Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

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1.0 INTRODUCTION

Anchorage to concrete Concrete Capacity Design (CCD) Method was first introduced in ACI 318-02 and ACI 349-01 Appendix D, followed by CSA A23.3-04 Annex D. Anchorage design provisions in ACI 318-08 and ACI 349-06 Appendix D, CSA A23.3-04 Annex D are similar except that ACI 349-06 imposes a more severe penalty on non-ductile anchor design (ACI 349-06 D3.6.3) and also ACI 349-06 provides additional provisions for shear transfer using friction and shear lugs.

Since ACI 318-02 the ACI has released ACI 318-05, ACI 318-08, and recently ACI 318-11. In ACI 318-08 the definition for Anchor Reinforcement is introduced, and the strength of Anchor Reinforcement used to preclude concrete breakout in tension and in shear is codified (ACI 318-08 D.5.2.9 and D.6.2.9.), guidance for detailing the Anchor Reinforcement is given in ACI 318-08 RD.5.2.9 and RD.6.2.9.

Since CSA A23.3-04 CSA has released several updates to catch up ACI’s revisions on anchorage design, with the latest CSA A23.3-04 (R2010, Reaffirmed 2010) partially incorporated Anchor Reinforcement (CSA A23.3-04 R2010 D.7.2.9). It’s expected that the same Anchor Reinforcement provisions as ACI 318-08 will be amended in the next revision of CSA A23.3-04 update.

This technical writing includes a series of design examples covering mainly the anchorage design of grouped anchors and studs, in both ACI 318-08 and CSA A23.3-04 R2010 code. The design examples are categorized in Anchor Bolt and Anchor Stud, with Anchor Reinforcement and without Anchor Reinforcement, with moment presence and without moment presence.

Anchor Bolt and Anchor Stud

The main difference between anchor bolt and anchor stud is the way how they attach to the base plate. For anchor bolt normally the anchor bolt holes on base plate are much bigger than anchor bolt diameter due to cast-in anchor bolt construction tolerance, while the anchor stud is rigidly welded to the base plate. This different approach of attachment will cause the difference on shear transfer mechanism during anchorage design (ACI 318-08 RD.6.2.1(b)).

Anchor Reinforcement and Supplementary Reinforcement

In all concrete failure modes, the tensile and shear concrete breakout strengths are most of the time the lowest strengths among all concrete failure modes. The concrete breakout strength limits the anchor design strength and make anchor bolt design not practical in many applications such as concrete pedestal, which has limited edge distances surrounding anchor bolts.

In ACI 318-08 the definition for Anchor Reinforcement is introduced, and the strength of Anchor Reinforcement used to preclude concrete breakout in tension and in shear is codified (ACI 318-08 D.5.2.9 and D.6.2.9.), guidance for detailing the Anchor Reinforcement is given in ACI 318-08 RD.5.2.9 and RD.6.2.9. The use of Anchor Reinforcement in many times is the only choice to make a practical anchor bolt design in applications such as concrete pedestal.
The use of supplementary reinforcement is similar to the anchor reinforcement, but it isn’t specifically designed to transfer loads. If supplementary reinforcement is used, the concrete strength reduction factor \( \phi \) is increased by 7% from 0.70 to 0.75, which is not that significant in terms of increasing concrete breakout strength.
**Anchor Ductility**

When an anchor’s overall design strength, for both tension and shear, is equal to the design strength of anchor rod steel element, and all potential concrete failure modes have design strengths greater than the anchor rod steel element design strength, this anchor design is considered as ductile anchor design.

Anchor’s ductility is its own characteristic related to anchor rod material, embedment depth, anchor bolt spacing and edge distances etc, and has nothing to do with the applied loadings. If high strength anchor rod material is used, it would be more difficult to achieve the ductile design as deeper embedment depth, larger edge distances are required for concrete failure modes design strengths to surpass anchor rod material design strength. The high strength anchor bolt material shall only be used when it’s necessary, such as for anchorages required pre-tensioned or subjected to dynamic impact load in cold temperature environment (A320 Grade L7). In most cases the anchorage design won’t benefit from the high strength bolt material as the concrete failure modes will govern, and the use of high strength bolt will make the anchor ductile design almost impossible.
For anchorage design in moderate to high seismic zone (ACI 318-08 SDC C and CSA A23.3-04 R2010  $I_{EF}S_a(0.2) >= 0.35$) ductile anchor design is mandatory as specified in ACI 318-08 D.3.3.4 and CSA A23.3-04 R2010 D.4.3.6.

For anchorage design in low seismic zone (ACI 318-08 SDC < C and CSA A23.3-04 R2010  $I_{EF}S_a(0.2) < 0.35$), the non-ductile anchor design is permitted, but when calculating anchor bolt force distribution, the plastic analysis approach is not permitted for non-ductile anchor as specified in ACI 318-08 D.3.1 and CSA A23.3-04 R2010 D.4.1.
2.0 DESIGN EXAMPLES

Example 01: Anchor Bolt + Anchor Reinf + Tension & Shear + ACI 318-08 Code

N_u = 20 kips (Tension) V_u = 25 kips
Concrete f_c' = 4 ksi  Rebar f_y = 60 ksi
Pedestal size 16" x 16"
Anchor bolt F1554 Grade 36 1.0" dia  Hex Head h_ref = 55"  h_a = 60"
Seismic design category >= C
Anchor reinforcement Tension → 8-No 8 ver. bar
Shear → 2-layer, 4-leg No 4 hor. bar

Provide built-up grout pad
ANCHOR BOLT DESIGN Combined Tension and Shear

Anchor bolt design based on
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D
PIP STE05121 Anchor Bolt Design Guide-2006

Assumptions
1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per
   ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft
7. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear

Anchor Bolt Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored tension for design</td>
<td>( N_u = 20.0 )</td>
<td>kips</td>
</tr>
<tr>
<td>Factored shear</td>
<td>( V_u = 25.0 )</td>
<td>kips</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>( f'c = 4.0 )</td>
<td>ksi</td>
</tr>
<tr>
<td>Anchor bolt material</td>
<td>F1554 Grade 36</td>
<td></td>
</tr>
<tr>
<td>Anchor tensile strength</td>
<td>( f_{utu} = 58 )</td>
<td>ksi</td>
</tr>
<tr>
<td>Anchor bolt diameter</td>
<td>( d_a = 1 )</td>
<td>in</td>
</tr>
<tr>
<td>Bolt sleeve diameter</td>
<td>( d_s = 3.0 )</td>
<td>in</td>
</tr>
<tr>
<td>Bolt sleeve height</td>
<td>( h_s = 10.0 )</td>
<td>in</td>
</tr>
<tr>
<td>Anchor bolt embedment depth</td>
<td>( h_{ef} = 55.0 )</td>
<td>in</td>
</tr>
<tr>
<td>Pedestal height</td>
<td>h = 60.0</td>
<td>in</td>
</tr>
<tr>
<td>Pedestal width</td>
<td>( b_c = 16.0 )</td>
<td>in</td>
</tr>
<tr>
<td>Pedestal depth</td>
<td>( d_c = 16.0 )</td>
<td>in</td>
</tr>
</tbody>
</table>

Diagram:
- Ver. Reinf For Tension
- Hor. Ties For Shear - 4 Legs
- Hor. Ties For Shear - 2 Legs
To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5\(h_{ef}\) from the outmost anchor's centerline. In this design 0.5\(h_{ef}\) value is limited to 8 in.

\[
0.5h_{ef} = 8.0 \text{ [in]}
\]

No of ver. rebar that are effective for resisting anchor tension

Ver. bar size No. 8 \(\downarrow\) 1.000 [in] dia single bar area \(A_s = 0.79\) [in\(^2\)]

To be considered effective for resisting anchor shear, hor. reinft shall be located within \(\min(0.5c_1, 0.3c_2)\) from the outmost anchor's centerline

\[
\min(0.5c_1, 0.3c_2) = 1.5 \text{ [in]}
\]

No of tie leg that are effective to resist anchor shear

No of tie layer that are effective to resist anchor shear

Hor. tie bar size No. 4 \(\downarrow\) 0.500 [in] dia single bar area \(A_s = 0.20\) [in\(^2\)]

For anchor reinft shear breakout strength calc suggest

Rebar yield strength \(f_y = 60\) [ksi] 60 = 414 [MPa]

No of bolt carrying tension \(n_t = 4\)
No of bolt carrying shear \(n_s = 4\)

For side-face blowout check use

No of bolt along width edge \(n_{bw} = 2\)
No of bolt along depth edge \(n_{bd} = 2\)

Anchor head type = Hex

Anchor effective cross sect area \(A_{se} = 0.606\) [in\(^2\)]
Bearing area of head \(A_{org} = 1.163\) [in\(^2\)]

Bolt 1/8" (3mm) corrosion allowance = No ?
Provide shear key ? = No ?
Seismic design category >= C = Yes ?
Provide built-up grout pad ? = Yes ?

Strength reduction factors

Anchor reinforcement \(\phi_s = 0.75\)
Anchor rod - ductile steel \(\phi_{ts} = 0.75\) \(\phi_{vs} = 0.65\)
Concrete - condition A \(\phi_{tc} = 0.75\) \(\phi_{vc} = 0.75\)
### CONCLUSION

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>Description</th>
<th>Ratio</th>
<th>OK</th>
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</thead>
<tbody>
<tr>
<td>Abchor Rod Embedment, Spacing and Edge Distance</td>
<td>OK ACI 318-08</td>
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<tr>
<td>Min Rquired Anchor Reinft. Development Length</td>
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<td>Overall</td>
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<td>Anchor Reinft Tensile Breakout Resistance ratio = 0.09 OK</td>
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</tr>
<tr>
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<td>Anchor Pullout Resistance ratio = 0.26 OK</td>
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<td>Strut Bearing Strength ratio = 0.59 OK</td>
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</tr>
<tr>
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<td>Tie Reinforcement ratio = 0.46 OK</td>
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<tr>
<td></td>
<td>Conc. Pryout Not Govern When hef &gt;= 12d_b</td>
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<tr>
<td>Tension Shear Interaction</td>
<td>ratio = 0.70 OK</td>
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<tr>
<td>Ductility</td>
<td>Tension Non-ductile</td>
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<td>Seismic Design Requirement</td>
<td></td>
<td>NG ACI 318-08 D.3.3.4</td>
</tr>
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<td>Anchor Rod Tensile Resistance ratio = 0.19 &gt; N_u</td>
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</tr>
<tr>
<td></td>
<td>Anchor Reinft Tensile Breakout Resistance</td>
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<tr>
<td></td>
<td>Min tension development length l_d =</td>
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<tr>
<td></td>
<td>Actual development length l_a =</td>
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<td>&gt; 12.0 OK</td>
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<tr>
<td></td>
<td>Anchor Rod Shear Resistance</td>
<td></td>
<td></td>
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<td></td>
<td>Anchor Reinft Shear Breakout Resistance</td>
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<td>Anchor Pullout Resistance</td>
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<td></td>
<td>Side Blowout Resistance</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic design strength reduction</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Calculation

#### Anchor Rod Tensile Resistance

\[
\phi_{ts} N_{sa} = \phi_{ts} n_t A_{se} f_{uta} = 105.4 \text{ [kips]} \quad D.5.1.2 (D-3)
\]

\[
\text{ratio} = 0.19 > N_u \quad \text{OK}
\]

#### Anchor Reinft Tensile Breakout Resistance

Min tension development length

\[
l_d = 47.4 \text{ [in]} \quad 12.2.1, 12.2.2, 12.2.4
\]

Actual development length

\[
l_a = h_{id} - c (2 \text{ in}) - 8 \text{ in} \times \tan 35 = 47.4 \text{ [in]}
\]

\[
> 12.0 \quad \text{OK} \quad 12.2.1
\]

#### Seismic design strength reduction

\[
N_{sa} = \phi_{ts} \times f_{u} \times n_t \times A_{se} \times (l_u / l_d, \text{if } l_a < l_d)
\]

\[
= 284.2 \text{ [kips]} \quad 12.2.5
\]

\[
\text{Seismic design strength reduction} = x 0.75 \quad \text{applicable} = 213.1 \text{ [kips]} \quad D.3.3.3
\]

\[
\text{ratio} = 0.09 > N_u \quad \text{OK}
\]
Anchor Pullout Resistance

Single bolt pullout resistance

\[
N_p = 8 A_{db} f' c
\]

\[
N_{cp} = \phi_{tc} N_p = \phi_{tc} n_t \psi c_p N_p
\]

Seismic design strength reduction

\[
\text{ratio} = 0.75 \quad \text{applicable} = 78.2 \quad [\text{kips}]
\]

\[
\psi_{c,p} = 1 \quad \text{for cracked conc}
\]

\[
\phi_{tc} = 0.70 \quad \text{pullout strength is always Condition B}
\]

Side Blowout Resistance

**Failure Along Pedestal Width Edge**

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal width edge

\[
N_{bw} = N_u x n_{bw} / n_t = 10.0 \quad [\text{kips}]
\]

\[
c = \min (c_1, c_3) = 5.0 \quad [\text{in}]
\]

Check if side blowout applicable

\[
h_{bf} = 55.0 \quad [\text{in}]
\]

\[
> 2.5c \quad \text{side bowout is applicable}
\]

Check if edge anchors work as a group or work individually

\[
s_{22} = 6.0 \quad [\text{in}] \quad s = s_2 = 6.0 \quad [\text{in}]
\]

Single anchor SB resistance

\[
\phi_{tc} N_{sb} = 40.9 \quad [\text{kips}]
\]

Multiple anchors SB resistance

\[
\phi_{tc} N_{sbg,w} = \begin{cases} 
(1 + s/6c) \times \phi_{tc} N_{sb} & \text{work as a group - applicable} \\
N_{bw} \times \phi_{tc} N_{sb} \times [1+(c_2 or c_4)/c] / 4 & \text{work individually - not applicable}
\end{cases}
\]

Seismic design strength reduction

\[
\text{ratio} = 0.75 \quad \text{applicable} = 36.8 \quad [\text{kips}]
\]

**Failure Along Pedestal Depth Edge**

Tensile load carried by anchors close to edge which may cause side-face blowout

along pedestal depth edge

\[
N_{bd} = N_u x n_{bd} / n_t = 10.0 \quad [\text{kips}]
\]

\[
c = \min (c_2, c_4) = 5.0 \quad [\text{in}]
\]

Check if side blowout applicable

\[
h_{bf} = 55.0 \quad [\text{in}]
\]

\[
> 2.5c \quad \text{side bowout is applicable}
\]

Check if edge anchors work as a group or work individually

\[
s_{11} = 6.0 \quad [\text{in}] \quad s = s_1 = 6.0 \quad [\text{in}]
\]

Single anchor SB resistance

\[
\phi_{tc} N_{sb} = 40.9 \quad [\text{kips}]
\]

Multiple anchors SB resistance

\[
\phi_{tc} N_{sbg,d} = \begin{cases} 
(1 + s/6c) \times \phi_{tc} N_{sb} & \text{work as a group - applicable} \\
N_{bd} \times \phi_{tc} N_{sb} \times [1+(c_1 or c_3)/c] / 4 & \text{work individually - not applicable}
\end{cases}
\]

Seismic design strength reduction

\[
\text{ratio} = 0.75 \quad \text{applicable} = 36.8 \quad [\text{kips}]
\]

Group side blowout resistance

\[
\phi_{tc} N_{sbg} = \phi_{tc} \min \left( \frac{N_{sbg,w}}{n_{bw}}, \frac{N_{sbg,d}}{n_{bd}} \right) = 73.7 \quad [\text{kips}]
\]

Govern Tensile Resistance

\[
N_r = \phi_{tc} \min (N_{sb}, N_{sb}, N_{cp}, N_{sbg}) = 73.7 \quad [\text{kips}]
\]
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear

\[ V_{sa} = V_{s,n} A_{sa} f_{ta} = 54.8 \text{ [kips]} \]

Resistance

Reduction due to built-up grout pads = 0.8, applicable = 43.9 [kips]

\[ \frac{\text{ratio}}{V_u} = 0.57 > \text{OK} \]

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

STM strength reduction factor \( \phi_s = 0.75 \)

\[ V_u = 2.250 \text{ [in]} \]

\[ d_s = 2.250 \text{ [in]} \]

\[ \theta = 45 \text{ [in]} \]

\[ d_t = 3.182 \text{ [in]} \]

Strut compression force

\[ C_s = 0.5 \frac{V_u}{\sin \theta} = 17.7 \text{ [kips]} \]

\[ f_{cc} = 0.85 f_c = 3.4 \text{ [ksi]} \]

\[ I_s = \min(8d_a, h_{ef}) = 8.0 \text{ [in]} \]

\[ A_{org} = l_s x d_a = 8.0 \text{ [in}^2] \]

\[ C_r = n_s x \phi_u x f_{oa} x A_{org} = 81.6 \text{ [kips]} \]

\[ > V_u \quad \text{OK} \]

* Bearing of anchor bolt

Anchor bearing length

\[ l_s = \min(8d_a, h_{ef}) = 8.0 \text{ [in]} \]

Anchor bearing area

\[ A_{org} = l_s x d_a = 8.0 \text{ [in}^2] \]

Anchor bearing resistance

\[ C_r = n_s x \phi_u x f_{oa} x A_{org} = 81.6 \text{ [kips]} \]

\[ > V_u \quad \text{OK} \]

* Bearing of ver reinft bar

Ver bar bearing area

\[ A_{org} = (l_s + 1.5 x d_t - d_s - d_o) x d_o = 11.8 \text{ [in}^2] \]

Ver bar bearing resistance

\[ C_r = \phi_u x f_{oa} x A_{org} = 30.0 \text{ [kips]} \]

\[ \frac{\text{ratio}}{C_s} = 0.59 > \text{OK} \]
Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength \( f_y \). Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook \( T_h \)
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar \( n = n_{leg} \times n_{lay} \) = 8

Pull out resistance at hook
\[
T_h = \phi_{t,c} 0.9 f_y e_h d_a = 3.0 \text{ [kips]} \quad \text{ACI 318-08}
\]
\[
e_h = 4.5 \text{ in} \quad d_a = 2.250 \text{ [in]}
\]

Single tie bar tension resistance
\[
T_r = \phi_s f_y A_s = 9.0 \text{ [kips]}
\]

Total tie bar tension resistance
\[
V_{rb} = 1.0 \times n \times T_r = 72.0 \text{ [kips]}
\]

Seismic design strength reduction
\[
\text{ratio} = 0.46 \quad > V_u \quad \text{OK}
\]

Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with \( h_{ef} \geq 12d_a \), the pryout failure will not govern

\[
12d_a = 12.0 \quad \text{[in]} \quad h_{ef} = 55.0 \quad \text{[in]} \quad > 12d_a \quad \text{OK}
\]

Govern Shear Resistance
\[
V_r = \min ( \phi_{v,s} V_{sa}, V_{rb} ) = 43.9 \text{ [kips]}
\]

Tension Shear Interaction

Check if \( N_u > 0.2 \phi N_n \) and \( V_u > 0.2 \phi V_n \)

\[
N_u / \phi N_n + V_u / \phi V_n = 0.84 \quad \text{D.7.3 (D-32)}
\]
\[
\text{ratio} = 0.70 \quad < 1.2 \quad \text{OK}
\]

Ductility Tension
\[
\phi_{t,s} N_{sa} = 105.4 \quad \text{[kips]}
\]
\[
> \min \{ N_{rb}, \phi_{t,c} ( N_{psn}, N_{shq} ) \} = 73.7 \quad \text{[kips]}
\]

Ductility Shear
\[
\phi_{v,s} V_{sa} = 43.9 \quad \text{[kips]}
\]
\[
< V_{rb} = 54.0 \quad \text{[kips]}
\]
Example 02: Anchor Bolt + Anchor Reinf + Tension & Shear + CSA A23.3-04 Code

\[ N_t = 89 \text{ kN (Tension) } \quad V_u = 111.2 \text{ kN} \]

Concrete \( f'_c = 27.6 \text{ MPa} \)
Rebar \( f_y = 414 \text{ MPa} \)

Pedestal size \( 406\text{mm} \times 406\text{mm} \)

Anchor bolt \( F1554 \text{ Grade 36 1.0" dia Hex Head } \)
\( h_{ef} = 1397\text{mm} \)
\( h_a = 1524\text{mm} \)

Seismic design \( I = F_a S_a(0.2) \geq 0.35 \)

Anchor reinforcement
- Tension \( \rightarrow 8-25\text{M ver. bar} \)
- Shear \( \rightarrow 2\)-layer, 4-leg 15\text{M hor. bar} \)

Provide built-up grout pad
ANCHOR BOLT DESIGN

Combined Tension and Shear

Anchor bolt design based on CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D and ACI 318-08 Metric Building Code Requirements for Structural Concrete and Commentary ACI318 M-08.

Assumptions
1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Anchor reinforcing strength is used to replace concrete tension/shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9
4. For tie reinforcing, only the top most 2 or 3 layers of ties (50mm from TOC and 2x75mm after) are effective
5. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcing
6. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear

Input Data

Factored tension for design

\[ N_u = 89.0 \text{ kN} = 20.0 \text{ kips} \]

Factored shear

\[ V_u = 111.2 \text{ kN} = 25.0 \text{ kips} \]

Factored shear for design

\[ V_u = 111.2 \text{ kN} \]

Concrete strength

\[ f'_c = 28 \text{ MPa} = 4.0 \text{ ksi} \]

Anchor bolt material

F1554 Grade 36

Anchor tensile strength

\[ f_{uta} = 58 \text{ ksi} \]

Anchor bolt diameter

\[ d_a = 1 \text{ in} = 25.4 \text{ mm} \]

Bolt sleeve diameter

\[ d_s = 76 \text{ mm} \]

Bolt sleeve height

\[ h_s = 254 \text{ mm} \]

Anchor bolt embedment depth

\[ h_{ef} = 1397 \text{ mm} \]

Pedestal height

\[ h = 1524 \text{ mm} \]

Pedestal width

\[ b_c = 406 \text{ mm} \]

Pedestal depth

\[ d_c = 406 \text{ mm} \]
<table>
<thead>
<tr>
<th>Bolt edge distance $c_1$</th>
<th>$c_1 = 127$ [mm]</th>
<th>114</th>
<th>OK</th>
<th>ACI318 M-08</th>
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<td>Page A-1 Table 1</td>
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<td>$s_2 = 152$ [mm]</td>
<td>102</td>
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</table>

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5$h_{ef}$ from the outmost anchor's centerline. In this design 0.5$h_{ef}$ value is limited to 200mm.

$$0.5h_{ef} = 200$$ [mm]

No of ver. rebar that are effective for resisting anchor tension $n_{v} = 8$

Ver. bar size $d_{v} = 25$ [mm] single bar area $A_{v} = 500$ [mm$^2$]

To be considered effective for resisting anchor shear, hor. reinft shall be located within min(0.5$c_1$, 0.3$c_2$) from the outmost anchor's centerline.

$$\text{min}(0.5c_1, 0.3c_2) = 38$$ [mm]

No of tie leg that are effective to resist anchor shear $n_{leg} = 4$

No of tie layer that are effective to resist anchor shear $n_{lay} = 2$

Hor. bar size $d_{h} = 15$ [mm] single bar area $A_{h} = 200$ [mm$^2$]

For anchor reinft shear breakout strength calc $100\%$ hor. tie bars develop full yield strength $\checkmark$?

Rebar yield strength $f_{y} = 414$ [MPa] 400 = 60.0 [ksi]

No of bolt carrying tension $n_{t} = 4$

No of bolt carrying shear $n_{s} = 4$

For side-face blowout check use

No of bolt along width edge $n_{bw} = 2$

No of bolt along depth edge $n_{bd} = 2$

Anchor head type $= \text{Hex}$

Bearing area of head $A_{ao} = 391$ [mm$^2$]

Bearing area of custom head $A_{hcg} = 750$ [mm$^2$] not applicable

Bolt 1/8" (3mm) corrosion allowance $= \text{No}$

Provide shear key $= \text{Yes}$

Seismic region where $I_{EF}\leq S_{h}(0.2)=0.35$ = A23.3-04 (R2010)

Provide built-up grout pad $= \text{Yes}$

Strength reduction factors

Anchor reinforcement factor $\phi_{as} = 0.75$

Steel anchor resistance factor $\phi_{s} = 0.85$

Concrete resistance factor $\phi_{c} = 0.65$

Resistance modification factors

Anchor rod - ductile steel $R_{ts} = 0.80$

Concrete - condition A $R_{tc} = 1.15$

$R_{ts} = 0.75$ D.5.4(a)

$R_{tc} = 1.15$ D.5.4(c)
CONCLUSION

Anchor Rod Embedment, Spacing and Edge Distance
Min Required Anchor Reinforcement Development Length
Overall

Tension
Anchor Rod Tensile Resistance
Anchor Reinforcement Tensile Breakout Resistance
Anchor Pullout Resistance
Side Blowout Resistance

Shear
Anchor Rod Shear Resistance
Anchor Reinforcement Shear Breakout Resistance
Strut Bearing Strength
Tie Reinforcement

Tension Shear Interaction

Ductility

Seismic Design Requirement

Caculation

Anchor Rod Tensile Resistance

Anchor Reinforcement Tensile Breakout Resistance
Min tension development length for ver. 25M bar
Actual development length

Seismic design strength reduction

LeFaSa(0.2)>0.35, A23.3-04 D.4.3.7 or D.4.3.8 must be satisfied for non-ductile design

Code Reference

A23.3-04 (R2010)
## Anchor Pullout Resistance

**Single bolt pullout resistance**

\[
N_{pr} = 8 \, A_{bol} \, f_y \, f_c' \, R_{t,c} = 107.7 \, [\text{kN}] 
\]

\[
N_{cpr} = n_t \, \psi_{c,p} \, N_{pr} = 430.7 \, [\text{kN}] 
\]

Seismic design strength reduction

\[
\text{ratio} = 0.28 \, > \, N_u \quad \text{OK} 
\]

\[
\psi_{c,p} = 1 \, \text{for cracked conc} 
\]

\[
R_{t,c} = 1.00 \quad \text{pullout strength is always Condition B} 
\]

## Code Reference

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<td>D.6.3.6</td>
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<tr>
<td>D.5.4(c)</td>
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</table>

## Side Blowout Resistance

### Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[
N_{bw} = N_u \times n_{bw} / n_t = 44.5 \, [\text{kN}] 
\]

\[
c = \min (c_1, c_3) = 127 \, [\text{mm}] 
\]

Check if side blowout applicable

\[
h_{bf} = 1397 \, [\text{mm}] 
\]

\[
> 2.5c \quad \text{side bowout is applicable} 
\]

Check if edge anchors work as a group or work individually

\[
s_{22} = 152 \, [\text{mm}] 
\]

\[
s = s_2 = 152 \, [\text{mm}] 
\]

Single anchor SB resistance

\[
N_{sbw} = 13.3c \sqrt{A_{bol} \, f_y} \sqrt{f_c'} \, R_{t,c} = 181.7 \, [\text{kN}] 
\]

Multiple anchors SB resistance

\[
N_{sbgr,w} = (1+s/6c) \times N_{sbw} = 217.9 \, [\text{kN}] 
\]

Work individually - not applicable

\[
= n_{bw} \times N_{sbw} \times [1+(c_1 or c_3) / c] / 4 = 0.0 \, [\text{kN}] 
\]

Seismic design strength reduction

\[
\text{ratio} = 0.27 \, > \, N_{bw} \quad \text{OK} 
\]

### Failure Along Pedestal Depth Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[
N_{bd} = N_u \times n_{bd} / n_t = 44.5 \, [\text{kN}] 
\]

\[
c = \min (c_2, c_4) = 127 \, [\text{mm}] 
\]

Check if side blowout applicable

\[
h_{bf} = 1397 \, [\text{mm}] 
\]

\[
> 2.5c \quad \text{side bowout is applicable} 
\]

Check if edge anchors work as a group or work individually

\[
s_{11} = 152 \, [\text{mm}] 
\]

\[
s = s_1 = 152 \, [\text{mm}] 
\]

Single anchor SB resistance

\[
N_{sbw} = 13.3c \sqrt{A_{bol} \, f_y} \sqrt{f_c'} \, R_{t,c} = 181.7 \, [\text{kN}] 
\]

Multiple anchors SB resistance

\[
N_{sbgr,d} = (1+s/6c) \times N_{sbw} = 217.9 \, [\text{kN}] 
\]

Work individually - not applicable

\[
= n_{bd} \times N_{sbw} \times [1+(c_1 or c_3) / c] / 4 = 0.0 \, [\text{kN}] 
\]

Seismic design strength reduction

\[
\text{ratio} = 0.27 \, > \, N_{bw} \quad \text{OK} 
\]

Group side blowout resistance

\[
N_{sbgr} = \min \left( \frac{N_{sbw} \times n_t \times N_{sbw} \times n_t}{n_{bw} \times n_{bw}} \right) = 326.9 \, [\text{kN}] 
\]

## Govern Tensile Resistance

\[
N_r = \min (N_{sr}, N_{sbw}, N_{cpr}, N_{sbgr}) = 323.1 \, [\text{kN}] 
\]
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear

\[ V_{sr} = n_s A_{se} f_y 0.6 f_{uta} R_{v,s} \]

Resistance

Reduction due to built-up grout pads = x 0.8, applicable

\[ V_u = 239.2 \text{ [kN]} \]

\[ \text{Ratio} = 0.58 \]

Code Reference

A23.3-04 (R2010)

D.7.1.2 (b) (D-21)

Anchor Reinft Shear Breakout Resistance

ACI318 M-08

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

STM strength reduction factor \( \phi_m = 0.75 \)

\[ \theta = 45 \]

\[ d_t = 81 \text{ [mm]} \]

Strut-and-Tie model geometry

\[ d_v = 57 \text{ [mm]} \]

\[ d_h = 57 \text{ [mm]} \]

Strut compression force

\[ C_s = 0.5 V_u / \sin \theta \]

\[ C_s = 78.6 \text{ [kN]} \]

Strut Bearing Strength

Strut compressive strength

\[ f_{oa} = 0.85 f_c \]

\[ f_{oa} = 23.5 \text{ [MPa]} \]

ACI318 M-08

\[ f_c = 23.5 \text{ [MPa]} \]

D.6.2.2

Bearing of anchor bolt

Anchor bearing length

\[ l_b = \min(8d_h, h_{st}) \]

\[ l_b = 203 \text{ [mm]} \]

Anchor bearing area

\[ A_{org} = l_b \times d_v \]

\[ A_{org} = 5161 \text{ [mm}^2\text{]} \]

Anchor bearing resistance

\[ C_r = n_b \times \phi_{ub} \times f_{oa} \times A_{org} \]

\[ C_r = 363.3 \text{ [kN]} \]

\[ C_r > V_u \]

ACI318 M-08

Bearing of ver reinft bar

Ver bar bearing area

\[ A_{org} = (l_b + 1.5 \times d_v - d_v/2 - d_v/2) \times d_v \]

\[ A_{org} = 7473 \text{ [mm}^2\text{]} \]

Ver bar bearing resistance

\[ C_r = \phi_{ub} \times f_{oa} \times A_{org} \]

\[ C_r = 131.5 \text{ [kN]} \]

\[ \text{Ratio} = 0.60 \]

\[ C_s > C_v \]
Tie Reinforcement

* For tie reinf, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength $f_y$. Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook $T_h$
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar $n = n_{leg} \times n_{lay} = 8$

Pull out resistance at hook $T_h = 0.9 \phi_b f'c e_h d_b R_{tc} = 16.3$ [kN] D.6.3.5 (D-17)

$e_h = 4.5 \ mm$ $d_b = 68 \ mm$

Single tie bar tension resistance $T_t = \phi_a f_y A_s = 62.1$ [kN]

Total tie bar tension resistance $V_{br} = 1.0 \times n \times T_t = 496.8$ [kN]

Seismic design strength reduction $= x 0.75 \ applicable = 372.6$ [kN] D.4.3.5

ratio $= 0.30 > V_u \ OK$

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} \geq 12 d_a$, the pryout failure will not govern.

