken to assure that the downward load producing frictional resistance occurs simultaneously with the shear load.

The frictional resistance can also be used in combination with shear lugs to resist the factored shear load. The frictional resistance should not be used in combination with the shear resistance of anchors unless there is a mechanism to keep the base plate from slipping before the anchors can resist the load (such as welding the anchor nut to the base plate.)

10.2 Calculating Resisting Friction Force

Per the LRFD Specification in the AISC LRFD Manual, the resisting friction force, $\mathbf{V_f}$, shall be computed as follows:

$$\mathbf{V_f} = \mu P$$

P = Normal Compression Force

μ = Coefficient of Friction

The materials used and embedment depth of the base plate determine the value of the coefficient of friction. (Refer to Figure F for a pictorial representation.)

a) $\mu = 0.90$ for concrete placed against as-rolled steel with the contact plane a full plate thickness below the concrete surface.

b) $\mu = 0.70$ for concrete or grout placed against as-rolled steel with the contact plate coincidental with the concrete surface.

c) $\mu = 0.55$ for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface.

11 Shear Lug Design

Normally, friction 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 need to transfer the excess shear force to the foundation by another means. If the total factored shear loads are transmitted through shear lugs or shear friction; the anchor bolts need not be designed for shear at all.

Using a shear lug, a plate or pipe stub section, welded perpendicularly to the bottom of the base plate allows for complete transfer of the force through this section, acting as a cantilever. Bearing is only applied on the portion of the plate adjacent to the concrete. Therefore, neglecting the part of the plate embedded in the top layer of grout, the bearing is uniformly distributed through the remaining height. The shear strength of the lug plate is not considered because it will not govern the design.

The shear lug should be designed for the applied shear portion not resisted by friction between the base plate and concrete foundation. Grout must completely surround the lug plate or pipe section and entirely fill the slot created in the concrete. When using a pipe section a hole approximately ½ inch (12 mm) in diameter should be drilled near the top of the section to allow the grout to fill the entire pocket. Another alternative is to put an inspection hole through the base plate over the pipe section.

11.1 Calculating Shear Load Applied to Shear Lug

The applied shear load, $\mathbf{V_{app}}$, used to design the shear lug should be computed as follows:

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

11.2 Design Procedure for Plate Shear Lugs

Design of a shear lug plate follows:

a) Calculate the required bearing area for the shear lug.

$$\mathbf{A_{req}} = V_{app} / (0.85 * \phi * f'c) \qquad \phi = 0.65$$

b) Determine the shear lug dimensions, assuming that bearing occurs only on the portion of the lug below the grout level. Assume a value of W, the lug width, based on the known base plate size to find H, the total height of the lug, including the grout thickness, G.

$$\mathbf{H} = (A_{req} / W) + G$$

c) Calculate the factored cantilever end moment acting on a unit length of the shear lug.

$$\mathbf{M_u} = (V_{app}/W) * (G + (H-G)/2)$$

d) Knowing the moment, the lug thickness can be found. The shear lug should not be thicker than the base plate.

t = $[(4 * M_u)/(.9*f_{ya})]^{.5}$

e) Design weld between plate section and base plate.

(f) Calculate the breakout strength of the shear lug in shear. The method shown below is from ACI 349-01, Appendix B, Section B.11.

V$_{cb}$ = $A_{Vc}*4*\phi*[f'_c]^{.5}$

Where,

A_{Vc} = the projected area of the failure half-truncated pyramid defined by projecting a 45 degree plane from the bearing edges of the shear lug to the free edge. The bearing area of the shear lug shall be excluded from the projected area.

ϕ = Concrete strength reduction factor = 0.85

11.3 Design Procedure for Pipe Shear Lugs

Design of a shear lug pipe section follows:

a) Calculate the required bearing area for the shear lug.

A$_{req}$ = V_{app} /(0.85ϕf′c) ϕ = 0.60

b) Determine the shear lug dimensions, assuming that bearing occurs only on the portion of the lug below the grout level. Assume the D, diameter of the pipe section, based on the known base plate size to find H, the total height of the pipe, including the grout thickness, G.

H = $(A_{req}/D) + G$

c) Calculate the factored cantilever end moment acting on the shear lug pipe.

M$_u$ = V_{app} * (G + (H-G)/2)

d) Check the applied shear force and the bending moment for pipe section failure. (LRFD 3rd Edition p 16.1-96, 16.1-102)

Shear Check –

$\boldsymbol{\phi_v V_n \geq V_{app}}$ ϕ_v = 0.9

$V_n = 0.6\ f_{ya}\ \pi(D^2 - (D\text{-}t)^2)/4$

Moment Check –

$\boldsymbol{\phi_b M_n \geq M_u}$ $\phi_b = 0.9$

M_n = is the lesser of:

$= S[\{600/(D/t)\} + f_{ya}]$ (local buckling moment)

$= Z * f_{ya}$ (plastic moment)

e) Design weld between pipe stub section and base plate.

f) Check the break out shear as shown in 11.2 f).

12 Pretensioning

Pretensioning of anchor bolts can be used to effectively "clap" the base plate to the foundation. This primary application for pretensioning is for large vibrating equipment, tall process columns and cantilever stacks. Pretensioning anchor bolts will virtually eliminate fatigue stresses in the anchor bolts and is also effective in eliminating shear forces on the bolts. Proper anchor bolt details, careful installation procedures and field quality control are essential to ensure that the anchor bolts are successfully tensioned.

12.1 Advantages

12.1.1 Can prevent stress reversals on anchors susceptible to fatigue weakening

12.1.2 May increase dampening for pulsating or vibrating equipment

12.1.3 Will decrease to some extent, the drift for process towers under wind or seismic load

12.1.4 Will increase the frictional shear resistance for process towers and other equipment

12.2 Disadvantages

12.2.1 Can be a costly process to accurately install.

12.2.2 No recognized code authority that gives guidance on the design and installation of pretensioned anchors. There is little research in this area.

12.2.3 Doubt as to the long-term load on the anchor due to creep of concrete under the pretension load

12.2.4 Usually when pretensioning, there is a sleeve around the anchor not filled with grout, thus there is no bearing resistance to shear on the anchor. Bearing can be accomplished by wrapping the anchor bolt with a bond-breaking tape and eliminating the sleeve or through the friction force developed by the pretensioning clamping force on the base plate.

12.2.5 Little assurance that the anchor is properly installed and pretensioned in the field

12.3 When to Pretension Bolts

Pretensioning should always be done when recommended by the equipment manufacturer. The manufacturer's instructions should be followed carefully. When not otherwise specified, anchors for turbines and reciprocating compressors should be torqued to the values shown in Table 4. When pretensioning is required, high strength anchors should be used. Pretensioned anchor bolts should also be considered for tall process columns or cantilever stacks, which can develop significant bolt tension under load wind or seismic loads.

12.4 Concrete Failure

In certain situations, the use of high strength anchors in concrete with high pretension forces may exceed the ultimate capacity of the concrete and prematurely breaking out the concrete in a side-face blowout type failure (see Figure RD.4.1 a(iv) of ACI 318-08, Appendix D). Whether this situation can occur depends on the depth of the anchor and other factors such as edge conditions and arrangement of the base plate. To ensure premature concrete failure does not occur, pretensioned anchors shall be designed so that the break out strength of the anchor in tension is greater than the maximum pretension 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 break out strength only needs to be designed for the amount that the external uplift exceeds the gravity plus pretensioning force loads.

12.5 Stretch Length

Pretensioning should only be implemented when the stretching (spring) length of the anchor extends down to the anchor head or anchor plate for the anchor bolt. On a typical anchor embedment, where there is no provision for a stretching length, if a pretensioning load is applied to the anchor, the anchor starts to shed its load to the concrete through its bond on the anchor. At that time, there exists a high bond stress at the first few inches of embedment. This bond will relieve itself over time and thereby reduce the pretension load on the anchor. Therefore, it is important to prevent bond between the anchor and concrete for pretensioned anchors. Refer to Figure G for a suggested detail to do this.

12.6 Pretensioning Methods

Methods used to apply pre-load area as follows:

12.6.1 **Hydraulic Jacking:** Hydraulic jacking is the most accurate method and is recommended if the pretension 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.

12.6.2 **Turn-of-Nut:** This method is preferable to the torque wrench method. The pretension load due to stretching of the anchor can be closely determined, but it is difficult to account for the compression of the concrete under the pretension load. This can usually be overcome by providing a pretension load that exceeds calculate anchor bolt tension by about 25%. Anchor bolt chairs should be checked for any increase in the pretension force above the maximum calculated bolt tension.

12.6.3 **Torque Wrench:** Torque wrench pretensioning provides only a rough measure of actual pretension load, but can be the method of choice if the amount of pretension load is not critical. This method relies on the approximate relationship between nut torque and bolt tension. This relationship is not very reliable and depends on the condition of the treads, coating system and fit between nut and bolt. Lubrication of the bolt and nut treads is essential to prevent galling and to better control the torque vs. tension relationship. Typical torque values are shown in Table 4.

12.6.4 **Load Indicator Washers:** This method is good if the amount of pretension desired is as much as the required load in slip-critical structural steel connections. These loads are typically very high and not normally required for anchors. Another limitation of load-indicating washers is that they may not be available for large bolt diameters.

12.7 Relaxation and Concrete Creep

12.7.1 Per ACI 355.1R, Section 3.2.2, "If headed anchors are preloaded, the initial force induced in the anchor is reduced with time due to creep of the highly stressed concrete under the anchor head. The final value of the tension force in the anchor depends primarily on the value of bearing stresses under the head, the concrete deformation and the anchorage depth. In typical cases the value of that final force will approach 40 to 80% of the initial preload (40% for short anchors, 80% for long anchors)." Retensioning the anchors about one week after the initial tensioning can reduce this. Per ACI 355.1R, the reduction of the initial preload can be reduced by about 30% by retensioning.

12.7.2 Anchor plates should be used to reduce the concrete bearing stresses, minimize the concrete creep and loss of anchor bolt pretension force.

12.8 Tightening Sequence

Pretensioned anchors should be tightened in two stages:

a) First stage should apply 50% of the full pretension load to all anchors.
b) Second stage should apply full pretension load to all anchors.

Anchors should be tightened in a criss-cross pattern. (Refer to Figure H.)

13 Installation

13.1 Anchor bolt are required to be places accurately by using a template and are must be set to the tolerances specified in the Section 7.5 of the AISC Code of Standard Practice for Buildings and Bridges." (Refer to SAES-Q-005) The oversized holes used in structural steel base plates are sized to accommodate anchor bolts that are set within the tolerances specified above.

