PIP STE05121
Application of ASCE Anchorage Design for Petrochemical Facilities
PURPOSE AND USE OF PROCESS INDUSTRY PRACTICES

In an effort to minimize the cost of process industry facilities, this Practice has been prepared from the technical requirements in the existing standards of major industrial users, contractors, or standards organizations. By harmonizing these technical requirements into a single set of Practices, administrative, application, and engineering costs to both the purchaser and the manufacturer should be reduced. While this Practice is expected to incorporate the majority of requirements of most users, individual applications may involve requirements that will be appended to and take precedence over this Practice. Determinations concerning fitness for purpose and particular matters or application of the Practice to particular project or engineering situations should not be made solely on information contained in these materials. The use of trade names from time to time should not be viewed as an expression of preference but rather recognized as normal usage in the trade. Other brands having the same specifications are equally correct and may be substituted for those named. All Practices or guidelines are intended to be consistent with applicable laws and regulations including OSHA requirements. To the extent these Practices or guidelines should conflict with OSHA or other applicable laws or regulations, such laws or regulations must be followed. Consult an appropriate professional before applying or acting on any material contained in or suggested by the Practice.

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# PIP STE05121
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1. **Scope**

This Practice provides guidelines for design engineers involved in design, fabrication, installation, and repair of anchorage for foundations and structures. This Practice supplements the *ASCE Anchorage Design for Petrochemical Facilities*, hereafter referred to as the *ASCE Anchorage Design Report*. The information on fabrication, installation, and repair of anchorages provided in this Practice is to be used by design engineers to develop specifications, drawings, scopes of work, etc. for fabricators, constructors, and maintenance personnel involved in fabrication, installation, and repair activities.

This Practice describes the design of anchorage based on the requirements in *ACI 318-11 / ACI 318M-11* and *ACI 318-14 / ACI 318M-14* including material selection, cast-in-place anchor design, post-installed anchor design, anchor installation and repair.

**Comment:** The information provided in the *ASCE Anchorage Design Report* is recommended for the design, fabrication, installation and repair of anchorage for foundations and structures. The *ASCE Anchorage Design Report* is in accordance with *ACI 318-08 / ACI 318M-08*. This Practice is in accordance with *ACI 318-11 / ACI 318M-11* and *ACI 318-14 / ACI 318M-14*. Differences with the *ASCE Anchorage Design Report* are noted in this Practice. Many anchor design tables that are not included in the *ASCE Anchorage Design Report* have been included in this Practice for use by engineers and designers.

**Comment:** The primary differences between *ACI 318-11 / ACI 318M-11* and *ACI 318-14 / ACI 318M-14* are as follows:

a. *ACI 318-14 / ACI 318M-14* have been reorganized so that all of the chapter, section, paragraph, equation, and table numbers have been changed.

b. The anchorage information that was in Appendix D of *ACI 318-11 / ACI 318M-11* is now in Chapter 17 of *ACI 318-14 / ACI 318M-14*.

c. *ACI 318-14 / ACI 318M-14* versions do not recognize the Alternative Load and Strength Reduction Factors shown in Appendix C of *ACI 318-11 / ACI 318M-11* and earlier *ACI 318 / ACI 318M* versions. The only Load Factors and Load Combinations that may now be used are those in what is now Chapter 5 of *ACI 318-14 / ACI 318M-14*.

d. When *ACI 318-14* or *ACI 318M-14* is the project standard, the user of the *ASCE Anchorage Design Report* should interpret “Appendix D” as “Chapter 17”. Table 6 in this Practice, shows the new chapter, section, paragraph, equation, and table numbers in the *ASCE Anchorage Design Report* which should be used when using *ACI 318-14* or *ACI 318M-14* as the project standard. Table 6 does not show the change if the only difference is using “Chapter 17” instead of “Appendix D”.

There are no other technical differences in regard to anchorage between the 2011 and 2014 versions of *ACI 318 / ACI 318M*. 
2. References

There is an extensive set of references at the end of each chapter in the ASCE Anchorage Design Report. These shall be considered part of this Practice. The references for the ASCE Anchorage Design Report and the revised ACI 318 and ACI 318M are shown below.

2.1 Industry Codes and Standards

- American Concrete Institute (ACI)
  - ACI 318-08 - Building Code Requirements for Structural Concrete and Commentary
  - ACI 318-11 - Building Code Requirements for Structural Concrete and Commentary
  - ACI 318-14 - Building Code Requirements for Structural Concrete and Commentary
  - ACI 318M-11 - Building Code Requirements for Structural Concrete and Commentary (Metric)
  - ACI 318M-14 - Building Code Requirements for Structural Concrete and Commentary (Metric)
  - ACI 355.3R-11 - Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

- American Institute of Steel Construction (AISC)
  - AISC Manual of Steel Construction

- American Society of Civil Engineers (ASCE)
  - Anchorage Design for Petrochemical Facilities, Task Committee on Anchorage Design, 2013 [Short title used herein is ASCE Anchorage Design Report.]

- Portland Cement Association (PCA)
  - PCA Notes on ACI 318-11 Building Code

3. Definitions

An extensive set of definitions and notations are in the ASCE Anchorage Design Report and are considered part of this Practice.

Comment: Traditionally, “anchors” as defined in the ASCE Anchorage Design Report and in ACI 318 / ACI 318M have been called “anchor bolts”. However, since there are many types of anchors that are not traditional bolts (i.e., bolts with a head on one end with a nut or nuts on the other end), the term “anchor” is used in this Practice to be consistent with the ASCE Anchorage Design Report and with recent versions of ACI 318 / ACI 318M. The definition of “anchor” in the ASCE Anchorage Design Report is “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.”
Following are definitions used in this Practice for anchorage types:

anchor: General term for the anchor bolt or anchor rod assembly. This does not include the concrete and rebar which are parts of the anchorage.

anchor bolt: Traditional bolt with a head on one end and a nut or nuts on the other end used for anchorage to concrete.

anchor plate: Circular plate bolted at the bottom of an anchor bolt or anchor rod to increase the pull out capacity of the anchor. Typically, this is required to make the anchorage ductile.

anchor rod assembly: Fabricated assembly that includes a rod threaded at the two ends, nuts, washers, and anchor plates if required.

anchorage: The complete system for anchoring into concrete. This includes the anchor, concrete into which the anchor is embedded, sleeves, and rebar transmitting the load from the anchor into the concrete.

The following acronyms are used in this Practice but are not in the ASCE Anchorage Design Report:

- **AC**: Diameter of a circular pattern of anchors (i.e., anchor circle), inches (mm)
- **AP**: Anchor Plate (see definition above)
- **AVc**: Projected concrete failure area in shear for a single anchor in an octagon foundation, inches (mm)
- **ca**: Distance from center of an anchor to the edge of concrete, inches (mm)
- **D**: Face to face dimension of an octagon, inches (mm)
- **Do**: Octagon equivalent circle diameter equal to 1.03D, inches (mm)
- **P**: Anchor rod projection above concrete (see Figure 1), inches (mm)
- **TB1**: Thread length at bottom of anchor rod with no anchor plate and dimension from bottom of anchor to top of bottom nut (see Figure 1, Type 2), inches (mm)
- **TB2**: Thread length at bottom of anchor rod with anchor plate and dimension from bottom of anchor to top of upper bottom nut (see Figure 1, Type 2), inches (mm)

4. Materials

4.1 Recommendations for materials for anchor bolts, studs, anchor rods, washers, nuts, and sleeves are included in ASCE Anchorage Design Report, Chapter 2. Recommendations for anchor material fabrication, corrosion protection, and considerations for extreme temperatures are also included in ASCE Anchorage Design Report, Chapter 2. These recommendations are considered part of this Practice with the following exceptions:

a. Section 2.2.1, delete the note in the box under the Section.

Comment: ASTM F1554 Grade 105 material can be used for ductile anchors. Even though Table 3 of ASTM F1554 shows a minimum of 12 percent elongation using an 8 in (203 mm) specimen (ACI 318 requires 14 percent elongation for ductile connections); Table 3 requires a minimum
of 15 percent elongation for 2 inch (50.8 mm) specifications which does meet the requirement for ductile anchors. The 2 inch (50.8 mm) test (versus the 8 inch (203 mm) test) is the only one with a consistent length to diameter ratio since it is machined to a 1/2 inch (12.7 mm) diameter with a gage length of 2 inches (50.8 mm) (i.e., a 4 to 1 ratio). Therefore the 2 inch (50.8 mm) test should be the only test used for determining the ACI 318 ductility/elongation requirements. This is supported by the fact that both PCA Notes on ACI 318-11 Table 34-1 and ACI 355.3R-11 Table A.1 specifically shows the 2 inch (50.8 mm) test as the base requirement for determining ductility/elongation requirements.

b. Table 2.1b, replace the last 3 lines in the table with the following:

<table>
<thead>
<tr>
<th>ASTM Specification</th>
<th>Fy, min, ksi (MPa)</th>
<th>Futa, min, ksi (MPa)</th>
<th>Diameter Range, in. (mm)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1554 Gr 55</td>
<td>55 (380)</td>
<td>75 (517)</td>
<td>⅛ to 2 (6.4 to 50)</td>
<td>Weldable with Specification’s Supplementary Requirement S1.</td>
</tr>
<tr>
<td>F1554 Gr 55</td>
<td>55 (380)</td>
<td>75 (517)</td>
<td>over 2 to 4 (50 to 102)</td>
<td>Weldable with Specification’s Supplementary Requirement S1. Not ductile due to reduction in area &lt; 30 percent.</td>
</tr>
<tr>
<td>F1554 Gr 105</td>
<td>105 (724)</td>
<td>125 (862)</td>
<td>⅛ to 4 (6.4 to 102)</td>
<td>None.</td>
</tr>
</tbody>
</table>

4.2 Threaded anchor rods should only be threaded at the top and bottom of the anchor rods where required for nuts. Threads should be excluded in the body of the anchor rod where the anchor rod is in contact with the concrete. This provides uniform tension stresses on the anchor rod when under tension, and maximizes the stretching length of the anchor rod.

4.3 Hot-dip galvanized anchors (i.e., anchor bolts, anchor rods and nuts) should use the UNC thread series. Nuts for galvanized anchors are tapped oversize to accommodate the extra 2 – 6 mils (0.05 – 0.15 mm) of zinc that is added to the threads of the anchor bolts and rods during the galvanizing process. Without this allowance, the nut cannot thread onto the anchor bolt or rod. If the 8 UN series is used instead of the UNC thread series, the finer threads for anchors with bolt or rod diameters greater than 1 ½ inches (40 mm) can result in premature thread stripping of the anchor bolts or rods, and reduce the capacity of theanchor bolts or rods. For simplicity, the UNC thread series should be used for all anchors that are to be hot-dip galvanized, and not just for anchors with bolt or rod diameters greater than 1 ½ inches (40 mm).

4.4 ASTM F1554 permits anchor suppliers to provide Grade 55 weldable anchors as a substitute for Grade 36 anchors. This substitution creates no problem except if ductile anchors are required. If ductile anchors are required, anchors should be both designed and specified as Grade 55, unless design calculations are performed to assure that Grade 36 anchors are acceptable.
5. **Cast-in-Place Anchorage Design**

5.1 **General**

Recommendations for the following are included in *ASCE Anchorage Design Report*, Chapter 3. These recommendations are considered part of this Practice except where changes are shown in Section 5.5.

a. Anchorage configuration and dimensions  
b. Strength design of anchorage  
c. Ductile design of anchorage  
d. Design of reinforcement to transmit loads from the anchors to the concrete  
e. Using frictional resistance to transmit shear loads into the concrete  
f. Transmitting shear force into anchorage  
g. Shear lug design  
h. Tensioning of anchors  
i. Welded anchors for embedded plates  
j. Vibratory loads considerations  
k. Seismic loads considerations  
l. Constructability considerations

5.2 **Anchorage Design Spreadsheets**

5.2.1 Electronic entry Anchorage Design Spreadsheets have been developed for use by PIP Member Companies only. The spreadsheets give shear and tensile capacities of an anchor or anchor group and the concrete surrounding it. They check for *ACI 318 / ACI 318M* code requirements and anchor ductility. The spreadsheets are provided in U.S. Customary units and Metric (SI) units.

5.2.2 The spreadsheets are in accordance with *ACI 318-11* and *ACI 318M-11*, Appendix D and *ACI 318-14* and *ACI 318M-14*, Chapter 17.

*Comment:* There are four versions, two in U.S. Customary units in accordance with *ACI 318-11* and *ACI 318-14* and two in Metric (SI) units in accordance with *ACI 318M-11* and *ACI 318M-14*.

5.2.3 The spreadsheets should be used in conjunction with the *ASCE Anchorage Design Report* and *ACI 318-11* or *ACI 318M-11*, Appendix D or *ACI 318-14* or *ACI 318M-14*, Chapter 17 as applicable.

5.2.4 The spreadsheets can be used to determine the following:

a. Shear and tensile capacities of an anchor or anchor group and the concrete around it  
b. Whether an anchorage is ductile

5.2.5 The spreadsheets can be accessed on the PIP Members website.
5.2.6 The spreadsheets can save time in laborious calculations, but are not a substitute for the engineer’s knowledge and expertise.

5.2.7 See Appendix, Example A, for an application of the Anchorage Design Spreadsheet.

5.3 **Anchorage Tension Rebar Check**

5.3.1 An electronic entry Anchorage Tension Rebar Check Spreadsheet has been developed for use by PIP Member Companies only.

5.3.2 The spreadsheet can be used to check the rebar sizes and development lengths for transferring tension from anchors to rebar.

5.3.3 The spreadsheet can be used in conjunction with the Anchorage Design Spreadsheet described in Section 5.2 to check the rebar if it is decided to make a column pedestal smaller and use rebar to replace the concrete breakout cone requirement.

5.3.4 The spreadsheet is in accordance with ACI 318-11 / ACI 318M-11 and ACI 318-14 / ACI 318M-14 codes.

5.3.5 The spreadsheets should be used in conjunction with the ASCE Anchorage Design Report and ACI 318-11, ACI 318M-11, ACI 318-14 or ACI 318M-14, as applicable.

5.3.6 The spreadsheet can be accessed on the PIP Members website.

5.3.7 The spreadsheet can save time in laborious calculations, but is not a substitute for the engineer’s knowledge and expertise.

5.4 **Anchorage Design for Column Pedestals**

5.4.1 MathCAD templates for Anchorage Design for Column Pedestals have been developed for use by PIP Member Companies only.

5.4.2 The MathCAD templates can be used to develop the full design of the anchors and reinforcement for a steel column anchored to a pedestal. The user can make changes in the template input to perform the design.

5.4.3 The MathCAD templates should be used in conjunction with the ASCE Anchorage Design Report and ACI 318-11, ACI 318M-11, ACI 318-14 or ACI 318M-14, as applicable.

5.4.4 The MathCAD templates can be used to determine the following:
   a. Anchor sizing
   b. Pull-out resistance of the anchorage in tension
   c. Side-face blowout resistance of the anchorage in tension
   d. Transfer of anchor load to vertical rebar
   e. Design of shear reinforcement
   f. Minimum distance to preclude splitting failure

5.4.5 The MathCAD templates can be accessed on the PIP Members website.
5.4.6 The MathCAD templates can save time in laborious calculations, but are not a substitute for the engineer’s knowledge and expertise.