$12d_a = 305 \ [mm] \ h'_{ef} = 1397 \ [mm] > 12d_a \ OK$

$12d_a = 305 \ [mm] \ h'_{ef} = 1397 \ [mm] > 12d_a \ OK$

Anchor Rod on Conc Bearing

$B_r = n_s x 1.4 x \phi_b x \min(8d_a, h'_{ef}) x d_a x f'c = 518.5$ [kN] 25.3.3.2

ratio $= 0.21 > V_u \ OK$

Govern Shear Resistance $V_r = \min (V_{br}, B_r) = 191.4$ [kN] A23.3-04 (R2010)

Tension Shear Interaction

Check if $N_r > 0.2 N_t$ and $V_r > 0.2 V_t$

Yes $N_r/N_t + V_r/V_t = 0.86$ D.8.2 & D.8.3

ratio $= 0.71 < 1.2 \ OK$

Ductility Tension

$N_{sr} = 425.3 \ [kN] > \min (N_{br}, N_{spr}, N_{sbpr}) = 323.1$ [kN] Non-ductile

Ductility Shear

$V_{sr} = 191.4$ [kN] $< \min (V_{br}, B_r) = 372.6$ [kN] Ductile
Example 03: Anchor Bolt + Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code

\[ M_u = 35 \text{ kip-ft} \quad N_u = 10 \text{ kips (Compression)} \quad V_u = 25 \text{ kips} \]

Concrete \( f'_c = 4 \text{ ksi} \) \quad Rebar \( f_y = 60 \text{ ksi} \)

Pedestal size \( 26'' \times 26'' \)

Anchor bolt \( F1554 \text{ Grade 36} \quad 1.25'' \text{ dia} \quad \text{Hex Head} \quad h_{ef} = 55'' \quad h_a = 60'' \)

Seismic design category < C

Anchor reinforcement Tension \( \rightarrow 2\)-No 8 ver. bar

Shear \( \rightarrow 2\)-layer, 2-leg No 4 hor. bar

Provide built-up grout pad
ANCHOR BOLT DESIGN  Combined Tension, Shear and Moment

Anchor bolt design based on
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D
PIP STE05121 Anchor Bolt Design Guide-2006

Assumptions
1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per
ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2” from TOC and 2x3” after) are effective
6. Strut-and-Tie model is used to anlyze the shear transfer and to design the required tie reinft
7. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis
and there is no redistribution of the forces between highly stressed and less stressed anchors
8. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output
9. Shear carried by only half of total anchor bolts due to oversized holes in column base plate

Anchor Bolt Data

Factored moment $M_u = 35.0$ [kip-ft] = 47.5 [kNm]
Factored tension /compression $N_u = -10.0$ [kips] in compression = -44.5 [kN]
Factored shear $V_u = 25.0$ [kips] = 111.2 [kN]
Factored shear for design $V_u = 25.0$ [kips] $V_u = 0$ if shear key is provided

| 2 BOLT LINE | 3 BOLT LINE | 4 BOLT LINE |
Code Reference

No of bolt line for resisting moment = 2 Bolt Line
No of bolt along outermost bolt line = 2

Outermost bolt line spacing $s_1$ = 16.0 [in] 5.0
Outermost bolt line spacing $s_2$ = 16.0 [in] 5.0
Internal bolt line spacing $s_{b1}$ = 10.5 [in] 5.0
Internal bolt line spacing $s_{b2}$ = 0.0 [in] 5.0

Column depth $d$ = 12.7 [in]
Concrete strength $f'_c$ = 4.0 [ksi] = 27.6 [MPa]
Anchor bolt material = F1554 Grade 36
Anchor tensile strength $f_{uta}$ = 58 [ksi] = 400 [MPa] ACI 318-08
Anchor is ductile steel element D.1

Anchor bolt diameter $d_a$ = 1.25 [in] = 31.8 [mm] PIP STE05121
Bolt sleeve diameter $d_s$ = 3.0 [in]
Bolt sleeve height $h_s$ = 10.0 [in]

Anchor bolt embedment depth $h_{ef}$ = 55.0 [in] 15.0 OK Page A -1 Table 1
Pedestal height $h$ = 60.0 [in] 58.0 OK
Pedestal width $b_c$ = 26.0 [in]
Pedestal depth $d_c$ = 26.0 [in]
Bolt edge distance $c_1$ = 5.0 [in] 5.0 OK Page A -1 Table 1
Bolt edge distance $c_2$ = 5.0 [in] 5.0 OK
Bolt edge distance $c_3$ = 5.0 [in] 5.0 OK
Bolt edge distance $c_4$ = 5.0 [in] 5.0 OK

Ver. Reinft For Tension  Hor. Ties For Shear - 4 Legs  Hor. Ties For Shear - 2 Legs
To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5\(h_{ef}\) from the outmost anchor's centerline. In this design 0.5\(h_{ef}\) value is limited to 8 in.

\[0.5h_{ef} = 8.0 \text{ [in]}\]

No of ver. rebar that are effective for resisting anchor tension

- No. = 2

Ver. bar size No. 1.000 \([\text{in}]\) dia single bar area \(A_s = 0.79 \text{ [in}^2\) \]

To be considered effective for resisting anchor shear, hor. reinft shall be located within \(\min(0.5c_1, 0.3c_2)\) from the outmost anchor's centerline

\[\min(0.5c_1, 0.3c_2) = 1.5 \text{ [in]}\]

No of tie leg that are effective to resist anchor shear

- \(n_{\text{leg}} = 2\)

No of tie layer that are effective to resist anchor shear

- \(n_{\text{lay}} = 2\)

Hor. tie bar size No. 0.500 \([\text{in}]\) dia single bar area \(A_s = 0.20 \text{ [in}^2\) \]

For anchor reinft shear breakout strength calc

- 100% hor. tie bars develop full yield strength

Rebar yield strength \(f_y = 60 \text{ [ksi]}\)

Total no of anchor bolt \(n = 4\)

No of bolt carrying tension \(n_t = 2\)

No of bolt carrying shear \(n_s = 2\)

For side-face blowout check use

- \(n_{\text{bw}} = 2\)

Anchor head type Hex

Anchor effective cross sect area \(A_{\text{se}} = 0.969 \text{ [in}^2\) \]

Bearing area of head \(A_{\text{avg}} = 1.817 \text{ [in}^2\) \]

- not applicable

Bolt 1/8" (3mm) corrosion allowance

- No

Provide shear key

- No

Seismic design category \(\geq C\)

- No

Provide built-up grout pad

- Yes
### Strength reduction factors

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<td>Anchor rod - ductile steel</td>
<td>$\phi_{t,s} = 0.75$</td>
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<tr>
<td>Concrete - condition A</td>
<td>$\phi_{t,c} = 0.75$</td>
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<td>Anchor reinforcement</td>
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<td>Anchor rod - ductile steel</td>
<td>$\phi_{v,s} = 0.65$</td>
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<tr>
<td>Concrete - condition A</td>
<td>$\phi_{v,c} = 0.75$</td>
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</table>

### Code Reference

- **Strength reduction factors**
  - Anchor reinforcement: $\phi_t = 0.75$
  - Anchor rod - ductile steel: $\phi_{t,s} = 0.75$
  - Concrete - condition A: $\phi_{t,c} = 0.75$
  - Anchor reinforcement: $\phi_{v,t} = 0.65$
  - Anchor rod - ductile steel: $\phi_{v,s} = 0.65$
  - Concrete - condition A: $\phi_{v,c} = 0.75$

### CONCLUSION

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<th>Requirement</th>
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<tr>
<td>Anchor Rod Embedment, Spacing and Edge Distance</td>
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<td>Min Required Anchor Reinforcement Development Length</td>
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<td>Overall</td>
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<td>Tension Anchor Rod Tensile Resistance</td>
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<td>Strut Bearing Strength</td>
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<td>Tie Reinforcement</td>
<td>ratio = 0.69</td>
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<td>Concrete Pryout Not Govern When $h_{ef} \geq 12d_a$</td>
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<td>Tension Shear Interaction</td>
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<td>Ductility</td>
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<td>Seismic Design Requirement</td>
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<tr>
<td>SDC &lt; C, ACI318-08 D.3.3 ductility requirement is NOT required</td>
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### Calculation

**Anchor Tensile Force**

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<th>ACI 318-08</th>
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<tr>
<td>Single bolt tensile force</td>
<td>T&lt;sub&gt;1&lt;/sub&gt; = 12.42 [kips] No of bolt for T&lt;sub&gt;1&lt;/sub&gt; n&lt;sub&gt;T1&lt;/sub&gt; = 2</td>
</tr>
<tr>
<td>T&lt;sub&gt;2&lt;/sub&gt; = 0.00 [kips] No of bolt for T&lt;sub&gt;2&lt;/sub&gt; n&lt;sub&gt;T2&lt;/sub&gt; = 0</td>
<td></td>
</tr>
<tr>
<td>T&lt;sub&gt;3&lt;/sub&gt; = 0.00 [kips] No of bolt for T&lt;sub&gt;3&lt;/sub&gt; n&lt;sub&gt;T3&lt;/sub&gt; = 0</td>
<td></td>
</tr>
<tr>
<td>Sum of bolt tensile force</td>
<td>$N_a = \sum n_i T_i = 24.8$ [kips]</td>
</tr>
<tr>
<td>Anchor Rod Tensile Resistance</td>
<td>$\psi_{t,s} N_{sa} = \psi_{t,s} A_{sa} f_{da} = 42.2$ [kips] D.5.1.2 (D-3)</td>
</tr>
<tr>
<td>Resistance</td>
<td>ratio = 0.29 &gt; T&lt;sub&gt;1&lt;/sub&gt; OK</td>
</tr>
</tbody>
</table>

**Anchor Reinforcement Tensile Breakout Resistance**

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>ACI 318-08</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min tension development length</td>
<td>$l_d = 47.4$ [in] 12.2.1, 12.2.2, 12.2.4 for ver. #8 bar</td>
</tr>
<tr>
<td>Actual development length</td>
<td>$l_a = h_{ef} - c (2 \text{ in}) - 8 \text{ in} \times \tan 35 = 47.4$ [in]</td>
</tr>
<tr>
<td>&gt; 12.0</td>
<td>OK 12.2.1</td>
</tr>
</tbody>
</table>
Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

Dongxiao Wu P. Eng.

Code Reference

ACI 318-08

Anchor Pullout Resistance

Seismic design strength reduction

Anchor Pullout Resistance

Single bolt pullout resistance

Seismic design strength reduction

Anchor Pullout Resistance

Pullout strength is always Condition B

Seismic design strength reduction

Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

Check if edge anchors work as a group or work individually

Single anchor SB resistance

Multiple anchors SB resistance

Seismic design strength reduction

Group side blowout resistance

Govern Tensile Resistance

\[
N_{cr} = \phi_{cr} \cdot \psi_{cr} \cdot \frac{f_{cy} \cdot A_{cr}}{l_2 / l_1} \cdot \frac{l_2}{l_1} = 71.0 \text{ [kips]} \\

\text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 71.0 \text{ [kips]} \\

\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
\]

\[
N_{cr} = \phi_{cr} \cdot \psi_{cr} \cdot \frac{f_{cy} \cdot A_{cr}}{l_2 / l_1} = 71.0 \text{ [kips]} \\

\text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 71.0 \text{ [kips]} \\

\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
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\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
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\text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 71.0 \text{ [kips]} \\

\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
\]

\[
N_{cr} = \phi_{cr} \cdot \psi_{cr} \cdot \frac{f_{cy} \cdot A_{cr}}{l_2 / l_1} = 71.0 \text{ [kips]} \\

\text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 71.0 \text{ [kips]} \\

\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
\]

\[
N_{cr} = \phi_{cr} \cdot \psi_{cr} \cdot \frac{f_{cy} \cdot A_{cr}}{l_2 / l_1} = 71.0 \text{ [kips]} \\

\text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 71.0 \text{ [kips]} \\

\text{ratio} = 0.35 \text{ not applicable} > N_u \text{ OK}
\]
**Note:** Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear

\[
\phi_{vs} V_{sa} = \phi_{vs} n_s 0.6 A_{sa} f_{uta} = 43.8 \text{ [kips]} \quad \text{D.6.1.2 (b) (D-20)}
\]

Resistance

Reduction due to built-up grout pads = x 0.8, applicable = 35.1 [kips] \text{D.6.1.3}

ratio = 0.71

\( > V_u \quad \text{OK} \)

Anchor Reinft Shear Breakout Resistance

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

STM strength reduction factor \( \phi_m = 0.75 \quad 9.3.2.6 \)

\[
\text{Strut-and-Tie model geometry} \quad d_a = 2.250 \text{ [in]} \quad d_b = 2.250 \text{ [in]}
\]

\( \theta = 45 \quad d_t = 3.182 \text{ [in]} \)

Strut compression force \( C_s = 0.5 V_u / \sin \theta = 17.7 \text{ [kips]} \)

\( \text{ACI 318-08} \)

Strut Bearing Strength

Strut compressive strength \( f_{oa} = 0.85 f_c = 3.4 \text{ [ksi]} \quad \text{A.3.2 (A-3)} \)

* Bearing of anchor bolt

Anchor bearing length \( l_a = \min(8 d_a, h_{ef}) = 10.0 \text{ [in]} \quad \text{D.6.2.2} \)

Anchor bearing area \( A_{org} = l_a \times d_a = 12.5 \text{ [in}^2] \)

Anchor bearing resistance \( C_r = n_s \times \phi_{ua} \times f_{oa} \times A_{org} = 63.8 \text{ [kips]} \quad > V_u \quad \text{OK} \)

* Bearing of ver reinft bar

Ver bar bearing area \( A_{org} = (l_b + 1.5 \times d_t - d_a/2 - d_b/2) \times d_b = 13.6 \text{ [in}^2] \)

Ver bar bearing resistance \( C_r = \phi_{ua} \times f_{oa} \times A_{org} = 34.8 \text{ [kips]} \quad > C_s \quad \text{OK} \)
Tie Reinforcement

* For tie reinf., only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength $f_y$. Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook $T_n$
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar
$$n = n_{leg} \times n_{lay} = 4$$

Pull out resistance at hook
$$T_n = \phi_{t,c} \times 0.9 \times f_y \times d_b$$
$$e_h = 4.5 \times d_b$$

Single tie bar tension resistance
$$T_r = \phi_s \times f_y \times A_s$$

Total tie bar tension resistance
$$V_{rb} = 1.0 \times n \times T_r$$

Seismic design strength reduction
$$\frac{V_{rb}}{\phi N_n} = 36.0 \text{ [kips]}$$

**Code Reference**

*ACI 318-08*

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} > 12d_a$, the pryout failure will not govern

$$12d_a = 15.0 \text{ [in]}$$
$$h_{ef} = 55.0 \text{ [in]}$$

Govern Shear Resistance
$$V_r = \min (\phi_{v,s} V_{sa}, V_{rb}) = 35.1 \text{ [kips]}$$

Tension Shear Interaction

Check if $N_n > 0.2 \phi N_p$ and $V_r > 0.2 \phi V_n$

$$N_n / \phi N_p + V_u / \phi V_n = 1.06$$

ratio = 0.89 < 1.2 \text{ OK}

Ductility Tension

$$\phi_{t,c} N_{sa} = 42.2 \text{ [kips]}$$

Ductility Shear

$$\phi_{v,s} V_{sa} = 35.1 \text{ [kips]}$$

$$V_r = 36.0 \text{ [kips]}$$
Example 04: Anchor Bolt + Anchor Reinf + Tension Shear & Moment + CSA A23.3-04 Code

\[ M_u = 47.4 \text{ kNm} \quad N_u = -44.5 \text{ kN (Compression)} \quad V_u = 111.2 \text{ kN} \]

Concrete \( f_c' = 27.6 \text{ MPa} \) \quad Rebar \( f_y = 414 \text{ MPa} \)

Pedestal size \( 660\text{mm} \times 660\text{mm} \)

Anchor bolt \( F1554 \text{ Grade 36} \quad 1.25'' \text{ dia} \quad \text{Hex Head} \quad h_{ef} = 1397\text{mm} \quad h_a = 1524\text{mm} \)

Seismic design \( I_E F_a S_o(0.2) < 0.35 \)

Anchor reinforcement
Tension \( \rightarrow \) 2-25M ver. bar
Shear \( \rightarrow \) 2-layer, 2-leg 15M hor. bar

Provide built-up grout pad
ANCHOR BOLT DESIGN  
Combined Tension, Shear and Moment

Anchor bolt design based on

- CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
- ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
- PIP STE05121 Anchor Bolt Design Guide-2006

**Assumptions**

1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
5. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft
6. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
7. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output
8. Shear carried by only half of total anchor bolts due to oversized holes in column base plate

**Anchor Bolt Data**

- Factored moment: $M_u = 47.4$ [kNm] = 35.0 [kip-ft]
- Factored tension /compression: $N_u = -44.5$ [kN] in compression = -10.0 [kips]
- Factored shear: $V_u = 111.2$ [kN] = 25.0 [kips]
- Factored shear for design: $V_u = 111.2$ [kN] $V_u = 0$ if shear key is provided
### Code Reference

- **No of bolt line for resisting moment**
  
  \[ \text{No of bolt line for resisting moment} = 2 \text{ Bolt Line} \]

- **No of bolt along outermost bolt line**
  
  \[ \text{No of bolt along outermost bolt line} = 2 \]

- **Outermost bolt line spacing**
  
  \[ s_1 = 406 [\text{mm}] \quad 127 \quad \text{OK} \]
  \[ s_2 = 406 [\text{mm}] \quad 127 \quad \text{OK} \]

- **Internal bolt line spacing**
  
  \[ s_{b1} = 267 [\text{mm}] \quad 127 \quad \text{OK} \]
  \[ s_{b2} = 0 [\text{mm}] \quad 127 \quad \text{OK} \]

- **Column depth**
  
  \[ d = 323 [\text{mm}] \]

- **Concrete strength**
  
  \[ f'_c = 28 [\text{MPa}] \quad 4.0 [\text{ksi}] \]

- **Anchor bolt material**
  
  \[ f_{tua} = 58 [\text{ksi}] \quad 400 [\text{MPa}] \quad \text{OK} \]

- **Anchor bolt diameter**
  
  \[ d_a = 1.25 [\text{in}] \quad 31.8 [\text{mm}] \quad \text{OK} \]

- **Bolt sleeve diameter**
  
  \[ d_s = 76 [\text{mm}] \quad \text{OK} \]

- **Bolt sleeve height**
  
  \[ h_s = 254 [\text{mm}] \quad \text{OK} \]

- **Anchor bolt embedment depth**
  
  \[ h_{ef} = 1397 [\text{mm}] \quad 381 \quad \text{OK} \]

- **Pedestal height**
  
  \[ h = 1524 [\text{mm}] \quad 1473 \quad \text{OK} \]

- **Pedestal width**
  
  \[ b_c = 660 [\text{mm}] \]

- **Pedestal depth**
  
  \[ d_c = 660 [\text{mm}] \]

- **Bolt edge distance**
  
  \[ c_1 = 127 [\text{mm}] \quad 127 \quad \text{OK} \]
  \[ c_2 = 127 [\text{mm}] \quad 127 \quad \text{OK} \]
  \[ c_3 = 127 [\text{mm}] \quad 127 \quad \text{OK} \]
  \[ c_4 = 127 [\text{mm}] \quad 127 \quad \text{OK} \]

### Diagrams

- **Ver. Reinfl For Tension**
- **Hor. Ties For Shear - 4 Legs**
- **Hor. Ties For Shear - 2 Legs**
To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5\(h_{ef}\) from the outmost anchor's centerline. In this design 0.5\(h_{ef}\) value is limited to 200mm.

\[
0.5h_{ef} = 200 \text{ [mm]}
\]

No of ver. rebar that are effective for resisting anchor tension

Ver. bar size

\[
d_b = 25 \text{ [mm]}
\]

Ventilation area \(A_v = 500 \text{ [mm}^2]\)

To be considered effective for resisting anchor shear, hor. reinf shall be located within \(\text{min}(0.5c_1, 0.3c_2)\) from the outmost anchor's centerline

\[
\text{min}(0.5c_1, 0.3c_2) = 38 \text{ [mm]}
\]

No of tie leg that are effective to resist anchor shear

Hor. bar size

\[
d_b = 15 \text{ [mm]}
\]

Single bar area \(A_b = 200 \text{ [mm}^2]\)

For anchor reinft shear breakout strength calc 100% hor. tie bars develop full yield strength

Rebar yield strength

\[
f_y = 414 \text{ [MPa] } 400 \text{ [ksi]}
\]

Total no of anchor bolt

No of bolt carrying tension

No of bolt carrying shear

For side-face blowout check use

No of bolt along width edge

Anchor head type

\[
A_{so} = 625 \text{ [mm}^2]\]

Bearing area of head

\[
A_{avg} = 1172 \text{ [mm}^2]\]

Bolt 1/8" (3mm) corrosion allowance

Provide shear key

Seismic region where \(I_{EF_aS_a(0.2)}\geq 0.35\)

Provide built-up grout pad

ACI318 M-08

RD.5.2.9

RD.6.2.9

ACI318 M-08

A23.3-04 (R2010)

D.4.3.5

D.7.1.3
**Strength reduction factors**

- Anchor reinforcement factor: \( \phi_{as} = 0.75 \)
- Steel anchor resistance factor: \( \phi_s = 0.85 \)
- Concrete resistance factor: \( \phi_c = 0.65 \)

**Resistance modification factors**

- Anchor rod - ductile steel: \( R_{t,s} = 0.80 \)  \( R_{v,s} = 0.75 \)
- Concrete - condition A: \( R_{t,c} = 1.15 \)  \( R_{v,c} = 1.15 \)

### CONCLUSION

- **Anchor Rod Embedment, Spacing and Edge Distance**: OK
- **Min Required Anchor Reinforcement Development Length**: OK
- **Overall**: OK

#### Tension

- Anchor Rod Tensile Resistance: ratio = 0.32  OK
- Anchor Reinft Tensile Breakout Resistance: ratio = 0.36  OK
- Anchor Pullout Resistance: ratio = 0.33  OK
- Side Blowout Resistance: ratio = 0.32  OK

#### Shear

- Anchor Rod Shear Resistance: ratio = 0.73  OK
- Anchor Reinft Shear Breakout Resistance
  - Strut Bearing Strength: ratio = 0.52  OK
  - Tie Reinforcement: ratio = 0.45  OK
- Conc. Pryout Not Govern When \( h_{ad} \geq 12d_a \): OK
- Anchor Rod on Conc Bearing: ratio = 0.27  OK

#### Tension Shear Interaction

- Anchor Rod Tensile Shear Interaction: ratio = 0.90  OK

#### Ductility

- Anchor Rod Tensile
  - Non-ductile: \( N_{tr} = A_{as} \phi_s f_{as} R_{t,s} = 170.0 \) [kN]  \( > T_1 \)  OK
  - Ductile

#### Seismic Design Requirement

- IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

### Calculation

#### Anchor Tensile Force

- Single bolt tensile force
  - \( T_1 = 55.2 \) [kN]  No of bolt for \( T_1 n_{T1} = 2 \)
  - \( T_2 = 0.0 \) [kN]  No of bolt for \( T_2 n_{T2} = 0 \)
  - \( T_3 = 0.0 \) [kN]  No of bolt for \( T_3 n_{T3} = 0 \)
- Sum of bolt tensile force
  - \( N_u = \sum n_i T_i = 110.3 \) [kN]

#### Anchor Rod Tensile Resistance

- \( N_{tr} = A_{as} \phi_s f_{as} R_{t,s} = 170.0 \) [kN]  D.6.1.2 (D-3)
- ratio = 0.32  OK

#### Anchor Reinft Tensile Breakout Resistance

- Min tension development length
  - \( l_d = 887 \) [mm]  12.2.3
  - for ver. 25M bar
### Code Reference

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>Actual development length $l_a = h_{df} - c$ (50mm) - 200mm x tan35 = 1207 [mm] A23.3-04 (R2010)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$&gt; 300$ OK</td>
</tr>
</tbody>
</table>

#### Anchor Pullout Resistance

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
<th>Value</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single bolt pullout resistance</td>
<td>$N_{pr} = \frac{\phi_c f_y n_a A_s (l_a / l_d, if l_a &lt; l_d)} x 1.0$ not applicable</td>
<td>310.5 kN</td>
<td>D.6.3.4 (D-16)</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>$\frac{\Psi_{c,p} N_{pr}}$</td>
<td>168.2 kN</td>
<td>D.6.3.1 (D-15)</td>
</tr>
<tr>
<td>Ratio</td>
<td>$0.36 &gt; \frac{N_u}{N}$</td>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

#### Side Blowout Resistance

**Failure Along Pedestal Width Edge**

- Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal width edge $N_{bw} = n_{11} T_1$ = 110.3 [kN] RD.5.4.2
- $c = \min (c_1, c_3)$ = 127 [mm] A23.3-04 (R2010)
- Check if side blowout applicable $h_{df} = 1397$ [mm] $> 2.5c$ side bowout is applicable D.6.4.1
- $s_{22} = 406$ [mm] $s = s_2 = 406$ [mm]
- Single anchor SB resistance $N_{sb,w} = 13.3 c \sqrt{A_{bs}} \phi_s f_y \sqrt{R_{t,c}}$ = 227.1 [kN] D.6.4.1 (D-18)
- Multiple anchors SB resistance $N_{sbgr,w} = (1+s/6c) x N_{sb,w}$ = 348.1 [kN] D.6.4.2 (D-19)
- $n_{bw} x N_{sb,w} x [1+c_2 or c_4] / c = 0.0$ [kN] D.6.4.1
- Seismic design strength reduction $x 1.0$ not applicable = 348.1 kN | D.4.3.5 |
- Ratio | $0.32 > \frac{N_{bw}}{N}$ | OK | |

**Group side blowout resistance** $N_{sbgr} = \frac{N_{sbgr,w}}{n_{bw}} = 348.1$ [kN]

**Govern Tensile Resistance** $N_r = \min (n_i N_{bw}, N_{fbw}, n_i N_{cpr}, N_{sbgr}) = 310.5$ [kN]
Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Code Reference
A23.3-04 (R2010)

Anchor Rod Shear

\[ V_{sr} = n_s A_{se} f_y \theta \frac{6}{f_{ut}} R_{u,s} \]

Resistance

\[ R_{v,s} = 191.2 \ [kN] \]

Reduction due to built-up grout pads

\[ = x \times 0.8 \ , \ \text{applicable} \]

\[ = 153.0 \ [kN] \]

\[ \text{ratio} = 0.73 \]

\[ > V_u \quad \text{OK} \]

Anchor Reinft Shear Breakout Resistance

ACI 318 M-08

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement.