13.2 Recommended Tightening if Anchor Pretensioning is not Required

Anchors should be brought to a snug tight condition. This is defined as the tightness that exists after a few impacts from an impact wrench of the full effort of a man using a spud wrench. At this point all surfaces should be in full contact (refer to ANSI A10.13, Section 9.6).

13.3 Anchor Installation for Heavy Machinery

Anchor bolts for heavy machinery shall comply with the machinery manufacturer's recommendations. Other requirements are contained in Standard SAES-Q-007 and PIP REIE 686.

Revision Summary

31 August 2002	New Saudi Aramco Best Practice.
24 May 2009	General revision to update Best Practice.

Table 1 – Minimum Anchor Dimensions
(Refer to Figure A)

	HEAVY HEX HEAD/NUT AISC-THREADED FASTENER	ANCHOR TYPE 2	ASCE ANCHOR BOLT REPORT MINIMUM DIMENSIONS (5.6)*				SLEEVES		
			h_{ef}	EDGE DISTANCE (C_{ai}[2])		SPACING			
ANCHOR DIA. d_o	WIDTH W_h	P1	$12d_o$	A307/A36 F1554 Grade 36	HIGH STRENGTH OR TORQUED BOLTS	$6d_o$	SHELL SIZE		h_e'
		$d_o + 1/2"$		$4d_o \geq 4.5"$	$6d_o \geq 4.5"$		Diameter d_s	Height h_s	$6d_o \geq 6"$
(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
1/2	1.00	1.00	6.0	4.5	4.5	3.0	2	5	6
5/8	1.25	1.13	7.5	4.5	4.5	3.25	2	7	6
3/4	1.44	1.25	9.0	4.5	4.5	4.5	2	7	6
7/8	1.69	1.38	10.5	4.5	5.3	5.3	2	7	6
1	1.88	1.50	12.0	4.5	6.0	6.0	3	10	6
1-1/8	2.06	1.63	13.5	4.5	6.8	6.8	3	10	7
1-1/4	2.31	1.75	15.0	5.0	7.5	7.5	3	10	8
1-3/8	2.50	1.88	16.5	5.5	8.3	8.3	4	15	8
1-1/2	2.75	2.00	18.0	6.0	9.0	9.0	4	15	9
1-3/4	3.19	2.25	21.0	7.0	10.5	10.5	4	15	11
2	3.63	2.50	24.0	8.0	12.0	12.0	4	18	12
2-1/4	4.06	2.75	27.0	9.0	13.5	13.5	4	18	14
2-1/2	4.50	3.00	30.0	10.0	15.0	15.0	6	24	15
2-3/4	4.94	3.25	33.0	11.0	16.5	16.5	6	24	17
3	5.31	3.50	36.0	12.0	18.0	18.0	6	24	18

*** NOTE:**

<u>IF SLEEVES ARE USED:</u>

EMBEDMENT SHALL BE THE LARGER OF $12d_o$ or ($h_s + h_e'$)

INCREASE EDGE DISTANCE BY $0.5(d_s - d_o)$

INCREASE SPACING BY ($d_s - d_o$)

Table 2 – Reinforcement Tensile Capacity and Development Length

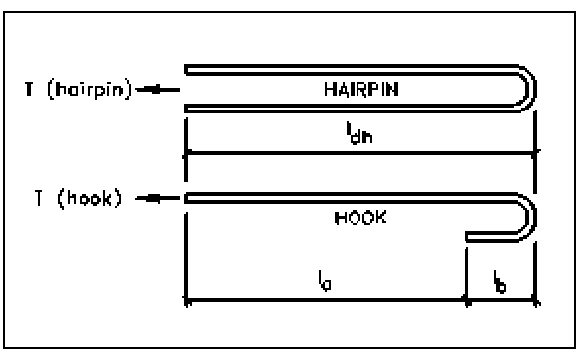
Reinforcement Yield Strength, f_y = 60 ksi
Compressive Strength of Concrete, f'_c = 4,000 psi
Minimum Reinforcement Cover (Regular Bars) ≥ d_b (ACI 318-08, 12.2.2)
Minimum Reinforcement Cover (Epoxy Coated Bars) ≥ $3d_b$
Minimum Reinforcing Spacing (Regular Bars) ≥ $2d_b$
Minimum Reinforcing Spacing (Epoxy Coated Bars) ≥ $6d_b$
Design Tensile Strength Reduction Factor, ø = 0.90 (ACI 318-08, 9.3)
Development Length & Factors as per ACI 318-08, 12.1, 12.2.2 & 12.4

			Development Lengths (l_d)			
Nominal Diameter d_b	Area A_r	Rebar Capacity $Ø(A_r)F_y$	Regular Bars Uncoated	Regular Bars Epoxy Coated	Top Bars Uncoated	Top Bars Epoxy Coated
(mm)	(cm^2)	(kN)	(mm)	(mm)	(mm)	(mm)
Ø6	0.28	10.5	305	305	305	352
Ø8	0.50	18.7	305	361	391	469
Ø10	0.79	29.2	376	451	489	587
Ø12	1.13	42.1	451	541	587	704
Ø14	1.54	57.3	526	632	684	821
Ø16	2.01	74.8	602	722	782	938
Ø18	2.54	94.6	677	812	880	1056
Ø20	3.14	116.9	940	1128	1222	1466
Ø22	3.80	141.5	1034	1241	1344	1613
Ø25	4.91	182.8	1175	1410	1528	1833
Ø28	6.16	229.3	1316	1579	1711	2053
Ø32	8.04	299.3	1504	1805	1955	2346

Table 3A – Hairpin Reinforcement Design and Details (Uncoated Bars)

Reinforcement Yield Strength, f_y = 60 ksi
Compressive Strength of Concrete, f'_c = 4,000 psi
Minimum Reinforcement Side Cover = 2.5 in. (ACI 318-08, 12.5.2)
Minimum Reinforcing Spacing = 3.0 in.
Development Length Reduction Factor (ACI 12.5.3) = 0.70
Design Tensile Strength Reduction (ACI 318-08, 9.3), ϕ = 0.90
Development Length & Reduction Factors as per ACI 12.2.1 & 12.2.4

Bar Size	BASIC DEVELOPMENT LENGTH	REINFORCING BAR CAPACITY	HAIRPIN AND HOOK DIMENSIONS				VERTICAL HAIRPIN		HORIZONTAL HAIRPIN	
			ACI 12.5.1 & Fig. R12.5.1			ACI Table 7.2	REGULAR BARS l_d (ACI 12.2.2)	CAPACITY ϕ*Ar*(fy)(1+la/ld)	TOP BARS l_d (ACI 12.2.2)	CAPACITY ϕ*Ar*(fy)(1+la/ld)
		ϕ*Ar*(f_y)	l_{dh} .7*l_{hb}≥150	l_a Fig. R12.5	l_b Fig. R12.5	Inside Hook				
	(mm)	(kN)	(mm)	(mm)	(mm)	(mm)	(mm)	(kN)	(mm)	(kN)
Ø6	114	10.5	150	63	88	36	305	13	305	13
Ø8	152	18.7	150	55	96	48	305	22	391	21
Ø10	190	29.2	150	47	104	60	376	33	489	32
Ø12	228	42.1	159	48	112	72	451	47	587	46
Ø14	266	57.3	186	66	120	84	526	65	684	63
Ø16	304	74.8	212	84	128	96	602	85	782	83
Ø18	341	94.6	239	95	144	108	677	108	880	105
Ø20	379	116.9	266	106	160	120	940	130	1222	127
Ø22	417	141.5	292	116	176	132	1034	157	1344	154
Ø25	474	182.8	332	132	200	150	1175	203	1528	199
Ø28	531	229.3	372	120	252	224	1316	250	1711	245
Ø32	607	299.3	425	137	288	256	1504	327	1955	320



HAIRPIN CAPACITY:

(1) Standard 180 Hook Capacity = Capacity of Straight Bar

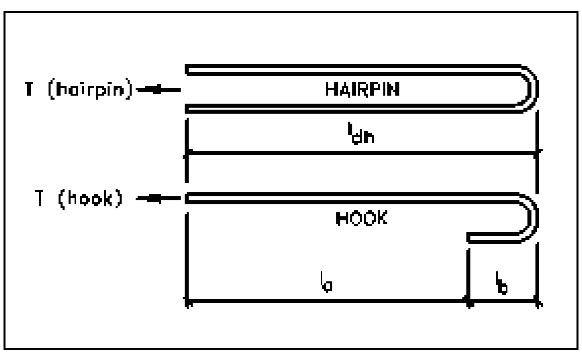
(2) Capacity of l_a portion of Hook = Bar Capacity $l_a = l_{dh} - l_b$

(3) Hairpin Capacity = Bar Capacity x $(1 + l_a/l_d)$ where l_d = Bar Development Length [$l_d > l_a$]

Table 3B – Hairpin Reinforcement Design and Details (Epoxy Coated Bars)

Reinforcement Yield Strength, f_y = 60 ksi
Compressive Strength of Concrete, f'_c = 4,000 psi
Minimum Reinforcement Side Cover = 2.5 in. (ACI 318-08, 12.5.2)
Minimum Reinforcing Spacing = 3.0 in.
Development Length Reduction Factor (ACI 12.5.3) = 0.70
Design Tensile Strength Reduction (ACI 318-08, 9.3), ϕ = 0.90
Development Length & Reduction Factors as per ACI 12.2.1 & 12.2.4

Bar Size	BASIC DEVELOPMENT LENGTH	REINFORCING BAR CAPACITY	HAIRPIN AND HOOK DIMENSIONS				VERTICAL HAIRPIN		HORIZONTAL HAIRPIN	
			ACI 12.5.1 & Fig. R12.5.1			ACI Table 7.2	REGULAR BARS	CAPACITY	TOP BARS	CAPACITY
		ϕ*Ar*(f_y)	l_{dh} .7*l_{hb}≥150	l_a Fig. R12.5	l_b Fig. R12.5	Inside Hook	ld (ACI 12.2.2)	ϕ*Ar*(fy)(1+la/ld)	ld (ACI 12.2.2)	ϕ*Ar*(fy)(1+la/ld)
	(mm)	(kN)	(mm)	(mm)	(mm)	(mm)	(mm)	(kN)	(mm)	(kN)
Ø6	137	10.5	150	63	88	36	305	13	352	12
Ø8	182	18.7	150	55	96	48	361	22	469	21
Ø10	228	29.2	159	56	104	60	451	33	587	32
Ø12	273	42.1	191	80	112	72	541	48	704	47
Ø14	319	57.3	223	104	120	84	632	67	821	65
Ø16	364	74.8	255	127	128	96	722	88	938	85
Ø18	410	94.6	287	143	144	108	812	111	1056	107
Ø20	455	116.9	319	159	160	120	1128	133	1466	130
Ø22	501	141.5	351	175	176	132	1241	161	1613	157
Ø25	569	182.8	398	198	200	150	1410	209	1833	203
Ø28	637	229.3	446	194	252	224	1579	258	2053	251
Ø32	728	299.3	510	222	288	256	1805	336	2346	328