5.4.7 See Appendix, Example D, for an example of the MathCAD template.

5.5 Changes in ASCE Anchorage Design Report, Chapter 3

5.5.1 The following changes to ASCE Anchorage Design Report, Chapter 3, are required for the Report to be in accordance with ACI 318-11 / ACI 318M-11 and for ACI 318-14 / ACI 318M-14 as well as to incorporate the most current recommended practice by PIP.

5.5.2 Section 3.2.3.1, second paragraph, modify as follows:

Change the term “elastomeric fill” to “nonbonding moldable material or elastomeric moldable non-hardening material”.

Replace the sentence, “In the case of larger diameter anchors, the only relevant application for using a sleeve is the case where the anchor will be tensioned.” with the following:

“In the case of larger diameter anchors, the only relevant applications for using a sleeve are to provide stretch length to meet the requirements for the anchor to be a ductile steel element or where the anchor will be tensioned.”

5.5.3 Paragraph following Table 3.1, modify as follows:

Change the term “approved elastomeric material” to “nonbonding moldable material or elastomeric moldable non-hardening material”.

Replace the sentence, “For full-length sleeves, the minimum $A_{brg}$ shall be calculated using ACI 318, equation D-15.” with the following:

“For full-length sleeves, the minimum $A_{brg}$ should be calculated using ACI 318-11 / ACI 318M-11, equation D-14 or ACI 318-14 / ACI 318M-14, equation 17.4.3.4”.

5.5.4 Section 3.4, second paragraph, modify as follows:

Replace the sentences, “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.)” with the following:

“An anchor that is characterized as a ductile steel element should have a stretch length of at least eight anchor diameters unless otherwise determined by analysis. (See ACI 318-11 / ACI 318M-11, paragraph D.3.3.4.3 (a) 3 or ACI 318-14 / ACI 318M-14 paragraph 17.2.3.4.3(a)(iii).) This stretch length can be obtained by the use of a sleeve that is longer than the stretch length filled with either nonbonding moldable material or elastomeric moldable non-hardening material or by wrapping the anchor with a non-bonding tape for the required stretch length.”
5.5.5 Section 3.4, third paragraph, modify as follows:

Replace the sentence, “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.” with the following:

“Where anchor ductility cannot be assured, brittle failure can be prevented by designing a ductile yielding mechanism in the attachment in flexure, shear, or bearing, or a combination of those conditions, and considering both material overstrength and strain hardening effects for the attachment. The anchor is then designed for the maximum tension and shear that can be transmitted to the anchor via this mechanism. (See ACI 318-11 / ACI 318M-11, paragraph D3.3.4.3 (b) and paragraph D3.3.5.3 (b) or ACI 318-14 / ACI 318M-14, paragraph 17.2.3.4.3 (b) and paragraph 17.2.3.5.3 (a).)”

5.5.6 Section 3.4, last paragraph, last sentence, modify as follows:

Replace the sentence, “Alternatively, the use of strength reduction factors similar to that discussed in ACI 318 Section D.3.3.6 is permitted” with the following:

“Alternatively; the use of the amplification factor, $O_0$, shown in ACI 318-11 / ACI 318M-11, Section D.3.3.5.3 (c) or ACI 318-14 / ACI 318M-14 Section 17.2.3.5.3 (c) is permitted”.

5.5.7 Paragraph 3.5.1.1, modify as follows:

Replace the part of the sentence near the center of the paragraph, “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…” with the following:

“Since supplementary reinforcement can improve the deformation capacity for the breakout mode, ACI 318-11 / ACI 318M-11, Sections D.4.3(c) and D.4.4(c) or ACI 318-14 / ACI 318M-14 Section 17.3.3 (c) permit…”

5.5.8 Paragraph 3.5.3.2.2, 3(a), modify as follows:

Replace the sentence, “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” with the following:

“Only the uppermost two layers of ties (the ASCE committee recommends two #4 ties) within 5 in. (127 mm) of the top of the pedestal as required by ACI 318-11 / ACI 318M-11, Section 7.10.5.7 or ACI 318-14 / ACI 318M-14 Section 10.7.6.1.6) are effective”.

5.5.9 Paragraph 3.9.6.a., modify as follows:

Replace the second and third sentences, “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.” with the following:

“ACI 318-11 / ACI 318M-11, Appendix D provides two equations for the calculation of shear strength, (D-28) and (D-29). Equation (D-28) is for
cast-in headed stud anchors.” or “ACI 318-14 / ACI 318M-14, Chapter 17 provides two equations for the calculation of shear strength, (17.5.1.2a) and (17.5.1.2b). Equation (17.5.1.2b) is for cast-in headed stud anchors.”

5.5.10 Paragraph 3.11.1 b., replace with the following:

“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 multiplied by 1.2. Note that the concrete strengths are not multiplied by 0.75φ for this comparison. (See ACI 318-11 / ACI 318M-11, Section D.3.3.4.3 (a) 1 or ACI 318-14 / ACI 318M-14 Section 17.2.3.4.3 (a) (i).)”

5.5.11 Paragraph 3.11.1 c., add the following at the end of the paragraph:

“The load transferred from the ductile yield mechanism should include material overstrength and strain hardening effects for the attachment and the anchor needs to be designed using the factor 0.75φ. (See ACI 318-11 / ACI 318M-11, Section D.3.3.4.3 (b) or ACI 318-14 / ACI 318M-14, Section 17.2.3.4.3(b).)”

5.5.12 Paragraph 3.11.1 d., replace with the following:

“d. Where yielding in the attached component or in the anchor cannot be achieved, it is acceptable to design the anchorage for Ω0 times the seismic loads transmitted by the attachment. The anchorage shall be designed using the strength reduction factor φ but the additional multiplier 0.75 does not need to be used, (See ACI 318-11 / ACI 318M-11, Sections D.3.3.4.3 (d) and D.3.3.5.3 (c) or ACI 318-14 / ACI 318M-14, Section 17.2.3.4.3 (d) and 17.2.3.5 (c).)”

5.5.13 Figure 3.25: Flow Chart for Seismic Design of Anchorage, replace with the following figures from this Practice:

Figure 5: Flowchart for Seismic Design of Anchorage – Tension

Figure 6: Flowchart for Seismic Design of Anchorage – Shear

Comment: The chart in the ASCE Anchorage Design Report has been divided into two flowcharts, since failure of an anchor in shear is no longer considered to be a viable ductile failure mechanism (see ACI 318-11 / ACI 318M-11 Commentary, Section RD.3.3.5, first paragraph or ACI 318-14 / ACI 318M-14 Commentary, Section R17.2.3.5, first paragraph).

5.5.14 Replace Table 3.3 Recommended Maximum Sizes for Anchor Holes in Base Plates and Maximum Fabricated Washer Sizes (including Notes) with Tables 5 and 5M from this Practice.

The diameter of anchor holes in base plates should be determined by the responsible engineer considering the following:

a. Larger holes can be used to accommodate inaccurate placement of anchors in the concrete but may require ASTM A36/A36M plate washers in addition to the ASTM F436/F436M washers to cover the holes and provide adequate bearing for the nut and ASTM F436/F436M washers. If the anchors are required to resist shear loads, there could be a fairly significant movement of
the base plate before the shear load is transferred to the anchors if large holes are used and the ASTM A36/A36M washers are not welded to the base plate. Field welding of the plate washers to the base plate is not recommended if the base plate is galvanized.

b. Smaller holes do not require additional ASTM A36/A36M plate washers and can minimize movement to resist shear loads but will require more accurate placement of the anchors in the concrete.

Tables 5 and 5M provide the responsible engineer with information to determine appropriate anchor hole diameters in base plates and whether ASTM A36/A36M plate washers are required.

5.5.15 Example 1 in the ASCE Anchorage Design Report refers to ACI 318-08. Example D in this Practice is an update of Example 1 incorporating revised references to ACI 318-14 instead of ACI 318-08. Member Companies can also use the MathCAD templates available on the PIP web site member area to design to ACI 318-11 / ACI 318M-11 and ACI 318-14 / ACI 318M-14. Note that reference changes to Example 1 in the ASCE Anchorage Design Report have not been included in Table 6 in this Practice as Example D in this Practice already shows the reference changes.

5.6 Anchor Plates

5.6.1 ACI 318-11 / ACI 318M-11 and ACI 318-14 / ACI 318M-14 Considerations

5.6.1.1 If anchors are required to be ductile, a plate (i.e., anchor plate) may need to be placed at the bottom of the anchor. This is because the pull out strength of a headed bolt, or nut on a threaded rod, $N_p$, is typically less than 1.2 times the nominal steel strength of the anchor if the concrete is considered as cracked ($\Psi_{c,P} = 1.0$).

5.6.1.2 If the concrete is considered as uncracked ($\Psi_{c,P} = 1.25$), it is very important to determine that there is no cracking of concrete in the anchor region during the design earthquake. Cracking due to shrinkage or temperature does not need to be considered but any kind of flexural cracking does need to be considered when the reinforcing steel has fully yielded. ACI 318-11 / ACI 318M-11, paragraph D.3.3.4.4 or ACI 318-14 / ACI 318M-14, paragraph 17.2.3.4.4 states “the anchor design tensile strength for resisting earthquake forces should be determined … assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.”

5.6.1.3 ACI 318-11 / ACI 318M-11 Commentary, Section RD.3.3.4.4 or ACI 318-14 / ACI 318M-14 Commentary, Section R17.2.3.4.4 states, “Because seismic design generally assumes that all or portions of the structure are loaded beyond yield, it is likely that the concrete is cracked throughout for the purpose of determining the anchor strength. In locations where it can be demonstrated that the concrete does not crack, uncracked concrete may be assumed for determining the anchor strength as governed by concrete failure modes.”
5.6.2 Standardization

5.6.2.1 See Figure 6 and Tables 4 and 4M of this Practice for guidance regarding standardizing anchor plates. Anchor plates are designed for ASTM F1554 Grade 36, ASTM F1554 Grade 55, ASTM F1554 Grade 105, ASTM A354 Grade BC, and ASTM A449 anchors. Two different concrete compressive strengths are used in the design $f'_c = 4,000$ psi (28 MPa) and $f'_c = 5,000$ psi (35 MPa).

5.6.2.2 Plates are designed for U.S. Customary anchors with anchor diameters from $\frac{1}{2}$ inch diameter to 4 inches and for Metric anchors with anchor diameters from 16 mm through 72 mm.

5.6.2.3 The anchor plates should be round as recommended by the ASCE Anchorage Design Report.

5.6.2.4 For another anchor material, the ultimate yield strength of the anchor material can be compared with a similar strength anchor material in the tables.

5.7 Tables to Aid Engineers

Tables are provided in this Practice as an aid for designing anchorage. The following are descriptions of the content of each table and guidance for their use:

a. Tables 1 and 1M show minimum dimensions of anchors and sleeves in U.S. Customary and Metric units, respectively.

b. Tables 2 and 2M show reinforcing steel capacities and development lengths for various concrete covers in U.S. Customary and Metric units, respectively. These are used for reinforcement design if anchors are designed to transfer their loads into reinforcing. The development lengths are for 4,000 psi (28 MPa) concrete, but development length multiplication factors are provided for other concrete strengths. The engineer needs to make sure that reinforcing steel is developed on both sides of a theoretical crack plane in the concrete.

c. Tables 3 and 3M show the details of hook and hairpin reinforcing steel along with their capacities in U.S. Customary and Metric units, respectively. The hook dimensions are provided for 4,000 psi (28 MPa) concrete, but factors are provided for other concrete strengths.

d. Tables 4 and 4M show anchor plate dimensions to provide full capacities of anchors in pull-out in U.S. Customary and Metric units, respectively. See section 5.6 for more information.

e. Tables 5 and 5M provide information to determine appropriate anchor hole diameters in base plates and whether ASTM A36/A36M plate washers are required in U.S. Customary and Metric units, respectively.

f. Table 6 shows the chapter, section, paragraph, equation, and table numbers which need to be changed in the ASCE Anchorage Design Report when using ACI 318-14 or ACI 318M-14 as the project standard.
6. Post-Installed Anchor Design

Recommendations for the following are included in *ASCE Anchorage Design Report*, Chapter 4. These recommendations are considered part of this Practice.

*Comment:* Some references in *ASCE Anchorage Design Report* refer to *ACI 318*, Appendix D. Starting with *ACI 318-14*, these references are in Chapter 17. Also, all of the definitions that were in Appendix D have been moved to *ACI 318-14*, Chapter 2.

a. Post-Installed Mechanical Anchors  
b. Post-Installed Bonded Anchors  
c. Considerations in Post-Installed Anchor Design  
d. Post-Installed Anchor Design  
e. Seismic Loading on Post-Installed Anchors  
f. Design of Post-Installed Anchors for High-Cycle Fatigue  
g. Post-Installed Anchor Qualification

7. Anchorage Installation and Repair

7.1 Recommendations for the following are included in *ASCE Anchorage Design Report*, Chapter 5. These recommendations are considered part of this Practice.

a. Post-Installed Anchor Installation  
b. Constructability Considerations  
c. Anchor Repair Procedures  

7.2 If anchors are provided at locations that require the structure or equipment base to slide for expansion and/or contraction purposes, two top nuts should be shown on the design drawings along with the following notes to specify how the nuts are to be installed.

a. Lower top nut shall be hand tightened and then backed off a half turn leaving approximately 1/16-inch (1.5 mm) clearance between lower top nut and structure or equipment base.  
b. Upper top nut shall then be installed and jammed against lower top nut.

8. Examples

8.1 Examples for the following are included in *ASCE Anchorage Design Report*, Appendix A. These examples are considered part of this Practice.

a. Example 1: Anchorage Design for Column Pedestals  

*Comment:* Example 1 shows the design of the anchors and reinforcement for a steel column anchored to a pedestal. Anchor sizing, pull-out resistance of the anchor in tension, side-face blowout resistance of the anchor in tension, transfer of anchor load to vertical rebar, design of shear reinforcement, and minimum distance to preclude splitting failure are all included in this example.
b. Example 2: Anchorage Design for Octagonal Pedestal

c. Example 3: Shear Lug Pipe Section Design

Comment: The titles in Examples 1 and 2 have been changed from “Anchor Design…” to “Anchorage Design…” to incorporate the definitions shown in Section 3 of this Practice.