STM strength reduction factor

\[ \phi_{st} = 0.75 \]

Strut-and-Tie model geometry

\[ d_s = 57 \ [mm] \]

\[ \theta = 45 \]

\[ d_t = 81 \ [mm] \]

Strut compression force

\[ C_s = 0.5 V_u \sin \theta \]

\[ = 78.6 \ [kN] \]

ACI 318 M-08

Strut Bearing Strength

Strut compressive strength

\[ f_{os} = 0.85 f_c \]

\[ = 23.5 \ [MPa] \]

ACI 318 M-08

* Bearing of anchor bolt

Anchor bearing length

\[ l_s = \min (8d_s, h_{ef}) \]

\[ = 254 \ [mm] \]

Anchor bearing area

\[ A_{org} = l_s \times d_s \]

\[ = 8065 \ [mm^2] \]

Anchor bearing resistance

\[ C_r = n_s \times \phi_{st} \times f_{os} \times A_{org} \]

\[ = 283.8 \ [kN] \]

\[ > V_u \quad \text{OK} \]

* Bearing of ver reinft bar

Ver bar bearing area

\[ A_{org} = (l_s + 1.5 \times d_t - d_s/2 - d_b/2) \times d_b \]

\[ = 8664 \ [mm^2] \]

Ver bar bearing resistance

\[ C_r = \phi_{st} \times f_{os} \times A_{org} \]

\[ = 152.4 \ [kN] \]

\[ \text{ratio} = 0.52 \]

\[ > C_s \quad \text{OK} \]
Tie Reinforcement

* For tie reinforcement, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength $f_y$. Use the pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook $T_h$
* Assume 100% of hor. tie bars can develop full yield strength.

**Code Reference**

A23.3-04 (R2010)

Total number of hor tie bar

$$n = n_{leg} \times n_{lay}$$

Pull out resistance at hook

$$T_h = 0.9 \cdot f_y \cdot e_h \cdot d_b \cdot R_{c,c}$$

Single tie bar tension resistance

$$T_r = \phi_c \cdot f_y \cdot A_s$$

Total tie bar tension resistance

$$V_{tr} = 1.0 \times n \times T_r$$

Seismic design strength reduction

$$\frac{n}{Vu} = 0.45$$

OK

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in-place headed anchors with $h_{ef} > 12d_a$, the pryout failure will not govern

$$12d_a = 381 \text{ [mm]}$$

$$h_{ef} = 1397 \text{ [mm]}$$

$$> 12d_a \ldots \text{OK}$$

CSA S16-09

Anchor Rod on Conc Bearing

$$B_r = n_s \times 1.4 \times \phi_c \times \min(8d_r, h_{ef}) \times d_a \times f'_{c'}$$

ratio = 0.27

< $V_u \ldots \text{OK}$

Govern Shear Resistance

$$V_r = \min (V_{sr}, V_{rbr}, B_r) = 153.0 \text{ [kN]}$$

Tension Shear Interaction

Check if $N_r > 0.2 \cdot N_t$ and $V_r > 0.2 \cdot V_t$

$$\frac{N_r}{N_t} + \frac{V_r}{V_t} = 1.08$$

< 1.2

OK

Ductility Tension

$$N_{sr} = 170.0 \text{ [kN]}$$

$$> \min (N_{sr}, N_{spr}, N_{spr}) = 168.2 \text{ [kN]}$$

Non-ductile

Ductility Shear

$$V_{sr} = 153.0 \text{ [kN]}$$

$$< \min (V_{rbr}, B_r) = 248.4 \text{ [kN]}$$

Ductile
Example 11: Anchor Bolt + No Anchor Reinf + Tension & Shear + ACI 318-08 Code

This example taken from Example 8 on page 71 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*

![Diagram of anchor bolt setup](image)

- $N_u = 12$ kips (tension), $V_u = 4$ kips, $f'c = 3$ ksi
- Anchor bolt $d_a=3/4$ in ASTM F1554 Grade 55, $h_d = 12$ in, $h_a = 24$ in, Anchor head $\to$ Hex
- Supplementary reinforcement: Tension $\to$ Condition B, Shear $\to$ Condition A, $\Psi_{c,V} = 1.2$
- Provide built-up grout pad
- Seismic is not a consideration
- Field welded plate washers to base plate at each anchor
ANCHOR BOLT DESIGN

Combined Tension and Shear

Anchor bolt design based on

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D

PIP STE05121 Anchor Bolt Design Guide-2006

Anchor Bolt Data

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>Code Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored tension for design</td>
<td>Nu = 12.0 [kips] = 53.4 [kN]</td>
</tr>
<tr>
<td>Factored shear</td>
<td>Vv = 4.0 [kips] = 17.8 [kN]</td>
</tr>
<tr>
<td>Factored shear for design</td>
<td>Vv = 4.0 [kips] Vv = 0 if shear key is provided</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>f'c = 3.0 [ksi] = 20.7 [MPa]</td>
</tr>
<tr>
<td>Anchor bolt material</td>
<td>F1554 Grade 55</td>
</tr>
<tr>
<td>Anchor bolt material (Anchor is ductile steel element)</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt diameter</td>
<td>da = 0.75 [in] = 19.1 [mm]</td>
</tr>
<tr>
<td>Bolt sleeve diameter</td>
<td>ds = 2.0 [in]</td>
</tr>
<tr>
<td>Bolt sleeve height</td>
<td>hs = 7.0 [in]</td>
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<tr>
<td>Anchor bolt embedment depth</td>
<td>hef = 12.0 [in] 9.0 OK</td>
</tr>
<tr>
<td>Concrete thickness</td>
<td>ha = 24.0 [in] 15.0 OK</td>
</tr>
<tr>
<td>Bolt edge distance c1</td>
<td>c1 = 4.0 [in] 4.5 Warn</td>
</tr>
<tr>
<td>Bolt edge distance c2</td>
<td>c2 = 4.0 [in] 4.5 Warn</td>
</tr>
<tr>
<td>Bolt edge distance c3</td>
<td>c3 = 100.0 [in] 4.5 OK</td>
</tr>
<tr>
<td>Bolt edge distance c4</td>
<td>c4 = 100.0 [in] 4.5 OK</td>
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<tr>
<td>c1 &gt; 1.5hef for at least two edges to avoid reducing of hef when Nu &gt; 0</td>
<td></td>
</tr>
<tr>
<td>Adjusted hef for design</td>
<td>hef = 12.00 [in] 9.0 Yes</td>
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<tr>
<td>Outermost bolt line spacing s1</td>
<td>s1 = 8.0 [in] 3.0 OK</td>
</tr>
<tr>
<td>Outermost bolt line spacing s2</td>
<td>s2 = 8.0 [in] 3.0 OK</td>
</tr>
<tr>
<td>Outermost bolt line spacing s3</td>
<td>s3 = 8.0 [in] 3.0 OK</td>
</tr>
</tbody>
</table>

ACI 318-08

D.5.2.3

Page A-1 Table 1

OK

F1554 Grade 55

Grade 55
Number of bolt at bolt line 1 \( n_1 = 2 \)
Number of bolt at bolt line 2 \( n_2 = 2 \)
Number of bolt carrying tension \( n_t = 4 \)
Oversized holes in base plate? \( \text{No} \)
Number of bolt carrying shear \( n_s = 2 \)
For side-face blowout check use
No of bolt along width edge \( n_{bw} = 2 \)
No of bolt along depth edge \( n_{bd} = 2 \)
Anchor head type \( = \text{Hex} \)
Anchor effective cross sect area \( A_{se} = 0.334 \text{[in}^2\text{]} \)
Bearing area of head \( A_{b} = 0.654 \text{[in}^2\text{]} \)
Bolt 1/8” (3mm) corrosion allowance \( \text{No} \)
Provide shear key? \( \text{No} \)
Seismic design category >= C \( \text{No} \)
Supplementary reinforcement
For tension \( \Psi_{c,V} = 1.2 \) Condition A
For shear
Provide built-up grout pad? \( \text{Yes} \)
Strength reduction factors
Anchor reinforcement \( \phi_s = 0.75 \) \( \phi_{s,t} = 0.75 \) \( \phi_{s,v} = 0.65 \) \( \phi_{s,0} = 0.75 \)
Anchor rod - ductile steel \( \phi_{s,t} = 0.75 \) \( \phi_{s,v} = 0.75 \)
Concrete \( \phi_{c,t} = 0.70 \) Cdn-B \( \phi_{c,v} = 0.75 \) Cdn-A
Assumptions
1. Concrete is cracked
2. Condition B - no supplementary reinforcement provided
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Tensile load acts through center of bolt group \( \Psi_{c,N} = 1.0 \)
5. Shear load acts through center of bolt group \( \Psi_{c,V} = 1.0 \)
6. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear

ACI 318-08
D.3.3.3
D.4.4 (c)
D.4.4 (a)
D.4.4 (c)
D.5.2.9 & D.6.2.9
D.5.2.4
D.6.2.5
AISC Design Guide 1
section 3.5.3
CONCLUSION

Anchor Rod Embedment, Spacing and Edge Distance

Overall ratio = 0.83 Warn

Tension
Anchor Rod Tensile Resistance ratio = 0.16 OK
Conc. Tensile Breakout Resistance ratio = 0.58 OK
Anchor Pullout Resistance ratio = 0.27 OK
Side Blowout Resistance ratio = 0.23 OK

Shear
Anchor Rod Shear Resistance ratio = 0.13 OK
Conc. Shear Breakout Resistance ratio = 0.41 OK
Conc. Pryout Shear Resistance ratio = 0.10 OK

Tension Shear Interaction
Tension Shear Interaction ratio = 0.83 OK

Ductility
Tension Non-ductile OK
Shear Non-ductile

Seismic Design Requirement OK D.3.3.4
SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

CALCULATION

Anchor Rod Tensile Resistance
\[ \phi_{t,s} N_{sa} = \phi_{t,s} n_t A_{sa} f_{uta}\]
\[ = 75.2 \text{ [kips]} \]
\[ \text{ratio} = 0.16 > N_u \text{ OK} \]

Conc. Tensile Breakout Resistance
\[ N_b = 24 \sqrt{h_{ef}} f_{ut}^2 \text{ if } h_{ud} < 11^\circ \text{ or } h_{ud} > 25^\circ = 55.1 \text{ [kips]} \]
\[ 16 \sqrt{h_{ef}} f_{ut}^2 \text{ if } 11^\circ \leq h_{ud} \leq 25^\circ \]
\[ \text{Projected conc failure area } 1.5h_{ef} = 18.00 \text{ [in]} \]
\[ A_{nc} = [\text{min}(c_1,1.5h_{ud})+\text{min}(c_2,1.5h_{ud})] x = 900.0 \text{ [in}^2] \]
\[ A_{nc,0} = 9 h_{ef}^2 = 1296.0 \text{ [in}^2] \]
\[ A_{nc} = \text{min}(A_{nc}, n_t A_{nc,0}) = 900.0 \text{ [in}^2] \]
\[ \text{Min edge distance } c_{min} = \text{min}(c_1, c_2, c_3, c_4) = 4.0 \text{ [in]} \]

Eccentricity effects \[ \Psi_{ec,N} = 1.0 \text{ for no eccentric load} \]
\[ \text{Edge effects } \Psi_{ed,N} = \text{min}(0.7+0.3c_{min}/1.5h_{ud}, 1.0) = 0.77 \]

Concrete cracking \[ \Psi_{c,N} = 1.0 \text{ for cracked concrete} \]

Concrete splitting \[ \Psi_{cp,N} = 1.0 \text{ for cast-in anchor} \]
### Concrete Breakout Resistance

\[ \phi_{tc} N_{cbg} = \phi_{tc} A_{bw} \psi_{nc} \psi_{sc} \psi_{nc} N_c = 20.5 \text{ [kips]} \]

Seismic design strength reduction = x 1.0, not applicable = 20.5 [kips]  
Seismic design strength reduction ratio = 0.58 > N_u  
OK

### Anchor Pullout Resistance

#### Single Bolt Pullout Resistance

\[ N_p = 8 A_{bg} f_c' \]

\[ \phi_{tc} N_{pn} = \phi_{tc} n_t \psi_{c,p} N_p = 43.9 \text{ [kips]} \]

Seismic design strength reduction = x 1.0, not applicable = 43.9 [kips]  
Seismic design strength reduction ratio = 0.27 > N_u  
OK

#### Anchor Pullout Resistance

\[ \phi_{tc} = 0.70 \] pullout strength is always Condition B

### Side Blowout Resistance

#### Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal width edge

\[ N_{bww} = N_u x n_{bw} / n_t = 6.0 \text{ [kips]} \]

\[ c = \min (c_1, c_3) = 4.0 \text{ [in]} \]

Check if side blowout applicable

\[ h_{id} = 12.0 \text{ [in]} \]

\[ > 2.5c \text{ side blowout is applicable} \]

Check if edge anchors work as a group or work individually

\[ s_{22} = 8.0 \text{ [in]} \]

\[ s = s_2 = 8.0 \text{ [in]} \]

Seismic design strength reduction = x 1.0, not applicable = 26.5 [kips]  
Seismic design strength reduction ratio = 0.23 > N_{buw}  
OK

#### Failure Along Pedestal Depth Edge

Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal depth edge

\[ N_{bud} = N_u x n_{bd} / n_t = 6.0 \text{ [kips]} \]

\[ c = \min (c_2, c_4) = 4.0 \text{ [in]} \]

Check if side blowout applicable

\[ h_{id} = 12.0 \text{ [in]} \]

\[ > 2.5c \text{ side blowout is applicable} \]

Check if edge anchors work as a group or work individually

\[ s_{11} = 8.0 \text{ [in]} \]

\[ s = s_1 = 8.0 \text{ [in]} \]

Seismic design strength reduction = x 1.0, not applicable = 26.5 [kips]  
Seismic design strength reduction ratio = 0.23 > N_{bud}  
OK
Group side blowout resistance \[ \phi_{t,c} N_{sb} = \phi_{t,c} \min \left( \frac{N_{ba,b}}{n_{ba}}, \frac{N_{ab,d}}{n_{ab}} \right) \]

Govern Tensile Resistance \[ N_r = \min [ \phi_{t,s} N_{sa}, \phi_{t,c} (N_{cb}, N_{pn}, N_{sb}) ] \]

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear \[ \phi_{v,s} V_{sa} = \phi_{v,s} n_s 0.6 A_{sa} f_{uta} = 39.1 \] [kips] D.6.1.2 (b) (D-20)

Resistance
Reduction due to built-up grout pads = x 0.8 , applicable = 31.3 [kips] D.6.1.3
ratio = 0.13 > \( V_u \) OK

Conc. Shear Breakout Resistance

Mode 1 Failure cone at front anchors, strength check against 0.5 x \( V_u \)
Mode 3 Failure cone at front anchors, strength check against 1.0 x \( V_u \), applicable when oversized holes are used in base plate

Bolt edge distance \[ c_1 = 4.0 \] [in]

Limiting \( c_1 \) when anchors are influenced by 3 or more edges = No D.6.2.4

Bolt edge distance - adjusted \[ c_1 = ca_1 \text{ needs NOT to be adjusted} \]

\[ c_2 = 4.0 \] [in] D.6.2.4

\[ 1.5c_1 = 6.0 \] [in]

\[ A_{vc} = \left[ \min(c_2, 1.5c_1) + s_2 + \min(c_4, 1.5c_1) \right] x \text{min}(1.5c_1, h_u) \]

\[ A_{vco} = 4.5c_1^2 \]

\[ A_{vc} = \min (A_{vc}, n_t A_{vco}) \]

\[ l_u = \min(8d_s, h_{ut}) \]

\[ V_0 = \left[ 7 \left( \frac{1}{d_1} \right)^{0.2} \sqrt{d_2} \right] \lambda \sqrt{c_1^2} \]

Eccentricity effects \[ \psi_{ec,v} = 1.0 \text{ shear acts through center of group} \]

Edge effects \[ \psi_{ed,v} = \min [0.7+0.3c/1.5c], 1.0] \]

Concrete cracking \[ \psi_{c,v} \]

Member thickness \[ \psi_{h,v} = \max [\sqrt{1.5c_1} / h_u, 1.0] \]
Conc shear breakout resistance

\[ V_{cbg1} = \phi_{c} \frac{A_{w}}{A_{as}} \psi_{w,c} \psi_{w,t} \psi_{w,v} \psi_{n,v} V_{c} = 4.9 \text{ [kips]} \]  

Mode 1 is used for checking

\[ V_{cbg1} \times 2.0 = 9.8 \text{ [kips]} \]

Mode 2  Failure cone at back anchors

**Code Reference**  
ACI 318-08  
D.6.2.1 (D-22)

Bolt edge distance  
\[ c_{a1} = c_{1} + s_{1} = 12.0 \text{ [in]} \]

Limiting \( c_{a} \) when anchors are influenced by 3 or more edges  
\[ = \text{No} \]  
D.6.2.4

Bolt edge distance - adjusted  
\[ c_{a1} = \text{ca1 needs NOT to be adjusted} = 12.0 \text{ [in]} \]  
D.6.2.4

\[ c_{2} = 4.0 \text{ [in]} \]

\[ 1.5c_{a1} = 18.0 \text{ [in]} \]

\[ A_{vc} = [\min(c_{2},1.5c_{a1}) + s_{2} + \min(c_{4},1.5c_{a1})] \times \min(1.5c_{a1}, h_{a}) = 540.0 \text{ [in}^2]\]  
D.6.2.1

\[ A_{vco} = 4.5c_{a1}^2 = 648.0 \text{ [in}^2]\]  
D.6.2.1 (D-23)

\[ A_{vc} = \min ( A_{vco} , n_{2} A_{vco} ) = 540.0 \text{ [in}^2]\]  
D.6.2.1

\[ l_{o} = \min( 8d_{a} , h_{af} ) = 6.0 \text{ [in]} \]  
D.6.2.2

\[ V_{b} = \left[ 7 \left( \frac{L}{d_{a}} \right)^{0.2} \sqrt{d_{a}} \right] A_{vc} 1.5^{c_{a1}} = 20.9 \text{ [kips]} \]  
D.6.2.2 (D-24)

Eccentricity effects  
\[ \psi_{w,c} = 1.0 \]  
shear acts through center of group  
D.6.2.5

Edge effects  
\[ \psi_{w,t} = \min [ (0.7+0.3c_{2}/1.5c_{a1}) , 1.0 ] = 0.77 \]  
D.6.2.6

Concrete cracking  
\[ \psi_{w,v} = 1.20 \]  
D.6.2.7

Member thickness  
\[ \psi_{h,v} = \max [ (\sqrt{1.5c_{a1}}/ h_{a}) , 1.0 ] = 1.00 \]  
D.6.2.8

Conc shear breakout resistance  
\[ V_{cbg2} = \phi_{c} \frac{A_{w}}{A_{as}} \psi_{w,c} \psi_{w,t} \psi_{w,v} \psi_{n,v} V_{c} = 12.0 \text{ [kips]} \]  
D.6.2.1 (D-22)

Min shear breakout resistance  
\[ V_{cbg} = \min ( V_{cbg1} , V_{cbg2} ) = 9.8 \text{ [kips]} \]

Seismic design strength reduction  
\[ = 1.0 \]  
not applicable  
D.3.3.3

\[ \text{ratio} = 0.41 \]  
\[ > V_{u} \]  
OK
### Conc. Pryout Shear Resistance

- $k_{cp} = 2.0$
- Factored shear pryout resistance: $\phi_{v,c} V_{cp} = \phi_{v,c} k_{cp} N_{cbg} = 41.1$ [kips]
  - $\phi_{v,c} = 0.70$
  - pryout strength is always Condition B
- Seismic design strength reduction: $\frac{V}{V_u} \times 1.0$ not applicable = 41.1 [kips] D.3.3.3
- $\phi_{v,c} = 0.70$ pryout strength is always Condition B

### Govern Shear Resistance

- $V_r = \min \{ \phi_{v,s} V_{sa}, \phi_{v,c} (V_{cpg}, V_{cp}) \} = 9.8$ [kips]

### Tension Shear Interaction

- Check if $\frac{\phi_t}{\phi_N} N_n + \frac{\phi_v}{\phi_V} V_n > 0.2$
  - $\phi_t N_n = 75.2$ [kips]
  - $\phi_t \min (N_{cbp}, N_{int}, N_{dbp}) = 20.5$ [kips]
  - $\phi_v V_n = 31.3$ [kips]
  - $\phi_v \min (V_{cpg}, V_{cp}) = 9.8$ [kips]

### Ductility Tension

- $\phi_{t,s} N_{sa} = 75.2$ [kips]
  - $\phi_{t,c} \min (N_{cbp}, N_{int}, N_{dbp}) = 20.5$ [kips] Non-ductile

### Ductility Shear

- $\phi_{v,s} V_{sa} = 31.3$ [kips]
  - $\phi_{v,c} \min (V_{cpg}, V_{cp}) = 9.8$ [kips] Non-ductile
Example 12: Anchor Bolt + No Anchor Reinft + Tension & Shear + CSA A23.3-04 Code

This example taken from Example 8 on page 71 of *ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D*

![Diagram of anchor bolt installation]

**Nu** = 53.4 kN (tension),  \( V_u = 17.8 \text{ kN}, \quad f_{c'} = 20.7 \text{ MPa} \)

- Anchor bolt \( d_a = 3/4 \text{ in} \) ASTM F1554 Grade 55  \( h_{ef} = 305 \text{ mm} \)
-  \( h_a = 610 \text{ mm} \)  Anchor head \( \rightarrow \) Hex
- Supplementary reinforcement: Tension \( \rightarrow \) Condition B  Shear \( \rightarrow \) Condition A  \( \Psi_{c,V} = 1.2 \)
- Provide built-up grout pad
- Seismic is not a consideration
- Field welded plate washers to base plate at each anchor

2011-12-16 Rev 1.0.0
ANCHOR BOLT DESIGN

Combined Tension and Shear

Anchor bolt design based on CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D

Dongxiao Wu P. Eng.

Factored tension for design

\[ N_u = 53.4 \text{ [kN]} = 12.0 \text{ [kips]} \]

Factored shear

\[ V_u = 17.8 \text{ [kN]} = 4.0 \text{ [kips]} \]

Factored shear for design

\[ V_u = 17.8 \text{ [kN]} \quad V_u = 0 \text{ if shear key is provided} \]

Concrete strength

\[ f'_c = 21 \text{ [MPa]} = 3.0 \text{ [ksi]} \]

Anchor bolt material

F1554 Grade 55

Anchor tensile strength

\[ f_{uta} = 75 \text{ [ksi]} = 517 \text{ [MPa]} \]

Anchor bolt diameter

\[ d_a = 0.75 \text{ [in]} = 19.1 \text{ [mm]} \]

Bolt sleeve diameter

\[ d_s = 51 \text{ [mm]} \]

Bolt sleeve height

\[ h_s = 178 \text{ [mm]} \]

Anchor bolt embedment depth

\[ h_{ef} = 305 \text{ [mm]} \]

Concrete thickness

\[ h_a = 610 \text{ [mm]} \]

Bolt edge distance \( c_1 \)

\[ c_1 = 102 \text{ [mm]} \]

Bolt edge distance \( c_2 \)

\[ c_2 = 102 \text{ [mm]} \]

Bolt edge distance \( c_3 \)

\[ c_3 = 2540 \text{ [mm]} \]

Bolt edge distance \( c_4 \)

\[ c_4 = 2540 \text{ [mm]} \]

\[ c_i > 1.5 h_{ef} \text{ for at least two edges to avoid reducing of } h_{ef} \text{ when } N_u > 0 \]

Adjusted \( h_{ef} \) for design

\[ h_{ef} = 305 \text{ [mm]} \]

Outermost bolt line spacing \( s_1 \)

\[ s_1 = 203 \text{ [mm]} \]

Outermost bolt line spacing \( s_2 \)

\[ s_2 = 203 \text{ [mm]} \]

\[ \min\left( c_i, 1.5 h_{ef}\right) \]

\[ s_1 \quad \min\left( c_i, 1.5 h_{ef}\right) \]

\[ h_{ef} \]

\[ D_2 \]

\[ C_1 \]

\[ S_1 \]

\[ S_1 \]

\[ C_3 \]

\[ C_3 \]

\[ d_s \]

\[ d_a \]
Number of bolt at bolt line 1 \( n_1 = 2 \)
Number of bolt at bolt line 2 \( n_2 = 2 \)
Number of bolt carrying tension \( n_t = 4 \)
Oversized holes in base plate? \( \text{No} \)
Number of bolt carrying shear \( n_s = 4 \)
For side-face blowout check use

No of bolt along width edge \( n_{bw} = 2 \)
No of bolt along depth edge \( n_{bd} = 2 \)

Anchor head type

\[
A_{sa} = 215 \, [\text{mm}^2]
\]
Bearing area of head \( A_{org} = 422 \, [\text{mm}^2] \)
Bearing area of custom head \( A_{org} = 210 \, [\text{mm}^2] \) not applicable

Bolt 1/8" (3mm) corrosion allowance

Provide shear key? \( \text{No} \)
Seismic region where \( I_{EFaSa}(0.2) \geq 0.35 \)

Supplementary reinforcement

For tension \( \Psi_{ec,N} = 1.0 \) Condition B
For shear \( \Psi_{ec,V} = 1.2 \) Condition A

Provide built-up grout pad? \( \text{Yes} \)

Strength reduction factors

Anchor reinforcement factor \( \phi_{as} = 0.75 \)
Steel anchor resistance factor \( \phi_s = 0.85 \)
Concrete resistance factor \( \phi_c = 0.65 \)

Resistance modification factors

Anchor rod - ductile steel \( R_{ts} = 0.80 \)
Concrete \( R_{tc} = 1.00 \) Cdn-B \( R_{tc} = 0.75 \) Cdn-A

Assumptions

1. Concrete is cracked
2. Condition B for tension - no supplementary reinforcement provided
3. Tensile load acts through center of bolt group \( \Psi_{ec,N} = 1.0 \)
4. Shear load acts through center of bolt group \( \Psi_{ec,V} = 1.0 \)
5. Anchor bolt washer shall be tack welded to base plate for all anchor bolts to transfer shear AISC Design Guide 1 section 3.5.3
### CONCLUSION

**Anchor Rod Embedment, Spacing and Edge Distance**

<table>
<thead>
<tr>
<th>Category</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall</td>
<td>0.86</td>
<td>Warn</td>
</tr>
<tr>
<td>Tension</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anchor Rod Tensile Resistance</td>
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</tr>
<tr>
<td>Conc. Tensile Breakout Resistance</td>
<td>0.62</td>
<td>OK</td>
</tr>
<tr>
<td>Anchor Pullout Resistance</td>
<td>0.29</td>
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</tr>
<tr>
<td>Side Blowout Resistance</td>
<td>0.24</td>
<td>OK</td>
</tr>
<tr>
<td>Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anchor Rod Shear Resistance</td>
<td>0.13</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Shear Breakout Resistance</td>
<td>0.41</td>
<td>OK</td>
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<tr>
<td>Conc. Pryout Shear Resistance</td>
<td>0.10</td>
<td>OK</td>
</tr>
<tr>
<td>Anchor Rod on Conc Bearing</td>
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</tr>
<tr>
<td>Tension Shear Interaction</td>
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</tr>
<tr>
<td>Ductility</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>Non-ductile</td>
<td>OK</td>
</tr>
<tr>
<td>Shear</td>
<td>Non-ductile</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Seismic Design Requirement**

D.4.3.6

\[ \text{iFeSa}(0.2) < 0.35, \text{A23.3-04 D.4.3.3 ductility requirement is NOT required} \]

### CALCULATION

**Anchor Rod Tensile Resistance**

\[ N_u = n_t A_{se} f_{se} R_{ts} \]