HAIRPIN CAPACITY:

(1) Standard 180 Hook Capacity = Capacity of Straight Bar

(2) Capacity of l_a portion of Hook = Bar Capacity $l_a = l_{dh} - l_b$

(3) Hairpin Capacity = Bar Capacity x $(1 + l_a/l_d)$ where l_d = Bar Development Length [$l_d > l_a$]

Table 4 – Pretension Load and Torque Recommendations*

Nominal Bolt Diameter (Inches)	Number of Threads (per inch)	Torque (foot-pounds)	Pretension Load (pounds)
3/4	10	100	9,060
7/8	9	160	12,570
1	8	245	16,530
1-1/8	8	355	21,840
1-1/4	8	500	27,870
1-1/2	8	800	42,150
1-3/4	8	1,500	59,400
2	8	2,200	79,560
2-1/4	8	3,180	102,690
2-1/2	8	4,400	128,760
2-3/4	8	5,920	157,770
3	8	7,720	189,720

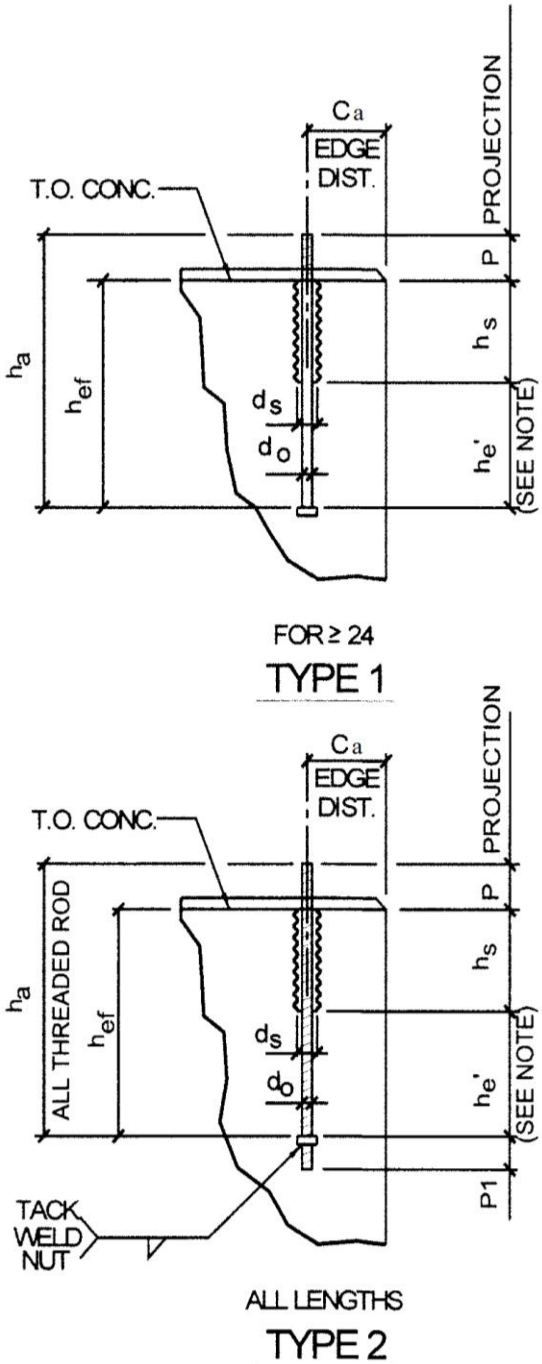
Nominal Bolt Diameter (mm)	Torque (newton-meters)	Pretension Load (kilograms)
M16	60	3,311
M24	100	7,447
M30	160	18,247
M52	245	37,136

Note 1: All torque values are based on anchor bolts with threads well-lubricated with oil.

Note 2: In all cases the elongation of the bolt will indicate the load on the bolt.

Note 3: Based upon 30 ksi internal Bolt Stress

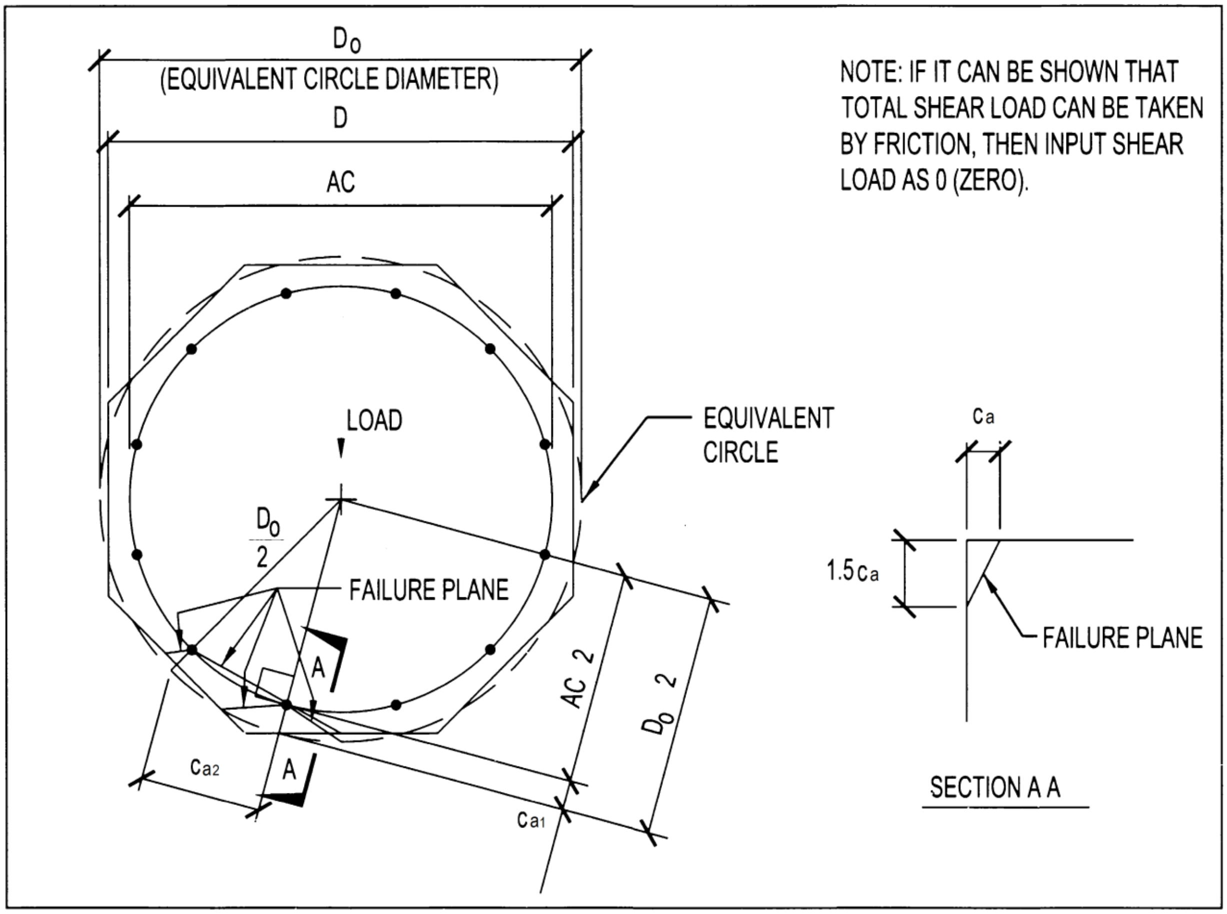
**From PIP REIE 686, Recommended Practices for Machinery Installation and Installation Design, Appendix A.*



NOTE: DISTANCE BETWEEN BOTTOM OF SLEEVE AND ANCHOR BEARING SURFACE, h_e', SHALL NOT BE LESS THAN $6d_o$ NOR 6-IN.

REFER TO TABLE 1 FOR MINIMUM DIMENSIONS

Figure A – Anchor Details



<u>Approximate solution</u>

$c_{a1} = D_o/2 - AC/2$

Calculate D_o so that equivalent circle has same area as octagon.

Note: Area of octagon = $.828D^2$

$\pi D_o^2/4 = .828D^2$

$\pi D_o^2 = \frac{.828D^2(4)}{\pi}$

$$D_o = \sqrt{\frac{.828D^2(4)}{\pi}} = 1.03D$$

For input into spreadsheet
$c_{a1} = 1.03D/2 - AC/2$
$c_{a2}, c_{a4} = [(1.03D/2)^2 - (AC/2)^2]^{1/2}$
$A_{vc} = 1.5c_1D$
$A_{vc}\,(\max) = n\ 4.5c_1^2$
n = Total number of bolts = 12

Pythagorean theorem:

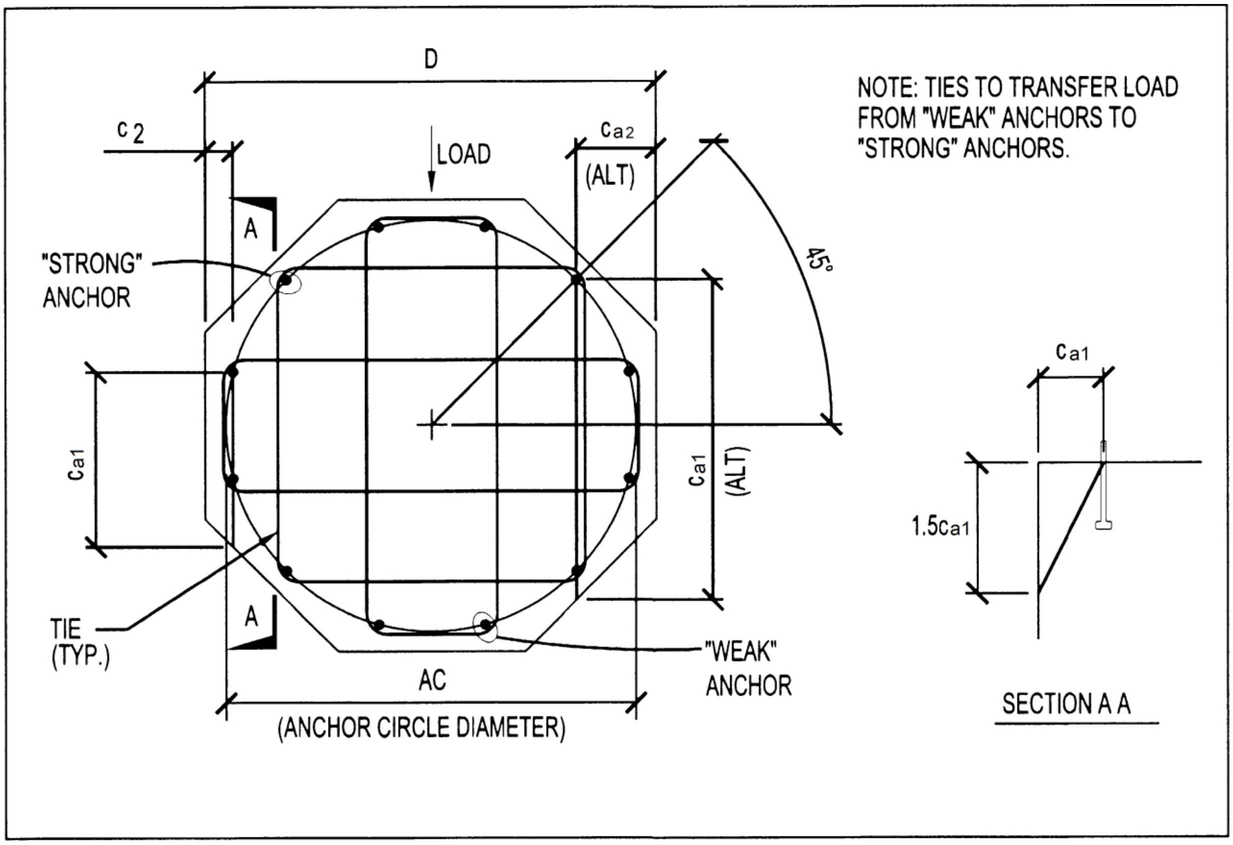
$c_{a2}^2 + (AC/2)^2 = (D_o/2)^2$

$c_{a2} = [(1.03D/2)^2 - (AC/2)^2]^{1/2}$

Failure planes overlap each other to go clear across pedestal.

$A_v = 1.5c_{a1}D$ (Max. $A_{vc} = nA_{vo} = n4.5c_{a1}^2$)

Figure B-1 – Concrete Breakout Strength of Anchors in Shear Octagon "Weak" Anchors



c_1 will vary with the number of anchors considered. Only anchors with an edge distance, c_1, greater than or equal to the c_1 for the chosen bolt shall be used for resisting shear.

For the case shown above, if the dimension marked c_1 is chosen, n = 6 bolts.
If the dimension marked c_1 (ALT) is chosen, n = 4 bolts.

For input into spreadsheet	Alternate
Ca_1 = As shown above	Ca_1 (ALT) = As shown above
$Ca_2 = (D-AC)/2$	$Ca_2\ (ALT) = (D-\cos(45^o)AC)/2$
$A_{vc} = 1.5c_1\ D$	$A_{vc} = 1.5c_1(ALT)\ D$
$A_{vc}\ (max) = n\ 4.5c_1^2$	$A_{vc}\ (max) = n\ 4.5(Ca_1(ALT))^2$
n = 6	n = 4

Figure B-2 – Concrete Breakout Strength of Anchors in Shear Octagon "Strong" Anchors ||