8.2 Appendix A of this Practice has the following examples which supplement the examples in the ASCE Anchorage Design Report:

a. Example A: Column Plate Connection Using Anchorage Design Spreadsheet

b. Example B: Column Plate Connection - Supplementary Tensile Reinforcing

c. Example C: Shear Lug Plate Section Design

d. Example D: Anchorage Design for Column Pedestals Using MathCAD Template

Comment: Example D is Example 1 from ASCE Anchorage Design Report, updated to ACI 318-14.
### Table 1 - Minimum Anchor Dimensions – U.S. Customary Units

(See Figure 1 for dimension locations)

<table>
<thead>
<tr>
<th>ANCHOR ROD DIAMETER</th>
<th>EFFECTIVE CROSS-SECTIONAL AREA OF ANCHOR ROD IN TENSION (Note 3)</th>
<th>HEAVY HEX HEAD/ NUT WIDTH</th>
<th>ANCHOR TYPE 2 THREAD LENGTH AT BOTTOM OF ANCHOR</th>
<th>ASCE ANCHORAGE DESIGN REPORT MINIMUM DIMENSIONS (Note 1)</th>
<th>SLEEVES (See Note 1 (d))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>h_{ef}</td>
<td>EDGE DISTANCE c_{a} (Note 2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12d_a</td>
</tr>
<tr>
<td>d_a</td>
<td>A_{sec,N}</td>
<td>W_h</td>
<td>TB1</td>
<td>TB2</td>
<td>4d_a ≥ 4.5''</td>
</tr>
<tr>
<td>in.</td>
<td>in^2</td>
<td>in.</td>
<td>in.</td>
<td>in.</td>
<td>in.</td>
</tr>
<tr>
<td>5/8</td>
<td>0.226</td>
<td>1.25</td>
<td>1.25</td>
<td>--</td>
<td>7.5</td>
</tr>
<tr>
<td>3/4</td>
<td>0.334</td>
<td>1.44</td>
<td>1.25</td>
<td>2.25</td>
<td>9.0</td>
</tr>
<tr>
<td>7/8</td>
<td>0.462</td>
<td>1.69</td>
<td>1.50</td>
<td>2.50</td>
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<td>1</td>
<td>0.606</td>
<td>1.88</td>
<td>1.75</td>
<td>3.00</td>
<td>12.0</td>
</tr>
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<td>1-1/4</td>
<td>0.969</td>
<td>2.31</td>
<td>2.00</td>
<td>3.50</td>
<td>15.0</td>
</tr>
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<td>1-1/2</td>
<td>1.405</td>
<td>2.75</td>
<td>2.25</td>
<td>4.00</td>
<td>18.0</td>
</tr>
<tr>
<td>1-3/4</td>
<td>1.900</td>
<td>3.19</td>
<td>2.50</td>
<td>4.75</td>
<td>21.0</td>
</tr>
<tr>
<td>2</td>
<td>2.500</td>
<td>3.63</td>
<td>2.75</td>
<td>5.25</td>
<td>24.0</td>
</tr>
<tr>
<td>2-1/4</td>
<td>3.250</td>
<td>4.06</td>
<td>3.00</td>
<td>5.75</td>
<td>27.0</td>
</tr>
<tr>
<td>2-1/2</td>
<td>4.000</td>
<td>4.50</td>
<td>3.50</td>
<td>6.50</td>
<td>30.0</td>
</tr>
<tr>
<td>2-3/4</td>
<td>4.930</td>
<td>4.94</td>
<td>3.75</td>
<td>7.00</td>
<td>33.0</td>
</tr>
<tr>
<td>3</td>
<td>5.970</td>
<td>5.31</td>
<td>4.00</td>
<td>7.75</td>
<td>36.0</td>
</tr>
</tbody>
</table>

**NOTES:**

1. If sleeves are used, the following dimensional modifications apply:
   (a) Embedment should be the greater of 12d_a or (h_s + h'_e)
   (b) Edge distance should be increased by 0.5(d_s - d_a)
   (c) Spacing should be increased by (d_s - d_a)
   (d) Partial length sleeves are not recommended for anchors greater than 1 in. See ASCE Anchorage Design Report, Section 3.2.3.1.

2. For machinery foundations, minimum edge distance from the edge of sleeves is the greater of 6 inches or 4 anchor rod diameters in accordance with PIP REIE686.

3. Effective Cross-Sectional Areas of Anchor Rods in Tension assumes UNC series threads. For cast-in-place anchors, the Effective Cross-Sectional Areas of Anchor Rods in Shear are the same areas as those in tension.

4. The TB2 dimension used for Type 2 anchors "WITH AP" assumes the anchor plate thickness shown in Table 4 for ASTM F1554 Grade 36 anchors in 4,000 psi concrete. If other thickness AP plates are used the TB2 dimension may need to be adjusted.
### Table 1M - Minimum Anchor Dimensions - Metric (SI) Units

(See Figure 1 for dimension locations)

<table>
<thead>
<tr>
<th>ANCHOR ROD DIAMETER</th>
<th>EFFECTIVE CROSS-SECTIONAL AREA OF ANCHOR ROD IN TENSION (Note 3)</th>
<th>HEAVY HEX HEAD/ NUT WIDTH</th>
<th>ANCHOR TYPE 2 THREAD LENGTH AT BOTTOM OF ANCHOR</th>
<th>ASCE ANCHORAGE DESIGN REPORT MINIMUM DIMENSIONS (Note 1)</th>
<th>SLEEVES (See Note 1 (d))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>h_{ef}</td>
<td>EDGE DISTANCE ( c_a ) (Note 2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>( 12d_a )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d_a ( \text{mm} )</td>
<td>A_{sec,N} ( \text{mm}^2 )</td>
<td>W_h ( \text{mm} )</td>
<td>TB1 ( \text{mm} )</td>
<td>TB2 ( \text{mm} )</td>
<td>4d_a ( \geq 114 \text{ mm} )</td>
</tr>
<tr>
<td>16</td>
<td>157</td>
<td>26</td>
<td>30</td>
<td>--</td>
<td>192</td>
</tr>
<tr>
<td>20</td>
<td>245</td>
<td>33</td>
<td>35</td>
<td>--</td>
<td>240</td>
</tr>
<tr>
<td>24</td>
<td>353</td>
<td>40</td>
<td>40</td>
<td>--</td>
<td>288</td>
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<tr>
<td>30</td>
<td>561</td>
<td>49</td>
<td>45</td>
<td>85</td>
<td>360</td>
</tr>
<tr>
<td>36</td>
<td>817</td>
<td>59</td>
<td>50</td>
<td>100</td>
<td>432</td>
</tr>
<tr>
<td>42</td>
<td>1121</td>
<td>68</td>
<td>60</td>
<td>110</td>
<td>504</td>
</tr>
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<td>56</td>
<td>2030</td>
<td>87</td>
<td>75</td>
<td>145</td>
<td>672</td>
</tr>
<tr>
<td>64</td>
<td>2676</td>
<td>97</td>
<td>85</td>
<td>165</td>
<td>768</td>
</tr>
<tr>
<td>72</td>
<td>3440</td>
<td>106</td>
<td>90</td>
<td>185</td>
<td>864</td>
</tr>
</tbody>
</table>

**NOTES:**

1. If sleeves are used, the following dimensional modifications apply:
   - (a) Embedment should be the greater of \( 12d_a \) or \( (h_s + h'_{e}) \)
   - (b) Edge distance should be increased by \( 0.5(d_s - d_a) \)
   - (c) Spacing should be increased by \( (d_s - d_a) \)
   - (d) Partial length sleeves are not recommended for anchors greater than 25 mm. See ASCE Anchorage Design Report, Section 3.2.3.1.

2. For machinery foundations, minimum edge distance from the edge of sleeves is the greater of 150 mm or 4 anchor rod diameters in accordance with PIP REIE686.

3. Effective Cross-Sectional Areas of Anchor Rods in Tension assumes UNC series threads. For cast-in-place anchors, the Effective Cross-Sectional Areas of Anchor Rods in Shear are the same areas as those in tension.

4. The TB2 dimension used for Type 2 anchors “WITH AP” assumes the anchor plate thickness shown in Table 4M for ASTM F1554 Grade 36 Anchors in 28 MPa concrete. If other thickness AP plates are used the TB2 dimension may need to be adjusted.
### Table 2 - Reinforcement Tensile Capacity and Tensile Development Length – U.S Customary Units

<table>
<thead>
<tr>
<th>BAR SIZE</th>
<th>BAR AREA (in²)</th>
<th>BAR CAPACITY</th>
<th>COVER = 1.5 in</th>
<th>COVER = 2 in</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A_r</td>
<td>φ<em>A_r</em>(f_y)</td>
<td>MIN. SPACING (2 * c_b)</td>
<td>MIN. SPACING (2 * c_b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Cover + half bar diam.)</td>
<td>(Cover + half bar diam.)</td>
</tr>
<tr>
<td>#3</td>
<td>0.11</td>
<td>5.94</td>
<td>1.69</td>
<td>3 3/8</td>
</tr>
<tr>
<td>#4</td>
<td>0.20</td>
<td>10.80</td>
<td>1.75</td>
<td>3 1/2</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>16.74</td>
<td>1.81</td>
<td>3 5/8</td>
</tr>
<tr>
<td>#6</td>
<td>0.44</td>
<td>23.76</td>
<td>1.88</td>
<td>3 3/4</td>
</tr>
<tr>
<td>#7</td>
<td>0.60</td>
<td>32.4</td>
<td>1.94</td>
<td>3 7/8</td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>42.66</td>
<td>2.00</td>
<td>4</td>
</tr>
<tr>
<td>#9</td>
<td>1.00</td>
<td>54.00</td>
<td>2.06</td>
<td>4 1/8</td>
</tr>
<tr>
<td>#10</td>
<td>1.27</td>
<td>68.58</td>
<td>2.14</td>
<td>4 1/4</td>
</tr>
<tr>
<td>#11</td>
<td>1.56</td>
<td>84.24</td>
<td>2.21</td>
<td>4 3/8</td>
</tr>
</tbody>
</table>

### Factors for Different Values of f'_c (Note: l_d shall not be less than 12 in.)

<table>
<thead>
<tr>
<th>f'_c (ksi)</th>
<th>DEVELOPMENT LENGTH FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,000</td>
<td>1.15</td>
</tr>
<tr>
<td>4,000</td>
<td>1.00</td>
</tr>
<tr>
<td>5,000</td>
<td>0.89</td>
</tr>
<tr>
<td>6,000</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Development lengths for "TOP" bars and "OTHER" bars are provided. In the typical case where bars that lap with anchors are vertical only, development length for only "OTHER" bars is needed. Development length for "TOP" bars is provided for the rare cases where an anchor is put in horizontally and for information for non-anchor use of rebar.

Development length for lap anchors shall not be less than 12 in. per Sec 12.2.1.1 ACI 318-11, 12.2.3 Min. of 12 in. per Sec 12.2.1 or ACI 318-14, Sec 25.4.2.3, Min. of 12 in. per Sec 25.4.2.1.
**Table 2M - Reinforcement Tensile Capacity and Tensile Development Length – Metric (SI) Units**

<table>
<thead>
<tr>
<th>BAR SIZE</th>
<th>Aᵣ</th>
<th>φ<em>Aᵣ</em>(fᵧ)</th>
<th>cᵦ</th>
<th>MIN. SPACING (2 * cᵦ)</th>
<th>TENSION DEVELOPMENT LENGTH, lₐ</th>
<th>cᵦ</th>
<th>MIN. SPACING (2 * cᵦ)</th>
<th>TENSION DEVELOPMENT LENGTH, lₐ</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mm²</td>
<td>kN</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
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<tr>
<td>#10</td>
<td>71</td>
<td>26.8</td>
<td>45</td>
<td>90</td>
<td>300</td>
<td>55</td>
<td>110</td>
<td>300</td>
</tr>
<tr>
<td>#13</td>
<td>129</td>
<td>48.8</td>
<td>46</td>
<td>95</td>
<td>500</td>
<td>56</td>
<td>115</td>
<td>300</td>
</tr>
<tr>
<td>#16</td>
<td>199</td>
<td>75.2</td>
<td>48</td>
<td>100</td>
<td>700</td>
<td>61</td>
<td>125</td>
<td>650</td>
</tr>
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<td>284</td>
<td>107.4</td>
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<td>100</td>
<td>800</td>
<td>63</td>
<td>125</td>
<td>750</td>
</tr>
<tr>
<td>#22</td>
<td>387</td>
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<td>51</td>
<td>105</td>
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<td>64</td>
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<td>#25</td>
<td>510</td>
<td>192.8</td>
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<td>105</td>
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<td>110</td>
<td>1100</td>
<td>68</td>
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<td>1370</td>
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<td>#32</td>
<td>819</td>
<td>309.6</td>
<td>56</td>
<td>115</td>
<td>1200</td>
<td>68</td>
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<td>58</td>
<td>120</td>
<td>1600</td>
<td>68</td>
<td>140</td>
<td>1370</td>
</tr>
</tbody>
</table>

**FACTORS FOR DIFFERENT VALUES OF f’c**
(Note: lₐ shall not be less than 300 mm)

<table>
<thead>
<tr>
<th>f’c (MPa)</th>
<th>DEVELOPMENT LENGTH FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>1.15</td>
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<tr>
<td>28</td>
<td>1.00</td>
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<tr>
<td>35</td>
<td>0.89</td>
</tr>
<tr>
<td>42</td>
<td>0.82</td>
</tr>
</tbody>
</table>

**BAR SIZE**

<table>
<thead>
<tr>
<th>COVER = 65 mm (and greater)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#10</td>
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<tr>
<td>#13</td>
</tr>
<tr>
<td>#16</td>
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<td>#19</td>
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<td>#22</td>
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<td>#25</td>
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<td>#29</td>
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<tr>
<td>#32</td>
</tr>
<tr>
<td>#36</td>
</tr>
</tbody>
</table>
Table 3 - Hairpin Reinforcement Design and Details – U.S. Customary Units

<table>
<thead>
<tr>
<th>REINFORCEMENT BAR SIZE</th>
<th>REINFORCEMENT BAR CAPACITY</th>
<th>MIN. INSIDE HOOK DIAMETER</th>
<th>1 1/2&quot; TO 2 1/2&quot; COVER</th>
<th>2 1/2&quot; COVER AND GREATER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>180 DEG. HOOK DEV. LENGTH</td>
<td>180 DEG. HOOK DEV. LENGTH</td>
</tr>
<tr>
<td></td>
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<td>l_a</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>HAIRPIN CAPACITY</td>
<td>HAIRPIN CAPACITY</td>
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<tr>
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<td>l_dh</td>
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</tr>
</tbody>
</table>

T (hairpin) = T (hook) x (1+l_a/l_d)

HAIRPIN CAPACITY:
(1) Standard 180° hook capacity = rebar capacity
(2) Capacity of l_a portion of hook = rebar capacity X (l_a/l_d) [l_d > l_a]
(3) Capacity of l_b portion of hook = rebar capacity - capacity of l_a portion
(4) Hairpin capacity = maximum of bar capacity X (1 + l_a/l_d) and bar capacity X 2 X l_d/l_d
   where l_d = bar development length [l_d > l_a]
   (l_d is shown in Table 2 for the corresponding cover and location of the rebar (other or top))
(5) Note that hairpin capacities are greater with less cover. This is because the minimum hairpin length is used. If the hairpin length is increased the capacities increase. Maximum hairpin length is l_d.