<table>
<thead>
<tr>
<th>Code Reference</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A23.3-04 (R2010)</td>
<td></td>
</tr>
<tr>
<td>D.6.2.2 (D-7)</td>
<td></td>
</tr>
<tr>
<td>D.6.2.2 (D-8)</td>
<td></td>
</tr>
</tbody>
</table>

\[ N_u = 303.1 \text{ [kN]} \]

**Conc. Tensile Breakout Resistance**

\[ N_{cr} = 10 \phi \sqrt{h_{ef} / h_p} R_{cr} \text{ if } h_{ef} \leq 275 \text{ or } h_{ef} \geq 625 \]

\[ = 160.5 \text{ [kN]} \]

\[ 3.9 \phi \sqrt{h_{ef} / h_p} R_{cr} \text{ if } 275 < h_{ef} < 625 \]

\[ A_{nc} = \min (A_{nc}, n_t A_{ndo}) \]

\[ A_{ndo} = 9 h_{ef}^2 \]

\[ A_{nc} = \min (A_{nc}, n_t A_{ndo}) \]

\[ = 5.8E+05 \text{ [mm}^2] \]

\[ \Psi_{ec,N} = 1.0 \text{ for no eccentric load} \]

\[ \Psi_{ed,N} = 0.77 \text{ for cracked concrete} \]

\[ \Psi_{sp,N} = 1.0 \text{ for cast-in anchor} \]

Projected conc failure area

\[ A_{nc} = \frac{1.5 h_{ef}}{} = 458 \text{ [mm]} \]

\[ A_{nc} = \left[ s_t + \min (c_t, 1.5 h_{ef}) \right] x \]

\[ \Psi_{ec,N} = 1.0 \text{ for no eccentric load} \]

\[ \Psi_{ed,N} = 0.77 \text{ for cracked concrete} \]

\[ \Psi_{sp,N} = 1.0 \text{ for cast-in anchor} \]

Min edge distance

\[ c_{min} = \min (c_t, c_p, c_c, c_s) \]

\[ = 102 \text{ [mm]} \]
### Code Reference

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>A23.3-04 (R2010)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete breakout resistance</td>
<td>$N_{cbgr} = \frac{A_{nb}}{A_{Ncb}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{eq,N} N_{yw}$</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>$x 1.0$ not applicable</td>
</tr>
<tr>
<td>ratio</td>
<td>$0.62$</td>
</tr>
</tbody>
</table>

### Anchor Pullout Resistance

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Single bolt pullout resistance</td>
<td>$N_{pr} = 8 A_{nb} f'c \bar{f}'c R_{t,c}$</td>
<td>$46.1$ [kN]</td>
</tr>
<tr>
<td>$N_{cbp} = n_p N_{pr}$</td>
<td>$184.3$ [kN]</td>
<td>D.6.3.1 (D-15)</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>$x 1.0$ not applicable</td>
<td>$184.3$ [kN]</td>
</tr>
<tr>
<td>ratio</td>
<td>$0.29$</td>
<td>$&gt; N_u$</td>
</tr>
<tr>
<td>$\Psi_{c,p} = 1$ for cracked conc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R_{t,c} = 1.00$ pullout strength is always Condition B</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Side Blowout Resistance

#### Failure Along Pedestal Width Edge

- **Tensile load carried by anchors close to edge which may cause side-face blowout**
  - **ACI318 M-08**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{nbw} = N_u \times n_{bw} / n_t$</td>
<td>$26.7$ [kN]</td>
</tr>
<tr>
<td>$c = \min (c_1, c_3)$</td>
<td>$102$ [mm]</td>
</tr>
<tr>
<td>Check if side blowout applicable</td>
<td></td>
</tr>
<tr>
<td>$h_{ef} = 305$ [mm]</td>
<td>A23.3-04 (R2010)</td>
</tr>
<tr>
<td>$&gt; 2.5c$</td>
<td>side blowout is applicable</td>
</tr>
<tr>
<td>Check if edge anchors work as a group or work individually</td>
<td></td>
</tr>
<tr>
<td>$s_{22} = 203$ [mm]</td>
<td>$s = s_2 = 203$ [mm]</td>
</tr>
<tr>
<td>a group or work individually</td>
<td></td>
</tr>
<tr>
<td>$&lt; 6c$</td>
<td>edge anchors work as a group</td>
</tr>
<tr>
<td>Single anchor SB resistance</td>
<td>$N_{sbr,w} = 13.3 c_3 \sqrt{A_{nb}} \bar{\phi}'c \sqrt{R_{t,c}} R_{t,c}$</td>
</tr>
<tr>
<td>Multiple anchors SB resistance</td>
<td></td>
</tr>
<tr>
<td>work as a group - applicable</td>
<td>$(1+s/6c) \times N_{sbr,w}$</td>
</tr>
<tr>
<td>work individually - not applicable</td>
<td>$n_{bw} \times N_{sbr,w} \times [1+(c_2 or c_4) / c] / 4$</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>$x 1.0$ not applicable</td>
</tr>
<tr>
<td>ratio</td>
<td>$0.24$</td>
</tr>
</tbody>
</table>

#### Failure Along Pedestal Depth Edge

- **Tensile load carried by anchors close to edge which may cause side-face blowout**
  - **ACI318 M-08**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{nbd} = N_u \times n_{bd} / n_t$</td>
<td>$26.7$ [kN]</td>
</tr>
<tr>
<td>$c = \min (c_2, c_4)$</td>
<td>$102$ [mm]</td>
</tr>
<tr>
<td>Check if side blowout applicable</td>
<td></td>
</tr>
<tr>
<td>$h_{ef} = 305$ [mm]</td>
<td>A23.3-04 (R2010)</td>
</tr>
<tr>
<td>$&gt; 2.5c$</td>
<td>side blowout is applicable</td>
</tr>
<tr>
<td>Check if edge anchors work as a group or work individually</td>
<td></td>
</tr>
<tr>
<td>$s_{11} = 203$ [mm]</td>
<td>$s = s_1 = 203$ [mm]</td>
</tr>
<tr>
<td>a group or work individually</td>
<td></td>
</tr>
<tr>
<td>$&lt; 6c$</td>
<td>edge anchors work as a group</td>
</tr>
<tr>
<td>Single anchor SB resistance</td>
<td>$N_{sbr,d} = 13.3 c_4 \sqrt{A_{nb}} \bar{\phi}'c \sqrt{R_{t,c}} R_{t,c}$</td>
</tr>
<tr>
<td>Multiple anchors SB resistance</td>
<td></td>
</tr>
<tr>
<td>work as a group - applicable</td>
<td>$(1+s/6c) \times N_{sbr,d}$</td>
</tr>
<tr>
<td>work individually - not applicable</td>
<td>$n_{bd} \times N_{sbr,d} \times [1+(c_1 or c_3) / c] / 4$</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>$x 1.0$ not applicable</td>
</tr>
<tr>
<td>ratio</td>
<td>$0.24$</td>
</tr>
</tbody>
</table>
Group side blowout resistance: $N_{sbgr} = \min \left( \frac{N_{sbgr}}{n_{bgr}}, \frac{N_{ns}}{n_{ns}} \right) = 221.1$ [kN]

Govern Tensile Resistance: $N_r = \min (N_{sr}, N_{br}, N_{cpr}, N_{sbgr}) = 85.5$ [kN]

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear.

Anchor Rod Shear: $V_{sr} = n_s A_s f_u 0.6 f_{u,t} R_v,s = 170.5$ [kN] D.7.1.2 (b) (D-21)

Reduction due to built-up grout pads: $x = 0.8$, applicable $V_r = 136.4$ [kN] D.7.1.3

ratio = 0.13 > $V_u$ OK

Concrete Shear Breakout Resistance:

Mode 1: Failure cone at front anchors, strength check against $0.5 \times V_u$

Mode 3: Failure cone at front anchors, strength check against $1.0 \times V_u$, applicable when oversized holes are used in base plate.

Bolt edge distance: $c_1 = 102$ [mm] D.7.2.4

Limiting $c_{a1}$ when anchors are influenced by 3 or more edges: $c_{a1} = 102$ [mm] D.7.2.4

Bolt edge distance - adjusted: $c_1 = c_{a1}$ needs NOT to be adjusted $c_2 = 153$ [mm] D.7.2.1

$A_v = \min (0.5c_t, h_u) x = 7.0E+04$ [mm$^2$] D.7.2.1

$A_{vo} = 4.5c_t^2 = 4.7E+04$ [mm$^2$] D.7.2.1 (D-24)

$A_v = \min (A_v, n_t A_{vo}) = 7.0E+04$ [mm$^2$] D.7.2.1

$l_u = \min (8d_s, h_u) = 152$ [mm] D.3

$V_{br} = 0.58 \left( \frac{l}{d_s} \right)^{0.2} \sqrt{d_s} \phi_1 \sqrt{l} c_{s,br}^2 R_v,s = 13.5$ [kN] D.7.2.2 (D-25)

Eccentricity effects: $\Psi_{ec,v} = 1.0$ shear acts through center of group D.7.2.5

Edge effects: $\Psi_{ed,v} = \min (0.7+0.3c_t/1.5c_t) = 0.90$ D.7.2.6

Concrete cracking: $\Psi_{c,v} = 1.20$ D.7.2.7

Member thickness: $\Psi_{h,v} = \max (\sqrt{1.5c_t} / h_u, 1.0) = 1.00$ D.7.2.8
Conc shear breakout

- Resistance:
  \[ V_{cbgr1} = \frac{A_{W_a}}{A_{W_a}} \psi_{e,c} \psi_{v,c} \psi_{h,c} \psi_{v,v} V_{ex} = 21.9 \text{ [kN]} \]

- Mode 1 is used for checking:
  \[ V_{cbgr1} = V_{cbg1} \times 2.0 = 43.8 \text{ [kN]} \]

**Mode 2** Failure cone at back anchors

\[ \psi_{e,c}, \psi_{v,c} = 1.0 \text{ shear acts through center of group} \]

\[ \psi_{ed}, \psi_{h,c} = \min\left\{\left(0.7 + 0.3c_{2}/1.5ca_{1}\right), 1.0\right\} = 0.77 \]

\[ \psi_{h,v} = \max\left\{\text{sqrt}(1.5ca_{1}/h_{a}), 1.0\right\} = 1.00 \]

**Concrete cracking**

\[ \psi_{c,v} = 1.20 \text{ D.7.2.7} \]

**Member thickness**

\[ \psi_{h,v} = \max\left\{\text{sqrt}(1.5ca_{1}/h_{a}), 1.0\right\} = 1.00 \text{ D.7.2.8} \]

**Conc shear breakout**

\[ V_{cbgr2} = \frac{A_{W_a}}{A_{W_a}} \psi_{e,c} \psi_{v,c} \psi_{h,c} \psi_{v,v} V_{ex} = 53.7 \text{ [kN]} \]

**Min shear breakout resistance**

\[ V_{cbgr} = \min\left(V_{cbgr1}, V_{cbgr2}\right) = 43.8 \text{ [kN]} \]

Seismic design strength reduction

\[ = x 1.0 \text{ not applicable} = 43.8 \text{ [kN]} \text{ D.4.3.5} \]

**Output**

\[ \text{ratio} = 0.41 > V_u \text{ OK} \]
### Conc. Pryout Shear Resistance

\[ k_{cp} = 2.0 \]

**Factored shear pryout resistance**

\[ V_{cpgr} = k_{cp} N_{cbgr} \quad = 171.0 \text{ [kN]} \]

\[ R_{v,c} = 1.00 \quad \text{pryout strength is always Condition B} \]

**Seismic design strength reduction**

\[ = x 1.0 \quad \text{not applicable} \quad = 171.0 \text{ [kN]} \]

\[ \text{ratio} = 0.10 \quad > V_u \quad \text{OK} \]

### Anchor Rod on Conc Bearing

\[ B_r = n_s x 1.4 \times \phi_c x \min(8d_a, h_{ef}) x d_a x f'_c \quad = 221.9 \text{ [kN]} \]

\[ \text{ratio} = 0.08 \quad > V_u \quad \text{OK} \]

### Govern Shear Resistance

\[ V_r = \min (V_{sr}, V_{cbgr}, V_{cpgr}, B_r) = 43.8 \text{ [kN]} \]

### Tension Shear Interaction

Check if \( N_u > 0.2 N_t \) and \( V_u > 0.2 V_r \)

\[ N_u/N_t + V_u/V_r = 1.03 \]

\[ \text{ratio} = 0.86 \quad < 1.2 \quad \text{OK} \]

### Ductility Tension

\[ N_{s,t} = 303.1 \text{ [kN]} \]

\[ > \min (N_{cbgr}, N_{cpr}, N_{sbgr}) = 85.5 \text{ [kN]} \]

**Non-ductile**

### Ductility Shear

\[ V_{s,t} = 136.4 \text{ [kN]} \]

\[ > \min (V_{cbgr}, V_{cpgr}, B_r) = 43.8 \text{ [kN]} \]

**Non-ductile**
Example 13: Anchor Bolt + No Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code

\[ M_u = 25 \text{ kip-ft} \quad N_u = 10 \text{ kips (Compression)} \quad V_u = 10 \text{ kips} \]

Concrete  \( f'_c = 5 \text{ ksi} \)

Anchor bolt  F1554 Grade 36  1.25" dia  Heavy Hex Head  \( h_{ef} = 16" \quad h_a = 20" \)

Oversized holes in base plate

Seismic design category < C

Supplementary reinforcement  Tension \( \rightarrow \) Condition A

Shear \( \rightarrow \) Condition A  \( \Psi_{CV} = 1.2 \)

Provide built-up grout pad
ANCHOR BOLT DESIGN Combined Tension, Shear and Moment

Anchor bolt design based on

**Code Abbreviation**
- ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D
- PIP STE05121 Anchor Bolt Design Guide-2006

**Assumptions**
1. Concrete is cracked
2. Condition A - supplementary reinforcement provided
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Shear load acts through center of bolt group $\Psi_{ec,V} = 1.0$
5. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
6. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output
7. Shear carried by only half of total anchor bolts due to oversized holes in column base plate

**Anchor Bolt Data**

- Factored moment $M_u = 25.0$ [kip-ft] = 33.9 [kNm]
- Factored tension /compression $N_u = -10.0$ [kips] in compression = -44.5 [kN]
- Factored shear $V_u = 10.0$ [kips] = 44.5 [kN]
- Factored shear for bolt design $V_u = 10.0$ [kips] $V_u = 0$ if shear key is provided

No of bolt line for resisting moment = 3 Bolt Line
No of bolt along outermost bolt line = 3
### Code Reference

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>PIP STE05121</th>
</tr>
</thead>
<tbody>
<tr>
<td>Page A-1 Table 1</td>
<td></td>
</tr>
</tbody>
</table>

#### Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

**Dongxiao Wu P. Eng.**

**Design Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Min Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outermost bolt line spacing ( s_1 )</td>
<td>16.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Outermost bolt line spacing ( s_2 )</td>
<td>16.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Internal bolt line spacing ( s_{b1} )</td>
<td>8.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Internal bolt line spacing ( s_{b2} )</td>
<td>0.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Column depth ( d )</td>
<td>12.7 [in]</td>
<td></td>
</tr>
<tr>
<td>Concrete strength ( f'_c )</td>
<td>5.0 [ksi] = 34.5 [MPa]</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt material</td>
<td>F1554 Grade 36</td>
<td></td>
</tr>
<tr>
<td>Anchor tensile strength ( f_{uta} )</td>
<td>58 [ksi] = 400 [MPa]</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt diameter ( d_a )</td>
<td>1.25 [in] = 31.8 [mm]</td>
<td></td>
</tr>
<tr>
<td>Bolt sleeve diameter ( d_s )</td>
<td>3.0 [in]</td>
<td></td>
</tr>
<tr>
<td>Bolt sleeve height ( h_s )</td>
<td>10.0 [in]</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt embedment depth ( h_{ef} )</td>
<td>16.0 [in] = 15.0</td>
<td></td>
</tr>
<tr>
<td>Concrete thickness ( h_a )</td>
<td>20.0 [in] = 19.0</td>
<td></td>
</tr>
<tr>
<td>Bolt edge distance ( c_1 )</td>
<td>6.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Bolt edge distance ( c_2 )</td>
<td>6.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Bolt edge distance ( c_3 )</td>
<td>100.0 [in]</td>
<td>5.0</td>
</tr>
<tr>
<td>Bolt edge distance ( c_4 )</td>
<td>100.0 [in]</td>
<td>5.0</td>
</tr>
</tbody>
</table>

- \( c_1 > 1.5h_{ef} \) for at least two edges to avoid reducing of \( h_{ef} \) when \( N_e > 0 \)
- Adjusted \( h_{ef} \) for design \( h_{ef}^{\text{adj}} = 16.00 \) [in] = 15.0 | OK | D.5.2.3

---

![Diagram of anchorage design](image-url)
Number of bolt at bolt line 1 \( n_1 = 3 \)
Number of bolt at bolt line 2 \( n_2 = 3 \)
Number of bolt carrying tension \( n_t = 5 \)
Oversized holes in base plate ? \( \text{Yes} \)
Total no of anchor bolt \( n = 8 \)
Number of bolt carrying shear \( n_s = 4 \)

Anchor head type = Heavy Hex
Anchor effective cross sect area \( A_{se} = 0.969 \, \text{[in}^2\text{]} \)
Bearing area of head \( A_{org} = 2.237 \, \text{[in}^2\text{]} \)
Bearing area of custom head \( A_{org} = 3.500 \, \text{[in}^2\text{]} \) not applicable

Bolt 1/8" (3mm) corrosion allowance ? No
Provide shear key ? No
Seismic design category \( \geq C \) ? Yes

Supplementary reinforcement
- For tension \( \Psi_{t,c} = 0.75 \) Cdn-A
- For shear \( \Psi_{v,c} = 0.65 \) Cdn-A

Strength reduction factors
- Anchor reinforcement \( \phi_s = 0.75 \) D.5.2.9 & D.6.2.9
- Anchor rod - ductile steel \( \phi_{t,s} = 0.75 \) \( \phi_{v,s} = 0.65 \) D.4.4 (a)
- Concrete \( \phi_{t,c} = 0.75 \) Cdn-A \( \phi_{v,c} = 0.75 \) Cdn-A D.4.4 (c)

CONCLUSION

<table>
<thead>
<tr>
<th>Anchor Rod Embedment, Spacing and Edge Distance</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall ratio = 0.81</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Tension**

| Anchor Rod Tensile Resistance | ratio = 0.12 | OK |
| Conc. Tensile Breakout Resistance | ratio = 0.39 | OK |
| Anchor Pullout Resistance | ratio = 0.08 | OK |
| Side Blowout Resistance | ratio = 0.13 | OK |

**Shear**

| Anchor Rod Shear Resistance | ratio = 0.14 | OK |
| Conc. Shear Breakout Resistance | ratio = 0.58 | OK |
| Conc. Pryout Shear Resistance | ratio = 0.11 | OK |

**Tension Shear Interaction**

| Tension Shear Interaction | ratio = 0.81 | OK |

**Ductility**

**Seismic Design Requirement**

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required
CALCULATION

Anchor Tensile Force

<table>
<thead>
<tr>
<th>Force (kips)</th>
<th>No of Bolt</th>
</tr>
</thead>
<tbody>
<tr>
<td>(T_1) = 4.86</td>
<td>3</td>
</tr>
<tr>
<td>(T_2) = 2.15</td>
<td>2</td>
</tr>
<tr>
<td>(T_3) = 0.00</td>
<td>0</td>
</tr>
</tbody>
</table>

Sum of bolt tensile force \(N_u = \sum n_i T_i = 18.9\) [kips]

Tensile bolts outer distance \(s_{tb}\) = 8.0 [in]

Eccentricity \(e'N\) -- distance between resultant of tensile load and centroid of anchors loaded in tension
\[e'N = 1.38\] [in] Fig. RD.5.2.4 (b)

Eccentricity modification factor
\[\Psi_{ec,N} = \frac{1}{1 + \frac{2\sigma_n}{3R_{ef}}} = 0.95\]

Anchor Rod Tensile
\[\phi_{t,a} N_{s,a} = \phi_{t,a} A_{s,a} f_{s,a} = 42.2\] [kips] D.5.1.2 (D-3)

Resistance ratio = 0.12 > \(T_1\) OK

Conc. Tensile Breakout Resistance
\[N_b = 24 \sqrt{\frac{h_{ef}}{e}} \] if \(h_{ef} < 11^*\) or \(h_{ef} > 25^*\) = 114.9 [kips] D.5.2.2 (D-7)
\[16 \sqrt{\frac{h_{ef}}{e}} \] if \(11^* \leq h_{ef} \leq 25^*\) D.5.2.2 (D-8)

Projected conc failure area
\[A_{NC} = [s_{tb} + \min(c_1, 1.5h_{ef}) + \min(c_2, 1.5h_{ef})][x] = 1748.0\] [in²]

Min edge distance
\[c_{min} = \min(c_1, c_2, c_3, c_4) = 6.0\] [in]

Eccentricity effects
\[\psi_{ec,N} = 0.95\]

Edge effects
\[\psi_{ed,N} = \min[(0.7 + 0.3 c_{min}/1.5h_{ef}), 1.0] = 0.78\]

Concrete cracking
\[\psi_{c,N} = 1.0\]

Concrete splitting
\[\psi_{cp,N} = 1.0\]

Concrete breakout resistance
\[\phi_{t,c} N_{cbg} = \phi_{t,c} A_{cb} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = 47.9\] [kips] D.5.2.1 (D-5)

Seismic design strength reduction ratio = 0.39 not applicable = 47.9 [kips] D.3.3.3

OK

Anchor Pullout Resistance

Single bolt pullout resistance
\[N_p = 8 A_{ubg} f'_c = 89.5\] [kips] D.5.3.4 (D-15)

Seismic design strength reduction ratio = 0.08 > \(T_1\) OK

\[\psi_{c,p} = 1\] for cracked conc D.5.3.6

\[\phi_{t,c} = 0.70\] pullout strength is always Condition B D.4.4(c)
Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[ N_{buw} = n_{t1} T_1 \]

along pedestal width edge

\[ c = \min \left( c_1, c_3 \right) \]

Check if side blowout applicable

\[ h_{ef} = 16.0 \text{ [in]} \]

> 2.5c side blowout is applicable

Check if edge anchors work as a group

\[ s_{22} = 8.0 \text{ [in]} \]

> 2.5c side blowout is applicable

Check if edge anchors work as a group

\[ s = s_2 = 16.0 \text{ [in]} \]

Single anchor SB resistance

\[ \phi_{t,c} N_{sb} = 14.6 \text{ [kips]} \]

Multiple anchors SB resistance

\[ \phi_{t,c} N_{sb,w} = (1 + s / 6c) \times \phi_{t,c} N_{sb} \]

work as a group - applicable

\[ \phi_{t,c} N_{sb,w} = 110.0 \text{ [kips]} \]

work individually - not applicable

Seismic design strength reduction

\[ = 0.0 \text{ [kips]} \]

\[ \text{ratio} = 0.13 > N_{buw} \]

OK

Group side blowout resistance

\[ \phi_{t,c} N_{sbg} = 183.3 \text{ [kips]} \]

Govern Tensile Resistance

\[ N_r = \min \left( \phi_{t,s} n_t N_{sa}, \phi_{t,c} (N_{cbg}, n_t N_{pn}, N_{sbg}) \right) = 47.9 \text{ [kips]} \]

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear

\[ \phi_{v,s} V_{sa} = 87.7 \text{ [kips]} \]

Resistance

Reduction due to built-up grout pads

\[ = 70.1 \text{ [kips]} \]

Conc. Shear Breakout Resistance

Mode 1 Failure cone at front anchors, strength check against 0.5 x \( V_u \)

Mode 3 Failure cone at front anchors, strength check against 1.0 x \( V_u \), applicable when oversized holes are used in base plate
Bolt edge distance \( c_{a1} = 6.0 \) [in]  
Limiting \( c_{a1} \) when anchors are influenced by 3 or more edges = No

Bolt edge distance - adjusted \( c_{a1} \) needs NOT to be adjusted = \( 6.0 \) [in] 
\( c_2 = 6.0 \) [in]  
\( 1.5c_{a1} = 9.0 \) [in]  
\( A_{VC} = \left[ \min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1}) \right] \times \min(1.5c_{a1}, h_u) \) 
\( A_{VCO} = 4.5c_{a1}^2 \) = \( 162.0 \) [in²]  
\( A_{VC} = \min(A_{VC}, n_1A_{VCO}) = 279.0 \) [in²]  
\( l_e = \min(8d_a, h_{ef}) = 10.0 \) [in]  
\( V_b = 12.3 \) [kips]  

Eccentricity effects \( \Psi_{ec,v} = 1.0 \) shear acts through center of group

Edge effects \( \Psi_{ed,v} = \min(0.7 + 0.3c_2/1.5c_1), 1.0 \) = 0.90

Concrete cracking \( \Psi_{c,v} = 1.20 \)

Member thickness \( \Psi_{h,v} = \max(\sqrt{1.5c_1 / h_u}, 1.0) \) = 1.00

Conc shear breakout resistance \( V_{cbg1} = 17.2 \) [kips]  
Mode 3 is used for checking \( V_{cbg1} \times 1.0 = 17.2 \) [kips] note

Mode 2 Failure cone at back anchors

Bolt edge distance \( c_{a1} = 22.0 \) [in]  
Limiting \( c_{a1} \) when anchors are influenced by 3 or more edges = No

Bolt edge distance - adjusted \( c_{a1} \) needs NOT to be adjusted = \( 22.0 \) [in]  
\( c_2 = 6.0 \) [in]  
\( 1.5c_{a1} = 33.0 \) [in]  

ACI 318-08
Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

Dongxiao Wu P. Eng.

Code Reference

ACI 318-08

\[ A_{VC} = [\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})] x = 1100.0 \text{ [in}^2\text{]} \] D.6.2.1

\[ A_{Vco} = 4.5c_{a1}^2 \] D.6.2.1 (D-23)

\[ A_{VC} = \min (A_{VC}, n_2 A_{Vco}) = 1100.0 \text{ [in}^2\text{]} \] D.6.2.1

\[ l_o = \min(8d_a, h_\text{ef}) = 10.0 \text{ [in]} \] D.6.2.2

\[ V_o = \left[ 7 \left( \frac{1}{d_a} \right)^{0.2} \sqrt{d_a} \right] \lambda \sqrt{c^{1.5}_{a1}} = 86.6 \text{ [kips]} \] D.6.2.2 (D-24)

Eccentricity effects
\[ \Psi_{ec,v} = 1.0 \text{ shear acts through center of group} \] D.6.2.5

Edge effects
\[ \Psi_{ed,v} = \min (0.7+0.3c_2/1.5c_{a1}, 1.0) = 0.75 \] D.6.2.6

Concrete cracking
\[ \Psi_{c,v} = \max (\sqrt{1.5c_{a1} / h_u}, 1.0) = 1.28 \] D.6.2.7

Member thickness
\[ \Psi_{h,v} = x 1.0 \text{ not applicable} \] D.3.3.3

Conc. Pryout Shear Resistance
\[ k_{cp} = 2.0 \] D.6.3

Factored shear pryout resistance
\[ \phi_{\nu,c} V_{cp} = \phi_{\nu,c} k_{cp} N_{cpg} = 89.5 \text{ [kips]} \] D.6.3 (D-31)

Seismic design strength reduction
\[ x 1.0 \text{ not applicable} \] D.3.3.3

Govern Shear Resistance
\[ V_r = \min [\phi_{\nu,s} V_{sa}, \phi_{\nu,c} (V_{cpg} \text{, } V_{cp} \text{, } V_{co})] = 17.2 \text{ [kips]} \]

Tension Shear Interaction
Check if \( N_n > 0.2 \phi N_n \text{ and } V_n > 0.2 \phi V_n \) Yes D.7.1 & D.7.2

\[ N_n / \phi N_n + V_n / \phi V_n = 0.98 \] D.7.3 (D-32)

ratio = 0.81 < 1.2 OK

Ductility Tension
\[ \phi_{\nu,s} N_{sa} = 42.2 \text{ [kips]} \]
\[ < \phi_{\nu,c} \min (N_{cpg} \text{, } N_{cp} \text{, } N_{co}) = 47.9 \text{ [kips]} \]

Ductility Shear
\[ \phi_{\nu,c} V_{sa} = 70.1 \text{ [kips]} \]
\[ > \phi_{\nu,c} \min (V_{cpg} \text{, } V_{cp} \text{, } V_{co}) = 17.2 \text{ [kips]} \]
Example 14: Anchor Bolt + No Anchor Reinf + Tension Shear & Moment + CSA A23.3-04 Code

\[ M_u = 33.9 \text{ kNm} \quad N_u = 44.5 \text{ kN (Compression)} \quad V_u = 44.5 \text{ kN} \]
Concrete \( f'_c = 34.5 \text{ MPa} \)
Anchor bolt \( F1554 \) Grade 36 1.25" dia Heavy Hex Head \( h_{ef} = 406\text{mm} \quad h_a = 508\text{mm} \)
Oversized holes in base plate
Seismic design \( I_E F_a S_a(0.2) < 0.35 \)
Supplementary reinforcement Tension \( \rightarrow \) Condition A
Shear \( \rightarrow \) Condition A \( \Psi_{c,V} = 1.2 \)

Provide built-up grout pad
ANCHOR BOLT DESIGN  Combined Tension, Shear and Moment

Anchor bolt design based on

- CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
- ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
- PIP STE05121 Anchor Bolt Design Guide-2006

**Assumptions**

1. Concrete is cracked
2. Condition A for tension - supplementary reinforcement provided
3. Shear load acts through center of bolt group
   \[ \psi_{ec,V} = 1.0 \]
4. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
5. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output
6. Shear carried by only half of total anchor bolts due to oversized holes in column base plate

**Anchor Bolt Data**

- Factored moment: \( M_u = 33.9 \) [kNm] = 25.0 [kip-ft]
- Factored tension/compression: \( N_u = -44.5 \) [kN] in compression = -10.0 [kips]
- Factored shear: \( V_u = 44.5 \) [kN] = 10.0 [kips]
- Factored shear for bolt design: \( V_u = 44.5 \) [kN] \( V_u = 0 \) if shear key is provided

No of bolt line for resisting moment = 3 Bolt Line
No of bolt along outermost bolt line = 3
Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

Dongxiao Wu P. Eng.