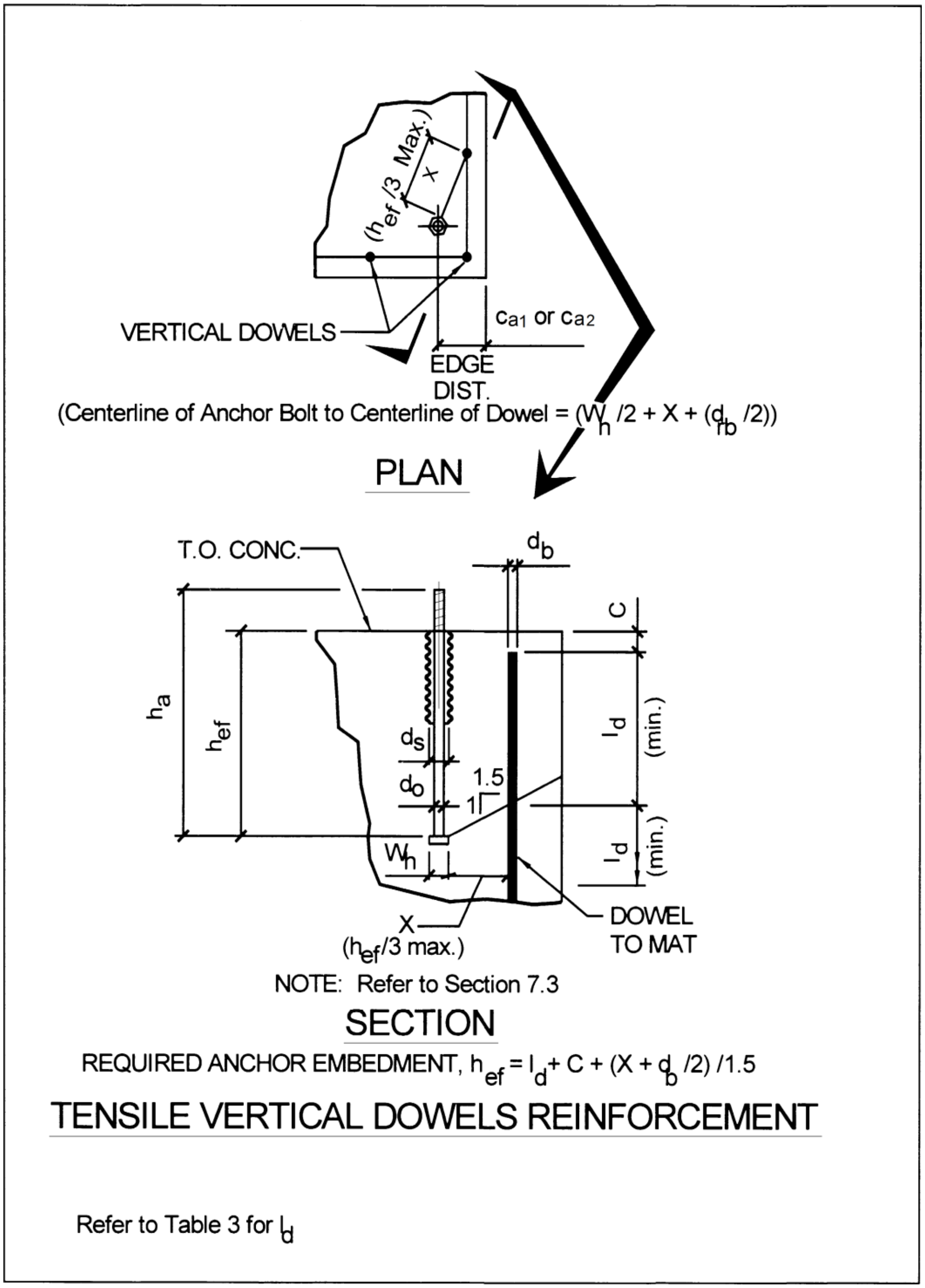


Figure C-1 – Tensile Reinforcement - Vertical Dowels

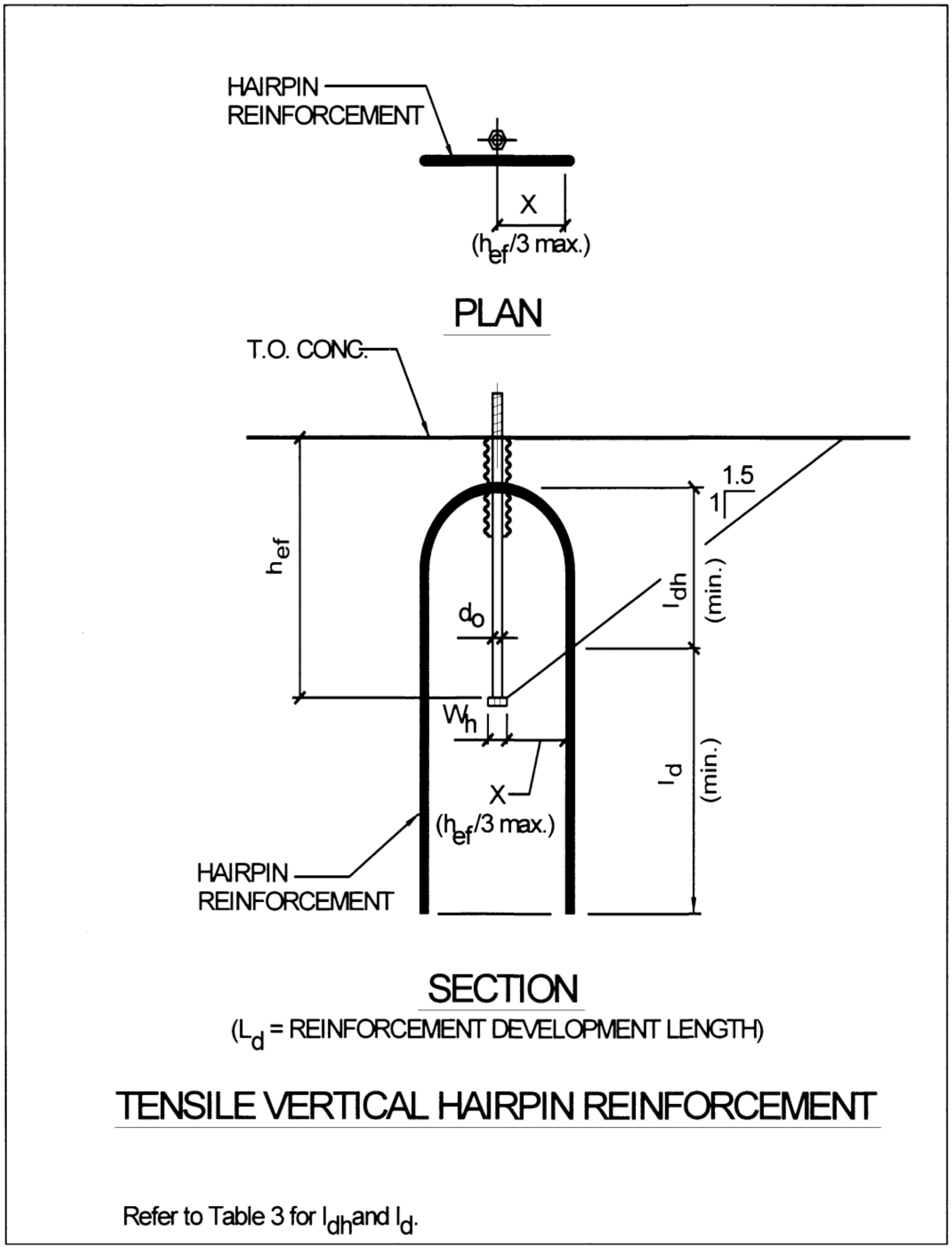


Figure C-2 – Tensile Reinforcement - Vertical Hairpin

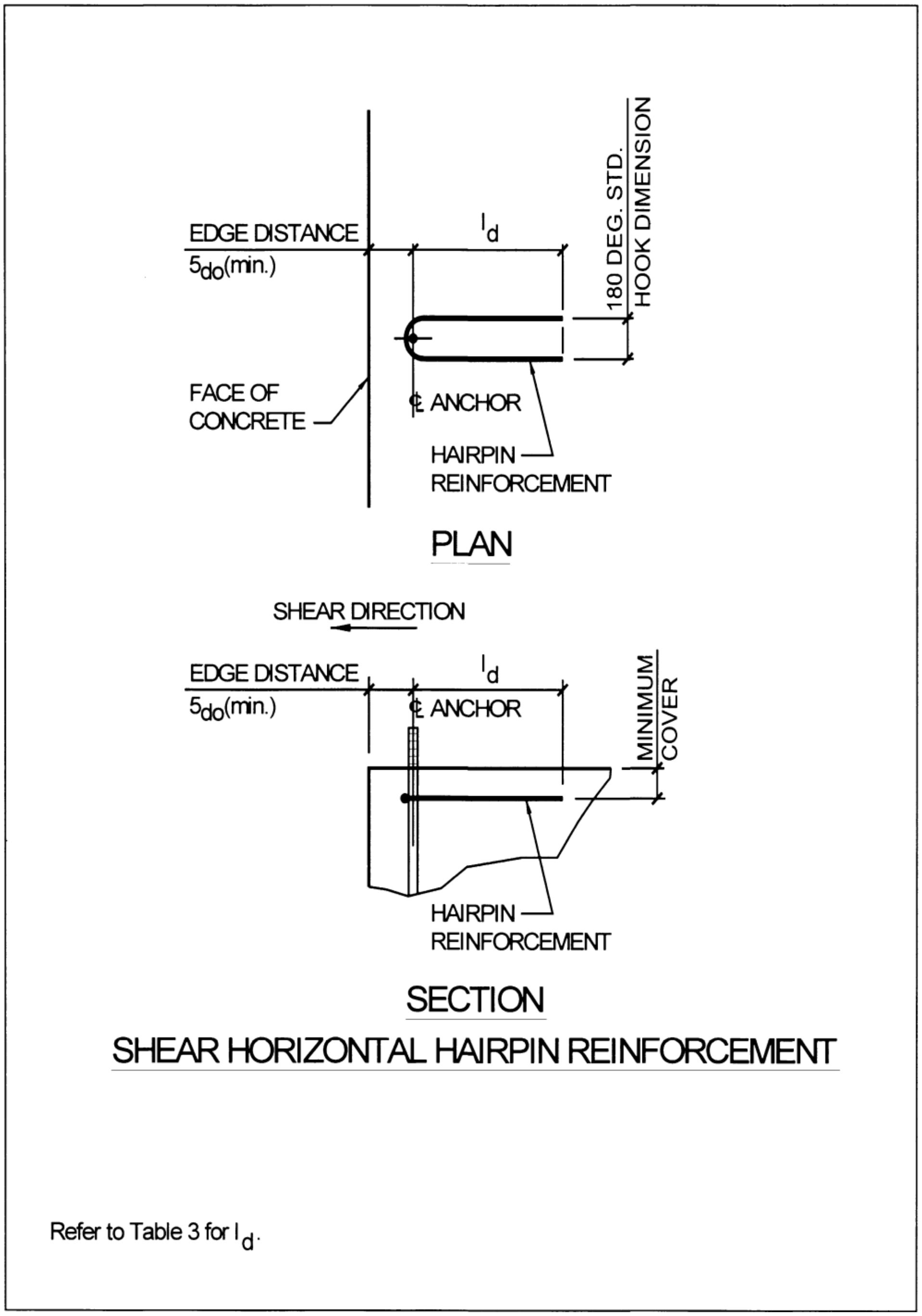


Figure D-1 – Shear Reinforcement - Horizontal Hairpin

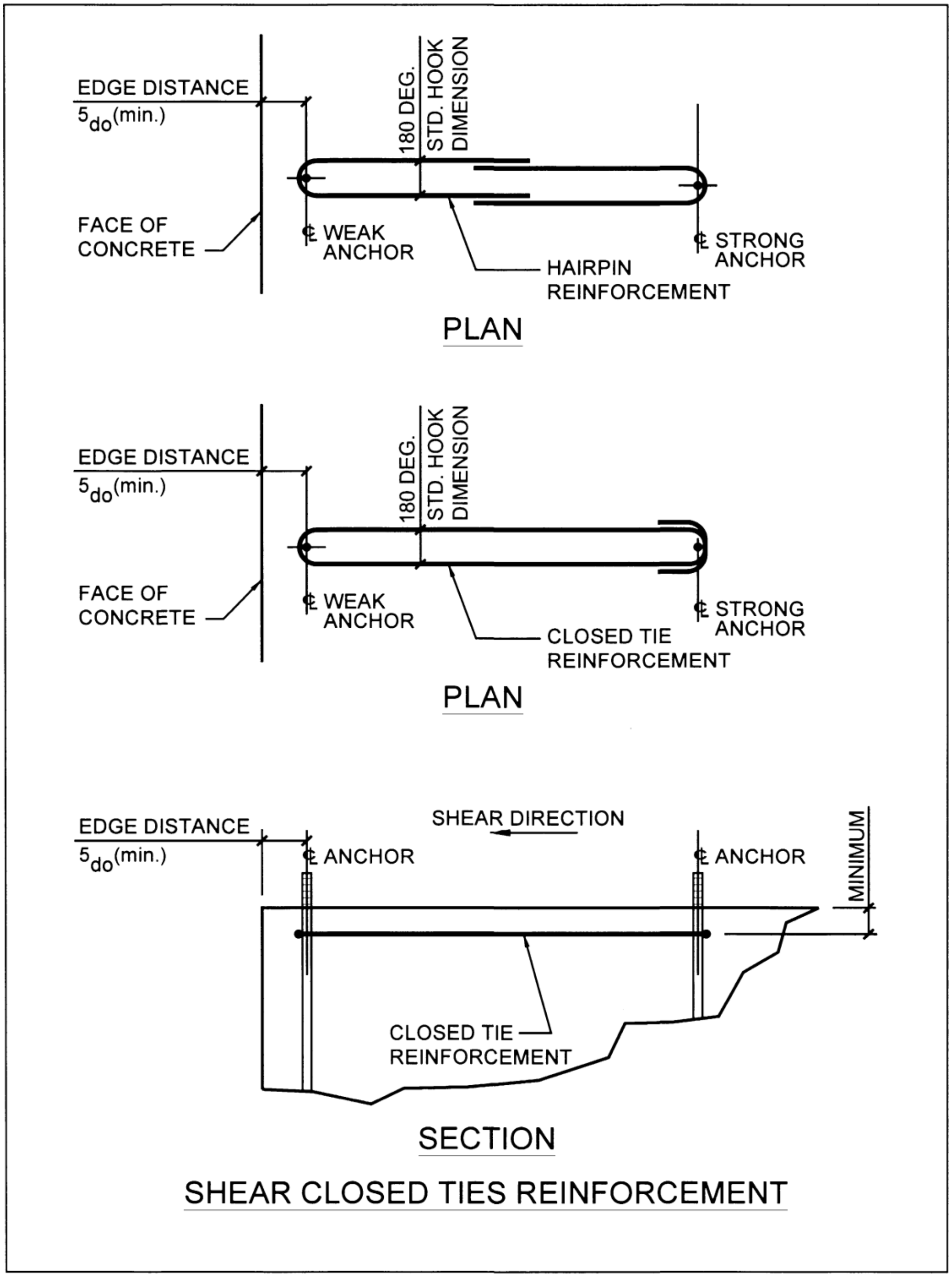


Figure D-2 – Shear Reinforcement - Closed Ties

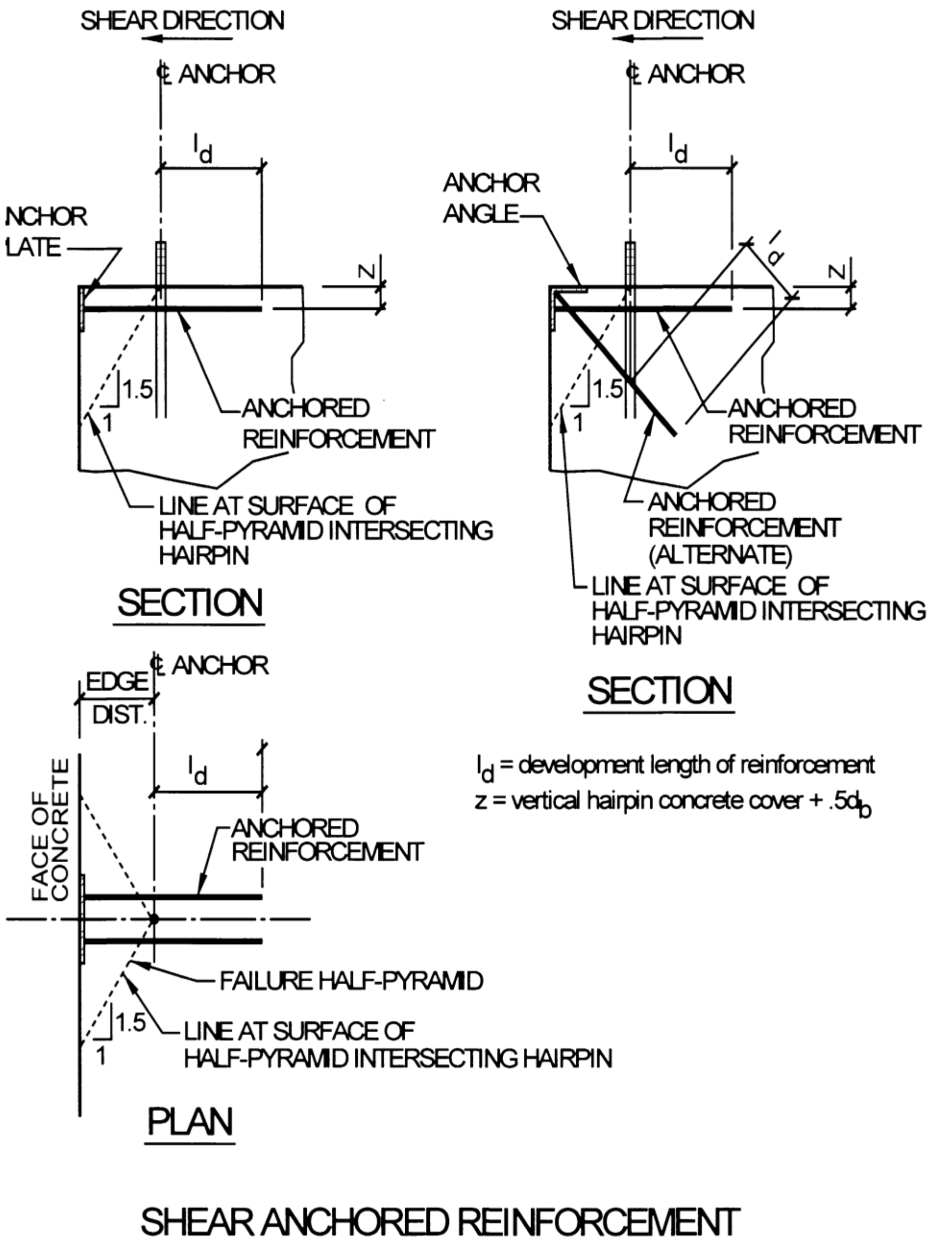
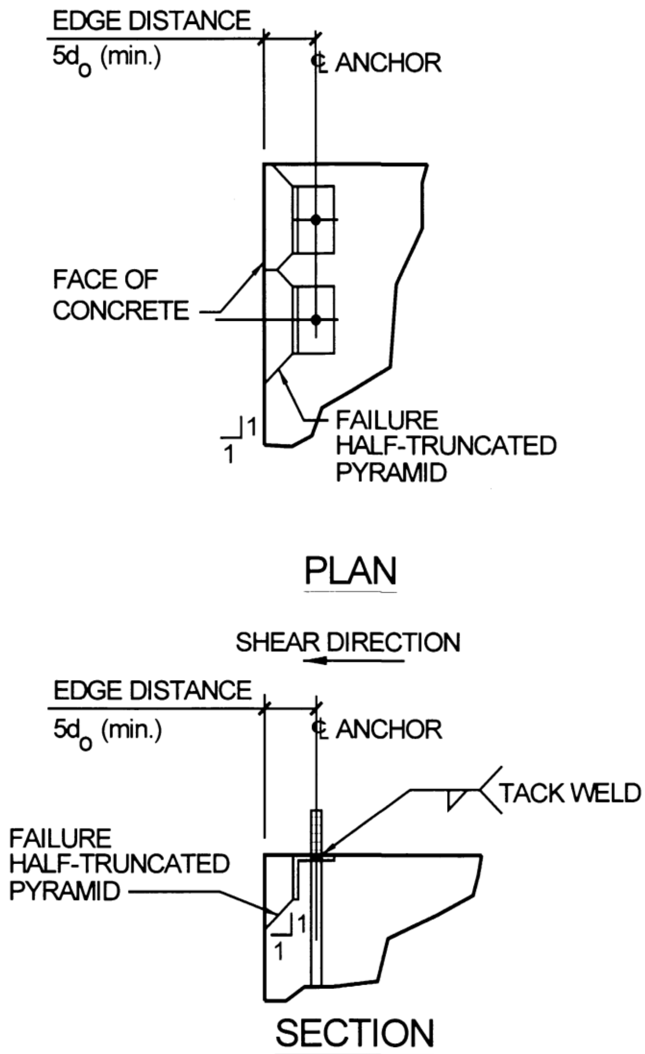


Figure D-3 – Shear Reinforcement - Anchored Reinforcement

Notes:

1. See Tables 3A & 3B for Rebar Capacities.
2. Anchor plate or Anchor angle must be designed for load from anchor.
3. Taking l_d from centerline of bolt is conservative.



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