FACTORS FOR DIFFERENT VALUES OF f'_c:

<table>
<thead>
<tr>
<th>f'_c (psi)</th>
<th>Development Length Factor (DFL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,000</td>
<td>1.15</td>
</tr>
<tr>
<td>4,000</td>
<td>1.00</td>
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<tr>
<td>5,000</td>
<td>0.89</td>
</tr>
<tr>
<td>6,000</td>
<td>0.82</td>
</tr>
</tbody>
</table>

l_b remains the same
l_dh = l_dh*(DFL)
l_a = l_dh - l_b
### Table 3M - Hairpin Reinforcement Design and Details – Metric (SI) Units

<table>
<thead>
<tr>
<th>REINFORCEMENT BAR SIZE</th>
<th>MIN. INSIDE-HOOK DIAMETER</th>
<th>40 to 65 mm COVER</th>
<th>65 mm COVER AND GREATER</th>
<th>HAIRPIN CAPACITY</th>
<th>HAIRPIN CAPACITY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td>$l_b$</td>
<td></td>
<td>$l_a$</td>
<td>$l_a$</td>
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<td></td>
<td>$l_{dh}$</td>
<td></td>
<td>$l_{dh}$</td>
<td>$l_{dh}$</td>
</tr>
<tr>
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<td>105</td>
<td>190</td>
<td>85</td>
</tr>
<tr>
<td>#13</td>
<td>49</td>
<td>78</td>
<td>117</td>
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<td>131</td>
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<tr>
<td>#16</td>
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<td>129</td>
<td>305</td>
<td>176</td>
</tr>
<tr>
<td>#19</td>
<td>107</td>
<td>114</td>
<td>152</td>
<td>362</td>
<td>210</td>
</tr>
<tr>
<td>#22</td>
<td>150</td>
<td>132</td>
<td>176</td>
<td>419</td>
<td>243</td>
</tr>
<tr>
<td>#25</td>
<td>197</td>
<td>150</td>
<td>200</td>
<td>476</td>
<td>276</td>
</tr>
</tbody>
</table>

### T (hairpin) = T (hook) x (1 + $l_a/l_d$)

- $l_b$ remains the same
- $l_{dh} = l_{dh} \times (DLF)$
- $l_a = l_{dn} - l_b$

**HAIRPIN CAPACITY:**

1. Standard 180° hook capacity = rebar capacity
2. Capacity of $l_a$ portion of hook = rebar capacity X ($l_a/l_d$)  
   [If $l_a > l_d$]
3. Capacity of $l_b$ portion of hook = rebar capacity - capacity of $l_a$ portion
4. Hairpin capacity = maximum of bar capacity X (1 + $l_a/l_d$) and bar capacity X 2 X $l_{dn}/l_d$
   where $l_d$ = bar development length [If $l_d > l_a$]
   ($l_a$ is shown in Table 2M for the corresponding cover and location of the rebar (other or top)]
5. Note that hairpin capacities are greater with less cover. This is because the minimum hairpin length is used. If the hairpin length is increased the capacities increase. Maximum hairpin length is $l_d$.  

### FACTORS FOR DIFFERENT VALUES OF $f'_c$:

<table>
<thead>
<tr>
<th>$f'_c$ (MPa)</th>
<th>Development Length Factor (DLF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>1.15</td>
</tr>
<tr>
<td>28</td>
<td>1.00</td>
</tr>
<tr>
<td>35</td>
<td>0.89</td>
</tr>
<tr>
<td>42</td>
<td>0.82</td>
</tr>
</tbody>
</table>
Anch Rod Diam.  | ASTM F1554 Grade 36  | ASTM F1554 Grade 55  (not ductile over 2 in. diam.)  | ASTM F1554 Grade 105  | ASTM A449  (1/2 thru 1 in. diam.)
---|---|---|---|---
inch  | 4,000 psi | 5,000 psi | 4,000 psi | 5,000 psi | 4,000 psi | 5,000 psi | 4,000 psi | 5,000 psi
---|---|---|---|---|---|---|---|---
1/2  | NR NR NR NR | 1/4  | NR NR | 1 1/4  | 1/4 | 1 1/4 | 1/4 | 1 1/4 | 1/4 | 1 1/4 | 1/4 | 1 1/4 | 1/4
5/8  | NR NR NR NR | 1/4 | NR NR | 1 1/4 | 3/8 | 1 1/2 | 3/8 | 1 1/2 | 3/8 | 1 1/2 | 3/8 | 1 1/2 | 3/8
3/4 | 1 1/4 | 1/4 | 1 1/2 | 1/4 | 1 1/2 | 1/4 | 1 1/2 | 1/4 | 1 1/2 | 1/4 | 1 1/2 | 1/4
7/8 | 1 3/4 | 1/4 | 1 1/2 | 1/4 | 1 3/4 | 1/4 | 1 3/4 | 1/4 | 1 3/4 | 1/4 | 1 3/4 | 1/4
---|---|---|---|---|---|---|---|---|---|---|---|---|---
1  | 1 3/4 | 1/4 | 1 3/4 | 1/4 | 2 | 1/4 | 2 | 1/4 | 2 | 1/4 | 5/8 | 2 1/4 | 5/8 | 2 1/4 | 1/2 | 2 | 3/8
1 1/8 | 2 1/4 | 1/4 | 2 1/4 | 1/4 | 2 1/4 | 3/8 | 2 1/4 | 3/8 | 2 1/4 | 3/8 | 2 1/2 | 5/8 | 2 1/2 | 1/2 | 2 | 1/4 | 3/8
1 1/4 | 2 1/4 | 3/8 | 2 1/4 | 1/4 | 2 1/2 | 3/8 | 2 1/4 | 3/8 | 2 1/4 | 3/8 | 2 1/2 | 3/8 | 2 1/2 | 3/8 | 2 1/2 | 1/2 | 2 | 1/4 | 2 1/2 | 3/8
1 3/8 | 2 1/2 | 3/8 | 2 1/2 | 1/4 | 2 3/4 | 1/4 | 2 1/2 | 3/8 | 3 1/4 | 7/8 | 3 7/8 | 3 | 7/8 | 3 | 5/8 | 2 3/4 | 1/2 | 3
1 1/2 | 2 3/4 | 3/8 | 2 3/4 | 1/4 | 3 | 1/2 | 2 3/4 | 3/8 | 3 1/2 | 1 | 3 1/4 | 7/8 | 3 1/4 | 3/4 | 3 | 5/8 | 5/8
2 | 3 1/2 | 1/2 | 3 1/2 | 3/8 | 4 | 5/8 | 3 1/2 | 1/2 | 4 3/4 | 1 3/8 | 4 1/4 | 1 1/4 | 4 | 3 3/4 | 3 3/4 | 5/8 | 3
2 1/4 | 4 | 1/2 | 4 | 3/8 | -- | -- | -- | -- | 5 1/4 | 15/8 | 4 3/4 | 1 3/8 | 4 1/2 | 7/8 | 4 1/4 | 3/4 | 4
2 1/2 | 4 1/2 | 1/2 | 4 1/2 | 1/2 | -- | -- | -- | -- | 5 3/4 | 13/4 | 5 1/4 | 1 1/2 | 5 | 7/8 | 4 1/2 | 3/4 | 4
2 3/4 | 5 | 5/8 | 5 | 5/8 | -- | -- | -- | -- | 6 1/2 | 2 | 6 | 7/8 | 5 1/2 | 1 | 5 | 3/4 | 4
3 | 5 1/2 | 3/4 | 5 1/4 | 5/8 | -- | -- | -- | -- | 7 | 2 1/4 | 6 1/2 | 2 | 6 | 1 1/8 | 5 1/2 | 7/8 | 3
4 | 7 1/4 | 7/8 | 7 | 3/4 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |

Notes:
All dimensions are in inches
NR = plate not required for ductility
4 inch diameter ASTM A449 anchors are not manufactured.
ASTM F1554 Grade 55 anchors over 2 inches in diameter are not ductile and should not be used where ductility is required.
See Figure 6 for nomenclature.
## Table 4M - Anchor Plates (Metric (SI) Units)

<table>
<thead>
<tr>
<th>Arch Rod Diam. (mm)</th>
<th>28 MPa</th>
<th>35 MPa</th>
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<tbody>
<tr>
<td></td>
<td>PL DIA</td>
<td>THK.</td>
</tr>
<tr>
<td></td>
<td>16</td>
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</tr>
<tr>
<td></td>
<td>20</td>
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<td>64</td>
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<tr>
<td></td>
<td>72</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Arch Rod Diam. (mm)</th>
<th>28 MPa</th>
<th>35 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PL DIA</td>
<td>THK.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>135</td>
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<tr>
<td></td>
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<td>165</td>
</tr>
<tr>
<td></td>
<td></td>
<td>175</td>
</tr>
</tbody>
</table>

**Notes:**
- All dimensions are in mm.
- See Figure 6 for nomenclature.
- All anchor plates are not ductile and should not be used where ductility is required.

### ASTM F1554
- Grade 55: $f_{tu} = 830$ MPa (12 thru 25 mm diam.)
- $f_{tu} = 720$ MPa (25.1 thru 38 mm diam.)
- $f_{tu} = 620$ MPa (38.1 thru 76 mm diam.)

### ASTM F1554 Grade 105
- $f_{tu} = 820$ MPa

### ASTM F1554 Grade 36
- $f_{tu} = 520$ MPa (not ductile over 50 mm diam.)

### ASTM F1554 Grade 36
- $f_{tu} = 400$ MPa
Table 5 – Recommended Anchor Hole Diameters in Base Plates and Minimum Fabricated Washer Sizes (U.S. Customary Units)

<table>
<thead>
<tr>
<th>Anchor Diameter, in.</th>
<th>(1) Minimum Hole Dia., in. (+/- 1/8 in. tol.) (Note 1)</th>
<th>(2) Minimum Hole Dia., in. (+/- 3/8 in. tol.) (Note 2)</th>
<th>(3) Maximum Hole Dia., in. (Note 3)</th>
<th>AISC Manual of Steel Construction Table 14-2 (Note 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8</td>
<td>15/16</td>
<td>1 3/8</td>
<td>15/16</td>
<td>NA</td>
</tr>
<tr>
<td>3/4</td>
<td>1 1/16</td>
<td>1 1/2</td>
<td>1 1/16</td>
<td>1 5/16</td>
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<td>7/8</td>
<td>1 3/16</td>
<td>1 5/8</td>
<td>1 5/16</td>
<td>1 9/16</td>
</tr>
<tr>
<td>1</td>
<td>1 5/16</td>
<td>1 3/4</td>
<td>1 1/2</td>
<td>1 13/16</td>
</tr>
<tr>
<td>1 1/8</td>
<td>1 7/16</td>
<td>1 7/8</td>
<td>1 11/16</td>
<td>NA</td>
</tr>
<tr>
<td>1 1/4</td>
<td>1 9/16</td>
<td>2</td>
<td>1 15/16</td>
<td>2 1/16</td>
</tr>
<tr>
<td>1 1/2</td>
<td>1 13/16</td>
<td>2 1/4</td>
<td>2 5/16</td>
<td>2 5/16</td>
</tr>
<tr>
<td>1 3/4</td>
<td>2 1/16</td>
<td>2 1/2</td>
<td>2 5/8</td>
<td>2 3/4</td>
</tr>
<tr>
<td>2</td>
<td>2 5/16</td>
<td>2 3/4</td>
<td>2 15/16</td>
<td>3 1/4</td>
</tr>
<tr>
<td>2 1/4</td>
<td>2 9/16</td>
<td>3</td>
<td>3 3/16</td>
<td>NA</td>
</tr>
<tr>
<td>2 1/2</td>
<td>2 13/16</td>
<td>3 1/4</td>
<td>3 9/16</td>
<td>3 3/4</td>
</tr>
<tr>
<td>3</td>
<td>3 5/16</td>
<td>3 1/2</td>
<td>4 5/16</td>
<td>NA</td>
</tr>
</tbody>
</table>

Notes:
1. Hole diameters shown in column (1) are based upon the PIP STS03001 anchor placement tolerance of +/- 1/8 in. between any two anchors within an anchor group.
2. Hole diameters shown in column (2) are based upon adding the tolerance between anchors in an anchor group (+/- 1/8 in.) to the tolerance of the center to center spacing between anchor groups (+/- 1/4 in.) for a total tolerance of +/- 3/8 in. between anchors. Note that if this tolerance is required for anchors 1 1/4 inch or smaller (highlighted in yellow), an ASTM A36 fabricated plate washer will have to be added because the maximum hole diameter is larger than can be accommodated by a standard ASTM F436 washer alone.
3. The hole diameters shown in column (3) are the largest hole diameter that can be accommodated by a standard ASTM F436 washer so that the washer will completely cover the hole when the anchor is at the edge of the hole. Hole diameters larger than this will require a fabricated ASTM A36 plate washer in addition to the ASTM F436 washer.
4. Recommendations from AISC Manual of Steel Construction Table 14-2 are included here as a convenience. Values marked NA in this table do not appear in AISC Manual of Steel Construction Table 14-2. Note that for anchors smaller than 1 inch, the AISC maximum hole is smaller than the minimum hole to accommodate a +/- 3/8 in. tolerance.
Table 5M – Recommended Anchor Hole Diameters in Base Plates and Minimum Fabricated Washer Sizes (Metric (SI) Units)

<table>
<thead>
<tr>
<th>Anchor Diameter, mm</th>
<th>(1) Minimum Hole Dia. mm (+/- 3 mm tol.) (Note 1)</th>
<th>(2) Minimum Hole Dia. mm (+/- 9 mm tol.) (Note 2)</th>
<th>(3) Maximum Hole Dia. mm (Note 3)</th>
<th>AISC Manual of Steel Construction Table 14-2 (Note 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. Hole Dia., mm</td>
<td>Min. Washer Size, mm</td>
<td>Min. Washer Thickness, mm</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>24</td>
<td>34</td>
<td>24</td>
<td>NA</td>
</tr>
<tr>
<td>20</td>
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<td>64</td>
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<td>82</td>
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<td>100</td>
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<tr>
<td>72</td>
<td>80</td>
<td>90</td>
<td>102</td>
<td>NA</td>
</tr>
</tbody>
</table>

Notes:
1. Hole diameters shown in column (1) are based upon the PIP STS03001 anchor placement tolerance of +/- 3 mm between any two anchors within an anchor group.
2. Hole diameters shown in column (2) are based upon adding the tolerance between anchors in an anchor group (+/- 3 mm) to the tolerance of the center to center spacing between anchor groups (+/- 6 mm) for a total tolerance of +/- 9 mm between anchors. Note that if this tolerance is required for anchors 36 mm or smaller (highlighted in yellow), an ASTM A36M fabricated plate washer will have to be added because the maximum hole diameter is larger than can be accommodated by a standard ASTM F436M washer alone.
3. The hole diameters shown in column (3) are the largest hole diameter that can be accommodated by a standard ASTM F436M washer so that the washer will completely cover the hole when the anchor is at the edge of the hole. Hole diameters larger than this will require a fabricated ASTM A36M plate washer in addition to the ASTM F436M washer.
4. Recommendations from AISC Manual of Steel Construction Table 14-2 are included here as a convenience. Values have been interpolated from U.S. Customary units in Table 14-2. Values marked NA in this table do not appear in AISC Manual of Steel Construction Table 14-2. Note that the AISC maximum hole for the 20 mm anchor is smaller than the minimum hole to accommodate a +/- 9 mm tolerance.
Table 6 – Chapter, Section, Paragraph, Equation, and Table Numbers in the ASCE Anchorage Design Report when ACI 318-14 or ACI 318M-14 is the Project Standard

<table>
<thead>
<tr>
<th>ASCE Anchorage Design Report Location</th>
<th>Reference in Report Using ACI 318-08</th>
<th>Reference for Projects Using ACI 318-14 or ACI 318M-14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pg. 28, Section 3.2.1, first paragraph</td>
<td>Section D.5.3.4</td>
<td>Section 17.4.3.4</td>
</tr>
<tr>
<td>Pg. 30, Section 3.2.2, next to last paragraph</td>
<td>Section D.8</td>
<td>Section 17.7</td>
</tr>
<tr>
<td>Pg. 35, Section 3.3.4, fourth paragraph</td>
<td>Appendix D Figure RD.6.2.1. (b)</td>
<td>Figure R17.5.2.1b</td>
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<td>Pg. 45, Section 3.5.3.1.3, first paragraph</td>
<td>Section D.5.4.1 (2 places)</td>
<td>Section 17.4.4 (2 places)</td>
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<td>Section D.5.4</td>
<td>Section 17.4.4</td>
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<td>Section D.6.2.9</td>
<td>Section 17.5.2.9</td>
</tr>
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<td>Section D.3.3.5</td>
<td>Section 17.2.3.5</td>
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<tr>
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<td>Section D.3.3.6</td>
<td>Section 17.2.3.6</td>
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<tr>
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<td>Section D.3.3</td>
<td>Section 17.2.3</td>
</tr>
</tbody>
</table>