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**Code Reference**

PIP STE05121

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**min required**

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**Adjusted Anchor Bolt Embedment Depth**

$h_{ef} = h_{ef} \cdot 1.25 = 406 \cdot 1.25 = 507.5$ mm

---

**Adjusted Anchor Bolt Embedment Depth**

$h_{ef} = h_{ef} = 406$ mm

---

**Concrete Thickness**

$h_a = 508$ mm

---

**Bolt Edge Distance**

$c_1 = 152$ mm
$c_2 = 152$ mm
$c_3 = 2540$ mm
$c_4 = 2540$ mm

---

**Adjustment for Design**

$h_{ef} = 406$ mm
**Number of bolt at bolt line 1** \( n_1 = 3 \)

**Number of bolt at bolt line 2** \( n_2 = 3 \)

**Total no of anchor bolt** \( n = 8 \)

**Number of bolt carrying tension** \( n_t = 5 \)

**Number of bolt carrying shear** \( n_s = 4 \)

**Oversized holes in base plate?** Yes

**Anchor head type** Heavy Hex

**Bearing area of head** \( A_{bh} = 625 \text{ mm}^2 \)

**Bearing area of custom head** \( A_{bh} = 1443 \text{ mm}^2 \) not applicable

**Bolt 1/8” (3mm) corrosion allowance?** No

**Provide shear key?** No

**Seismic region where \( \frac{i F_a S_a}{0.2} \geq 0.35 \)?** D.4.3.5

**Supplementary reinforcement**

For tension

\[ \gamma_{ci} = 1.2 \text{ Condition A} \]

For shear

\[ \gamma_{s1} = 1.2 \text{ Condition A} \]

**Provide built-up grout pad?** Yes

**Strength reduction factors**

**Anchor reinforcement factor** \( \phi_{as} = 0.75 \)

**Steel anchor resistance factor** \( \phi_s = 0.85 \)

**Concrete resistance factor** \( \phi_c = 0.65 \)

**Resistance modification factors**

**Anchor rod - ductile steel** \( R_{tc} = 0.80 \)

**Concrete** \( R_{tc} = 1.15 \text{ Cdn-A} \)

**CONCLUSION**

**Anchor Rod Embedment, Spacing and Edge Distance**

Overall ratio = 0.81 OK

**Tension**

Anchor Rod Tensile Resistance ratio = 0.13 OK

Conc. Tensile Breakout Resistance ratio = 0.39 OK

Anchor Pullout Resistance ratio = 0.08 OK

Side Blowout Resistance ratio = 0.13 OK

**Shear**

Anchor Rod Shear Resistance ratio = 0.15 OK

Conc. Shear Breakout Resistance ratio = 0.58 OK

Conc. Pryout Shear Resistance ratio = 0.12 OK

Anchor Rod on Conc Bearing ratio = 0.04 OK

**Tension Shear Interaction**

Tension Shear Interaction ratio = 0.81 OK

**Ductility**

Tension Ductile OK

Shear Non-ductile OK

**Seismic Design Requirement**

IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 duality requirement is NOT required

D.4.3.6
### CALCULATION

#### Anchor Tensile Force

**Single bolt tensile force**
- $T_1 = 21.6$ [kN]  
  No of bolt for $T_1$ $n_{T1} = 3$
- $T_2 = 9.6$ [kN]  
  No of bolt for $T_2$ $n_{T2} = 2$
- $T_3 = 0.0$ [kN]  
  No of bolt for $T_3$ $n_{T3} = 0$

**Sum of bolt tensile force**

\[
N_u = \sum n_i T_i = 83.9 \text{ [kN]} 
\]

**Tensile bolts outer distance $s_{tb}$**

$s_{tb} = 203$ [mm]

**Eccentricity $e'_N$—distance between resultant of tensile load and centroid of anchors loaded in tension**

$e'_N = 35$ [mm]  
Figure D.8 (b)

**Eccentricity modification factor**

\[
\Psi_{ec,N} = \frac{1}{1 + \frac{2e_n}{3h_w}} = 0.95 \quad \text{D.6.2.4 (D-9)}
\]

**Anchor Rod Tensile**

$N_{fr} = A_{se} \phi f_{ub}R_{ct,s} = 170.0$ [kN]  
**Resistance ratio** = 0.13 > $T_1$  
**OK**

#### Conc. Tensile Breakout Resistance

\[
N_{fr} = 10 \phi \sqrt[3]{h_w^{0.5}} R_{ct} \text{ if } h_w \leq 275 \text{ or } h_w \geq 625
\]

\[
3.9 \phi \sqrt[3]{h_w^{0.5}} R_{ct} \text{ if } 275 < h_w < 625
\]

$N_{fr} = 382.8$ [kN]  
**D.6.2.2 (D-7)**

**Projector conc failure area**

- $h_{ef} = 609$ [mm]
- $A_{No} = [s_{tb} + \min(c_1, 1.5h_{w}) + \min(c_3, 1.5h_{w})] 
  \times [s_{tb} + \min(c_2, 1.5h_{w}) + \min(c_4, 1.5h_{w})]$  
  $= 1.1E+06$ [mm$^2$]  
- $A_{Nco} = 9 h_{ef}^2 = 1.5E+06$ [mm$^2$]  
- $A_{Nc} = \min(A_{No}, A_{Nco}) = 1.1E+06$ [mm$^2$]  
**D.6.2.1**

**Min edge distance**

$c_{min} = \min(c_1, c_2, c_3, c_4) = 152$ [mm]

**Eccentricity effects**

$\Psi_{ec,N} = 0.95$  
**D.6.2.4 (D-9)**

**Edge effects**

$\Psi_{ed,N} = \min(0.7 + 0.3c_{min}/1.5h_{w}), 1.0 \ } = 0.78$  
**D.6.2.5**

**Concrete cracking**

$\Psi_{c,N} = 1.0$ for cracked concrete  
**D.6.2.6**

**Concrete splitting**

$\Psi_{cp,N} = 1.0$ for cast-in anchor  
**D.6.2.7**

**Concrete breakout resistance**

\[
N_{dfr} = \frac{A_{No}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_{fr} = 213.0 \text{ [kN]} \quad \text{D.6.2.1 (D-5)}
\]

**Seismic design strength reduction**

- $x \times 1.0$ not applicable  
- $= 213.0$ [kN]  
**D.4.3.5**

**Anchor Pullout Resistance**

**Single bolt pullout resistance**

$N_{pr} = 8 A_{sb} \phi f_{u} R_{ct,c} = 261.2$ [kN]  
**D.6.3.4 (D-16)**

$N_{dpr} = \Psi_{cp} N_{pr} = 261.2$ [kN]  
**D.6.3.1 (D-15)**

**Seismic design strength reduction**

$\times x \times 1.0$ not applicable  
$= 261.2$ [kN]  
**D.4.3.5**

**Anchor pullout strength is always Condition B**

- $R_{ct,c} = 1.00$  
- $\Psi_{cp} = 1$ for cracked conc  
- $\Psi_{c,p} = 1$ for cracked conc  
- $\Psi_{cp} = 1$ for cracked conc  
- $\Psi_{c,p} = 1$ for cracked conc  
- $\Psi_{cp} = 1$ for cracked conc  
- $\Psi_{c,p} = 1$ for cracked conc  
**D.5.4(c)**
Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[ N_{b\text{uw}} = n_{T1} T_1 \]

\[ c = \min (c_1, c_3) \]

Check if side blowout applicable

\[ h_{nf} = 406 \text{ [mm]} \] side bowout is applicable

Check if edge anchors work as a group or work individually

\[ s_{22} = 203 \text{ [mm]} \]

\[ s = s_2 = 406 \text{ [mm]} \]

Single anchor SB resistance

\[ N_{sbr, w} = 13.3 c_{\sqrt{A_{org}}} \phi_{\text{f}} \sqrt{T_1} R_t \]

Multiple anchors SB resistance

work as a group - applicable

\[ N_{sbgr, w} = (1+s/6c) \times N_{sbr, w} \]

work individually - not applicable

\[ n_{bw} \times N_{sbr, w} \times [1+(c_2 \text{ or } c_4)/c] / 4 \]

Seismic design strength reduction

\[ \text{ratio} = 0.13 \]

\[ N_{sbgr, w} \]

Group side blowout resistance

\[ N_{sbgr} = \frac{n_{sbr}}{n_{T1}} \]

Govern Tensile Resistance

\[ N_r = \min (n_1, N_{b\text{uw}}, n_1 N_{sbr}, n_1 N_{sbgr}) = 213.0 \text{ [kN]} \]

Note: Anchor bolt sleeve portion must be tape wrapped and grouted to resist shear

Anchor Rod Shear

\[ V_{sr} = n_s A_{as} \phi_s 0.6 f_{\text{ut}} R_v \]

Resistance

Reduction due to built-up grout pads

\[ \text{ratio} = 0.8, \text{ applicable} \]

\[ V_r = 306.0 \text{ [kN]} \]

Conc. Shear Breakout Resistance

Mode 1 Failure cone at front anchors, strength check against 0.5 x \( V_u \)

Mode 3 Failure cone at front anchors, strength check against 1.0 x \( V_u \), applicable when oversized holes are used in base plate
**Code Reference**

Bolt edge distance

\[ c_1 = 152 \text{ [mm]} \]

Limiting \( c_{ca} \) when anchors are influenced by 3 or more edges

\[ c_{ca} = \text{No} \]

Bolt edge distance - adjusted

\[ c_{a1} = \text{ca1 needs NOT to be adjusted} \]

\[ c_2 = 152 \text{ [mm]} \]

\[ 1.5c_1 = 229 \text{ [mm]} \]

\[ A_{VC} = \left[ \min(c_{2}, 1.5c_1) + s_2 + \min(c_{4}, 1.5c_1) \right] \times \min(1.5c_1, h_a) \]

\[ A_{VCO} = 4.5c_1^2 \]

\[ A_{VC} = \min(A_{VCO}, n_1A_{VCO}) \]

\[ l_a = \min(8d_a, h_{ef}) \]

\[ c_2 = 152 \text{ [mm]} \]

\[ 1.5c_1 = 229 \text{ [mm]} \]

\[ c_v = \min(c_{2}, 1.5c_1) + s_2 + \min(c_{4}, 1.5c_1) \times 1.8 \times 10^5 \text{ [mm}^2\] \]

\[ c_v = 4.5c_1^2 \]

\[ A_{VCO} = 4.5c_1^2 \]

\[ A_{VC} = \min(A_{VCO}, n_1A_{VCO}) \]

\[ l_a = \min(8d_a, h_{ef}) \]

\[ 0.58 \left( \frac{l_a^2}{d_a} \right)^{1/2} \sqrt{\frac{d_a}{h_a}} \sqrt{c_{sa}^4 R_{vc}} = 41.1 \text{ [kN]} \]

Eccentricity effects

\[ \Psi_{e,v} = 1.0 \text{ shear acts through center of group} \]

Edge effects

\[ \Psi_{e,v} = \min\left[ (0.7+0.3c_2/1.5c_1), 1.0 \right] = 0.90 \]

Concrete cracking

\[ \Psi_{e,v} = 1.20 \]

Member thickness

\[ \Psi_{h,v} = \max\left[ \sqrt{1.5c_1/h_a}, 1.0 \right] = 1.00 \]

Conc shear breakout resistance

\[ V_{cbgr1} = \frac{A_{VC}}{A_{VCO}} \Psi_{e,v} \Psi_{e,v} \Psi_{c,v} \Psi_{h,v} V_{br} = 76.4 \text{ [kN]} \]

Mode 3 is used for checking

\[ V_{cbgr1} = V_{cbgr1} \times 1.0 = 76.4 \text{ [kN]} \]

**Mode 2** Failure cone at back anchors

\[ c_{sa} = c_1 + s_1 = 558 \text{ [mm]} \]

Limiting \( c_{sa} \) when anchors are influenced by 3 or more edges

\[ c_{sa} = \text{No} \]

Bolt edge distance - adjusted

\[ c_{sa} = \text{ca1 needs NOT to be adjusted} \]

\[ c_2 = 152 \text{ [mm]} \]

\[ 1.5c_{sa} = 838 \text{ [mm]} \]
### Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

**Dongxiao Wu P. Eng.**

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<tr>
<td>A23.3-04 (R2010)</td>
<td>$A_{vc} = \left[ \min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a4}) \right] \times \min(1.5c_{a1}, h_u) = 7.1E+05$ [mm²]</td>
<td></td>
</tr>
<tr>
<td>D.7.2.1</td>
<td>$A_{vco} = 4.5c_{a1}^2 = 1.4E+06$ [mm²]</td>
<td></td>
</tr>
<tr>
<td>D.7.2.1 (D-24)</td>
<td>$A_v = \min\left( A_{vc}, n_2 A_{vco} \right) = 7.1E+05$ [mm²]</td>
<td></td>
</tr>
<tr>
<td>D.7.2.1</td>
<td>$l_0 = \min(8d_a, h_{ef}) = 254$ [mm]</td>
<td></td>
</tr>
<tr>
<td>D.3</td>
<td>$V_{cr} = 0.58 \left( \frac{1}{d_a} \right)^{0.2} \sqrt{d_a} \phi_t \sqrt{f_t} c_{v1} R_{v,c} = 288.2$ [kN]</td>
<td></td>
</tr>
<tr>
<td>D.7.2.2 (D-25)</td>
<td>$7E+05$ [mm²]</td>
<td></td>
</tr>
</tbody>
</table>

**Eccentricity effects**
- $\Psi_{ec,v} = 1.0$ shear acts through center of group
- D.7.2.5

**Edge effects**
- $\Psi_{ed,v} = \min\left( 0.7 + 0.3c_2/1.5c_{a1}, 1.0 \right) = 0.75$
- D.7.2.6

**Concrete cracking**
- $\Psi_{c,v} = 1.20$
- D.7.2.7

**Member thickness**
- $\Psi_{h,v} = \max\left( \text{sqrt}(1.5c_{a1}/h_u), 1.0 \right) = 1.28$
- D.7.2.8

**Conc shear breakout**
- $V_{dgr2} = \frac{A_{vc}}{A_{vco}} \Psi_{ec,v} \Psi_{at,v} \Psi_{c,v} \Psi_{h,v} V_{sr} = 169.4$ [kN]  
- D.7.2.1 (D-23)

**Min shear breakout resistance**
- $V_{dgr} = \min\left( V_{dgr1}, V_{dgr2} \right) = 76.4$ [kN]

**Seismic design strength reduction**
- $V_{u} = 76.4$ [kN]  
- D.4.3.5

**ratio**
- $0.58 > V_u \quad \text{OK}$

**Conc. Pryout Shear Resistance**
- $k_{cp} = 2.0$
- D.7.3

**Factored shear pryout resistance**
- $V_{cpgr} = k_{cp} N_{cpgr} = 370.4$ [kN]  
- D.7.3 (D-32)

**Seismic design strength reduction**
- $R_{v,c} = 1.00$ pryout strength is always Condition B
- D.5.4(c)

**ratio**
- $0.12 > V_u \quad \text{OK}$

**Anchor Rod on Conc Bearing**
- $B_r = n_r x 1.4 x d_0 x \min(8d_a, h_{ef}) x d_a x f_t' = 1021.5$ [kN]  
- CSA S16-09

**Govern Shear Resistance**
- $V_r = \min\left( V_{sr}, V_{cbgr}, V_{cpgr}, B_r \right) = 76.4$ [kN]

**Tension Shear Interaction**
- $A23.3-04$ (R2010)

**Check if $N_r > 0.2 N$ and $V_r > 0.2 V_r$**
- Yes
- D.8.2 & D.8.3

**ratio**
- $0.81 < 1.2 \quad \text{OK}$

**Ductility Tension**
- $N_{sr} = 170.0$ [kN]
- $< \min\left( N_{cbgr}, N_{cpgr}, N_{dgr} \right) = 213.0$ [kN]
- Ductile

**Ductility Shear**
- $V_{sr} = 306.0$ [kN]
- $> \min\left( V_{cbgr}, V_{cpgr}, B_r \right) = 76.4$ [kN]
- Non-ductile
**Example 21: Welded Stud + Anchor Reinft + Tension & Shear + ACI 318-08 Code**

N_t = 20 kips (Tension)  \quad V_u = 25 kips

Concrete \quad f_c' = 4 ksi  \quad Rebar \quad f_y = 60 ksi

Pedestal size \quad 16" x 16"

Anchor stud \quad AWS D1.1 Grade B \quad 1.0" dia \quad h_{ef} = 55" \quad h_a = 60"

Seismic design category \geq C

Anchor reinforcement
- Tension \rightarrow 8-No 8 ver. bar
- Shear \rightarrow 2-layer, 4-leg No 4 hor. bar

No built-up grout pad for embedded plate.

**Note:** The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment \( h_{ef} \) is required for anchor reinforcement to develop resistance on both sides of the failure plane.
STUD ANCHOR DESIGN  Combined Tension and Shear

Anchor bolt design based on ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D

Pipe STE05121 Anchor Bolt Design Guide-2006

Assumptions

1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Anchor reinft strength is used to replace concrete tension/shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
6. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

Input Data

set N_u = 0 if it's compression

Factored tension

\[ N_u = 20.0 \text{ [kips]} = 89.0 \text{ [kN]} \]

Factored shear

\[ V_u = 25.0 \text{ [kips]} = 111.2 \text{ [kN]} \]

Concrete strength

\[ f_c' = 4.0 \text{ [ksi]} = 27.6 \text{ [MPa]} \]

Stud material

AWS D1.1 Grade B

Stud tensile strength

\[ f_{uta} = 65 \text{ [ksi]} = 448 \text{ [MPa]} \]

Stud diameter

\[ d_a = 1 \text{ [in]} = 25.4 \text{ [mm]} \]

Stud shank area

\[ A_{se} = 0.79 \text{ [in}^2\text{]} = 507 \text{ [mm}^2\text{]} \]

Stud head bearing area

\[ A_{brg} = 1.29 \text{ [in}^2\text{]} = 831 \text{ [mm}^2\text{]} \]

Stud embedment depth

\[ h_{ef} = 55.0 \text{ [in]} = 12.0 \]

Stud embedment depth

\[ h_{ef} = 55.0 \text{ [in]} = 12.0 \]

OK

Page A -1 Table 1

Pedestal height

\[ h = 60.0 \text{ [in]} = 58.0 \]

Pedestal width

\[ b_c = 16.0 \text{ [in]} \]

Pedestal depth

\[ d_c = 16.0 \text{ [in]} \]
Stud edge distance $c_1 = 5.0$ [in] $4.5$ OK Code Reference
Stud edge distance $c_2 = 5.0$ [in] $4.5$ OK PIP STE05121
Stud edge distance $c_3 = 5.0$ [in] $4.5$ OK Page A-1 Table 1
Stud edge distance $c_4 = 5.0$ [in] $4.5$ OK

Outermost stud line spacing $s_1 = 6.0$ [in] $4.0$ OK Page A-1 Table 1
Outermost stud line spacing $s_2 = 6.0$ [in] $4.0$ OK

To be considered effective for resisting anchor tension, vertical reinforcing bars shall be located within $0.5h_{ef}$ from the outmost stud's centerline. In this design $0.5h_{ef}$ value is limited to 8 in.

$$0.5h_{ef} = 8.0$$ [in]

No of vertical rebar that are effective for resisting anchor tension

$$n_v = 8$$

Ver. bar size No. $8 \downarrow$ 1.000 [in] dia single bar area $A_s = 0.79$ [in$^2$]

To be considered effective for resisting anchor shear, horizontal reinforcement shall be located within $\min(0.5c_1, 0.3c_2)$ from the outmost stud's centerline

$$\min(0.5c_1, 0.3c_2) = 1.5$$ [in]

No of tie legs that are effective to resist anchor shear

$$n_{leg} = 4$$

No of tie layers that are effective to resist anchor shear

$$n_{lay} = 2$$

Hor. tie bar size No. $4 \downarrow$ 0.500 [in] dia single bar area $A_s = 0.20$ [in$^2$]

For anchor reinf shear breakout strength calc

100% hor. tie bars develop full yield strength

Rebar yield strength

$$f_y = 60$$ [ksi] $60 = 414$ [MPa]

Total no of welded stud

$$n = 4$$

Number of stud carrying tension

$$n_t = 4$$

Number of stud carrying shear

$$n_s = 4$$

For side-face blowout check use

No of stud along width edge

$$n_{bw} = 2$$

No of stud along depth edge

$$n_{bd} = 2$$

Seismic design category $\geq C$

Yes

Provide built-up grout pad?

No

Strength reduction factors

Anchor reinforcement

$$\phi_s = 0.75$$

Anchor rod - ductile steel

$$\phi_{rs} = 0.75 \quad \phi_{vs} = 0.65$$

Concrete - condition A

$$\phi_{cc} = 0.75 \quad \phi_{vc} = 0.75$$

ACI 318-08

D.3.3.3

D.6.1.3

D.5.2.9 & D.6.2.9

D.4.4(a)

D.4.4(c)
CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance  OK
Min Rquired Anchor Reinf. Development Length  ratio = 0.25  OK
Overall  ratio = 0.60  OK

Tension

Stud Tensile Resistance  ratio = 0.13  OK
Anchor Reinf Tensile Breakout Resistance  ratio = 0.09  OK
Stud Pullout Resistance  ratio = 0.23  OK
Side Blowout Resistance  ratio = 0.26  OK

Shear

Stud Shear Resistance  ratio = 0.19  OK
Anchor Reinf Shear Breakout Resistance
  Strut Bearing Strength  ratio = 0.59  OK
  Tie Reinforcement  ratio = 0.46  OK

Conc. Pryout Not Govern When h_d >= 12d

Tension Shear Interaction

Tension Shear Interaction  ratio = 0.60  OK

Seismic Design Requirement

Non-ductile

D.3.3.4
SDC>= C, ACI318-08 D.3.3.5 or D.3.3.6 must be satisfied for non-ductile design

Calculation

Stud Tensile Resistance

\[ \phi_{ts} N_{sa} = \phi_{ts} n_c A_{se} f_{tsa} \]
\[ = 153.2 \text{ [kips]} \]
\[ \text{ratio} = 0.13 > N_u \text{ [kips]} \]
\[ \text{OK} \]

Anchor Reinf Tensile Breakout Resistance

Min tension development length

\[ l_d = \text{[in]} \] 12.2.1, 12.2.2, 12.2.4

for ver. #8 bar

Actual development length

\[ l_a = h_d - c (2 \text{ in}) - 8 \text{ in} \times \tan 35^\circ \]
\[ = 47.4 \text{ [in]} \]
\[ > 12.0 \text{ [in]} \]
\[ \text{OK} \] 12.2.1

Conc. Pryout Not Govern When h_d >= 12d

Seismic design strength reduction

\[ x 0.75 \text{ applicable} \]
\[ = 213.1 \text{ [kips]} \]
\[ \text{D.3.3.3} \]
\[ \text{OK} \]
**Stud Pullout Resistance**

**Single bolt pullout resistance**

\[
N_p = 8 A_{drg} f'_c \]

\[= 41.2 \text{ [kips]} \quad \text{ACI 318-08} \quad D.5.3.4 \ (D-15) \]

Seismic design strength reduction

\[= x \times 0.75 \quad \text{applicable} \quad = 86.6 \text{ [kips]} \quad \text{D.3.3.3} \]

Code Reference

\[\Psi_{c,p} = 1 \text{ for cracked conc} \quad D.5.3.6 \]

\[\phi_{t,c} = 0.70 \quad \text{pullout strength is always Condition B} \quad D.4.4(c) \]

**Side Blowout Resistance**

**Failure Along Pedestal Width Edge**

Tensile load carried by anchors close to edge which may cause side-face blowout

\[N_{bw} = N_u \times n_{wb} / n_t \]

\[= 10.0 \text{ [kips]} \quad \text{RD.5.4.2} \]

- Check if side blowout applicable
  - \[h_{bf} = 55.0 \text{ [in]} \]
  - \[> 2.5c \text{ side blowout is applicable} \quad D.5.4.1 \]

- Check if edge anchors work as a group or work individually
  - \[s_{22} = 6.0 \text{ [in]} \]
  - \[s = s_2 = 6.0 \text{ [in]} \]

**Single anchor SB resistance**

\[\phi_{t,c} N_{sb} = \phi_{t,c} \left(160 c \sqrt{A_{drg}} \right) \lambda \sqrt{f'_{c}} \]

\[= 43.1 \text{ [kips]} \quad \text{D.5.4.1 (D-17)} \]

**Multiple anchors SB resistance**

- work as a group - applicable
  - \[= (1+s/6c) \times \phi_{t,c} N_{sb} \]
  - \[= 51.7 \text{ [kips]} \quad \text{D.5.4.2 (D-18)} \]

- work individually - not applicable
  - \[= n_{wb} \times \phi_{t,c} N_{sb} \times [1+(c_2 \text{ or } c_4) / c] / 4 \]
  - \[= 0.0 \text{ [kips]} \quad \text{D.5.4.1} \]

Seismic design strength reduction

\[= x \times 0.75 \quad \text{applicable} \quad = 38.8 \text{ [kips]} \quad \text{D.3.3.3} \]

**Failure Along Pedestal Depth Edge**

Tensile load carried by anchors close to edge which may cause side-face blowout

\[N_{bud} = N_u \times n_{bd} / n_t \]

\[= 10.0 \text{ [kips]} \quad \text{RD.5.4.2} \]

- Check if side blowout applicable
  - \[h_{bf} = 55.0 \text{ [in]} \]
  - \[> 2.5c \text{ side blowout is applicable} \quad D.5.4.1 \]

- Check if edge anchors work as a group or work individually
  - \[s_{11} = 6.0 \text{ [in]} \]
  - \[s = s_1 = 6.0 \text{ [in]} \]

**Single anchor SB resistance**

\[\phi_{t,c} N_{sb} = \phi_{t,c} \left(160 c \sqrt{A_{drg}} \right) \lambda \sqrt{f'_{c}} \]

\[= 43.1 \text{ [kips]} \quad \text{D.5.4.1 (D-17)} \]

**Multiple anchors SB resistance**

- work as a group - applicable
  - \[= (1+s/6c) \times \phi_{t,c} N_{sb} \]
  - \[= 51.7 \text{ [kips]} \quad \text{D.5.4.2 (D-18)} \]

- work individually - not applicable
  - \[= n_{bd} \times \phi_{t,c} N_{sb} \times [1+(c_1 \text{ or } c_3) / c] / 4 \]
  - \[= 0.0 \text{ [kips]} \quad \text{D.5.4.1} \]

Seismic design strength reduction

\[= x \times 0.75 \quad \text{applicable} \quad = 38.8 \text{ [kips]} \quad \text{D.3.3.3} \]

**Group side blowout resistance**

\[\phi_{t,c} N_{sb} = \phi_{t,c} \min \left( N_{sbw} n_t \frac{n_{wb}}{n_{td}} \right) \]

\[= 77.5 \text{ [kips]} \quad \text{[kips]} \]

**Govern Tensile Resistance**

\[N_r = \phi_{t,c} \min \left( N_p, N_{bw}, N_{cp}, N_{sbw} \right) \]

\[= 77.5 \text{ [kips]} \quad \text{[kips]} \]
**Stud Shear Resistance**

\[ \phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta} = 132.7 \text{ [kips]} \]