SHEAR ANGLES REINFORCEMENT

Figure D-4 – Shear Reinforcement - Shear Angles

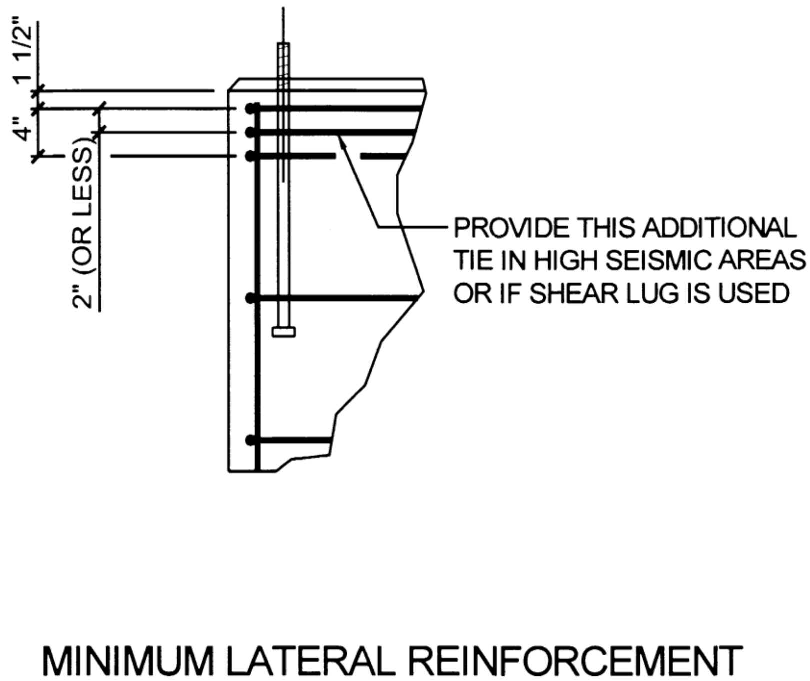


Figure E – Minimum Lateral Reinforcement - Pedestal

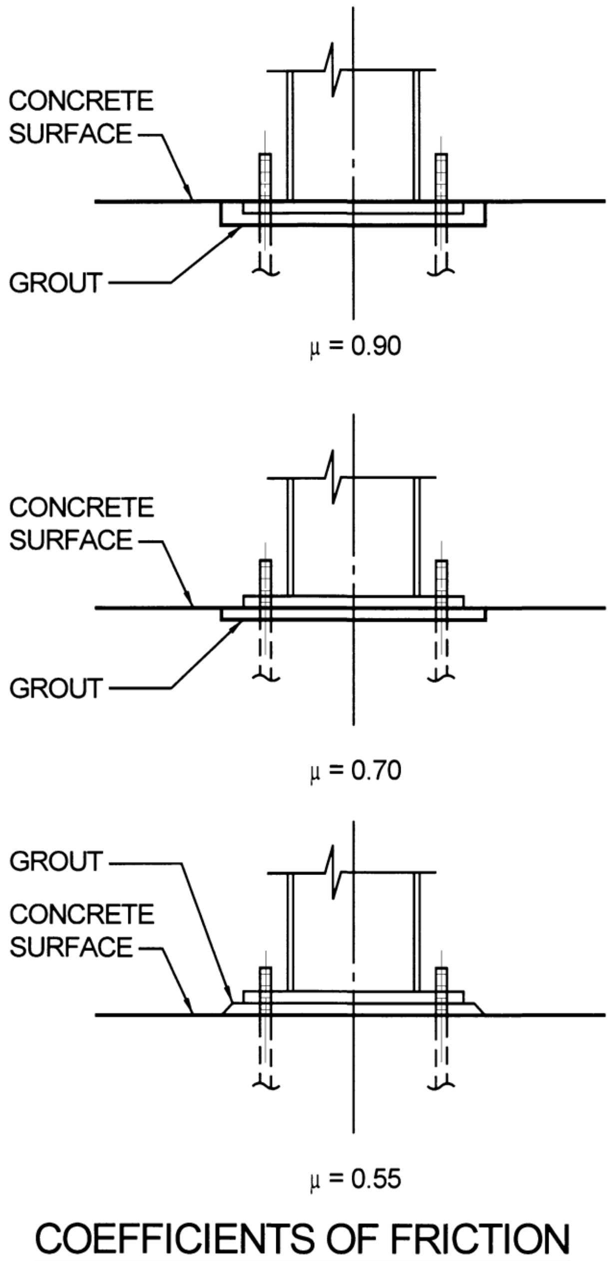
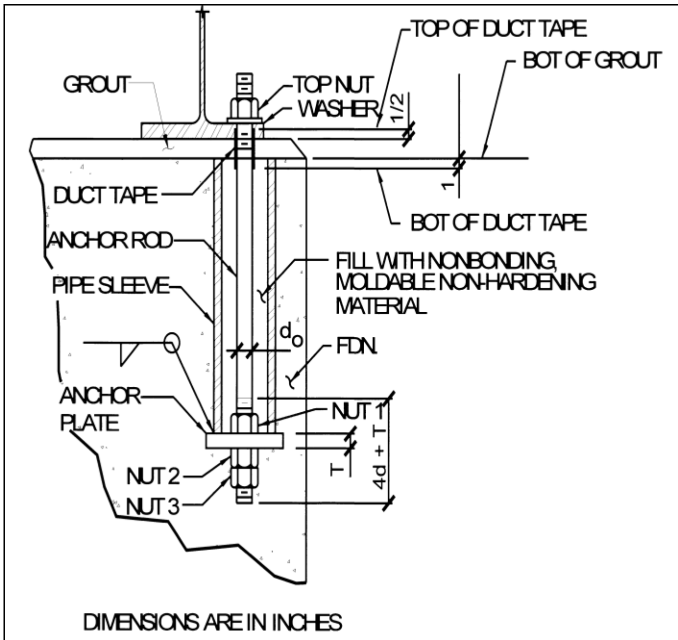


Figure F – Coefficients of Friction



Notes:

1. Materials:
   Anchor Plate: ASTM A36
   Anchor Rod: ASTM A36 or F1554 or 36.
   Nuts: ASTM A563 GR A Heavy Hex
   Washer: ASTM F436
   Pipe Sleeve: ASTM A53 SCH 40
2. Weld shall be in accordance with AWS D1.1.
3. Fabrication Sequence:
   A. Position Anchor Rod to obtain the specified projection above the Anchor Plate.
   B. Holding Nut 1, tighten Nut 2 to a snug tight condition.
   C. Hold Nut 2, tighten Nut 3 to a snug tight condition.
   D. Position and weld the pipe sleeve.