Pgs. 143 - 148, Example 2, (References are in order of the references in the example.)

| D.4.4 | 17.3.4 |
| D.5.1 | 17.4.1 |
| D.4.4a | 17.3.4a |
| Eq D-3 | (17.4.1.2) |
| D.5.2 | 17.4.2 |
| D.5.2.3 | 17.4.2.3 |
| Eq D-6 | (17.4.2.1c) |
| D.5.2.1 | 17.4.2.1 |
| Eq D-7 | (17.4.2.2a) |
| Eq D-11 | (17.4.2.5b) |
| D.5.2.6 | 17.4.2.6 |
| D.5.2.7 | 17.4.2.7 |
| Eq D-4 | (17.4.2.1a) |
| D.4.4c (Condition B) | 17.3.3c (Condition B) |
| D.5.3 | 17.4.3 |
| Eq D-14 | (17.4.3.1) |
| Eq D-15 | (17.4.3.4) |
| D.4.4c | 17.3.c |
| D.5.4 | 17.4.4 |
| Eq D-17 | (17.4.4.1) |
| D.5.4.1 | 17.4.4.1 |
| D.4.4c | 17.3.c |
| D.4.1.2 | Table 17.3.1.1 |
| D.4.1.1 | 17.3.1.1 |

Pg. 149, Example 3, third line

| D.4.4 (Condition A) | 17.3.3. (Condition A) |
NOTES:
1. Distance between bottom of sleeve and anchor bearing surface, $h_e$, should not be less than $6d_a$ nor 6 inches (150 mm).
2. See Table 1 (U.S. Customary units) or Table 1M (Metric units) for minimum dimensions.
Figure 2 - Concrete Breakout Strength of Anchors in Shear - Octagon "Weak" Anchors

Approximate solution

\[ c_{a1} = \frac{D_o}{2} - \frac{AC}{2} \]

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

Note: Area of octagon = 0.828D²

\[ \pi D_o^2/4 = 0.828D^2 \]

\[ D_o^2 = 0.828D^2(4) \]

\[ D_o = \sqrt{\frac{0.828D^2}{\pi} \left(\frac{4}{\pi}\right)} = \frac{1.03D}{2} \]

Pythagorean Theorem:

\[ c_{a2}^2 + \left(\frac{AC}{2}\right)^2 = \left(\frac{D_o}{2}\right)^2 \]

\[ c_{a2} = \left(\left(\frac{1.03D}{2}\right)^2 - \left(\frac{AC}{2}\right)^2\right)^{1/2} \]

For input into PIP STE05121

Anchorage Design Spreadsheet, available to PIP Members only.

\[ c_{a1} = 1.03D/2 - AC/2 \]

\[ c_{a2}, c_{a4} = \left(\left(\frac{1.03D}{2}\right)^2 - \left(\frac{AC}{2}\right)^2\right)^{1/2} \]

\[ A_{Vc} = 1.5c_{a1}D \]

\[ A_{Vc} \text{ (max)} = n 4.5c_{a1}^2 \]

\[ n = \text{Total number of bolts} = 12 \]

Failure planes overlap each other to go clear across pedestal.

\[ A_{Vc} = 1.5c_{a1}D \quad \text{(Max. } A_{Vc} = nA_{Vco} = n4.5c_{a1}^2) \]
Figure 3 - Concrete Breakout Strength of Anchors in Shear - Octagon "Strong" Anchors

Notes for Figure 3:

c\textsubscript{a1} varies with the number of anchors considered. Only anchors with an edge distance, c\textsubscript{a1}, greater than or equal to the c\textsubscript{a1} for the chosen anchor should be used for resisting shear.

For the case shown above, if the dimension marked c\textsubscript{a1} is chosen, n = 6 anchors. If the dimension marked c\textsubscript{a1} (ALT) is chosen, n = 4 anchors.

<table>
<thead>
<tr>
<th>For input into PIP STE05121 Anchorage Design Spreadsheet, available to PIP Members only</th>
<th>Alternate</th>
</tr>
</thead>
<tbody>
<tr>
<td>c\textsubscript{a1} = As shown above</td>
<td>c\textsubscript{a1} (ALT) = As shown above</td>
</tr>
<tr>
<td>c\textsubscript{a2} = (D-AC)/2</td>
<td>c\textsubscript{a2} (ALT) = (D-Cos(45(^\circ))AC)/2</td>
</tr>
<tr>
<td>A\textsubscript{Vc} = 1.5c\textsubscript{a1} D</td>
<td>A\textsubscript{Vc} = 1.5c\textsubscript{a1}(ALT) D</td>
</tr>
<tr>
<td>A\textsubscript{Vc} (max) = n 4.5c\textsubscript{a1}\textsuperscript{2}</td>
<td>A\textsubscript{Vc} (max) = n 4.5(c\textsubscript{a1}(ALT))\textsuperscript{2}</td>
</tr>
<tr>
<td>n = 6</td>
<td>n = 4</td>
</tr>
</tbody>
</table>
Figure 4 - Pretensioned Anchors for Turbines and Reciprocating Compressors

Notes:
1. Materials:
   - Anchor plate: ASTM A36/A36M
   - Anchor rod: ASTM A36/A36M or ASTM F1554 Grade 36.
   - Nuts: ASTM A563 Grade A heavy hex or ASTM A563M Property Class 9 heavy hex
   - Washer: ASTM F436/F436M
   - Pipe sleeve: ASTM A53/A53M STD Weight Class (Schedule No. 40)
2. Weld should be in accordance with AWS D1.1/D1.1M.
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.

<table>
<thead>
<tr>
<th>ANCHOR ROD DIAMETER</th>
<th>NOMINAL PIPE SLEEVE DIAMETER</th>
<th>ANCHOR PLATE DIAMETER</th>
<th>ANCHOR PLATE THICKNESS (T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.</td>
<td>in.</td>
<td>in.</td>
<td>in.</td>
</tr>
<tr>
<td>3/4</td>
<td>20</td>
<td>1 1/2</td>
<td>3</td>
</tr>
<tr>
<td>7/8</td>
<td>22</td>
<td>2</td>
<td>3 1/2</td>
</tr>
<tr>
<td>1</td>
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<td>3 1/2</td>
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<tr>
<td>1 1/8</td>
<td>28</td>
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<tr>
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<td>3</td>
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<tr>
<td>1 3/4</td>
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<td>64</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>2 3/4</td>
<td>70</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>75</td>
<td>6</td>
<td>9</td>
</tr>
</tbody>
</table>
Figure 5 - Flowchart for Seismic Design of Anchorage – Tension
(Page 1 of 2)

STEP (1): Determine the controlling strength design load combination for the connection.

STEP (2): Is the anchorage part of a structure assigned to Seismic Design Category C, D, E, or F?

YES

STEP (3): Design anchorage for the controlling strength design load combination using \( 0.75N_a \).

NO

STEP (4): Is the tensile component of the seismic force \( > 20\% \) of the total tensile force for the same load combination?

YES

STEP (5): Are anchors post-installed?

YES

STEP (6): Are anchors pre-qualified for seismic in accordance with ACI 355.2 or ACI 355.4?

YES

STEP (7): Choose a qualified post-installed anchor

NO

NO

NO

STEP (8): Does yielding of an attached "yielding mechanism" (e.g. baseplate) occur at a force less than the anchorage capacity?**

YES

STEP (9): Design anchorage for forces that are required to yield the yielding mechanism using \( 0.75N_a \).

NO

NO

Go to STEP 10 on Page 2

* Note that the "yielding mechanism" must yield at a force larger than the controlling load case.
Figure 5 - Flowchart for Seismic Design of Anchorage – Tension
(Page 2 of 2)

1. If anchors are subject to both tension and shear loads acting simultaneously, comply with ACI 318-11 Section D.7 or ACI 318-14 Section 17.6 in the design of the anchors (STEPS 3, 9, 11, and 16).
2. The subscript arf stands for anchor reinforcement.
3. The section and paragraph numbers for metric versions of ACI 318, ACI 318M-11 and ACI 318M-14, are the same as the corresponding U.S. customary versions.
Figure 6 - Flowchart for Seismic Design of Anchorage – Shear
(PAGE 1 OF 2)

STEP (1): Determine the controlling strength design load combination for the connection.

STEP (2): Is the anchorage part of a structure assigned to Seismic Design Category C, D, E, or F?

NO

STEP (3): Design anchorage for the controlling strength design load combination using $\phi V_s$.

YES

STEP (4): Is the shear component of the seismic force > 20% of the total shear force for the same load combination?

NO

YES

STEP (7): Choose a qualified post-installed anchor.

NO

STEP (6): Are anchors pre-qualified for seismic in accordance with ACI 355.2 or ACI 355.4?

YES

STEP (5): Are anchors post-installed?

NO

YES

STEP (8): Does yielding of an attached "yielding mechanism" (e.g. anchor tension) occur at a force less than the anchor shear capacity? *

YES

STEP (9): Design anchorage for forces that are required to yield the yielding mechanism using $0.75 \phi V_s$.

NO

Go to STEP 10 on Page 2

* Note that the "yielding mechanism" must yield at a force larger than the controlling load case.
Figure 6 - Flowchart for Seismic Design of Anchorage – Shear

From STEP (8) on Page 1

STEP (10): Is the connection to be designed with E increased by \( \Omega_b \)?

YES

STEP (11): Design the anchorage with E increased by \( \Omega_b \) using \( \Phi V_n \)

NO

STEP (12): Is rebars used to develop anchor forces per ACI 318-11 D.6.2.8 or ACI 318-14 17.5.2.9?

NO

STEP (13): Use 0.75\( V_{ref} \) for anchor reinforc. design to satisfy concrete breakout requirements

YES

STEP (14): Does pryout (\( V_m \)) control the strength of the anchorage?

NO

STEP (15): Design for controlling load case using \( \Phi V_n \)

YES

1. If failure in breakout (\( V_{e} \)) consider adding anchor reinforcement and going to STEP (13).
2. If failure in pryout (\( V_{p} \)) consider modifying bolt spacing, edge distances, pedestal size, or \( E \).
3. If none of these are practical go to STEP (11) and design for E increased by \( \Omega_b \).

1. Consider modifying bolt spacing, edge distances, pedestal size, or \( E \).
2. If not practical go to Step (11) and design for E increased by \( \Omega_b \).

Notes: 1. If anchors are subject to both tension and shear loads acting simultaneously, comply with ACI 318-11 Section D.7 or ACI 318-14 Section 17.6 in the design of the anchors (STEPS 3, 9, 11, and 15).
2. The subscript arf stands for anchor reinforcement.
3. The section and paragraph numbers for metric versions of ACI 318, ACI 318M-11 and ACI 318M-14, are the same as the corresponding U.S. customary versions.
Figure 7 - Anchor Plates
Appendix A

Examples
Example A - Column Plate Connection Using Anchorage Design Spreadsheet

Base Plate Connection Data
W12 x 45 column
Four anchors on 16" x 16" spacing
Base plate 1 1/2" x 1'-10" x 1'-10" with vertical stiffener plates
Factored base loads (gravity plus wind - maximum uplift condition)
Shear ($V_{ua}$) = 17 kips
Moment ($M_{u}$) = 140 kip-feet
Tension ($N_{ua}$) = 17 kips
Low-seismic area (ductility not required)
$f'_c = 4,000$ psi, ASTM F1554 Grade 36 anchor material

$$\Sigma M_P = 0$$

$$T = \frac{(140 \text{ k-ft x 12 + 17 k x 8.625")}}{(11 + 8 - 2.67)}$$

$$T = 109 \text{ k for 2 anchors}$$

$$P = 109 - 17 = 92 \text{ kips}$$

Resisting friction load ($V_f$) = $m \cdot P$

$$m = 0.55 \text{ (ASCE Anchorage Design Report Figure 3.20)}$$

$$V_f = 0.55 \times 92 = 50 \text{ kips} > 17 \text{ kips}$$
Therefore, anchors are not required to resist shear.

$$X = 2.67$$
(Refer to Blodgett - Design of Welded Structures - Figure 17 [Similar].)
Note: Other theories for determining "X" are equally valid.

By trial and error using the Anchorage Design Spreadsheet, available to PIP Member Companies only, the following is determined.

Nominal Anchor Rod Diameter = 1 1/2"  Anchor Embedment = 21" (12 anchor rod diameters)
Pedestal Size = 5' 0" x 5' 0"  ($c_{a1} = 22", c_{a2} = 22", c_{a3} = 38", c_{a4} = 22", s_{2} = 16", s_{1} = 0")

Because only two anchors resist tension, $s_{1}$ input as 0".

The Anchorage Design Spreadsheet input and output sheets are attached for this condition.

This is a very large pedestal. For a smaller pedestal, supplementary tensile reinforcing may be used to resist the load. See Example B.
Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_Input

<table>
<thead>
<tr>
<th>Company</th>
<th>PIP</th>
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</thead>
<tbody>
<tr>
<td>Project</td>
<td>Example A - Column Plate Connection using Anchor Design Spreadsheet</td>
</tr>
<tr>
<td>Subject</td>
<td></td>
</tr>
<tr>
<td>Name</td>
<td></td>
</tr>
<tr>
<td>Checked by</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td>7/4/2016</td>
</tr>
<tr>
<td>Check Date</td>
<td></td>
</tr>
<tr>
<td>Project #</td>
<td>PIP STE05121</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>1</td>
</tr>
<tr>
<td>Total Sheets</td>
<td>1</td>
</tr>
</tbody>
</table>

**LOADING CONDITIONS**

- Calculations are per ACI 318-14 Chapter 17.
- Use factored loads for $N_{ud}$ and $V_{ud}$ per ACI 318-14 Chapter 5.
- Factored tensile load (kips) = $N_{ud} = 92$
- Factored shear load (kips) = $V_{ud} = 0$
- Is there a built-up grout pad? No

**ANCHOR DATA, EMBEDMENT, AND THICKNESS OF MEMBER**

- Anchor material type = F1554 Gr 35
- Nominal anchor diameter (in.) = 1.1/2
- Note: Calculations limit h eff to <=C(a)(max)/1.5 or if 3 s/d or more are less than 1.5 then maximum of C(a)(max)/1.5 and 1/3s/d or S2 = 14.67 in., see ACI 17.4.2.3
- Thickness of member in which anchor is anchored, (in.) = 2
- Number of anchors in tension = n(tension) = 2
- Number of anchors in shear = n(shear) = 4

**CONCRETE FAILURE AREAS**

- Do you want to manually input the value of $A_{rc}$? Yes
  - $A_{rc} = 333$
  - $A_{rc} = 2040$

**DESIGN CONSIDERATIONS**

- Tensile Ductility required? No
- Seismic Category C, D, E, or F? No
- Specified concrete strength (psi) = f'(c) = 4000
- Cracking modification factor, $f_{c}^{'}$ = 1.4
- Located in region where there isn't cracking at service loads (ft < fr)

**ECCENTRICITY**

- Eccentricity of tensile force on group of tensile anchors (in.):
  - Direction of s (1 = s1, 2 = s2): 0
  - Eccentricity of shear force on group of anchors (in.): 0

**EDGE DISTANCES AND SPACING**

- Edge Distance, in. = 22.0
- Spacing, in. = 38.00
- Edge Distance, in. = 22.0
- Spacing, in. = 16.00

**SUMMARY OF RESULTS**

- Tensile ductility not req'd
- Interaction of Tensile and Shear Forces
- Results
  - Equation 1: $3N_u = 113.7$ kips $\Rightarrow N_u = 92$ kips
  - Equation 2: $3V_u = 69.7$ kips $\Rightarrow V_u = 0$ kips
  - $N_u/(3N_u) + V_u/(3V_u) = 0.81 + 0.00 = 0.81 \leq 1.0$
- OK
- Applicable Interaction Eq. = 1
- 113.7
Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_Output

Anchorage Design Spreadsheet (U.S. Units - ACI 318-14) Revision 0, March 16

<table>
<thead>
<tr>
<th>Company</th>
<th>PIP</th>
<th>Project</th>
<th>Project #</th>
<th>PIP STE05121</th>
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<tbody>
<tr>
<td>Subject</td>
<td>Example A - Column Plate Connection using Anchor Design Spreadsheet</td>
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<td>Name</td>
<td>Date 7/4/2016</td>
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<td>Check Date</td>
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<td></td>
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</table>

BOLT PARAMETERS

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<tr>
<th>Grade</th>
<th>F1554 Gr 36</th>
<th>$f_{ps}$</th>
<th>36 ksi</th>
<th>$A_{le}$</th>
<th>21.00 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>1 1/2 in.</td>
<td>$f_{sta}$</td>
<td>58 ksi</td>
<td>$A_{sh}$</td>
<td>2</td>
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<tr>
<td></td>
<td>1.500 in.</td>
<td></td>
<td></td>
<td>$A_{s}$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$A_{eq}$</td>
<td>3.118 sq. in.</td>
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</table>

LOAD CONDITIONS

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<tr>
<th>Tensile Load, $N_u$</th>
<th>92.0 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Load, $V_u$</td>
<td>0.0 kips</td>
</tr>
</tbody>
</table>

REINFORCEMENT

- Reinforcement NOT designed to carry tensile load |
- Reinforcement NOT designed to carry shear load

DESIGN CONSIDERATIONS

- Ductility NOT req'd for tension
- Concrete Strength, $f' c = 4000$ psi
- Cracking Modification Factor, $\beta_{cr} = 1.4$
- Seismic Category A or B
- No Grout Pad
- Eccentricities $e' = 0.00$ in. $e'' = 0.00$ in.