Reduction due to built-up grout pads = x 1.0 , not applicable

\[ V_u = 132.7 \text{ [kips]} \]

**Anchor Reinf Shear Breakout Resistance**

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

STM strength reduction factor \( \phi_st = 0.75 \)

**Strut-and-Tie model geometry**

- \( d_v = 2.250 \text{ [in]} \)
- \( d_h = 2.250 \text{ [in]} \)
- \( \theta = 45^\circ \)
- \( d_t = 3.182 \text{ [in]} \)

**Strut compression force**

\[ C_s = 0.5 \frac{V_u}{\sin \theta} = 17.7 \text{ [kips]} \]

**Strut Bearing Strength**

Strut compressive strength

\[ f_{oa} = 0.85 f_c = 3.4 \text{ [ksi]} \]

**Anchor bearing length**

\[ l_b = \min(8d_v, h_{ut}) = 8.0 \text{ [in]} \]

**Anchor bearing area**

\[ A_{org} = l_b \times d_v = 8.0 \text{ [in}^2\text{]} \]

**Anchor bearing resistance**

\[ C_r = n_s \times f_{oa} \times A_{org} = 81.6 \text{ [kips]} \]

**Ver bar bearing area**

\[ A_{org} = (l_b + 1.5 \times d_t - d_v/2 - d_h/2) \times d_v = 11.8 \text{ [in}^2\text{]} \]

**Ver bar bearing resistance**

\[ C_r = \phi_{st} \times f_{oa} \times A_{org} = 30.0 \text{ [kips]} \]

**Strut-and-Tie model geometry**

- \( d_v = 2.250 \text{ [in]} \)
- \( d_h = 2.250 \text{ [in]} \)
- \( \theta = 45^\circ \)
- \( d_t = 3.182 \text{ [in]} \)

**Strut compression force**

\[ C_s = 0.5 \frac{V_u}{\sin \theta} = 17.7 \text{ [kips]} \]

**Calculations**

- Stud Shear Resistance: \( \phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta} = 132.7 \text{ [kips]} \)
- Anchor Reinf Shear Breakout Resistance: STM strength reduction factor \( \phi_{st} = 0.75 \)
- Strut-and-Tie model geometry
- Strut compression force: \[ C_s = 0.5 \frac{V_u}{\sin \theta} = 17.7 \text{ [kips]} \]
- Strut Bearing Strength: Strut compressive strength \[ f_{oa} = 0.85 f_c = 3.4 \text{ [ksi]} \]
- Anchor bearing length
- Anchor bearing area
- Anchor bearing resistance
- Ver bar bearing area
- Ver bar bearing resistance
Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength $f_y$. Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook $T_h$
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar $n = n_{leg} (\text{leg}) \times n_{lay} (\text{layer}) = 8$

ACI 318-08

Pull out resistance at hook

\[ T_h = \phi_{t,c} 0.9 f'_c e_h d_a \]
\[ e_h = 4.5 d_b \]
\[ = 3.0 \text{ [kips]} \quad \text{D.5.3.5 (D-16)} \]
\[ = 2.250 \text{ [in]} \]

Single tie bar tension resistance

\[ T_r = \phi_s f_y A_s \]
\[ = 9.0 \text{ [kips]} \]

Total tie bar tension resistance

\[ V_{rb} = 1.0 x n x T_r \]
\[ = 72.0 \text{ [kips]} \]

Seismic design strength reduction

\[ \text{ratio} = 0.46 \]
\[ > V_u \quad \text{OK} \quad \text{D.3.3.3} \]

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} \geq 12d_a$, the pryout failure will not govern

\[ 12d_a = 12.0 \quad \text{[in]} \]
\[ h_{ef} = 55.0 \quad \text{[in]} \]
\[ > 12d_a \quad \text{OK} \]

Govern Shear Resistance

\[ V_r = \min \left( \phi_{v,s} V_{sa}, V_{rb} \right) = 54.0 \quad \text{[kips]} \]

Tension Shear Interaction

Check if $N_u > 0.2 N_t$ and $V_u > 0.2 V_n$

\[ N_u / N_t + V_u / V_n = 0.72 \quad \text{D.7.3 (D-32)} \]
\[ \text{ratio} = 0.60 \]
\[ < 1.2 \quad \text{OK} \]

Ductility Tension

\[ \phi_{t,s} N_{ta} = 153.2 \quad \text{[kips]} \]
\[ > \min \left[ N_{rb}, \phi_{t,c} \left( N_{pn}, N_{snlq} \right) \right] = 77.5 \quad \text{[kips]} \]

Non-ductile

Ductility Shear

\[ \phi_{t,s} N_{sa} = 132.7 \quad \text{[kips]} \]
\[ > V_{rb} \quad \text{Non-ductile} \]

\[ = 54.0 \quad \text{[kips]} \]
Example 22: Welded Stud + Anchor Reinft + Tension & Shear + CSA A23.3-04 Code

\[ N_u = 89 \text{ kN (Tension)} \quad V_u = 111.2 \text{ kN} \]

Concrete  \( f'_c = 27.6 \text{ MPa} \)  
Rebar  \( f_y = 414 \text{ MPa} \)

Pedestal size  406mm x 406mm

Anchor stud  AWS D1.1 Grade B  1.0” dia  
\( h_{ef} = 1397 \text{ mm} \)  
\( h_a = 1524 \text{ mm} \)

Seismic design  \( I_c F_n S_a(0.2) \geq 0.35 \)

Anchor reinforcement  
Tension \( \rightarrow \) 8-25M ver. bar  
Shear \( \rightarrow \) 2-layer, 4-leg 15M hor. bar  

No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment \( h_{ef} \) is required for anchor reinforcement to develop resistance on both sides of the failure plane.
STUD ANCHOR DESIGN

Combined Tension and Shear

Anchor bolt design based on

- CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
- ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
- PIP STE05121 Anchor Bolt Design Guide-2006

**Code Reference**

- CSA-A23.3-04 (R2010)
- ACI318 M-08
- PIP STE05121

**Assumptions**

1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (50mm from TOC and 2x75mm after) are effective
5. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft

**Input Data**

- Factored tension
  - $N_u = 89.0$ [kN] = 20.0 [kips]
- Factored shear
  - $V_u = 111.2$ [kN] = 25.0 [kips]
- Concrete strength
  - $f'_c = 28$ [MPa] = 4.0 [ksi]
- Stud material
  - AWS D1.1 Grade B
- Stud tensile strength
  - $f_{uta} = 65$ [ksi] = 448 [MPa] $A23.3-04$ (R2010)
- Stud diameter
  - $d_a = 1$ [in] = 25.4 [mm]
- Stud shank area
  - $A_{so} = 0.79$ [in$^2$] = 507 [mm$^2$]
- Stud head bearing area
  - $A_{brg} = 1.29$ [in$^2$] = 831 [mm$^2$]

**Anchor bolt embedment depth**

- $h_{ef} = 1397$ [mm] = 305 [OK]

**Pedestal dimensions**

- Height $h = 1524$ [mm] = 1473 [OK]
- Width $b_c = 406$ [mm]
- Depth $d_c = 406$ [mm]

**Ver. Reinft For Tension**

**Hor. Ties For Shear - 4 Legs**

**Hor. Ties For Shear - 2 Legs**
Stud edge distance $c_1 = 127$ [mm] 115  OK  Code Reference
Stud edge distance $c_2 = 127$ [mm] 115  OK  PIP STE05121
Stud edge distance $c_3 = 127$ [mm] 115  OK  Page A-1 Table 1
Stud edge distance $c_4 = 127$ [mm] 115  OK

Outermost stud line spacing $s_1 = 152$ [mm] 102  OK  Page A-1 Table 1
Outermost stud line spacing $s_2 = 152$ [mm] 102  OK

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within $0.5\text{h}_{ef}$ from the outmost anchor's centerline. In this design $0.5\text{h}_{ef}$ value is limited to 200mm.

$0.5\text{h}_{ef} = 200$ [mm]

No of ver. rebar that are effective for resisting anchor tension $n_v = 8$

Ver. bar size $d_v = 25$  single bar area $A_v = 500$ [mm²]

To be considered effective for resisting anchor shear, hor. reinft shall be located within $\min(0.5c_1, 0.3c_2)$ from the outmost anchor's centerline $\min(0.5c_1, 0.3c_2) = 38$ [mm]

No of tie leg that are effective to resist anchor shear $n_{leg} = 4$

No of tie layer that are effective to resist anchor shear $n_{lay} = 2$

Hor. bar size $d_h = 15$  single bar area $A_h = 200$ [mm²]

For anchor reinft shear breakout strength calc $\text{100% hor. tie bars develop full yield strength}$

Rebar yield strength $f_y = 414$ [MPa] 400 = 60.0 [ksi]

Total no of welded stud $n = 4$
No of stud carrying tension $n_t = 4$
No of stud carrying shear $n_s = 4$

For side-face blowout check use
No of stud along width edge $n_{bw} = 2$
No of stud along depth edge $n_{bd} = 2$

Seismic region where $I_F S_p (0.2) = 0.35$ $\text{Yes}$ $\text{No}$$\text{No Input for Side-Face Blowout Check Use}$

Provide built-up grout pad $\text{Yes}$ $\text{No}$

Strength reduction factors
Anchor reinforcement factor $\phi_a = 0.75$
Steel anchor resistance factor $\phi_s = 0.85$ 8.4.3 (a)
Concrete resistance factor $\phi_c = 0.65$ 8.4.2

Resistance modification factors
Anchor rod - ductile steel $R_{ts} = 0.80$  $R_{as} = 0.75$ D.5.4(a)
Concrete - condition A $R_{tc} = 1.15$  $R_{ac} = 1.15$ D.5.4(c)
### CONCLUSION

| Anchor Rod Embedment, Spacing and Edge Distance | OK |
| Min Required Anchor Reinforcement Development Length | ratio = 0.25 | OK |
| Overall | ratio = **0.60** | OK |

#### Tension

| Stud Tensile Resistance | ratio = 0.14 | OK |
| Anchor Reinforcement Tensile Breakout Resistance | ratio = 0.10 | OK |
| Stud Pullout Resistance | ratio = 0.25 | OK |
| Side Blowout Resistance | ratio = 0.26 | OK |

#### Shear

| Stud Shear Resistance | ratio = 0.19 | OK |
| Anchor Reinforcement Shear Breakout Resistance | Strut Bearing Strength | ratio = 0.60 | OK |
| Tie Reinforcement | ratio = 0.30 | OK |
| Concrete Pryout Not Govern When $h_{ef} >= 12d_a$ | OK |
| Stud on Concrete Bearing | ratio = 0.21 | OK |

#### Tension Shear Interaction

| Tension Shear Interaction | ratio = 0.46 | OK |

#### Ductility

| Tension | Non-ductile |
| Shear | Non-ductile |

#### Seismic Design Requirement

| $I_e F_s a(0.2) >= 0.35$, A23.3-04 D.4.3.7 or D.4.3.8 must be satisfied for non-ductile design | NG |

#### Calculation

**Stud Tensile Resistance**

$$N_{sr} = n_t A_{se} f_y \phi_s$$

$$= 617.7 \text{ [kN]}$$

$$= 1242.0 \text{ [kN]}$$

$$\text{ratio} = 0.14 > N_u$$

**Anchor Reinforcement Tensile Breakout Resistance**

Min tension development length

$$l_d = 887 \text{ [mm]}$$

for ver. 25M bar

Actual development length

$$l_a = h_{ef} - c (50mm) - 200mm \times \tan{35} = 1207 \text{ [mm]}$$

$$\text{ratio} = 0.10 > N_u$$

Seismic design strength reduction

$$= x 0.75 \text{ applicable}$$

$$= 931.5 \text{ [kN]}$$

$$\text{ratio} = 0.10 > N_u$$
### Stud Pullout Resistance

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>A23.3-04 (R2010)</th>
</tr>
</thead>
</table>

- **Single bolt pullout resistance**
  \[ N_{pr} = 8 A_{arg} \phi_c f'c R_{t,c} \]
  \[ = 119.3 \text{ [kN]} \]
  D.6.3.1 (D-15)
- **Seismic design strength reduction**
  \( x = 0.75 \) applicable
  \[ = 357.9 \text{ [kN]} \]
  D.4.3.5
- **Ratio**
  \( = 0.25 \)
  \( > N_u \) 
  OK
- **Pullout strength is always Condition B**
  D.5.4(c)

### Side Blowout Resistance

#### Failure Along Pedestal Width Edge

- **Tensile load carried by anchors close to edge which may cause side-face blowout**
  ACI318 M-08
- **Along pedestal width edge**
  \[ N_{bw} = N_a x n_{bw} / n_t \]
  \[ = 44.5 \text{ [kN]} \]
  RD.5.4.2
- **Check if side blowout applicable**
  \( h_{bf} = 1397 \text{ [mm]} \)
  D.23.04-04 (R2010)
  \( > 2.5c \)
  side blowout is applicable
  D.6.4.1
- **Check if edge anchors work as a group or work individually**
  \( s_{22} = 152 \text{ [mm]} \)
  \( s = s_2 = 152 \text{ [mm]} \)
  D.6.4.2
- **Single anchor SB resistance**
  \[ N_{sbr,w} = 13.3c \sqrt{A_{arg} \phi_c f'c R_{t,c}} \]
  \[ = 191.3 \text{ [kN]} \]
  D.6.4.1 (D-18)
- **Multiple anchors SB resistance**
  work as a group - applicable
  \( = (1+s/6c) x N_{sbr,w} \)
  \[ = 229.4 \text{ [kN]} \]
  D.6.4.2 (D-19)
  work individually - not applicable
  \( n_{bw} x N_{sbr,w} x [1+(c_2 or c_3) / c] / 4 \)
  \[ = 0.0 \text{ [kN]} \]
  D.6.4.1
- **Seismic design strength reduction**
  \( x = 0.75 \) applicable
  \[ = 172.1 \text{ [kN]} \]
  D.4.3.5
- **Ratio**
  \( = 0.26 \)
  \( > N_{bw} \) 
  OK

#### Failure Along Pedestal Depth Edge

- **Tensile load carried by anchors close to edge which may cause side-face blowout**
  ACI318 M-08
- **Along pedestal depth edge**
  \[ N_{bd} = N_a x n_{bd} / n_t \]
  \[ = 44.5 \text{ [kN]} \]
  RD.5.4.2
- **Check if side blowout applicable**
  \( h_{bf} = 1397 \text{ [mm]} \)
  D.23.04-04 (R2010)
  \( > 2.5c \)
  side blowout is applicable
  D.6.4.1
- **Check if edge anchors work as a group or work individually**
  \( s_{11} = 152 \text{ [mm]} \)
  \( s = s_1 = 152 \text{ [mm]} \)
  D.6.4.2
- **Single anchor SB resistance**
  \[ N_{sbr,d} = 13.3c \sqrt{A_{arg} \phi_c f'c R_{t,c}} \]
  \[ = 191.3 \text{ [kN]} \]
  D.6.4.1 (D-18)
- **Multiple anchors SB resistance**
  work as a group - applicable
  \( = (1+s/6c) x N_{sbr,d} \)
  \[ = 229.4 \text{ [kN]} \]
  D.6.4.2 (D-19)
  work individually - not applicable
  \( n_{bd} x N_{sbr,d} x [1+(c_2 or c_3) / c] / 4 \)
  \[ = 0.0 \text{ [kN]} \]
  D.6.4.1
- **Seismic design strength reduction**
  \( x = 0.75 \) applicable
  \[ = 172.1 \text{ [kN]} \]
  D.4.3.5
- **Ratio**
  \( = 0.26 \)
  \( > N_{bd} \) 
  OK

#### Group side blowout resistance

\[ N_{sbg} = \min \left( \frac{N_{sbr,w} x n_{bw}}{n_{bw}} , \frac{N_{sbr,d} x n_{bd}}{n_{bd}} \right) \]
\[ = 344.1 \text{ [kN]} \]

#### Govern Tensile Resistance

\[ N_r = \min ( N_{sr}, N_{tbr}, N_{spg}, N_{sbgr} ) = 344.1 \text{ [kN]} \]
Stud Shear Resistance

\[ V_{st} = n_s A_{se} \phi_s f_{ta} R_{v,s} = 579.1 \text{[kN]} \]

Reduction due to built-up grout pads

\[ V_u = V_{st} \times 1.0 \text{, not applicable} = 579.1 \text{[kN]} \]

\[ \frac{\text{ratio}}{\text{OK}} = 0.19 > V_u \]

Anchor Reinforcement Shear Breakout Resistance

ACI 318 M-08

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement.

STM strength reduction factor

\[ \phi_s = 0.75 \]

Strut-and-Tie model geometry

\[ d_v = 57 \text{[mm]} \quad d_h = 57 \text{[mm]} \]

\[ \theta = 45 \quad d_t = 81 \text{[mm]} \]

Strut compression force

\[ C_s = 0.5 \frac{V_u}{\sin \theta} = 78.6 \text{[kN]} \]

ACI 318 M-08

Strut Bearing Strength

Strut compressive strength

\[ f_{ta} = 0.85 f_c = 23.5 \text{[MPa]} \]

* Bearing of anchor bolt

Anchor bearing length

\[ l_s = \min(8d_s, h_{lu}) = 203 \text{[mm]} \]

Anchor bearing area

\[ A_{org} = l_s \times d_s = 5161 \text{[mm}^2] \]

Anchor bearing resistance

\[ C_t = n_s \times \phi_s \times f_{ta} \times A_{org} = 363.3 \text{[kN]} \]

\[ > V_u \quad \text{OK} \]

* Bearing of ver reinforement bar

Ver bar bearing area

\[ A_{org} = (l_v + 1.5 \times d_v - d_v/2 - d_v/2) \times d_b = 7473 \text{[mm}^2] \]

Ver bar bearing resistance

\[ C_t = \phi_u \times f_{ta} \times A_{org} = 131.5 \text{[kN]} \]

\[ \text{ratio} = 0.60 > C_s \quad \text{OK} \]
Tie Reinforcement

* For tie reinforcing, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective.
* For enclosed tie, at hook location, the tie cannot develop full yield strength $f_y$. Use pullout resistance in tension of a single J-bolt as per A23.3-04 Annex D Eq. (D-17) as the max force can be developed at hook $T_h$.
* Assume 100% of horizontal tie bars can develop full yield strength.

Total number of horizontal tie bar

$$ n = n_{leg \ (leg)} \times n_{lay \ (layer)} = 8 $$  

A23.3-04 (R2010)

Pull out resistance at hook

$$ T_h = 0.9 \phi_c f'c_e d_b R_{t,c} $$  
$$ e_h = 4.5 \ d_b $$  

D.6.3.5 (D-17)  

Single tie bar tension resistance

$$ T_r = \phi_a f_y A_s $$  

Pullout resistance at hook $T_h = 0.9 \phi_c f'c_e e_h d_b R_{t,c}$  

$e_h = 4.5 \ d_b$  

A23.3-04 (R2010)  

Total tie bar tension resistance $V_{tr} = 1.0 \times n \times T_r$  

Seismic design strength reduction $\rho = 0.75$ applicable  

$$ V_{tr} = 62.1 \ [kN] $$  

Total tie bar tension resistance $V_{tr} = 496.8 \ [kN]$  

Seismic design strength reduction $\rho = 0.30$ applicable  

$$ V_{tr} = 372.6 \ [kN] $$  

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in-place headed anchors with $h_{eff} > 12d_a$, the pryout failure will not govern.

$$ 12d_a = 305 \ [mm] \quad h_{eff} = 1397 \ [mm] $$  

$$ > 12d_a \quad OK $$  

CSA S16-09

Stud on Conc Bearing

$$ B_r = n_s \times 1.4 \times \phi_e \times \min(8d_a, h_{eff}) \times d_a \times f'c_e $$  

$\rho = 0.21$  

$$ B_r = 518.5 \ [kN] $$  

$$ > V_u \quad OK $$  

A23.3-04 (R2010)

Govern Shear Resistance

$$ V_r = \min (V_{tr}, V_{eff}, B_r) = 372.6 \ [kN] $$  

Tension Shear Interaction

Check if $N_r > 0.2 N_i$ and $V_r > 0.2 V_i$  

Yes  

$$ \frac{N_r}{N_i} + \frac{V_r}{V_i} = 0.56 $$  

$$ \rho = 0.46 < 1.2 $$  

D.8.2 & D.8.3  

D.8.4 (D-35)

Ductility Tension

$$ N_{tr} = 617.7 \ [kN] $$  

$$ > \min (N_{tr}, N_{tip}, N_{tab}) = 344.1 \ [kN] $$  

Non-ductile

Ductility Shear

$$ V_{tr} = 579.1 \ [kN] $$  

$$ > \min (V_{tr}, B_r) = 372.6 \ [kN] $$  

Non-ductile
Example 23: Welded Stud + Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code

\[ M_u = 35 \text{ kip-ft} \quad N_u = 10 \text{ kips (Compression)} \quad V_u = 25 \text{ kips} \]

Concrete \( f_c' = 4 \text{ ksi} \) \hspace{1cm} Rebar \( f_y = 60 \text{ ksi} \)

Pedestal size \( 26" \times 26" \)

Anchor stud \( \text{AWS D1.1 Grade B} \quad 1.0" \text{ dia} \quad h_{ef} = 55" \quad h_a = 60" \)

Seismic design category < C

Anchor reinforcement
- Tension \( \rightarrow 2\)-No 8 ver. bar
- Shear \( \rightarrow 2\)-layer, 2-leg No 4 hor. bar

No built-up grout pad for embedded plate.

Note: The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment \( h_{ef} \) is required for anchor reinforcement to develop resistance on both sides of the failure plane.
### STUD ANCHOR DESIGN

Combined Tension, Shear and Moment

Anchor bolt design based on

- **ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D**
- **PIP STE05121 Anchor Bolt Design Guide-2006**

### Code Abbreviation

**ACI 318-08**

**PIP STE05121**

### Code Reference

**ACI 318-08**

### Assumptions

1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Load combinations shall be as per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318-08 Appendix D clause D.5.2.9 and D.6.2.9
5. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
6. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft
7. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
8. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output

### Anchor Stud Data

<table>
<thead>
<tr>
<th>Description</th>
<th>Factored Moment</th>
<th>Factored Tension /Compression</th>
<th>Factored Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>M&lt;sub&gt;u&lt;/sub&gt;</td>
<td>35.0 [kip-ft]</td>
<td>-10.0 [kips] in compression</td>
<td>25.0 [kips]</td>
</tr>
<tr>
<td>N&lt;sub&gt;u&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V&lt;sub&gt;u&lt;/sub&gt;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Code References

- **ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D**
- **PIP STE05121 Anchor Bolt Design Guide-2006**
- **ACI 318-08**
- **D.4.4 (c)**
- **D.4.4**
- **D.5.2.9 & D.6.2.9**
- **D.3.1**
- **D.3.1**

### Diagrams

- Diagrams showing the Anchor Stud Data with labeled anchor bolts and forces.

2 BOLT LINE  
3 BOLT LINE  
4 BOLT LINE
<table>
<thead>
<tr>
<th>Description</th>
<th>Min Required</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of bolt line for resisting moment</td>
<td>2 Bolt Line</td>
<td></td>
</tr>
<tr>
<td>No of bolt along outermost bolt line</td>
<td>2</td>
<td>PIP STE05121</td>
</tr>
<tr>
<td>Outermost stud line spacing $s_1$</td>
<td>$s_1 = 16.0$ [in] 4.0</td>
<td>OK</td>
</tr>
<tr>
<td>Outermost stud line spacing $s_2$</td>
<td>$s_2 = 16.0$ [in] 4.0</td>
<td>OK</td>
</tr>
<tr>
<td>Internal stud line spacing $s_{b1}$</td>
<td>$s_{b1} = 10.5$ [in] 4.0</td>
<td>OK</td>
</tr>
<tr>
<td>Internal stud line spacing $s_{b2}$</td>
<td>$s_{b2} = 0.0$ [in] 4.0</td>
<td>OK</td>
</tr>
<tr>
<td>Column depth</td>
<td>$d = 12.7$ [in]</td>
<td></td>
</tr>
<tr>
<td>Concrete strength</td>
<td>$f'_c = 4.0$ [ksi] = 27.6 [MPa]</td>
<td></td>
</tr>
<tr>
<td>Stud material</td>
<td>AWS D1.1 Grade B</td>
<td>ACI 318-08</td>
</tr>
<tr>
<td>Stud tensile strength</td>
<td>$f_{uta} = 65$ [ksi] = 448 [MPa]</td>
<td></td>
</tr>
<tr>
<td>Stud diameter</td>
<td>$d_s = 1$ [in] = 25.4 [mm]</td>
<td>PIP STE05121</td>
</tr>
<tr>
<td>Stud shank area</td>
<td>$A_{so} = 0.79$ [in$^2$] = 507 [mm$^2$]</td>
<td></td>
</tr>
<tr>
<td>Stud head bearing area</td>
<td>$A_{brg} = 1.29$ [in$^2$] = 831 [mm$^2$]</td>
<td></td>
</tr>
<tr>
<td>Stud embedment depth</td>
<td>$h_{ef} = 55.0$ [in] 12.0</td>
<td>OK</td>
</tr>
<tr>
<td>Pedestal height</td>
<td>$h = 60.0$ [in] 58.0</td>
<td>OK</td>
</tr>
<tr>
<td>Pedestal width</td>
<td>$b_c = 26.0$ [in]</td>
<td></td>
</tr>
<tr>
<td>Pedestal depth</td>
<td>$d_c = 26.0$ [in]</td>
<td></td>
</tr>
<tr>
<td>Stud edge distance $c_1$</td>
<td>$c_1 = 5.0$ [in] 4.5</td>
<td>OK</td>
</tr>
<tr>
<td>Stud edge distance $c_2$</td>
<td>$c_2 = 5.0$ [in] 4.5</td>
<td>OK</td>
</tr>
<tr>
<td>Stud edge distance $c_3$</td>
<td>$c_3 = 5.0$ [in] 4.5</td>
<td>OK</td>
</tr>
<tr>
<td>Stud edge distance $c_4$</td>
<td>$c_4 = 5.0$ [in] 4.5</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Ver. Reinf For Tension**

**Hor. Ties For Shear - 4 Legs**

**Hor. Ties For Shear - 2 Legs**
To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5h_{ef} from the outmost anchor's centerline. In this design 0.5h_{ef} value is limited to 8 in.

\[ 0.5h_{ef} = 8.0 \text{ [in]} \]

No of ver. rebar that are effective for resisting anchor tension \( n_v = 2 \)

Ver. bar size No. \( 8 \) 1.000 [in] dia single bar area \( A_s = 0.79 \text{ [in}^2\text{]} \)

To be considered effective for resisting anchor shear, hor. reinft shall be located within \( \min(0.5c_1, 0.3c_2) \) from the outmost anchor's centerline

\[ \min(0.5c_1, 0.3c_2) = 1.5 \text{ [in]} \]

No of tie leg that are effective to resist anchor shear \( n_{leg} = 2 \)

No of tie layer that are effective to resist anchor shear \( n_{lay} = 2 \)

Hor. tie bar size No. \( 4 \) 0.500 [in] dia single bar area \( A_s = 0.20 \text{ [in}^2\text{]} \)

For anchor reinft shear breakout strength calc suggest 100% hor. tie bars develop full yield strength

Rebar yield strength \( f_y = 60 \text{ [ksi]} \) 60 = 414 [MPa]

Total no of welded stud \( n = 4 \)
Number of stud carrying tension \( n_t = 2 \)
Number of stud carrying shear \( n_s = 2 \)

For side-face blowout check use

No of stud along width edge \( n_{bw} = 2 \)

Seismic design category \( \geq C \) = No ?
Provide built-up grout pad ? = No ?