ANCHOR ROD DIAMETER db	NOMINAL PIPE SLEEVE DIAMETER	ANCHOR PLATE. DIMENSIONS	ANCHOR PLATE THICKNESS (T)
(in.)	(in.)	(in.)	(in.)
3/4	1-1/2	2 1/2 x 2 1/2	5/8
7/8	2	3 x 3	7/8
1	2	3 x 3	7/8
1-1/8	2-1/2	3 1/2 x 3 1/2	1
1-1/4	2-1/2	3 1/2 x 3 1/2	1
1-1/2	3	4 1/2 x 4 1/2	1 1/4
1-3/4	3-1/2	5 x 5	1 1/2
2	4	5 1/2 x 5 1/2	1 3/4
2-1/4	5	6 1/2 x 6 1/2	2
2-1/2	5	7 x 7	2 1/4
2-3/4	5	7 x 7	2 1/4
3	6	8 x 8	2 1/2

Figure G – Pretensioned Anchors for Turbines and Reciprocating Compressors

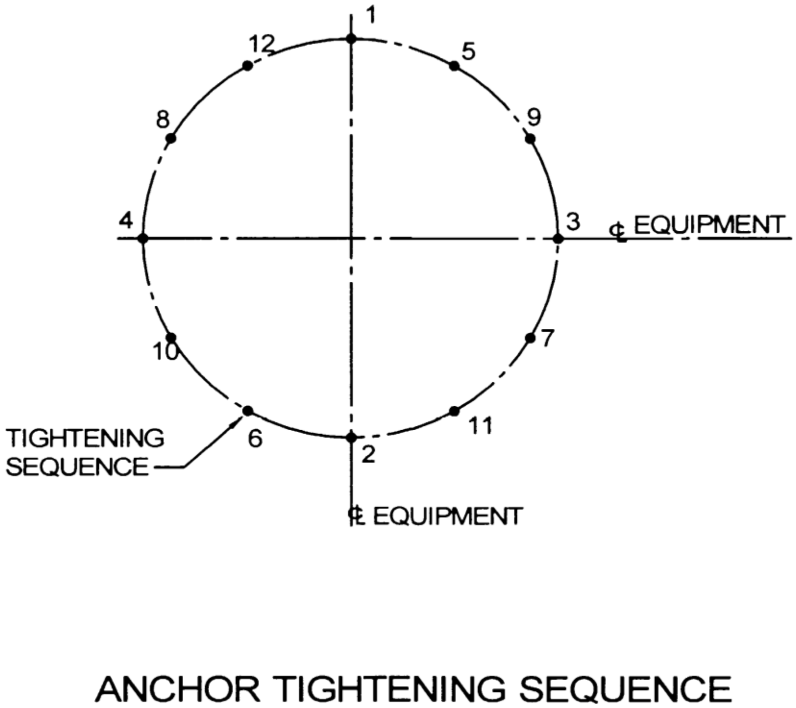


Figure H – Anchor Tightening Sequence

Example 1 – Column Plate Connection Using Spreadsheet

Base Plate Connection Data

W12x45 Column

Four Anchors on 8" x 16" spacing.

Base Plate 1½" x 14" x 1'-10" with vertical stiffener plates

Factored Base Loads (Gravity plus Wind - Maximum Uplift Condition)

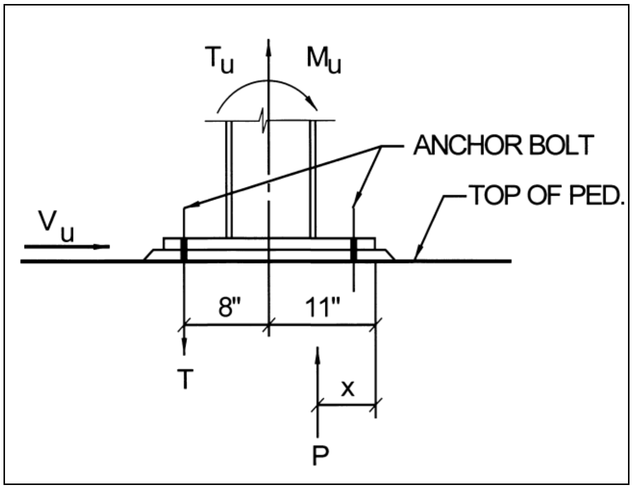
Shear (V_{ua}) = 17 kips

Moment (M_u) = 146 kip-feet

Tension (N_{ua}) = 17 kips

Low Seismic Area (ductility not required)

f'_c = 3000 psi, A36 Anchor Material



$\Sigma M_P = 0$

T = (146 k-ft x 12 + 17 k x 8.625")/(11 + 8 - 2.67)
T = 116 k for 2 bolts
P = 116 - 17 = 99 kips

Resisting friction load (V_f) = μ P

μ = 0.55 (PIP STE05121 - Figure F)
V_f = 0.55 x 99 = 54 kips > 17 kips

Therefore, anchors are not required to resist shear.

X = 2.67
(Ref.: Blodgett - *Design of Welded Structures* - Figure 17 [Similar])

BY TRIAL AND ERROR USING THE SPREADSHEET (THIS ONLY TAKES A FEW MINUTES), THE FOLLOWING IS DETERMINED:

Nom. Anchor Diameter = 1¾ inches   Anchor Embedment = 21 inches (12 anchor diameters).
Pedestal Size = 6' 4" x 4' 4" (c_{1a} = 30", c_{2a} = 23", c_{3a} = 46", c_{4a} = 23", s_2 = 6", s_1 = 0")

(Since only 2 bolts resist tension s_1 must be input as 0".)

The input and output sheets are attached for this condition.

This is a very large pedestal. If a smaller pedestal is required or desired, supplementary tensile reinforcing can be used to resist the load. See Example 2.

Example 2 – Column Plate Connection - Supplementary Tensile Reinforcing

Same data as example 1. Use supplementary tensile reinforcing to reduce pedestal size.

Shear (Vu) = 17 kips
Moment (Mu) = 146 kip-feet
Tension (Nu) = 17 kips

Per Example 1:
T = 116k on 2 bolts
Friction will take shear load.
Nom. Anchor Diameter = 1¾ inches

Assume a 2'-0" x 2'-6" Pedestal.

ASSUME ANCHORS ARE RESISTED BY 3 HAIRPINS.

116k / 3 = 38.7 kips
Per Table 3 of PIP STE05122 1- #7 hairpin resists 48.37 kips. OK.
l_{dh} (min) = 15.3 inches per Table 3 of PIP STE05122.

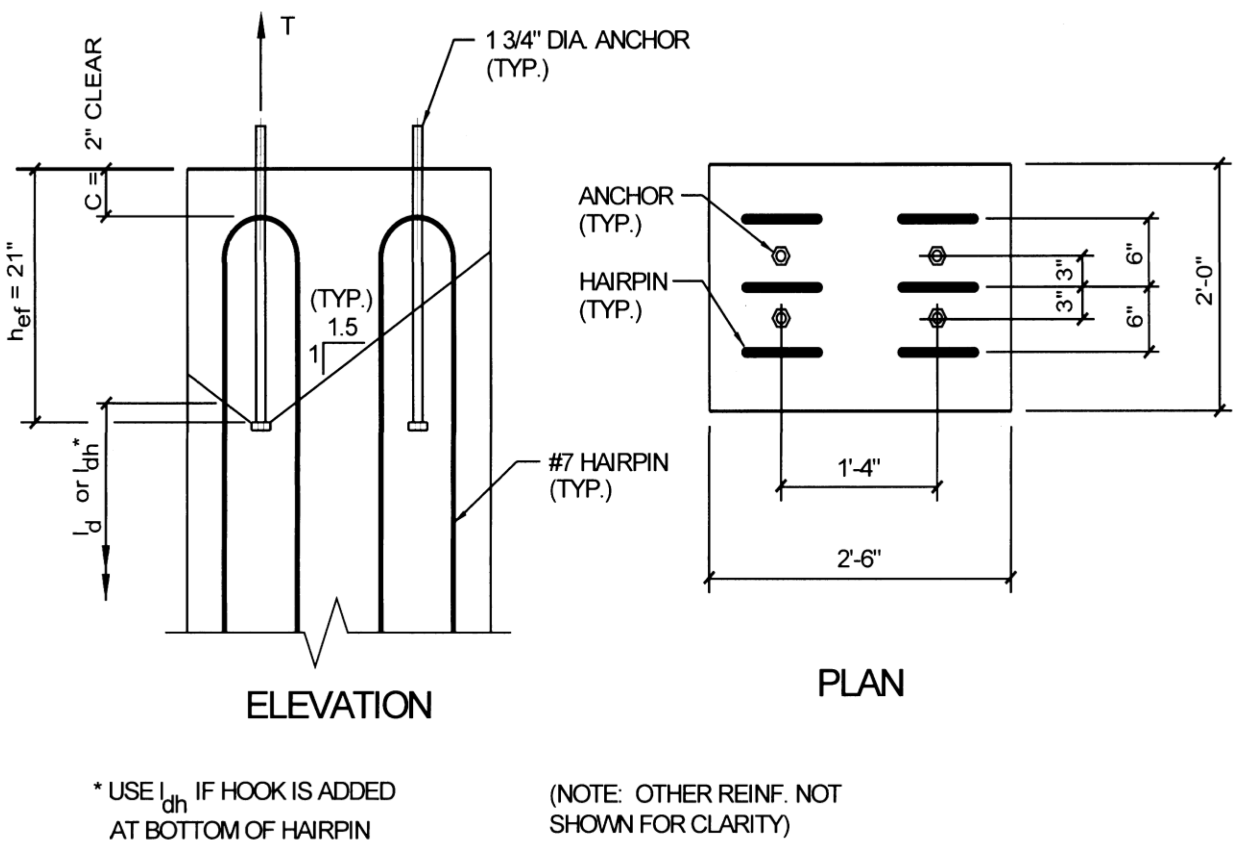
Space hairpins 3 inches away from each anchor. Required $h_{ef} = C + l_{dh} + 3/1.5$. See Figure C-2.

Where C = Concrete cover = 2 inches.