DESIGN FOR TENSION

<table>
<thead>
<tr>
<th>Steel Strength</th>
<th>$N_u$(tot)</th>
<th>153.6 kips</th>
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</thead>
<tbody>
<tr>
<td>Concrete breakout strength of anchor(s)</td>
<td>$N_{tp}$ or $N_{tdg}$</td>
<td>151.6 kips</td>
</tr>
<tr>
<td>Pullout strength of anchor(s)</td>
<td>$n N_{tp}$</td>
<td>279.4 kips</td>
</tr>
<tr>
<td>Concrete sideface blowout strength of headed anchor(s)</td>
<td>$N_{dp}$ or $N_{sdg}$</td>
<td>(governing)</td>
</tr>
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</table>

DESIGN FOR SHEAR

<table>
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<tr>
<th>Steel Strength</th>
<th>$V_u$(tot)</th>
<th>196.3 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete breakout strength of anchor(s), Perpendicular to edge</td>
<td>$V_{tp}$ or $V_{tdg}$</td>
<td>92.9 kips</td>
</tr>
<tr>
<td>Concrete pryout strength of anchor(s)</td>
<td>$V_{dp}$</td>
<td>303.2 kips</td>
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EDGE DISTANCES, SPACINGS, FAILURE AREAS

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<tr>
<th>Tension</th>
<th>Shear</th>
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<tr>
<td>$C_{t1}$</td>
<td>22.00 in.</td>
</tr>
<tr>
<td>$C_{t2}$</td>
<td>22.00 in.</td>
</tr>
<tr>
<td>$C_{t3}$</td>
<td>38.00 in.</td>
</tr>
<tr>
<td>$C_{t4}$</td>
<td>22.00 in.</td>
</tr>
<tr>
<td>$S_{t1}$</td>
<td>0.00 in.</td>
</tr>
<tr>
<td>$S_{t2}$</td>
<td>18.00 in.</td>
</tr>
</tbody>
</table>

| $A_{t1}$ or $A_{t2}$ | 2640 sq. in. | 3240 sq. in. |

SUMMARY OF RESULTS

<table>
<thead>
<tr>
<th>TENSION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Capacity</td>
</tr>
<tr>
<td>Concrete Capacity</td>
</tr>
</tbody>
</table>

- Tensile ductility not req'd

<table>
<thead>
<tr>
<th>SHEAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Capacity</td>
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<td>Concrete Capacity</td>
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</table>

INTERACTION OF TENSILE AND SHEAR FORCES

<table>
<thead>
<tr>
<th>Steel Capacity</th>
<th>122.7 kips</th>
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</thead>
<tbody>
<tr>
<td>Concrete Capacity</td>
<td>113.7 kips</td>
</tr>
</tbody>
</table>

- $N_u = 92.0$ kips
- $V_u = 0.0$ kips
- $N_u/(0.8N_{tp}) + V_u/(0.8V_{dp}) = 0.81 + 0 = 1.0$

*Multiplied by 0.75 if Seismic Category C, D, E, or F
## Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_Calculations

### Anchorage Design Spreadsheet (U.S. Units - ACI 318-14) Revision 0, March 16

### Calculations

**Selected Bolt:** 1 1/2 in. F1554 Gr 36

- $d_a = 1.500$ in.
- $A_{ss} = 1.410$ sq. in.
- $f_{u} = 36$ ksi
- $n_{(b)} = 2$
- $n_{(sh)} = 4$

**Bolt:**

- $t_{b} = 21.0$ in.
- $A_{sb} = 3.118$ sq. in.
- $f_{u} = 58$ ksi

Note: Figures in parenthesis and in red refer to equations or paragraphs in ACI 318-14, Chapter 17.

### Steel Strength in Tension

- Total capacity of bolts considering eccentricity:
  - $N_{sb}^{(tot)} = \begin{cases} s_{1} &= 0.0 \text{ in.} \\ N_{sb}^{(tot)} &= \text{if } (e'/e_{n} = 0, \text{nt } (N_{sb}), \text{else } 2e/(e + 2e_{n}^{*}) \times N_{sb}^{*} \text{ if } n_{t} = 4, 2, \text{else 1}) = 163.6 \text{ kips} \end{cases}$

### 1. Concrete breakout strength of anchor in tension:

- Use $h_{n} = 14.67$ in. (17.4.2.3) (See Supplementary Calculations below)
- $A_{nc}(\text{calc}) = 2640.0$ sq. in. Use $A_{nc} = 2640.0$ sq. in.
- $A_{nca} = 9h_{n}^{2} = 19360.0$ sq. in. (17.4.2.1c)

- $\beta_{n} = \left[ 1/1 + 2h_{n}^{2}/h_{c}^{2} \right] = 1.00$ (17.4.2.4)
- $c_{c-min} = 22.0$ in. $\beta_{c} = 1.00$ (17.4.2.5a or 17.4.2.5b)
- $\beta_{c,p} = 1.00$ (17.4.2.7)

- Calculated $N_{sb}^{(c)} = (A_{nc}/A_{nca})\beta_{n}\beta_{n}\beta_{c}\beta_{c,p}N_{sb} = 151.6 \text{ kips}$ (17.4.2.1a or 17.4.2.1b)

### 2. Pullout strength of anchor in tension (crushing of concrete under head):

- $\beta_{c} = 1.4$ (17.4.3.6)
- $N_{sb} = 8A_{nc}f_{c}^{0.6} = 99.8 \text{ kips}$ (17.4.3.4)

- Total capacity of bolts considering eccentricity: 279.4 kips

### 3. Concrete side-face blowout strength of headed anchor in tension:

- $c_{u} = 22.0$ in. $c_{u} = 22.0$ in. $c_{t}/c_{u} = 1.00$

- Side-face blowout strength does not apply (if $h_{t} = 2.5 \times \text{min edge distance}$)

- $N_{sb}$ for one anchor = 190(1/A_{nca})\beta_{c}^{0.6}(f_{c})^{0.6} = 0 \text{ (17.4.4.1)}$ For one anchor $N_{sb}$ (modified) = $N_{sb}$[1 + $c_{t}/c_{u}$] = $N_{sb}$[1 + 1] = $N_{sb}$ (17.4.4.1)

- $\beta = 1$ (Normal Weight Concrete) For two anchors $N_{sb}$ (modified) = $N_{sb}$

- Side blowout group effects do not apply.

- $N_{sb} = \left( 1 + \alpha_{t}/\beta_{c} \right) N_{sb} = \text{NA}$ (17.4.4.2) $s_{p} = 16$

- $N_{sb}^{(governing)}$ (anchors at side face) = $N_{sb}$ $s_{p} = 0$

- $N_{sb}$ or $N_{sb}^{(l)}$ (total capacity for all anchors, see note below) = $N_{sb}$

Have increased the overall blow out strength where there are 4 or more anchors to account for the anchors on the far side of the pedestal taking some of the load. Where eccentricity is zero, the far anchors take half the tension so the magnification factor is 2. The magnification factor is 2*(sperm)/(sperm + 2e'N), where e'N is the eccentricity in tension and sperm is the bolt spacing perpendicular to the edge where the blowout would occur. Note that 0 <= e'N <= sperm/2. See note 4 if there are more than 4 bolts.
Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_ Calculations

Anchorage Design Spreadsheet (U.S. Units - ACI 318-14) Revision 0, March 16

Steel Strength of Fastener in Shear:

Capacity of single bolt: \( V_{sa} = 0.6A_{sys}V_{sa}^{*} \) (0.8 if there is a grout pad) = 49.1 kips \( \text{(17.5.1.2b & 17.5.1.3)} \)

Total capacity of bolts considering eccentricity: \( V_{tot} = 2V_{sa}(s + 2e_s) \times V_{sa}^{*} \) (0.8) = 199.3 kips

1. Concrete breakout strength of anchor in shear:

\( A_{hcr} = 3240.0 \) sq in.

\( A_{hcu} = 3240.0 \) sq in.

Use min. \( A_{hcu} = 18200.0 \) sq in. \( \text{(17.5.2.1)} \)

\( A_{hcu} \) (max) = 18200.0 sq in.

\( l_e = \min (d_{sy}, h_u) = 12.0 \) in.

\( c_{ct} = \frac{33.00}{31.2} \) in.

\( V_{d} = \min \left[ \frac{7I_d (d_{sy})^{2}}{7I_d (d_{sy})^{2}} \right] \left[ (c_{ct})^{1.5} \right. \left. \text{and } (\frac{f_{c}}{f_{ct}})^{1.5} \right] = 93.5 \) kips \( \text{(17.5.2.2a and 17.5.2.2b)} \)

\( \frac{f_{at}}{f_{ct}} = 1 \) (17.5.2.7)

\( \frac{f_{at}}{f_{ct}} = 1.0 \) (17.5.2.8)

\( V_{cap} = V_{cap} = (A_{hcu}/N_{cap}) \left[ (c_{at})^{1.5}. \left( (c_{ct})^{1.5} \right) \right] = 92.9 \) kips \( \text{(17.5.2.1a or 17.5.2.1b)} \)

Shear perpendicular to edge < Applies

Shear parallel to edge, \( \beta_{act} = 1.0 \) < NA

2. Concrete pryout strength of anchor in shear:

\( k_{ip} = 2 \) (17.5.3.1)

\( N_{ip} = 205.0 \) kips or \( V_{ip} = k_{ip}N_{ip} = 303.2 \) kips \( \text{(17.5.3.1a or 17.5.3.1b)} \)

Summary of Results:

Tension:

\( \beta_{act} = 0.75 \) for steel \( \beta_{act} = 0.75 \) \( \text{(17.3.3)} \)

Steel capacity = \( \frac{N_{ip}}{V_{ip}}(tot) = 122.7 \) kips

Concrete capacity = \( \frac{N_{ip}}{V_{ip}}(tot) = 113.7 \) kips

Governing mode of concrete failure: Concrete breakout strength of anchor in tension Tension Ductility Req'd? No

Tensile ductility not req'd

Shear:

\( \beta_{act} = 0.65 \) for steel \( \beta_{act} = 0.75 \) \( \text{(17.3.3)} \)

\( \beta_{act} = 0.70 \) for concrete (pryout) \( \text{(17.3.3)} \)

Steel capacity \( \beta_{act} V_{cap} = 122.7 \) kips

Conc. capacity = \( \beta_{act}V_{cap} = 69.7 \) kips

Governing mode of concrete failure: Concrete breakout strength of anchor in shear
## Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_ Calculations

### Anchorage Design Spreadsheet

(U.S. Units - ACI 318-14) Revision 0, March 15

#### Calculations

Interaction of tensile and shear forces:

\[
\begin{align*}
\text{\( \text{\(\Sigma N\)} = 113.7 \text{ kips} \quad \text{\( \Sigma V_N\)} = 69.7 \text{ kips} \)} \\
0.20\Sigma N = 22.7 \text{ kips} & \quad (17.6.2) \\
0.20\Sigma V_N = 13.9 \text{ kips} & \quad (17.6.1) \\
N_{ua} = 92.0 \text{ kips} & \quad V_{ua} = 0.0 \text{ kips} \\
N_{u}(\Sigma N) = 0.81 & \quad V_{u}(\Sigma V_N) = 0.00 \\
N_{u}(\Sigma N) + V_{u}(\Sigma V_N) = 0.81 & \quad \text{Less than 1.0 or 1.2?} \quad \text{<= 1.0} \\
\end{align*}
\]

Potential Interaction Equations:

- Equation 1: \( \Sigma N \gg N_{ua} \) (Table 17.3.1.1)
- Equation 2: \( \Sigma V_N \gg V_{ua} \) (Table 17.3.1.1)
- Equation 3: \( N_{u}(\Sigma N) + V_{u}(\Sigma V_N) \leq 1.2 \) (17.6.3)

Applicable equation = 1  \quad 113.69  

**OK**

### Supplementary Calculations

#### Calculation of \( c_{u,\text{max}} \) and \( h_{uf} \)

- \( h_{ef} \) from input = 21.0
  - \( 1.5h_{ef} = 31.5 \)

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Input</th>
<th>Find ( c_{u,\text{max}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge distance (( c_{u})) =</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Edge distance (( c_{u})) =</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Edge distance (( c_{u})) =</td>
<td>38</td>
<td>0</td>
</tr>
<tr>
<td>Edge distance (( c_{u})) =</td>
<td>22</td>
<td>1</td>
</tr>
<tr>
<td>Total sides &lt; 1.5 ( h_{uf} ) =</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>( c_{u,\text{max}} ) =</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anchor spacing (( s_u)) =</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Anchor spacing (( s_u)) =</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>( c_{u,\text{max}}/1.5 ) =</td>
<td>14.67</td>
<td></td>
</tr>
<tr>
<td>( 1/3*\text{max}(s, \text{ or } s_u) ) =</td>
<td>5.33</td>
<td></td>
</tr>
<tr>
<td>( h_{uf} ) used in calculations =</td>
<td>14.67</td>
<td></td>
</tr>
</tbody>
</table>

* \( c_{u,\text{max}} \) is not applicable if there are fewer than 3 sides within 1.5 \( h_{uf} \) of the anchors. If there are 3 sides within 1.5 \( h_{uf} \) of the anchors, then \( c_{u,\text{max}} \) is the maximum of these 3 edge distances. For pedestals with 4 sides less than 1.5\( h_{uf} \), \( c_{u,\text{max}} \) should be the second largest edge distance less than 1.5\( h_{uf} \). If all 4 edge distances were being used to find \( c_{u,\text{max}} \), it has been determined that the calculated capacity of the concrete would increase by as much as 40% when the fourth edge is brought from outside of 1.5\( h_{uf} \) to inside of 1.5\( h_{uf} \), even though \( h_{uf} \) is being decreased. ACI has been contacted about this problem. The PIP CSA Function Team has agreed that this meets the intent of the code and is conservative.
Anchorage Design Spreadsheet (ACI 318-14) U.S. Customary Units - Example A_ Calculations

For Calculation for $A_{vc}$ and $c_{st}(max)$

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness ($h_1$)</td>
<td>60</td>
</tr>
<tr>
<td>Anchor Spacing ($s_h$)</td>
<td>16</td>
</tr>
<tr>
<td>Edge distance ($c_{d0}$)</td>
<td>28</td>
</tr>
<tr>
<td>Edge distance ($c_{da}$)</td>
<td>20</td>
</tr>
<tr>
<td>$\max(c_{d0},c_{da})/1.5$</td>
<td>18.67</td>
</tr>
<tr>
<td>$s_2/3$</td>
<td>5.33</td>
</tr>
<tr>
<td>$h_1/1.5$</td>
<td>40</td>
</tr>
<tr>
<td>$\max c_{t1}$</td>
<td>40.00</td>
</tr>
<tr>
<td>$c_{st}$ (used)</td>
<td>30.00</td>
</tr>
</tbody>
</table>

Note: The calculated value for $c_{t1}$ is used.
Example B - Column Plate Connection - Supplementary Tensile Reinforcing

Same data as Example A. Use supplementary tensile reinforcing to reduce pedestal size.