ACI 318-08

Code Reference

ACI 318-08

Dongxiao Wu P. Eng.
### CONCLUSION

Abchor Rod Embedment, Spacing and Edge Distance

<table>
<thead>
<tr>
<th>Min Rquired Anchor Reinft. Development Length</th>
<th>Overall</th>
</tr>
</thead>
<tbody>
<tr>
<td>ratio = 0.25 OK</td>
<td>ratio = 0.94 OK</td>
</tr>
</tbody>
</table>

**Tension**

<table>
<thead>
<tr>
<th>Stud Tensile Resistance</th>
<th>ratio = 0.32 OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Reinft Tensile Breakout Resistance</td>
<td>ratio = 0.35 OK</td>
</tr>
<tr>
<td>Stud Pullout Resistance</td>
<td>ratio = 0.43 OK</td>
</tr>
<tr>
<td>Side Blowout Resistance</td>
<td>ratio = 0.38 OK</td>
</tr>
</tbody>
</table>

**Shear**

<table>
<thead>
<tr>
<th>Stud Shear Resistance</th>
<th>ratio = 0.38 OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Reinft Shear Breakout Resistance</td>
<td>ratio = 0.59 OK</td>
</tr>
<tr>
<td>Strut Bearing Strength</td>
<td>ratio = 0.69 OK</td>
</tr>
<tr>
<td>Conc. Pryout Not Govern When $h_{ef} &gt;= 12d_a$</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Tension Shear Interaction**

<table>
<thead>
<tr>
<th>Tension Shear Interaction</th>
<th>ratio = 0.94 OK</th>
</tr>
</thead>
</table>

**Ductility**

<table>
<thead>
<tr>
<th>Tension Non-ductile</th>
<th>Shear Non-ductile</th>
</tr>
</thead>
</table>

**Seismic Design Requirement**

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required

### CALCULATION

**Stud Tensile Force**

<table>
<thead>
<tr>
<th>Single stud tensile force</th>
<th>$N_a = \sum n_i T_i = 24.8$ [kips]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_1 = 12.42$ [kips] No of stud for $T_1 n_T1 = 2$</td>
<td></td>
</tr>
<tr>
<td>$T_2 = 0.00$ [kips] No of stud for $T_2 n_T2 = 0$</td>
<td></td>
</tr>
<tr>
<td>$T_3 = 0.00$ [kips] No of stud for $T_3 n_T3 = 0$</td>
<td></td>
</tr>
</tbody>
</table>

**Stud Tensile Resistance**

<table>
<thead>
<tr>
<th>$\phi_{ts} N_{sa} = \phi_{ts} A_{sa} f_{u,sa}$</th>
<th>= 38.3 [kips] D.5.1.2 (D-3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ratio = 0.32 &gt; $T_1$</td>
<td>OK</td>
</tr>
</tbody>
</table>

**Anchor Reinft Tensile Breakout Resistance**

| Min tension development length $l_d = 47.4$ [in] 12.2.1, 12.2.2, 12.2.4 |
|-----------------------------|------------------------|
| for ver. #8 bar |

<table>
<thead>
<tr>
<th>Actual development length $l_a = h_{ef} - c (2$ in) - 8 in * tan35</th>
<th>= 47.4 [in]</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 12.0</td>
<td>OK</td>
</tr>
</tbody>
</table>
Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

Dongxiao Wu P. Eng.

5 of 7

Code Reference

\[ \text{Seismic design strength reduction} = x 1.0 \quad \text{not applicable} = 71.0 \quad \text{kips} \]

D.3.3.3

Seismic design strength reduction = x 1.0 not applicable = 71.0 [kips] D.3.3.3

ratio = 0.35 > N_u

OK

Stud Pullout Resistance

Single bolt pullout resistance

\[ N_p = \phi_{t,c} N_p \]

Seismic design strength reduction

\[ = x 1.0 \quad \text{not applicable} = 28.9 \quad \text{kips} \]

D.3.3.3

ratio = 0.43 > T_1

OK

\[ \Psi_{c,p} = 1 \quad \text{for cracked conc} \]

D.5.3.6

\[ \phi_{t,c} = 0.70 \quad \text{pullout strength is always Condition B} \]

D.4.4(c)

Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal width edge

\[ N_{bw} = n_{T_1} T_1 \]

D.5.4.2

\[ c = \min ( c_1, c_2 ) = 5.0 \quad \text{in} \]

RD.5.4.2

Check if side blowout applicable

\[ h_{st} = 55.0 \quad \text{in} \]

\[ > 2.5c \quad \text{side blowout is applicable} \]

D.5.4.1

Check if edge anchors work as a group or work individually

\[ s_{22} = 16.0 \quad \text{in} \]

\[ s = 16.0 \quad \text{in} \]

D.5.4.2

Single anchor SB resistance

\[ \phi_{t,c} N_{ab} = \phi_{t,c} \left( 160 c / \sqrt{A_{oc}} \right) \lambda \sqrt{l_p} \]

D.5.4.1 (D-17)

\[ = 43.1 \quad \text{kips} \]

D.5.4.2 (D-18)

Multiple anchors SB resistance

\[ \phi_{t,c} N_{ab,g,w} = \left( 1 + s / 6c \right) x \phi_{t,c} N_{ab} \]

D.5.4.1 (D-17)

\[ = 66.0 \quad \text{kips} \]

D.5.4.2 (D-18)

Seismic design strength reduction

\[ = x 1.0 \quad \text{not applicable} = 66.0 \quad \text{kips} \]

D.3.3.3

ratio = 0.38 > N_{bw}

OK

Group side blowout resistance

\[ \phi_{t,c} N_{abg} = \phi_{t,c} N_{ab} x n_{T_1} \]

D.5.4.1 (D-17)

\[ = 66.0 \quad \text{kips} \]

D.5.4.2 (D-18)

Govern Tensile Resistance

\[ N_r = \phi_{t,c} \min ( n_1 N_s, N_b, n_1 N_{sp}, N_{abg} ) \]

D.5.4.1 (D-17)

\[ = 57.7 \quad \text{kips} \]
### Stud Shear Resistance

\[ \phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{se} f_{uta} = 66.4 \quad [\text{kips}] \quad D.6.1.2 \text{ (a)(D-19)} \]

Reduction due to built-up grout pads = \( x \times 1.0 \), not applicable = 66.4 [kips] \( D.6.1.3 \)

\[ \text{ratio} = 0.38 \quad > V_u \quad \text{OK} \]

### Anchor Reinforcement Shear Breakout Resistance

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement.

STM strength reduction factor \( \phi_{st} = 0.75 \) \( 9.3.2.6 \)

### Strut-and-Tie Model Geometry

- \( d_v = 2.250 \quad [\text{in}] \)
- \( d_h = 2.250 \quad [\text{in}] \)
- \( \theta = 45 \)
- \( d_t = 3.182 \quad [\text{in}] \)

**Strut Compression Force**

\[ C_s = 0.5 \frac{V_u}{\sin \theta} = 17.7 \quad [\text{kips}] \]

**ACI 318-08**

### Strut Bearing Strength

**Strut Compressive Strength**

\[ f_{oa} = 0.85 f'_c = 3.4 \quad [\text{ksi}] \quad A.3.2 \text{ (A-3)} \]

* Bearing of anchor bolt

**Anchor Bearing Length**

\[ l_{e} = \min(8d_v, h_{cf}) = 8.0 \quad [\text{in}] \quad D.6.2.2 \]

**Anchor Bearing Area**

\[ A_{org} = l_e \times d_v = 8.0 \quad [\text{in}^2] \]

**Anchor Bearing Resistance**

\[ C_r = n_s \times \phi_{u} \times f_{oa} \times A_{org} = 40.8 \quad [\text{kips}] \quad > V_u \quad \text{OK} \]

* Bearing of ver. reinforc. bar

**Ver. Bar Bearing Area**

\[ A_{org} = (l_e + 1.5 \times d_v - d_b/2) \times d_b = 11.8 \quad [\text{in}^2] \]

**Ver. Bar Bearing Resistance**

\[ C_r = \phi_{u} \times f_{oa} \times A_{org} = 30.0 \quad [\text{kips}] \]

**Ratio**

\[ \text{ratio} = 0.59 \quad > C_s \quad \text{OK} \]
Tie Reinforcement

For tie reinforcement, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective.

For enclosed tie, at hook location the tie cannot develop full yield strength $f_y$. Use the pullout resistance in tension of a single hooked bolt as per ACI318-08 Eq. (D-16) as the max force can be developed at hook $T_h$.

Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar

$$n = n_{leg} \times n_{lay} = 4$$

Pull out resistance at hook

$$T_h = \phi_{t,c} \times 0.9 \times f_y \times e_h \times d_t = 3.0 \text{ [kips]}$$

$$e_h = 4.5 \times d_t = 2.250 \text{ [in]}$$

Single bar tension resistance

$$T_r = \phi_s \times f_y \times A_s = 9.0 \text{ [kips]}$$

Total tie bar tension resistance

$$V_{rb} = 1.0 \times n \times T_r = 36.0 \text{ [kips]}$$

Seismic design strength reduction

$$\text{ratio} = 0.69 > V_u \quad \text{OK}$$

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} \geq 12d_a$, the pryout failure will not govern.

$$12d_a = 12.0 \text{ [in]} \quad h_{ef} = 55.0 \text{ [in]}$$

$$> 12d_a \quad \text{OK}$$

Govern Shear Resistance

$$V_r = \min (\phi_{t,s} V_{sa}, V_{rb}) = 36.0 \text{ [kips]}$$

Tension Shear Interaction

Check if $N_s > 0.2\phi_{t,s}N_{sa} \text{ and } V_{sr} > 0.2\phi V_n \text{. Yes}$

$$N_s/N_{sa} + V_{sr}/V_n = 1.12 \quad \text{D.7.1 & D.7.2}$$

$$\text{ratio} = 0.94 < 1.2 \quad \text{OK}$$

Ductility Tension

$$\phi_{t,c} N_{sa} = 38.3 \text{ [kips]}$$

$$> \phi_{t,c} \min (N_{sc}, N_{pc}, N_{hcg}) = 28.9 \text{ [kips]}$$

Non-ductile

Ductility Shear

$$\phi_{t,s} N_{sa} = 66.4 \text{ [kips]}$$

$$V_{rb} = 36.0 \text{ [kips]}$$

Non-ductile
Example 24: Welded Stud + Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code

\[ M_u = 47.4 \text{ kNm} \quad N_u = 44.5 \text{ kN (Compression)} \quad V_u = 111.2 \text{ kN} \]

Concrete \( f'_c = 27.6 \text{ MPa} \) \quad Rebar \( f_y = 414 \text{ MPa} \)

Pedestal size \( 660\text{mm} \times 660\text{mm} \)

Anchor stud \( \text{AWS D1.1 Grade B} \quad 1.0'' \text{ dia} \quad h_{ef} = 1397\text{mm} \quad h_a = 1524\text{mm} \)

Seismic design \( I_E F_s S_0(0.2) < 0.35 \)

Anchor reinforcement
- Tension \( \rightarrow 2\text{-}25M \text{ ver. bar} \)
- Shear \( \rightarrow 2\text{-}layer, 2\text{-}leg 15M \text{ hor. bar} \)

No built-up grout pad for embedded plate.

**Note:** The stud length used in this example may not be commercially available and it's for illustration purpose only.

Deep anchor stud embedment \( h_{ef} \) is required for anchor reinforcement to develop resistance on both sides of the failure plane.
STUD ANCHOR DESIGN  
Combined Tension, Shear and Moment

Anchor bolt design based on CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
PIP STE05121 Anchor Bolt Design Guide-2006

Assumptions
1. Concrete is cracked
2. Condition A - supplementary reinforcement is provided
3. Anchor reinft strength is used to replace concrete tension / shear breakout strength as per ACI318 M-08 Appendix D clause D.5.2.9 and D.6.2.9
4. For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
5. Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinft
6. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
7. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output

Anchor Stud Data
Factored moment \( M_u = 47.4 \text{ [kNm]} = 35.0 \text{ [kip-ft]} \)
Factored tension /compression \( N_u = -44.5 \text{ [kN]} \text{ in compression} = -10.0 \text{ [kips]} \)
Factored shear \( V_u = 111.2 \text{ [kN]} \text{ in tension} = 25.0 \text{ [kips]} \)
No of bolt line for resisting moment

No of bolt along outermost bolt line

Outermost stud line spacing $s_1$

Outermost stud line spacing $s_2$

Internal stud line spacing $s_{b1}$

Internal stud line spacing $s_{b2}$

Column depth $d$

Concrete strength $f'_c$

Anchor bolt material

Anchor tensile strength $f_{uta}$

Stud diameter $d_s$

Stud shank area $A_{so}$

Stud head bearing area $A_{brg}$

Anchor bolt embedment depth $h_{ef}$

Pedestal height $h$

Pedestal width $b_c$

Pedestal depth $d_c$

Stud edge distance $c_1$

Stud edge distance $c_2$

Stud edge distance $c_3$

Stud edge distance $c_4$

Vertical Reinft For Tension

Horizontal Ties For Shear - 4 Legs

Horizontal Ties For Shear - 2 Legs

Warn: $s_{b1} = 0.5 \times s_1 = 203.0$ [mm]
To be considered effective for resisting anchor tension, vertical reinforcing bars shall be located within $0.5h_{ef}$ from the outmost anchor's centerline. In this design $0.5h_{ef}$ value is limited to 200mm.

$$0.5h_{ef} = 200 \text{ [mm]}$$

No of vertical rebar that are effective for resisting anchor tension

Ver. bar size $d_b = 25 \text{ [mm]}$

single bar area $A_s = 500 \text{ [mm}^2\text{]}$

To be considered effective for resisting anchor shear, horizontal reinforcement shall be located within $\min(0.5c_1, 0.3c_2)$ from the outmost anchor's centerline

$$\min(0.5c_1, 0.3c_2) = 38 \text{ [mm]}$$

No of tie legs that are effective to resist anchor shear

No of tie layers that are effective to resist anchor shear

Hor. bar size $d_h = 15 \text{ [mm]}$

single bar area $A_s = 200 \text{ [mm}^2\text{]}$

For anchor reinforcement shear breakout strength calculation

100% hor. tie bars develop full yield strength

Rebar yield strength $f_y = 414 \text{ [MPa]}$

$400 = 60.0 \text{ [ksi]}$

Total no of welded stud $n = 4$

No of stud carrying tension $n_t = 2$

No of stud carrying shear $n_s = 2$

For side-face blowout check use

No of stud along width edge $n_{bw} = 2$

Seismic region where $I_kF_aS_a(0.2) >= 0.35$

Provide built-up grout pad

ACI318 M-08

RD.5.2.9

A23.3-04 (R2010)

D.4.3.5

D.7.1.3
<table>
<thead>
<tr>
<th>Strength reduction factors</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor reinforcement factor</td>
<td>$\phi_{as} = 0.75$</td>
</tr>
<tr>
<td>Steel anchor resistance factor</td>
<td>$\phi_s = 0.85$</td>
</tr>
<tr>
<td>Concrete resistance factor</td>
<td>$\phi_c = 0.65$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance modification factors</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor rod - ductile steel</td>
<td>$R_{ts} = 0.80$</td>
</tr>
<tr>
<td>Concrete - condition A</td>
<td>$R_{tc} = 1.15$</td>
</tr>
</tbody>
</table>

**CONCLUSION**

- Anchor Rod Embedment, Spacing and Edge Distance: OK
- Min Rquired Anchor Reinft. Development Length: OK
- Overall ratio = 0.76 (12.2.1)

**Tension**

- Stud Tensile Resistance: $\phi_s = 0.85$ (OK)
- Anchor Reinft Tensile Breakout Resistance: $\phi_s = 0.85$ (OK)
- Stud Pullout Resistance: $\phi_s = 0.85$ (OK)
- Side Blowout Resistance: $\phi_s = 0.85$ (OK)

**Shear**

- Stud Shear Resistance: $\phi_s = 0.85$ (OK)
- Anchor Reinft Shear Breakout Resistance: $\phi_s = 0.85$ (OK)
  - Strut Bearing Strength: $\phi_s = 0.85$ (OK)
  - Tie Reinforcement: $\phi_s = 0.85$ (OK)
- Conc. Pryout Not Govern When $h_{ef} >= 12d_a$: OK
- Stud on Conc Bearing: $\phi_s = 0.85$ (OK)

**Tension Shear Interaction**

- Tension Shear Interaction: $\phi_s = 0.85$ (OK)

**Ductility**

- Tension Non-ductile Shear Non-ductile: OK (D.4.3.6)

**Seismic Design Requirement**

- IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required

**CALCULATION**

**Anchor Tensile Force**

<table>
<thead>
<tr>
<th>Single stud tensile force</th>
<th>No of stud for $T_1$ $n_{T1}$ = 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_1 = 55.2$ [kN]</td>
<td>No of stud for $T_2$ $n_{T2}$ = 0</td>
</tr>
<tr>
<td>$T_2 = 0.0$ [kN]</td>
<td>No of stud for $T_3$ $n_{T3}$ = 0</td>
</tr>
<tr>
<td>$T_3 = 0.0$ [kN]</td>
<td></td>
</tr>
</tbody>
</table>

- Sum of stud tensile force $N_u = \sum n_i T_i = 110.3$ [kN]

- Stud Tensile Resistance $N_{sr} = A_{se} \phi_s f_{usat} R_{ts}$ = 154.4 [kN] (D.6.1.2 (D-3))
  - ratio = 0.36 $> T_1$: OK
### Anchor Reinft Tensile Breakout Resistance

Min tension development length: 
\[ l_d = \text{[mm]} \]  

Actual development length: 
\[ l_a = h_{df} - c (50) - 200 \times \tan 35 = 1207 \text{[mm]} > 300 \text{OK} \]

Seismic design strength reduction: 
\[ \text{ratio} = 0.36 < \text{Nu} \text{OK} \]

### Stud Pullout Resistance

Single bolt pullout resistance: 
\[ N_{sp} = 8 A_{bol} f_y f' c R_{tc} = 119.3 \text{[kN]} \]

Seismic design strength reduction: 
\[ \text{ratio} = 0.46 < T_1 \text{OK} \]

\[ \Psi_{c,p} = 1 \text{ for cracked conc} \]

R_{tc} = 1.00 pullout strength is always Condition B

### Side Blowout Resistance

**Failure Along Pedestal Width Edge**

Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal width edge: 
\[ N_{buw} = n_{1s} T_1 = 110.3 \text{[kN]} \]

Check if side blowout applicable: 
\[ h_{df} = 1397 \text{[mm]} > 2.5c \text{side blowout is applicable} \]

Check if edge anchors work as a group or work individually: 
\[ s = s_2 = 406 \text{[mm]} \]

Single anchor SB resistance: 
\[ N_{sbw} = 13.3c \sqrt{A_{bol}} f_y f' c R_{tc} = 191.3 \text{[kN]} \]

Multiple anchors SB resistance: 
\[ N_{sbgr,w} = (1+s/6c) \times N_{sbw} = 293.2 \text{[kN]} \]

Seismic design strength reduction: 
\[ \text{ratio} = 0.38 < N_{buw} \text{OK} \]

Group side blowout resistance: 
\[ N_{sbgr} = \frac{N_{sbgr,w}}{n_{1s}} = 293.2 \text{[kN]} \]

### Govern Tensile Resistance

\[ N_r = \min (n_i N_{spr}, n_i N_{cr}, n_i N_{cpr}, N_{sbgr}) = 238.6 \text{[kN]} \]
Stud Shear Resistance

\[ V_s = n_s A_s \phi_a f_{ut} \]  
\[ = 289.5 \text{ [kN]} \]  

Reduction due to built-up grout pads  
\[ = x \times 1.0, \text{ not applicable} \]  
\[ = 289.5 \text{ [kN]} \]  
\[ > V_u \]  
\[ \text{OK} \]

Anchor Reinforcement Shear Breakout Resistance

Strut-and-Tie model is used to analyze the shear transfer and to design the required tie reinforcement.

STM strength reduction factor  
\[ \phi_s = 0.75 \]  
\[ 9.3.2.6 \]

Strut-and-Tie model geometry
\[ d_s = 57 \text{ [mm]} \]  
\[ d_b = 57 \text{ [mm]} \]  
\[ \theta = 45 \]  
\[ d_t = 81 \text{ [mm]} \]

Strut compression force
\[ C_s = 0.5 V_u / \sin \theta \]  
\[ = 78.6 \text{ [kN]} \]

* Strut Bearing Strength

Strut compressive strength  
\[ f_{ct} = 0.85 f_c \]  
\[ = 23.5 \text{ [MPa]} \]  
\[ A.3.2 (A-3) \]

* Bearing of anchor bolt

Anchor bearing length  
\[ l_a = \min(8d_s, h_{ut}) \]  
\[ = 203 \text{ [mm]} \]  
\[ D.6.2.2 \]

Anchor bearing area  
\[ A_{org} = l_a \times d_s \]  
\[ = 5161 \text{ [mm}^2\text{]} \]

Anchor bearing resistance  
\[ C_r = n_s \times \phi_a \times f_{ut} \times A_{org} \]  
\[ = 181.6 \text{ [kN]} \]  
\[ > V_u \]  
\[ \text{OK} \]

* Bearing of ver reinforcement bar

Ver bar bearing area  
\[ A_{org} = (l_a + 1.5 \times d_s - d_s / 2 - d_t / 2) \times d_b \]  
\[ = 7473 \text{ [mm}^2\text{]} \]

Ver bar bearing resistance  
\[ C_r = \phi_a \times f_{ut} \times A_{org} \]  
\[ = 131.5 \text{ [kN]} \]  
\[ > C_s \]  
\[ \text{OK} \]
Tie Reinforcement

* For tie reinft, only the top most 2 or 3 layers of ties (2" from TOC and 2x3" after) are effective
* For enclosed tie, at hook location the tie cannot develop full yield strength f_y. Use the pullout resistance in tension of a single J-bolt as per A23.3.04 Annex D Eq. (D-17) as the max force can be developed at hook $T_h$
* Assume 100% of hor. tie bars can develop full yield strength.

Total number of hor tie bar $n = n_{leg} (\text{leg}) \times n_{lay} (\text{layer}) = 4$

Pull out resistance at hook $T_h = 0.9 \phi c f_y e_h d_b R_{c,c}$

Single tie bar tension resistance $T_r = \phi c f_y x A_s$

Total tie bar tension resistance $V_{tr} = x 1.0 \times n \times T_r$

Seismic design strength reduction $= x 1.0$ not applicable

Conc. Pryout Shear Resistance

The pryout failure is only critical for short and stiff anchors. It is reasonable to assume that for general cast-in place headed anchors with $h_{ef} > 12d_a$, the pryout failure will not govern

$12d_a = 305$ [mm] $h_{ef} = 1397$ [mm] $> 12d_a \quad \text{OK}$ CSA S16-09

Stud on Conc Bearing

$B_r = n_s \times 1.4 \times \phi c \times \min(8d_a, h_{ef}) \times d_b \times f_y'$

$\text{ratio} = 0.43 < V_u \quad \text{OK}$

Govern Shear Resistance

$V_r = \min (V_{sr}, V_{rbr}, B_r) = 248.4$ [kN]

Tension Shear Interaction

Check if $N_i > 0.2 N_i$ and $V_i > 0.2 V_i$

$N_i / N_i + V_i / V_i = 0.91 < 1.2 \quad \text{OK}$ A23.3.04 (R2010)

Ductility Tension

$N_{sr} = 154.4$ [kN] $> \min (N_{sr}, N_{cpr}, N_{dpr}) = 119.3$ [kN] Non-ductile

Ductility Shear

$V_{sr} = 289.5$ [kN] $> \min (V_{sr}, B_r) = 248.4$ [kN] Non-ductile
Example 31: Welded Stud + No Anchor Reinft + Tension & Shear + ACI 318-08 Code

\[ N_u = 20 \text{ kips (Tension)} \quad V_u = 10 \text{ kips} \]

Concrete \( f'_c = 4.5 \text{ ksi} \)

Anchor stud
AWS D1.1 Grade B 1.0" dia \( h_{ef} = 12" \) \( h_a = 15" \)

Seismic design category < C

Supplementary reinforcement
Tension \( \rightarrow \) Condition A
Shear \( \rightarrow \) Condition A \( \psi_{c,V} = 1.2 \)

No built-up grout pad for embedded plate.

**Note:** The stud length used in this example may not be commercially available and it's for illustration purpose only.
STUD ANCHOR DESIGN  
Combined Tension and Shear

Anchor bolt design based on

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D

Code Abbreviation

ACI 318-08

Input Data

set $N_u = 0$ if it's compression

Factored tension $N_u = 20.0$ [kips] = 89.0 [kN]
Factored shear $V_u = 10.0$ [kips] = 44.5 [kN]

Concrete strength $f'c = 4.5$ [ksi] = 31.0 [MPa]

Stud material

AWS D1.1 Grade B

Stud tensile strength $f_{uta} = 65$ [ksi] = 448 [MPa]

ACI 318-08

Stud is ductile steel element

D.1

Stud diameter $d_a = 1$ [in] = 25.4 [mm]

Stud shank area $A_{sa} = 0.79$ [in$^2$] = 507 [mm$^2$]

Stud head bearing area $A_{brg} = 1.29$ [in$^2$] = 831 [mm$^2$]

Concrete thickness $h_a = 15.0$ [in] 15.0

Stud embedment depth $h_{ef} = 12.0$ [in] 12.0

OK

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$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of $h_{ef}$ when $N>0$

No

Warn

Adjusted $h_{ef}$ for design $h_{ef} = 5.33$ [in] 12.0

D.5.2.3

Outermost stud line spacing $s_1 = 16.0$ [in] 4.0

OK

Page A-1 Table 1

Outermost stud line spacing $s_2 = 16.0$ [in] 4.0

OK

Page A-1 Table 1
Number of stud at bolt line 1 \( n_1 = 2 \)
Number of stud at bolt line 2 \( n_2 = 2 \)
Total no of welded stud \( n = 4 \)
Number of stud carrying tension \( n_t = 4 \)
Number of stud carrying shear \( n_s = 2 \)

For side-face blowout check use
No of stud along width edge \( n_{bw} = 2 \)
No of stud along depth edge \( n_{bd} = 2 \)

Seismic design category \( = C \)?

Supplementary reinforcement

For tension \( \Psi_{c,V} = 1.2 \) Condition A

For shear

Provide built-up grout pad \( = \) ?

Strength reduction factors

Anchor reinforcement \( \phi_s = 0.75 \)
Anchor rod - ductile steel \( \phi_{ts} = 0.75 \) \( \phi_{vs} = 0.65 \)
Concrete \( \phi_{tc} = 0.75 \) Cdn-A \( \phi_{vc} = 0.75 \) Cdn-A

Assumptions

1. Concrete is cracked
2. Condition A - supplementary reinforcement provided
3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2
4. Tensile load acts through center of bolt group \( \Psi_{c,N} = 1.0 \)
5. Shear load acts through center of bolt group \( \Psi_{c,V} = 1.0 \)
## CONCLUSION

### Abchon Rod Embedment, Spacing and Edge Distance

<table>
<thead>
<tr>
<th></th>
<th>Overall</th>
<th>ratio</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tension</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stud Tensile Resistance</td>
<td>ratio = 0.13</td>
<td>OK</td>
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</tr>
<tr>
<td>Conc. Tensile Breakout Resistance</td>
<td>ratio = 0.57</td>
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<tr>
<td>Stud Pullout Resistance</td>
<td>ratio = 0.15</td>
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<td>Side Blowout Resistance</td>
<td>ratio = 0.00</td>
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<td><strong>Shear</strong></td>
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<td>Stud Shear Resistance</td>
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<td>D.3.3.4</td>
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</table>

SDC< C, ACI318-08 D.3.3 ductility requirement is NOT required.