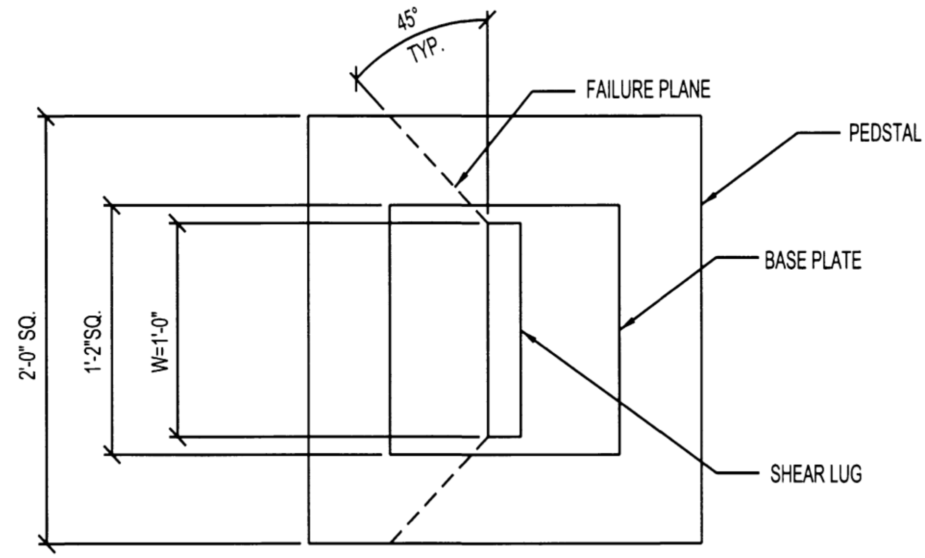
$h_{ef} = 2 + 15.3 + 4.61/1.5 = 20.4$ inches
min. $h_{ef} = 12\ d_0 = 12 \times 1.75 = 21$ inches.

Final Design

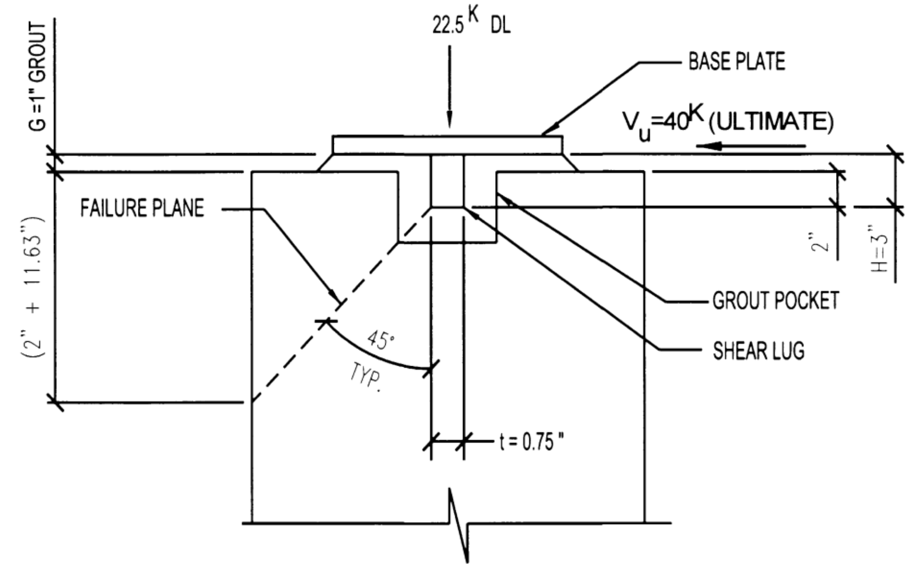
Use h_{ef} = 21 inches



Example 3 – Shear Lug Plate Section Design



PLAN



SECTION

Example 3 – Shear Lug Plate Section Design (cont'd)

Design a shear lug plate for a 14 inch square base plate, subject to a factored axial dead load of 22.5 kips, factored live load of 65 kips, and a factored shear load of 40 kips. The base plate and shear lug have F_y = 36 ksi and f'_c = 3 ksi. The contact plane between the grout and base plate is assumed to be 1 inch above the concrete. A 2 ft 0 in square pedestal is assumed. Ductility is not required.

$V_{app} = V_{ua} - V_f = 40 - (.55)(22.5) = 27.6$ kips

Bearing Area = $A_{req} = V_{app} / (0.85\ \phi\ f'_c)$ = 27.6 kips / (0.85*0.6*3 ksi) = 16.67 in²

Based on base plate size, assume the plate width, W, will be 12 in.

Height of plate = H = A_{req} / W + G = 16.67 in² /12 in + 1 in = 2.39 in. Use 3 in.

Ultimate Moment = $M_u = (V_{app} / W) * (G + (H - G)/2)$

= (27.6 kips / 12 in) * (1 in + (3 in-1 in)/2) = 4.61 k-in / in

Thickness = t = $[(4 * M_u)/(.9* f_{ya})]^{½}$ = $[(4*4.55 \text{ kip-in})/(.9*36 \text{ ksi})]^{½}$ = .754" Use 0.75"

This 12 in x 3 in x 0.75 in plate will be sufficient to carry the applied shear load and resulting moment. Design of the weld between the plate section and the base plate is left to the reader.

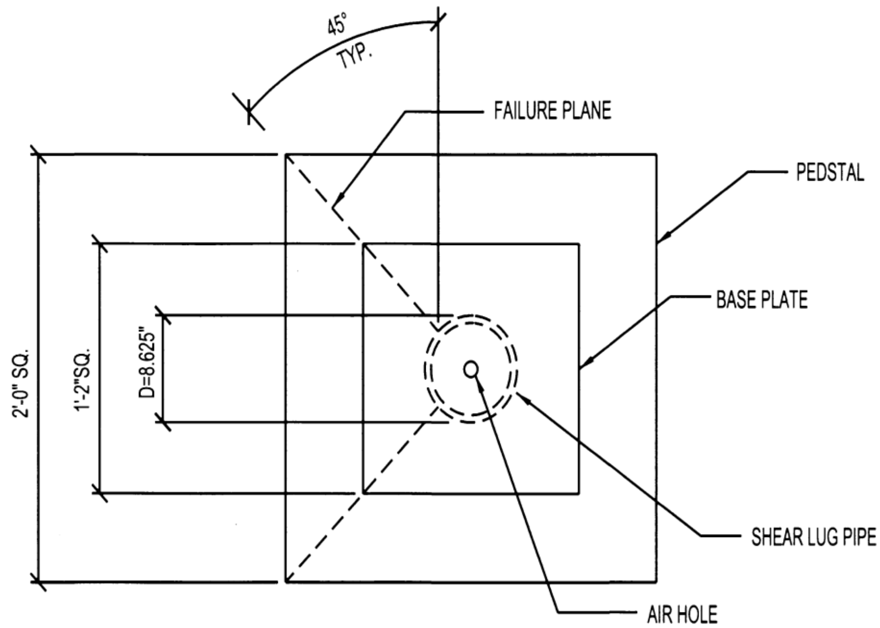
Check concrete breakout strength of the shear lug in shear.

Distance from shear lug to edge of concrete = (24 - .75) / 2 = 11.63 in.

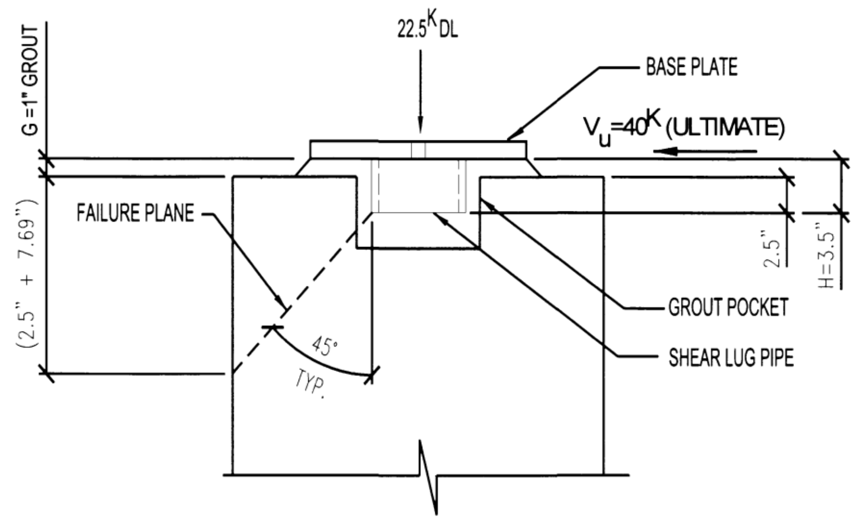
A_{Vc} = 24 * (2+11.63) – (12 * 2) = 303 in²

Vcb = A_V*4*ϕ*[f'c].5 = 303 * 4 * 0.85 * [3000].5 = 56400 lb = 56.4 kips > 27.3 kips OK

Example 4 – Shear Lug Pipe Section Design



PLAN



SECTION

Example 4 – Shear Lug Pipe Section Design (cont'd)

Design a shear lug pipe section for a 14 inch square base plate, subject to a factored axial dead load of 22.5 kips, factored live load of 65 kips, and a factored shear load of 40 kips. The base plate and shear lug have Fy = 36 ksi and f'c = 3 ksi. The contact plane between the grout and base plate is assumed to be 1 inch above the concrete. A 2 ft 0 in pedestal is assumed. Ductility is not required.

$V_{app} = V_{ua} – V_f = 40 – (.55)(22.5) = 27.6$ kips

Bearing Area = $A_{req} = V_{app} / (0.85\ \phi\ f'_c)$ = 27.6 kips / (0.85*0.6*3ksi) = 16.67 in²

Based on base plate size, assume the pipe diameter will be 8-in. nominal std. weight pipe.
D = 8.625 in., t = .322 in., S = 16.81 in³, Z = 22.2 in³

Height of pipe = H = A_{req} / D + G = 16.67 in² / 8.625 in + 1 in = 2.92 in Use 3.5 in

Ultimate Moment = $M_u = V_{app} * (G + (H – G)/2)$
= 27.63 kips * (1 in + (3.5 in - 1 in)/2) = 62.17 k-in

Check Moment:

$M_n = S [\{600/(D/t)\} + f_{ya}]$ = 16.81 in³ *[600/(8.625 in./0.322 in.) + 36ksi]
= 982 k-in

or $M_n = Z * f_{ya}$ = 22.2 in³ * 36 ksi = 799 k-in
$\phi_b = 0.9$
$\phi_b M_n$ = (0.9)*(799 k-in) = 719 k-in > 62.17 k-in ok

Check Shear:

$V_n = 0.6\ F_y\ \pi(D^2 – (D-2t)^2)/4$ = 0.6* 36ksi * π* [8.625² – (8.625 – 0.322)²] in² / 4
= 181.4 kips
$\phi_v = 0.9$
$\phi_v V_n$ = (0.9)*(181.4 kips) = 163.2 kips > 27.6 kips ok

This 3.5 inch long x 8-in. diameter nominal std. weight pipe will be sufficient to carry the applied shear load and resulting moment.

Check Failure Plane of Pedestal.

Distance from edge of pipe to edge of concrete = (24 – 8.625) / 2 = 7.69 in.

A_{vc} = 24*(2.5 + 7.69) – 8.62*2.5 = 223 in²

$V_{cb} = A_v*4*\phi*[f'_c]^{.5}$ = 323 * 4 * 0.85 * [3000]$^{.5}$ = 41500 lb = 41.5 kips > 27.3 kips ok