Shear \( V_{ua} \) = 17 kips
Moment \( M_{u} \) = 146 kip-feet
Tension \( N_{ua} \) = 17 kips

Per Example A:

T = 116 kips on two anchors
Friction will take shear load.
Nominal anchor rod diameter = 1-1/2"

Assume a 2'-6" x 2'-6" pedestal.

Assume anchors are resisted by three hairpins.

\[ 116 \text{ kips} / 3 = 38.7 \text{ kips} \]

Per Table 3 of PIP STE05121, one #8 hairpin resists 55.5 kips (Other Bars) OK.

\[ l_{dh} \text{ (min)} = 19.0" \text{ per Table 3. Width of hairpin (to center of each leg) = 6 bar diameters minimum inside bend diameter + 1 rebar dia = 7" (See ACI 318-14 Table 25.3.1).} \]

Space hairpins 3" away from each anchor.

Distance from anchor to leg of hairpin = \[ 3^2 + (7.0/2)^2 \] \( ^{0.5} = 4.61" \]

Required \( h_{ef} = C + l_{dh} + 4.61/1.5 \). See Figure 5.

Where \( C = \text{concrete cover} = 2" \)

\[ h_{ef} = 2 + 15.3 + 4.61/1.5 = 20.4" \]

min. \( h_{ef} = 12 d_0 = 12 \times 1.75 = 21" \)

Final Design

Use \( h_{ef} = 21" \)
1 3/4" DIA. ANCHOR (TYP.)

ELEVATION

* USE \( l_{th} \) IF HOOK IS ADDED AT BOTTOM OF HAIRPIN

( NOTE: OTHER REINF. NOT SHOWN FOR CLARITY )

PLAN

\[ h_0 = 21" \]
Example C - Shear Lug Plate Section Design

[Diagram of shear lug plate section design with labels and dimensions]
Example C - Shear Lug Plate Section Design (Continued)

Design a shear lug plate for a 14-in. 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 $f_{ya} = 36$ ksi and $f_c' = 4$ ksi. The contact plane between the grout and base plate is assumed to be 1 in. above the concrete. A 2-ft 0-in. square pedestal is assumed. Ductility is not required.

$$V_{app} = V_{ua} - V_l = 40 - (0.55)(22.5) = 27.6 \text{ kips}$$

Bearing area $= A_{req} = V_{app} / (0.85 * \phi * f_c') = 27.6 \text{ kips} / (0.85 * 0.65 * 4 \text{ ksi}) = 12.5 \text{ in.}^2$

On the basis of base plate size, assume the plate width, $W$, will be 12 in.

Height of plate $= H = A_{req} / W + G = 12.5 \text{ in.}^2 / 12 \text{ in.} + 1 \text{ in.} = 2.04 \text{ in.}$

Use 2.5 in.

Ultimate moment $= M_u = (V_{app} / W) * (G + (H - G) / 2) = (27.6 \text{ kips} / 12 \text{ in.}) * (1 \text{ in.} + (2.5 \text{ in.} - 1 \text{ in.}) / 2) = 4.03 \text{ k-in.} / \text{ in.}$

Thickness $= t = [(4 * M_u) / (\phi * f_{ya})]^{0.5} = [(4 * 4.03 \text{ kip-in.}) / (0.9 * 36 \text{ ksi})]^{0.5} = 0.705 \text{ in.}$

Use 0.75 in.

This 12-in. x 2.5-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 engineer.

Check concrete breakout strength of the shear lug in shear.

Distance from shear lug to edge of concrete $= (24 - 0.75) / 2 = 11.63 \text{ in.}$

$$A_{Vc} = 24 * (1.5 + 11.63) - (12 * 1.5) = 297 \text{ in.}^2$$

$$V_{cb} = A_{Vc} * 4 * \phi * (f_c')^{0.5} = 297 * 4 * 0.85 * (4000)^{0.5} = 63865 \text{ lb} = 63.9 \text{ kips} > 27.6 \text{ kips} \quad \text{OK}$$
Example D - Anchorage Design for Column Pedestals Using MathCAD Template
US Customary Unit Version

Design the anchorage for the steel column located at the top of concrete pedestals shown in Figure A1-1. Anchors resist tension and shear forces. Design is per ACI 318-14, ASCE 7-10, and ASCE Anchorage Design for Petrochemical Facilities, Task Committee on Anchorage Design, dated 2013. This will be referred to as the ASCE Anchorage Design Report in these calculations.

Maximum total factored loads (per Chapter 5 of ACI 318-14 and Section 2.3 of ASCE 7-10):

- Tension: $N_{ua \text{ total}} = 80 \text{ kip}$
- Maximum shear in the X-direction: $V_{ua \text{ total} X} = 20 \text{ kip}$
- Maximum shear in the Y-direction: $V_{ua \text{ total} Y} = 20 \text{ kip}$

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 318-14.
Pedestal Data:

Specified compressive strength of concrete: $f_c = 4000$-ps

Height: Pedestal height = 28-in
Vertical dowel concrete cover: Concrete_cover = 1.5-in
Vertical dowel concrete side cover: Side_cover = 2-in

Note: In many cases, the height of the pedestal is a design constraint.

Cross-section dimensions: $b_1 = 24$-in, $b_2 = 24$-in

Edge Distance: $c_{a1} = 8$-in, $c_{a2} = 8$-in

Anchor Spacing: $s_1 = 8$-in, $s_2 = 8$-in

Anchors:

Specification: ASTM F1554, Gr 36

$\sigma_y = 36,000$-ksi

ASTM F1554, Gr 36 is a ductile steel (See: Table 2.1 of the ASCE Anchorage Design Report).

Therefore: $\sigma_T = 0.75$ (tension loads), $\sigma_Y = 0.65$ (shear loads) (17.3.3)

Anchor Rod Diameter:

$d_a = 1.25$-in

Embedment Depth (12 $d_a$ min):

$h_{er} = 24$-in

Reinforcing bars:

Grade 60 steel: $f_y = 60,000$-ksi

Vertical (longitudinal rebars): $\text{BarNumberVert} = 6$

Shear reinforcement: $\text{BarNumberTie} = 4$

<table>
<thead>
<tr>
<th>SIZE</th>
<th>PERIM.</th>
<th>WT/FT</th>
<th>DIA.</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in</td>
<td>lb/ft</td>
<td>in</td>
<td>in²</td>
</tr>
<tr>
<td>3</td>
<td>1.178</td>
<td>0.376</td>
<td>0.375</td>
<td>0.11</td>
</tr>
<tr>
<td>4</td>
<td>1.571</td>
<td>0.668</td>
<td>0.500</td>
<td>0.20</td>
</tr>
<tr>
<td>5</td>
<td>1.963</td>
<td>1.043</td>
<td>0.625</td>
<td>0.31</td>
</tr>
<tr>
<td>6</td>
<td>2.356</td>
<td>1.502</td>
<td>0.750</td>
<td>0.44</td>
</tr>
<tr>
<td>7</td>
<td>2.749</td>
<td>2.044</td>
<td>0.875</td>
<td>0.60</td>
</tr>
<tr>
<td>8</td>
<td>3.142</td>
<td>2.670</td>
<td>1.000</td>
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</tr>
<tr>
<td>9</td>
<td>3.544</td>
<td>3.400</td>
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<td>1.00</td>
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<tr>
<td>10</td>
<td>3.990</td>
<td>4.303</td>
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<td>1.27</td>
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<tr>
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<td>4.430</td>
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<td>1.56</td>
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<td>14</td>
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<td>7.650</td>
<td>1.693</td>
<td>2.25</td>
</tr>
<tr>
<td>18</td>
<td>7.081</td>
<td>13.600</td>
<td>2.257</td>
<td>4.00</td>
</tr>
</tbody>
</table>

$\text{BarNumberVert} = 6$, $d_{b,\text{vert}} = d_{b,\text{vert}} = 0.75$-in, $A_{\text{vert}} = A_{\text{vert}} = 0.44$-in²

$\text{BarNumberTie} = 4$, $d_{b,\text{tie}} = d_{b,\text{tie}} = 0.5$-in, $A_{\text{tie}} = A_{\text{tie}} = 0.20$-in²
Design assumptions:

1. The tension and the shear forces in the anchors are transferred to the longitudinal rebar and shear reinforcement, respectively, which will be designed as anchor reinforcement. Therefore, the concrete breakout strength in tension and shear (17.4.2 and 17.5.2) are not checked. The concrete pullout strength in shear (17.5.3) 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 (17.5.2.1)

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, the force in each anchor is:

$$N_{ua} = \frac{N_{ua\text{ total}}}{4} = 20 \text{kip}$$

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:

$$V_{ua} = \frac{V_{ua\text{ total}}}{2} = 10 \text{kip}$$

If there is any shear in the X-direction acting simultaneously, it may be added vectorially here.

Anchor Rod Diameter: $d_{av} = 1.25 \text{ in}$

<table>
<thead>
<tr>
<th>$d_a$</th>
<th>$A_{se}$</th>
<th>$A_{se\gamma}$</th>
<th>$n_{eff}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.500</td>
<td>0.142</td>
<td>0.467</td>
<td>6.0</td>
</tr>
<tr>
<td>0.625</td>
<td>0.226</td>
<td>0.671</td>
<td>7.5</td>
</tr>
<tr>
<td>0.750</td>
<td>0.334</td>
<td>0.911</td>
<td>9.0</td>
</tr>
<tr>
<td>0.875</td>
<td>0.462</td>
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<td>10.5</td>
</tr>
<tr>
<td>1.000</td>
<td>0.606</td>
<td>1.501</td>
<td>12.0</td>
</tr>
<tr>
<td>1.125</td>
<td>0.763</td>
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</tr>
<tr>
<td>1.250</td>
<td>0.969</td>
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<tr>
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<td>4.930</td>
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<td>5.970</td>
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<td>11.080</td>
<td>19.923</td>
<td>48.0</td>
</tr>
</tbody>
</table>

$$A_{se\gamma} = A_{se\gamma} = 0.969 \text{ in}^2$$

$$A_{se\gamma} = A_{se\gamma}$$
The steel strength of one anchor in tension: 

\[ N_{sa} = A_{se,N} f_{uta} \]  
\[ \phi_T = 0.75 \]  
\[ \phi_T N_{sa} = 42.151 \text{ kip} \]  
\[ N_{ua} = 20 \text{ kip} \]

Result = "OK" if \( \phi_T N_{sa} \geq N_{ua} \)

Result = "NG" otherwise

The steel strength of one anchor in shear: 

\[ V_{sa} = 0.8 \cdot 0.6 \cdot A_{se,V} f_{uta} \]  
\[ \phi_V = 0.65 \]  
\[ \phi_V V_{sa} = 17.535 \text{ kip} \]  
\[ V_{ua} = 10 \text{ kip} \]

Result = "OK" if \( \phi_V V_{sa} \geq V_{ua} \)

Result = "NG" otherwise

Note: Shear strength of anchors with base plate supported on grout pads shall be multiplied by 0.8 (17.5.1.3).

\[ N_{ua} = 20 \text{ kip} \]  
\[ 0.2 \cdot \phi_T N_{sa} = 8.43 \text{ kip} \]  
\[ V_{ua} = 10 \text{ kip} \]  
\[ 0.2 \cdot \phi_V V_{sa} = 3.51 \text{ kip} \]

Since \( N_{ua} > 0.2 \phi_T N_{sa} \) and \( V_{ua} > 0.2 \phi_V V_{sa} \), check interaction equation (17.6.3):

\[ \text{Ratio} = \frac{N_{ua}}{\phi_T N_{sa}} + \frac{V_{ua}}{\phi_V V_{sa}} = 1.04 \]

Result = "OK" if \( \text{Ratio} \leq 1.2 \)

"NG" otherwise

The recommended minimum embedment depth of the non-sleeve anchor per Section 3.2.2 of the ASCE Anchorage Design Report is 12 anchor rod diameters:

\[ h_{em, min} = 15 \text{ in} \]

\[ h_{ef} = 24 \text{ in} \] (from input)

Check to determine if it is sufficient for the required development length of the vertical reinforcing.
2. Check the pullout resistance of anchor in tension (17.4.3.4)

Section 17.4.3.4 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 ($N_{ua}$).

Use a heavy hex nut (or the anchor head)

Not bearing area: $A_{N_b} - A_{brg} = 2.24$ in$^2$ (from anchor table)

The pullout resistance in tension of a single headed anchor:

Assume concrete cracks: $\psi_p = 1$ (17.4.3.6)

$$N_{pn} = \psi_p \cdot \beta \cdot A_{brg} \cdot f_c = 71.6 \text{ kip} \quad (17.4.3.1 \text{ and } 17.4.3.4)$$

Strength reduction factor for anchor governed by pullout:

$$\phi_p = 0.70 \quad \text{(Condition B per 17.3.3.(c))}$$

Note that even though supplementary reinforcement is provided, Condition B applies for pullout strength.

Therefore:

$$\phi_p N_{pn} = 50.1 \text{ kip}$$

$$N_{ua} = 20 \text{ kip}$$

Result = 

<table>
<thead>
<tr>
<th>Condition</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_p N_{pn} \geq N_{ua}$</td>
<td>OK</td>
</tr>
<tr>
<td>$\phi_p N_{pn} &lt; N_{ua}$</td>
<td>NG</td>
</tr>
</tbody>
</table>

Result = “OK”

3.a. Check side-face blowout resistance of anchor in tension per 17.4.4 (Method 1)

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):

$$\phi_{sb} = 0.75 \quad (17.3.3.(c))$$

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

$$N_{sb} = 160 \cdot c_{a1} \cdot \sqrt{A_{brg}} \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \frac{lbf}{\text{in}^2} = 121.1 \text{ kip} \quad (17.4.4.1, \lambda=1)$$

Corner effect:

<table>
<thead>
<tr>
<th>$c_{a1}$</th>
<th>$c_{a2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>24 in</td>
<td>8 in</td>
</tr>
</tbody>
</table>

Consider corner effect if $c_{a2} < 3 \cdot c_{a1}$

Consider = “Consider corner effect” if $c_{a2} < 3 \cdot c_{a1}$

Consider = “Do not consider corner effect” otherwise
Modified for comer effect:

\[ N_{sbm} = N_{sb} \frac{b_2}{0.25} = 60.5 \text{kip} \quad (17.4.4.1) \]

\[ N_{sa} = 56.2 \text{kip} \]

Result = "OK" if \( N_{sbm} \geq N_{sa} \\
"NG" otherwise

Result = "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.

\[ \phi_{sb} \cdot N_{sbm} = 45.4 \text{kip} \\
N_{ua} = 20 \text{kip} \]

Result = "OK" if \( \phi_{sb} \cdot N_{sbm} > N_{ua} \\
"NG" otherwise

Result = "OK"

Close spacing effect:

\[ s_1 < 6 \cdot c_{a1} \rightarrow \text{Consider close spacing} \]

\[ s_1 = 8 \text{ in} \]

\[ 6 \cdot c_{a1} = 48 \text{ in} \]

Consider = "Consider close spacing" if \( s_1 < 6 \cdot c_{a1} \\
"Do not consider spacing" otherwise

Consider = "Consider close spacing"

\[ N_{sbg} = \left( 1 + \frac{s_1}{6 \cdot c_{a1}} \right) N_{sb} \quad (17.4.4.2) \\
N_{sbg} = 141.3 \text{ kip} \]

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

\[ \phi_{sb} \cdot N_{sbg} = 105.9 \text{ kip} \\
2 \cdot N_{ua} = 40 \text{ kip} \]

Result = "OK" if \( \phi_{sb} \cdot N_{sbg} > 2 \cdot N_{ua} \\
"NG" otherwise

Result = "OK"
3.b. Check side-face blowout resistance of anchor in tension per Section 3.5.3.1.4 of the ASCE Anchorage Design Report (Method 2)

If the pedestal fails using Section 3.a (Method 1) and increasing the size of the pedestal is not feasible, then additional ties can be provided and the pedestal can be designed using the Stut-and-Tie Method (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 harpins should be designed to carry \( F \). Since the majority of action occurs at the bearing surface of the anchor head, it is recommended that anchor reinforcement be placed as closely as possible to the bearing surface of the anchor head.

![Diagram](image)

**Figure A1-3**

Bearing pressure at the anchor head:

\[
 p = \frac{N_{ua}}{A_{bg}} \quad p = 8.94 \text{ ksi}
\]

Section 3.5.3.1.3 of the ASCE Anchorage Design Report (some simplifications of the formulas have been made):

\[
\alpha = \max \left( 0.25, 0.1 \cdot \frac{p}{f_c} + 0.037 \cdot \frac{p}{f_c} \right) \quad \alpha = 0.25
\]

\[ N_{ua} = 20 \text{ kip} \]

\[ F = \alpha \cdot N_{ua} \]

\[ F = 5 \text{ kip} \]

Assuming the ties will be fully-developed, the required number of layers of #4 ties to resist \( F \) is:

For anchor reinforcement design: \( \phi_s = 0.75 \) \hspace{1cm} (17.4.2.9)

\[
 n_{\text{layers}} = \frac{F}{\phi_s (A_{te} f_y)} \quad n_{\text{layers}} = 0.566
\]
This is the number of layers of ties 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 $N_{\text{ua total}}$ in the pedestal (i.e., Section 4.1 of this example problem).

4. Transfer of Anchor Load to Vertical Rebars

4.1 Amount of vertical reinforcing steel

It is prudent and a good practice to place anchor reinforcement as close as practical to the anchor (See Section 3.5.3.1.1 of the ASCE Anchorage Design Report). In order to be considered effective for resisting anchor tension, vertical reinforcing steel must be located within a distance from the edge of the anchor head or bearing plate of:

$$d_{\text{max}} = \frac{h_{\text{ef}}}{3} = 8 \text{ in}$$

The number of pedestal vertical rebars that are effective for resisting anchor tension (See Figure A1-2) is:

$$N_{\text{ef}} = 3$$

Determine the required number of vertical rebar to resist $N_{\text{ua}}$:

*Note: Ductile design is not considered (low seismic risk)*

$$N_{\text{ua}} = 20 \text{ kip} \quad \text{Bar Number Vert} = 6 \quad \text{(from input)} \quad A_{\text{vert}} = 0.44 \text{ in}^2$$

For anchor reinforcement design:

$$d_{\text{max}} = 0.75 \quad \text{(17.4.2.9)}$$

$$n_{\text{required}} = \frac{N_{\text{ua}}}{\phi_s f_y A_{\text{vert}}} = 1.01$$

Provided number of effective bars (from Fig A1-2)

$$n_{\text{provided}} = 3$$
Note: Often for a new design, vertical reinforcing is designed to develop 1.2 times the nominal steel strength of the anchor in order to provide for potential loading in excess of current design loading and/or to provide the capacity for a ductile design in the future.

ALL REBARS THAT ARE LOCATED LESS THAN $d_{min}$ FROM THE EDGE OF THE ANCHOR HEAD CAN BE EFFECTIVE FOR RESISTING ANCHOR TENSION

Figure A1-2

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, $l_d$ ($\geq 12$ in), is determined based on Sections 25.4.2.2 and 25.4.2.4 of ACI 318-14 as follows:

- Bar location factor: $\psi_l = 1$ (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:

\[
I_d = \left( 1 - \frac{\frac{f_y}{\text{psi}}}{25.4} \right) \left( \frac{\psi_b \cdot \psi_e}{\frac{f_c}{\text{psi}}} \right) d_{\text{vert}} \quad I_d = 28.5 \text{ in}
\]

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

\[
\text{Concrete cover} = 1.5 \text{ in} \quad d_{\text{actual}} = 6.95 \text{ in} \quad \text{(see Figure A1-2, input by engineer)}
\]

\[
I_d_{\text{available}} = h_{\text{ef}} - \text{Concrete cover} - d_{\text{actual}} \cdot \tan(35^\circ) \quad I_d_{\text{available}} = 17.6 \text{ in}
\]

However, since the provided number of effective rebar is significantly more than the required number of rebars (per Section 4.1 above) and for low seismic risks, \( I_d \) can be reduced using the excess reinforcement factor per 25.4.10 (but cannot be less than 12 in. per 25.4.2.1).

\[
I_d_{\text{reduced, calc}} = I_d \left( \frac{n_{\text{required}}}{n_{\text{provided}}} \right) = 9.54 \text{ in}
\]

\[
I_d_{\text{reduced, min}} = 12 \text{ in}
\]

\[
I_d_{\text{reduced}} = \max(I_d_{\text{reduced, calc}}, I_d_{\text{reduced, min}}) = 12 \text{ in}
\]

Result = “OK” if \( I_d_{\text{available}} \geq I_d_{\text{reduced}} \)  
Result = “NG” otherwise

5. Design of shear reinforcement

5. a. Use the concept of concrete breakout cone (Method 1)

Calculate the available development length \( I_{da} \):

Based on geometry of pedestal, see Figure A1-4 (engineer inputs these values):

Layer A: \( I_{da A L} = 13.1 \text{ in} \quad I_{da A R} = 7.9 \text{ in} \)
Layer B: \( I_{da B L} = 11 \text{ in} \quad I_{da B R} = 10 \text{ in} \)

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

\[
\psi_e = 1
\]

\[
d_{\text{b, tie}} = 0.5 \text{ in}
\]

\[
I_{dh} = \left[ 0.02 \cdot \psi_e \left( \frac{f_y}{\text{psi}} \right) \right] \frac{f_c}{\sqrt{\text{psi}}} d_{\text{b, tie}} = 9.5 \text{ in} \quad \text{(Section 25.4.3.1 of ACI 318-14)}
\]
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.

$$l_{da\text{-min}} = \min(l_{da\ A\ L}, l_{da\ A\ R}, l_{da\ B\ L}, l_{da\ B\ R}) = 7.9\ \text{in}$$

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

$$f_s = \frac{l_{da\text{-min}}}{l_{da}} \cdot f_y = 50\ \text{ksi}$$

Number of tie legs crossing the failure plane (see Figure A1-4):

$$n_{\text{legs\ provd}} = 6$$

Number of tie legs required:

$$V_{ua\ total\ X} = 20\ \text{kip} \quad A_{se} = 0.196\ \text{in}^2 \quad \phi_s = 0.75$$

$$n_{\text{legs\ reqd}} = \frac{V_{ua\ total\ X}}{\phi_s f_s A_{tie}} = 2.7$$

Result = "OK" if $n_{\text{legs\ provd}} \geq n_{\text{legs\ reqd}}$

"NG" otherwise

Result = "OK"
5. b. Use the concept of Strut-and-Tie Method approach (Section 3.5.3.2.2 of the ASCE Anchorage Design Report) (Method 2).

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.

![Force distribution in the truss model for V=10 kip (per bolt):](image1)

![Force distribution in the truss model after dividing by φ=0.75 (Section 22.2.1(g) of the ACI 318-14; φ for the strut-and-tie model is 0.75):](image2)

In this example, the Strut-and-Tie Model that requires internal ties (i.e., T2) is used. As explained in Section 3.5.3.2.2 of the ASCE Anchorage Design 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., T1) 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 23.2.7 of ACI 318-14).

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

The truss model and member forces are shown in Figure A1-5.
5.3.3 Check strength of bearing

Assume concrete strength for checking the strength of bearing and compression struts: $f_{oe} = 0.85 f_c$

5.3.3.a Bearing of the anchor

Bearing area:

$A_{bearing, anch} = 8 \cdot d_a \cdot d_b$

$A_{bearing, anch} = 12.5 \text{ in}^2$

Strength:

$f_{oe} = 0.85 \cdot (f_c)$

$f_{oe} = 3400\text{ psi} > \frac{13.3\text{kip}}{A_{bearing, anch}} = 1064 \text{ psi} \quad \text{OK!}$

5.3.3.b Bearing of the reinforcing bars

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

The clear distance between the nodes B and D, $l_{BD}$:

$l_{BD} = \sqrt{(5.625 \text{ in})^2 + (4 \text{ in})^2} - \frac{d_a}{2} - \frac{d_{b, vert}}{2}$

$l_{BD} = 5.902 \text{ in}$

Bearing area:

$A_{bearing, rebar} = (8 \cdot d_a + 1.5 \cdot l_{BD} - \text{Concrete cover}) \cdot d_{b, vert}$

$A_{bearing, rebar} = 13 \text{ in}^2$

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 23.9.2 of ACI 318-14 is conservatively used:

$\beta_n = 0.6$

Strength:

$f_{oe} = 0.85 \cdot \beta_n \cdot (f_c)$

$f_{oe} = 2040 \text{ psi} > \frac{9.56\text{kip}}{A_{bearing, rebar}} = 735 \text{ psi} \quad \text{OK!}$

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

![Diagram showing force distribution in the truss model](Image)

*Figure A1-6*
5.0.4. Check strength of struts

Since it is assumed that the strength of strut is the same as the bearing strength ($f_{oe} = 0.85 \beta_r f_c'$) 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 strength at the anchor and rebar are OK, the strength of struts does not need to be checked.

5.0.5. Select tie reinforcement

Assumptions (See Section 3.5.3.2.2 of the ASCE Anchorage Design Report):

1. Only the top most two layers of ties (within 5" of pedestal as required by Section 10.7.6.1.6 of ACI 318-14), 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 tie $T$ is:

$$T_1 = A_{oe} 20 \text{ kips} \quad T_1 = 3.9 \text{ kips}$$

$$T_{hook} = T_1$$

---

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 $T_{hook}$:

Total resistance:  

$$R_{tot\_ab} = A_{tie} \cdot f_y + T_{hook}$$

$$R_{tot\_ab} = 15.7 \text{ kip} \quad > \quad 5.54 \text{ kip} \quad \text{OK !}$$

Tie c (see Figure A1-6):

Tie c is resisted by a hairpin.

Diameter of hairpin:  

$$d_{hairpin} = 0.5\text{in}$$

Yield stress of hairpin:  

$$f_{y\_hairpin} = 60 \text{ ksi}$$

Check the stress that can be developed in the hairpin:

Check available length of the hairpin:  

$$l_{dha} = 24\text{in} - 2\text{-Side\_cover} - 2d_{b\_tie}$$

$$l_{dha} = 19\text{ in}$$

Required straight development length for a fully developed hairpin:

Bar location factor:  

$$\lambda_b = 1.3$$

Coating factor:  

$$\lambda_c = 1$$  

(for uncoated bar)

Concrete density factor:  

$$\lambda_v = 1$$  

(for normal concrete)

For # 6 and smaller bars, use:  

$$l_{d\_hairpin} = \left[ \frac{f_y(\psi_b \psi_e)}{25 \lambda_v f_c} \right] d_{hairpin}$$

$$l_{d\_hairpin} = 24.7\text{ in}$$  

cannot be less than 12 in per Section 25.4.2.1 of ACI 318-14.

The stress that can be developed in the hairpin:

$$f_{s\_hairpin} = \frac{l_{dha}}{l_{d\_hairpin}} f_{y\_hairpin}$$

$$f_{s\_hairpin} = 46.154 \text{ ksi}$$

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

Total resistance:  

$$R_{tot\_c} = 2A_{s\_hairpin} f_{s\_hairpin}$$  

(Note: 2 legs per hairpin)

$$R_{tot\_c} = 18.125 \text{ kip} \quad > \quad 15.6 \text{ kip} \quad \text{OK !}$$
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 (17.7.1): \( d_{\text{min, untorqued}} - 4d_3 = 5 \text{ in} < \min (c_1, s_2) \)  OK!

II. Minimum edge distance (17.7.2):

For untorqued cast-in anchors, the minimum edge distances shall be based on minimum cover requirements for reinforcement specified in Section 20.6.1 of ACI 318-14.

\[ c_{\text{min, untorqued}} = \text{cover} = 1.5 \text{ in} < \min (c_{a1}, c_{a2}) \]  OK!
Notes for Anchorage Design for Column Pedestals: MathCAD Template

The following assumptions are used for this template:

- New pedestal design
- Untorqued, cast-in anchors
- No sleeve is used
- Low seismic risk non-ductile design
- Tension force is distributed equally among all anchors
- Washers are welded to the base plate to transfer the load to all anchors. However, to be conservative, it is assumed that only the two “strong” anchors take the shear load.
- The shears in the X-direction and Y-direction do not act simultaneously.

The template covers various options.

- For the check for side-face blowout, the user can check side-face blowout per ACI 318-14 Section 17.4.4 where the check is done via concrete resistance (Section 3.1 of the template); or by the strut-and-tie-method (STM) per Section 3.5.3.1.4 of the ASCE Anchorage Design Report where additional rebar ties are used to resist the blowout (Section 3.2 of the template).
- For shear reinforcement design, the user can use the concept of a concrete breakout cone, (Section 5.1 of the template); or the STM concept presented in Section 3.5.3.2.2 of the ASCE Anchorage Design Report (Section 5.2 of the template).

The template is meant to be revised to be used for other situations not covered by the assumptions in Section 1 or for using a set option for side-face blowout or shear reinforcement design as described in Section 2.

This template can also be used for ductile design, however extensive revisions will need to be made. If there is a great need by more than one company, a new template can be made to do ductile design.

Users who revise this template to cover specific situations that are not covered by this template and that may have general interest for other structural engineers are invited to send their revised template to the PIP CSA Function Team Leader. These templates will be considered for inclusion in the PIP Implementation Resource Center (IRC) on the PIP website.