## CALCULATION

### Stud Tensile Resistance

\[
\phi_{ts} N_{sa} = \phi_{ts} n_t A_{se} f_{s}\text{a} = 153.2 \text{ [kips] D.5.1.2 (D-3)}
\]

\[
\text{ratio} = 0.13 > N_u \quad \text{OK}
\]

### Conc. Tensile Breakout Resistance

\[
N_b = 24 \sqrt{h_e f_{t}\text{a}^2} \quad \text{if } h_e < 11^* \text{ or } h_e > 25^* = 19.8 \text{ [kips] D.5.2.2 (D-7)}
\]

\[
16 \sqrt{h_e f_{t}\text{a}^2} \quad \text{if } 11^* \leq h_e \leq 25^* = 19.8
\]

Projected conc failure area

\[
A_{fc} = [s_1 + \min(c_1, 1.5h_{ed}) + \min(c_2, 1.5h_{ed})]x = 676.0 \text{ [in}^2]\]

\[
A_{fc} = 9 h_{ed}^2 = 256.0 \text{ [in}^2]\]

Min edge distance

\[
c_{min} = \min(c_1, c_2, c_3, c_4) = 5.0 \text{ [in]}\]

Eccentricity effects

\[
\Psi_{esc,N} = 1.0 \text{ for no eccentric load D.5.2.4}
\]

Edge effects

\[
\Psi_{ed,N} = \min(0.7 + 0.3c_{min}/1.5h_{ed}, 1.0) = 0.89 \text{ D.5.2.5}
\]

Concrete cracking

\[
\Psi_{c,N} = 1.0 \text{ for cracked concrete D.5.2.6}
\]

Concrete splitting

\[
\Psi_{sp,N} = 1.0 \text{ for cast-in anchor D.5.2.7}
\]
### Concrete Breakout Resistance

\[
\phi_{lt,c} N_{cbg} = \phi_{lt,c} \frac{A_d}{A_{ego}} \psi_{acN} \psi_{scN} \psi_{ct,N} N_c = 34.9 \text{ [kips]}
\]

- Seismic design strength reduction: \(x = 1.0\) not applicable
- \(\phi_{lt,c} N_{cbg} = 34.9\) [kips]
- Ratio: 0.57 > \(N_u\) **OK**

### Stud Pullout Resistance

- Single bolt pullout resistance: \(N_p = 8 A_{bog} f'_c\)
  \[
  \phi_{lt,c} N_{pn} = \phi_{lt,c} n_1 \psi_{c,bp} N_p = 129.9 \text{ [kips]}
  \]
- Seismic design strength reduction: \(x = 1.0\) not applicable
- \(\phi_{lt,c} N_{pn} = 129.9\) [kips]
- Ratio: 0.15 > \(N_u\) **OK**

- \(\Psi_{c,p} = 1\) for cracked conc

### Side Blowout Resistance

#### Failure Along Pedestal Width Edge

- Tensile load carried by anchors close to edge which may cause side-face blowout
  
  along pedestal width edge: \(N_{buw} = N_u \times n_{bw} / n_t\)
  
  - \(c = \min (c_1, c_3)\)
  - \(c = 5.0\) [in]
  
  - Check if side blowout applicable: \(h_{bf} = 12.0\) [in]
    
    - \(< 2.5c\) side blowout is NOT applicable
      
      - \(s_{22} = 0.0\) [in] \(s = s_2 = 0.0\) [in]

- Single anchor SB resistance: \(\phi_{lt,c} N_{sb} = 0.0\) [kips]

- Multiple anchors SB resistance: \(\phi_{lt,c} N_{sbg,w} =\)
  
  - work as a group - not applicable: \((1+s/6c) \times \phi_{lt,c} N_{sb}\) \(= 0.0\) [kips]
  - work individually - not applicable: \(n_{bw} \times \phi_{lt,c} N_{sb} \times (1+(c_2 or c_4)/c) / 4\) \(= 0.0\) [kips]

- Seismic design strength reduction: \(x = 1.0\) not applicable
- \(\phi_{lt,c} N_{sb} = 0.0\) [kips] **OK**

#### Failure Along Pedestal Depth Edge

- Tensile load carried by anchors close to edge which may cause side-face blowout
  
  along pedestal depth edge: \(N_{bud} = N_u \times n_{bd} / n_t\)
  
  - \(c = \min (c_2, c_4)\)
  - \(c = 5.0\) [in]
  
  - Check if side blowout applicable: \(h_{bf} = 12.0\) [in]
    
    - \(< 2.5c\) side blowout is NOT applicable
      
      - \(s_{11} = 0.0\) [in] \(s = s_1 = 0.0\) [in]

- Single anchor SB resistance: \(\phi_{lt,c} N_{sb} = 0.0\) [kips]

- Multiple anchors SB resistance: \(\phi_{lt,c} N_{sbg,d} =\)
  
  - work as a group - not applicable: \((1+s/6c) \times \phi_{lt,c} N_{sb}\) \(= 0.0\) [kips]
  - work individually - not applicable: \(n_{bd} \times \phi_{lt,c} N_{sb} \times (1+(c_1 or c_3)/c) / 4\) \(= 0.0\) [kips]

- Seismic design strength reduction: \(x = 1.0\) not applicable
- \(\phi_{lt,c} N_{sb} = 0.0\) [kips] **OK**
**Group side blowout resistance**

\[ \phi_{t,c} N_{sbg} = \phi_{t,c} \min \left( \frac{N_{bgw,n}}{n_{bw}}, \frac{N_{bgd,n}}{n_{bd}} \right) = 0.0 \text{ [kips]} \]

**Govern Tensile Resistance**

\[ N_r = \min \{ \phi_{t,s} N_{sa}, \phi_{c,c} (N_{dbg}, N_{dpn}, N_{sbg}) \} = 34.9 \text{ [kips]} \]

**Stud Shear Resistance**

\[ \phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{sa} f_{uta} = 66.4 \text{ [kips]} \]

Reduction due to built-up grout pads = \( x \times 1.0 \), not applicable

\[ \text{ratio} = 0.15 > V_u \quad \text{OK} \]

**Concrete Shear Breakout Resistance**

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment

This applies to welded stud case so only Mode 2 is considered for shear checking in Case 2

**Mode 2** Failure cone at back anchors

Bolt edge distance \( c_a1 = c_t + s_t \) = 21.0 [in]

Limiting \( c_a1 \) when anchors are influenced by 3 or more edges = Yes

Bolt edge distance - adjusted \( c_a1 \) needs to be adjusted = 10.0 [in]

\[ c_2 = 5.0 \text{ [in]} \]

\[ 1.5c_{a1} = 15.0 \text{ [in]} \]

\[ A_{Vc} = [\min(c_2, 1.5c_{a1}) + s_2 + \min(c_4, 1.5c_{a1})] x \min(1.5c_{a1}, h_a) = 390.0 \text{ [in}^2] \]

\[ A_{Vco} = 4.5c^2 \]

\[ A_{Vc} = \min (A_{Vco}, n_2 A_{Vcd}) = 390.0 \text{ [in}^2] \]

\[ l_9 = \min (8d_a, h_{at}) = 8.0 \text{ [in]} \]

\[ V_b = \left[ 8 \left( \frac{L}{d_a} \right) \sqrt{d_a} \right] A \sqrt{f_{c_{a1}}} c_{a1}^{1.5} = 25.7 \text{ [kips]} \]
### Code Reference

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>ACI 318-08</th>
</tr>
</thead>
</table>

### Eccentricity effects
\[ \Psi_{ec,v} = 1.0 \quad \text{shear acts through center of group} \]  
**D.6.2.5**

### Edge effects
\[ \Psi_{ed,v} = \min \left( \frac{0.7 + 0.3c_2/1.5c_1}{0.80} \right) \]  
**D.6.2.6**

### Concrete cracking
\[ \Psi_{c,v} = 1.20 \]  
**D.6.2.7**

### Member thickness
\[ \Psi_{h,v} = \max \left( \frac{\sqrt{1.5c_1}}{h_a}, 1.0 \right) \]  
**D.6.2.8**

### Conc. shear breakout resistance
\[ V_{cbg2} = \phi_{c,v} \frac{A_{cbg}}{A_{vb}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_b = 16.1 \text{ [kips]} \]  
**D.6.2.1 (D-22)**

### Seismic design strength reduction
\[ = x \times 1.0 \quad \text{not applicable} \]  
**D.3.3.3**

### Conc. Pryout Shear Resistance
\[ k_{cp} = 2.0 \]  
**D.6.3**

### Factored shear pryout resistance
\[ \phi_{c,v} V_{cp} = \phi_{c,v} k_{cp} N_{cbg} = 65.1 \text{ [kips]} \]  
**D.6.3 (D-31)**

\[ \phi_{c,v} = 0.70 \quad \text{pryout strength is always Condition B} \]  
**D.4.4(c)**

### Seismic design strength reduction
\[ = x \times 1.0 \quad \text{not applicable} \]  
**D.3.3.3**

### Govern Shear Resistance
\[ V_r = \min \left( \phi_{v,s} V_{sa}, \phi_{v,c} (V_{cbg}, V_{cpg}) \right) = 16.1 \text{ [kips]} \]

### Tension Shear Interaction
Check if \( N_t > 0.2N_s \) and \( V_u > 0.2V_n \):

\[ N_t/N_s + V_u/V_n = 1.20 \]  
**D.7.3 (D-32)**

\[ \text{ratio} = 1.00 \quad < 1.2 \quad \text{OK} \]

### Ductility Tension
\[ \phi_{t,s} N_{sa} = 153.2 \text{ [kips]} \]

\[ > \phi_{t,c} \min (N_{cbg}, N_{cp}, N_{cbg}) = 34.9 \text{ [kips]} \]

**Non-ductile**

### Ductility Shear
\[ \phi_{v,s} V_{sa} = 66.4 \text{ [kips]} \]

\[ > \phi_{v,c} \min (V_{cbg}, V_{cpg}) = 16.1 \text{ [kips]} \]

**Non-ductile**
Example 32: Welded Stud + No Anchor Reinft + Tension & Shear + CSA A23.3-04 Code

\[ N_t = 89 \text{ kN} \quad \text{(Tension)} \quad V_u = 44.5 \text{ kN} \]

Concrete  \( f'_c = 31 \text{ MPa} \)

Anchor stud  AWS D1.1 Grade B  1.0" dia  \( h_{ef} = 305 \text{mm} \)  \( h_a = 381 \text{mm} \)

Seismic design  \( I_E F_a S_a(0.2) < 0.35 \)

Supplementary reinforcement  Tension \( \rightarrow \) Condition A  
Shear \( \rightarrow \) Condition A  \( \psi_{c,v} = 1.2 \)

No built-up grout pad for embedded plate.

**Note:** The stud length used in this example may not be commercially available and it's for illustration purpose only.
### STUD ANCHOR DESIGN  
**Combined Tension and Shear**

Anchor bolt design based on:

- **CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D**
- **ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary**
- **PIP STE05121 Anchor Bolt Design Guide-2006**

**Input Data**

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<tr>
<th>Parameter</th>
<th>Code Abbreviation</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored tension ( N_u )</td>
<td>CSA-A23.3-04 (R2010)</td>
<td>A23.3-04 (R2010)</td>
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<tr>
<td>Factored shear ( V_u )</td>
<td>ACI 318 M-08</td>
<td>ACI318 M-08</td>
</tr>
<tr>
<td>Concrete strength ( f'_c )</td>
<td>PIP STE05121</td>
<td>PIP STE05121</td>
</tr>
<tr>
<td>Anchor tensile strength ( f_{uta} )</td>
<td>A23.3-04 (R2010)</td>
<td>D.2</td>
</tr>
</tbody>
</table>

**Factored Force Input**

- \( N_u = 89.0 \) kN = 20.0 kips
- \( V_u = 44.5 \) kN = 10.0 kips

**Concrete Properties**

- \( f'_c = 31 \) MPa = 4.5 ksi
- \( f_{uta} = 65 \) ksi = 448 MPa

**Anchor Bolt Material**

- Stud is ductile steel element

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Code Abbreviation</th>
<th>Code Reference</th>
</tr>
</thead>
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<tr>
<td>Stud diameter ( d_a )</td>
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<td>D.2</td>
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<tr>
<td>Stud shank area ( A_{se} )</td>
<td>PIP STE05121</td>
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<tr>
<td>Stud head bearing area ( A_{brg} )</td>
<td>A23.3-04 (R2010)</td>
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</tr>
</tbody>
</table>

- \( d_a = 1 \) in = 25.4 mm
- \( A_{se} = 0.79 \) in\(^2\) = 507 mm\(^2\)
- \( A_{brg} = 1.29 \) in\(^2\) = 831 mm\(^2\)

**Minimum Required Anchor Bolt Embedment Depth**

- \( h_{ef} = 305 \) mm = 305 OK

**Concrete Thickness**

- \( h_a = 381 \) mm = 381 OK

**Stud Edge Distance**

- \( c_1 = 127 \) mm = 115 OK
- \( c_2 = 127 \) mm = 115 OK
- \( c_3 = 127 \) mm = 115 OK
- \( c_4 = 127 \) mm = 115 OK

- \( c_i > 1.5h_{ef} \) for at least two edges to avoid reducing of \( h_{ef} \) when \( N_u > 0 \) No D.6.2.3

**Adjusted \( h_{ef} \) for Design**

- \( h_{ef} = 135 \) mm = 305 Warn D.6.2.3

**Outermost Stud Line Spacing**

- \( s_1 = 406 \) mm = 102 OK PIP STE05121
- \( s_2 = 406 \) mm = 102 OK

**Design Diagram**

- [Diagram showing anchor bolt design with dimensions and forces]
No of stud at bolt line 1  \( n_1 = 2 \)
No of stud at bolt line 2  \( n_2 = 2 \)
Total no of welded stud  \( n = 4 \)
No of stud carrying tension  \( n_t = 4 \)
No of stud carrying shear  \( n_s = 2 \)

For side-face blowout check use
No of stud along width edge  \( n_{bw} = 2 \)
No of stud along depth edge  \( n_{bd} = 2 \)

Seismic region where \( I_{EFaS_S(0.2)} \geq 0.35 \):

- Supplementary reinforcement
  - For tension  \( \Psi = \text{Condition A} \)
  - For shear  \( \Psi_{c,V} = 1.2 \)  \( \text{Condition A} \)
  - Provide built-up grout pad?  \( \text{No} \)

Strength reduction factors
- Anchor reinforcement factor  \( \phi_a = 0.75 \)
- Steel anchor resistance factor  \( \phi_s = 0.85 \)
- Concrete resistance factor  \( \phi_c = 0.65 \)

Resistance modification factors
- Anchor rod - ductile steel  \( R_{ts} = 0.80 \)  \( R_{ts} = 0.75 \)
- Concrete  \( R_{tc} = 1.15 \)  \( R_{tc} = 1.15 \)

Assumptions
1. Concrete is cracked
2. Condition A for tension - supplementary reinforcement provided
3. Tensile load acts through center of bolt group  \( \Psi_{ec,N} = 1.0 \)
4. Shear load acts through center of bolt group  \( \Psi_{ec,V} = 1.0 \)
## CONCLUSION

### Anchor Rod Embedment, Spacing and Edge Distance

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<thead>
<tr>
<th>Category</th>
<th>Ratio</th>
<th>Status</th>
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</thead>
<tbody>
<tr>
<td>Overall</td>
<td>1.01</td>
<td>NG</td>
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<tr>
<td>Tension</td>
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<tr>
<td>Stud Tensile Resistance</td>
<td>0.14</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Tensile Breakout Resistance</td>
<td>0.58</td>
<td>OK</td>
</tr>
<tr>
<td>Stud Pullout Resistance</td>
<td>0.17</td>
<td>OK</td>
</tr>
<tr>
<td>Side Blowout Resistance</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td>Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stud Shear Resistance</td>
<td>0.15</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Shear Breakout Resistance</td>
<td>0.63</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Pryout Shear Resistance</td>
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<tr>
<td>Stud on Conc Bearing</td>
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<td>Tension Shear Interaction</td>
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<tr>
<td>Seismic Design Requirement</td>
<td>OK</td>
<td>D.4.3.6</td>
</tr>
</tbody>
</table>

*Note: The NS5.3-12 seismic design requirement is NOT met.*

### CALCULATION

#### Stud Tensile Resistance

\[ N_{st} = n_t A_{st} f_{st} R_{ts} \]

\( \phi_r = 0.14 > 1 \) OK

#### Conc. Tensile Breakout Resistance

\[ N_{cb} = 10 \phi_r \sqrt{f_{ct}^2 h_{cf}^2 + R_{cs}} \]

\( \phi_r = 0.58 \)

### Ductility

<table>
<thead>
<tr>
<th>Category</th>
<th>Tension</th>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Design Requirement</td>
<td>Non-ductile</td>
<td>Non-ductile</td>
</tr>
</tbody>
</table>

*Note: The NS5.3-12 seismic design requirement is NOT met.*

### Seismic Design Requirement

\( \text{Ie} \text{F} \text{S} \text{a}(0.2) < 0.35 \), A23.3-04 D.4.3.3 ductility requirement is NOT required
## Concrete Breakout Resistance

Concrete breakout resistance

\[ N_{cbgr} = \frac{A_{ho}}{A_{no}} \Psi_{sc,n} \Psi_{ed,n} \Psi_{cp,n} N_f \]

\[ = 153.7 \text{ [kN]} \]  

Seismic design strength reduction

\[ = x \times 1.0 \text{ not applicable} \]

\[ = 153.7 \text{ [kN]} \]  

Seismic design strength reduction ratio

\[ = 0.58 > N_u \quad \text{OK} \]

## Stud Pullout Resistance

### Single Bolt Pullout Resistance

Seismic design strength reduction

\[ = x \times 1.0 \quad \text{not applicable} \]

\[ = 536.0 \text{ [kN]} \]  

Stud pullout resistance

\[ N_{p} = 8 A_{bol} \phi_c f'_{c} R_{t,c} \]

\[ = 134.0 \text{ [kN]} \]  

Seismic design strength reduction ratio

\[ = 0.17 > N_u \quad \text{OK} \]

### Side Blowout Resistance

#### Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[ N_{bw} = N_u \times n_{bw} / n_t \]

\[ = 44.5 \text{ [kN]} \]  

Check if side blowout applicable

\[ h_{ef} = 305 \text{ [mm]} \]

< 2.5c side blowout is NOT applicable

Check if edge anchors work as a group or work individually

\[ s = s_2 = 0 \text{ [mm]} \]

< 6c side blowout is NOT applicable

### Single Anchor SB Resistance

\[ N_{sbr, w} = 0.0 \text{ [kN]} \]  

### Multiple Anchors SB Resistance

\[ N_{sbgr, w} = (1 + s / 6c) x N_{sbr, w} \]

\[ = 0.0 \text{ [kN]} \]  

\[ \phi \]

## Failure Along Pedestal Depth Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[ N_{bd} = N_u \times n_{bd} / n_t \]

\[ = 44.5 \text{ [kN]} \]  

Check if side blowout applicable

\[ h_{ef} = 305 \text{ [mm]} \]

< 2.5c side blowout is NOT applicable

Check if edge anchors work as a group or work individually

\[ s = s_1 = 0 \text{ [mm]} \]

< 6c side blowout is NOT applicable

### Single Anchor SB Resistance

\[ N_{sbr, d} = 0.0 \text{ [kN]} \]  

### Multiple Anchors SB Resistance

\[ N_{sbgr, d} = (1 + s / 6c) x \phi_{c} \sqrt[4]{R_{t,c}} \]

\[ = 0.0 \text{ [kN]} \]  

Seismic design strength reduction ratio

\[ = 0.00 < N_{bw} \quad \text{OK} \]
Group side blowout resistance

\[ N_{sbgr} = \min \left( \frac{N_{nbow}}{n_n}, \frac{N_{sbbr}}{n_s} \right) \] = 0.0 [kN]  

Code Reference: A23.3-04 (R2010)

Govern Tensile Resistance

\[ N_r = \min (N_{sr}, N_{br}, N_{cpr}, N_{sbgr}) \] = 153.7 [kN]

Conc. Shear Breakout Resistance

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Only Case 2 needs to be considered when anchors are rigidly connected to the attachment

This applies to welded stud case so only Mode 2 is considered for shear checking in Case 2
### Code Reference

**A23.3-04 (R2010)**

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>Description</th>
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<tbody>
<tr>
<td>D.7.2.5</td>
<td>Eccentricity effects</td>
</tr>
<tr>
<td>D.7.2.6</td>
<td>Edge effects</td>
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<tr>
<td>D.7.2.7</td>
<td>Concrete cracking</td>
</tr>
<tr>
<td>D.7.2.8</td>
<td>Member thickness</td>
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<td>D.7.2.9</td>
<td>Conc shear breakout resistance</td>
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<td>D.7.3</td>
<td>Conc. Pryout Shear Resistance</td>
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<td>D.7.3</td>
<td>Stud on Conc Bearing</td>
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<td>D.8.2 &amp; D.8.3</td>
<td>Tension Shear Interaction</td>
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<td>D.8.4 (D-35)</td>
<td>Ductility Tension</td>
</tr>
<tr>
<td>D.8.4 (D-35)</td>
<td>Ductility Shear</td>
</tr>
</tbody>
</table>

#### Eccentricity effects

\[ \Psi_{ec,v} = 1.0 \] shear acts through center of group

#### Edge effects

\[ \Psi_{ed,v} = \min \left(0.7 + 0.3c_2/1.5ca_1, 1.0 \right) = 0.80 \]

#### Concrete cracking

\[ \Psi_{c,v} = 1.20 \]

#### Member thickness

\[ \Psi_{h,v} = \max \left(\sqrt{1.5ca_1 / h_a}, 1.0 \right) = 1.00 \]

#### Conc shear breakout resistance

\[ V_{cbgr} = \frac{A_{sw}}{A_{sw}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_{sr} = 70.6 \text{ kN} \]

#### Seismic design strength reduction

\[ = \times 1.0 \text{ not applicable} = 70.6 \text{ kN} \]

#### Conc. Pryout Shear Resistance

\[ k_{cp} = 2.0 \]

#### Factored shear pryout resistance

\[ V_{cpp} = k_{cp} N_{cbgr} = 267.3 \text{ kN} \]

\[ R_{V,c} = 1.00 \text{ pryout strength is always Condition B} \]

#### Seismic design strength reduction

\[ = \times 1.0 \text{ not applicable} = 267.3 \text{ kN} \]

#### Stud on Conc Bearing

\[ B_r = n_x \times 1.4 \times \phi_c \times \min(8d_a, h_d) \times d_a \times f'_c = 291.2 \text{ kN} \]

#### Govern Shear Resistance

\[ V_r = \min \left( V_{sr}, V_{cbgr}, V_{cpp}, B_r \right) = 70.6 \text{ kN} \]

#### Tension Shear Interaction

Check if \( N_r > 0.2 N_t \text{ and } V_{sr} > 0.2 V_t \)

Yes

\[ N_r / N_t + V_{sr} / V_t = 1.21 \]

#### Ductility Tension

\[ N_{dr} = 617.7 \text{ kN} \]

\[ > \min \left( N_{cbgr}, N_{cpp}, N_{cbgr} \right) = 153.7 \text{ kN} \]

#### Ductility Shear

\[ V_{sr} = 289.5 \text{ kN} \]

\[ > \min \left( V_{cbgr}, V_{cpp}, B_r \right) = 70.6 \text{ kN} \]
Example 33: Welded Stud + No Anchor Reinft + Tension Shear & Moment + ACI 318-08 Code

This example taken from Example 10 on page 82 of ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

\[ M_u = 30 \text{ kip-ft} \quad N_u = 0 \text{ kips} \quad V_u = 20 \text{ kips} \quad f_{c'} = 4.5 \text{ ksi} \]

Anchor stud \( d_a = 7/8 \text{ in} \quad h_{ef} = 9 \text{ in} \quad h_a = 18 \text{ in} \)

Supplementary reinforcement
- Tension \( \rightarrow \) Condition B
- Shear \( \rightarrow \) Condition A

\[ \Psi_{c,V} = 1.2 \]

Field welded plate washers to base plate at each anchor

Notes:
There are two locations in this calculation which are different from calculation in ACI 355.3R-11 Example 10
1. Concrete tension breakout \( A_{nc} = 1215 \text{ in}^2 \), different from \( A_{nc} = 1519 \text{ in}^2 \), value in ACI 355.3R-11 page 86. We assume the moment may apply in both directions. When moment causes tensile anchors being close to the edge side, the \( A_{nc} \) value is consequently reduced.
2. Concrete shear breakout \( c_{a1} \) reduction from 27" to 12" in ACI 355.3R-11 page 90 is not correct. It doesn't comply with both edge distances \( c_{a2,1} < 1.5c_{a1} \) and \( c_{a2,2} < 1.5c_{a1} \). Refer to ACI 318-11 Fig. RD.6.2.4 for more details.
STUD ANCHOR DESIGN

Combined Tension, Shear and Moment

Anchor bolt design based on:

ACI 318-08 Building Code Requirements for Structural Concrete and Commentary Appendix D

PIP STE05121 Anchor Bolt Design Guide-2006

Assumptions

1. Concrete is cracked

ACI 318-08

2. Condition B - no supplementary reinforcement provided

D.4.4 (c)

3. Load combinations shall be per ACI 318-08 Chapter 9 or ASCE 7-05 Chapter 2

D.4.4

4. Shear load acts through center of bolt group \( \Psi_{Ec,V} = 1.0 \)

D.6.2.5

5. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors

D.3.1

6. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output

Anchor Stud Data

Factored moment \( M_u = 30.0 \) [kip-ft] = 40.7 [kNm]

Factored tension /compression \( N_u = 0.0 \) [kips] = 0.0 [kN]

Factored shear \( V_u = 20.0 \) [kips] = 89.0 [kN]

No of bolt line for resisting moment = 3 Bolt Line

No of bolt along outermost bolt line = 3
### Code Reference

**PIP STE05121**

### Outermost stud line spacing \( s_1 \)

- \( s_1 = 21.0 \text{ [in]} \)
- Min required: 3.5
- Status: OK

### Outermost stud line spacing \( s_2 \)

- \( s_2 = 21.0 \text{ [in]} \)
- Min required: 3.5
- Status: OK

### Internal stud line spacing \( s_{b1} \)

- \( s_{b1} = 10.5 \text{ [in]} \)
- Min required: 3.5
- Status: OK

### Internal stud line spacing \( s_{b2} \)

- \( s_{b2} = 0.0 \text{ [in]} \)
- Min required: 3.5
- Status: OK

### Column depth \( d \)

- \( d = 13.9 \text{ [in]} \)

### Concrete strength \( f'_c \)

- \( f'_c = 4.5 \text{ [ksi]} \) = 31.0 [MPa]

### Stud material

- AWS D1.1 Grade B

### Stud tensile strength \( f_{uta} \)

- \( f_{uta} = 65 \text{ [ksi]} \) = 448 [MPa]  
  
  **ACI 318-08**

  **Stud is ductile steel element D.1**

### Stud diameter \( d_a \)

- \( d_a = 0.875 \text{ [in]} \) = 22.2 [mm]

### Stud shank area \( A_{so} \)

- \( A_{so} = 0.60 \text{ [in}^2\text{]} \) = 388 [mm²]

### Stud head bearing area \( A_{brg} \)

- \( A_{brg} = 0.88 \text{ [in}^2\text{]} \) = 570 [mm²]

### Stud embedment depth \( h_{ef} \)

- \( h_{ef} = 9.0 \text{ [in]} \) = 10.5

### Concrete thickness \( h_a \)

- \( h_a = 18.0 \text{ [in]} \) = 12.0

### Stud edge distance \( c_1 \)

- \( c_1 = 6.0 \text{ [in]} \) = 4.5

### Stud edge distance \( c_2 \)

- \( c_2 = 6.0 \text{ [in]} \) = 4.5

### Stud edge distance \( c_3 \)

- \( c_3 = 100.0 \text{ [in]} \) = 4.5

### Stud edge distance \( c_4 \)

- \( c_4 = 100.0 \text{ [in]} \) = 4.5

\( c_i > 1.5h_{ef} \) for at least two edges to avoid reducing of \( h_{ef} \) when \( N_y > 0 \)

- \( c_i > 1.5h_{ef} \) for at least two edges to avoid reducing of \( h_{ef} \) when \( N_y > 0 \)

- Yes

### Adjusted \( h_{ef} \) for design

- \( h_{ef} = 9.00 \text{ [in]} \) = 10.5

**Warn**

### Diagrams

- Schematic diagrams illustrating the anchorage system with dimensions labeled.

---

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### Code Reference

<table>
<thead>
<tr>
<th>Description</th>
<th>Formula</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of stud at bolt line 1</td>
<td>( n_1 = 3 )</td>
<td></td>
</tr>
<tr>
<td>Number of stud at bolt line 2</td>
<td>( n_2 = 3 )</td>
<td></td>
</tr>
<tr>
<td>Total no of welded stud</td>
<td>( n = 8 )</td>
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</tr>
<tr>
<td>Number of stud carrying tension</td>
<td>( n_t = 5 )</td>
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</tr>
<tr>
<td>Number of stud carrying shear</td>
<td>( n_s = 3 )</td>
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</tr>
<tr>
<td>Seismic design category</td>
<td>&gt;= C</td>
<td>D.3.3.3</td>
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<tr>
<td>Supplementary reinforcement</td>
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<td></td>
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<tr>
<td>For tension</td>
<td>( \psi_{c,t} = 1.2 )</td>
<td>Condition A</td>
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<tr>
<td>For shear</td>
<td></td>
<td>D.6.2.7</td>
</tr>
<tr>
<td>Provide built-up grout pad</td>
<td></td>
<td>D.6.1.3</td>
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<tr>
<td>Strength reduction factors</td>
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<tr>
<td>Anchor reinforcement</td>
<td>( \phi_s = 0.75 )</td>
<td>D.5.2.9 &amp; D.6.2.9</td>
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<td>( \phi_{t,s} = 0.75 ) ( \phi_{v,s} = 0.65 )</td>
<td>D.4.4 (a)</td>
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<tr>
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<td>( \phi_{c,t} = 0.70 ) Cdn-B</td>
<td>( \phi_{v,c} = 0.75 ) Cdn-A D.4.4 (c)</td>
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### Conclusion

**Anchor Rod Embedment, Spacing and Edge Distance**

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<thead>
<tr>
<th>Description</th>
<th>Ratio</th>
<th>Status</th>
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<tr>
<td>Overall</td>
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<tr>
<td><strong>Tension</strong></td>
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<tr>
<td>Stud Tensile Resistance</td>
<td>0.21</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Tensile Breakout Resistance</td>
<td>0.64</td>
<td>OK</td>
</tr>
<tr>
<td>Stud Pullout Resistance</td>
<td>0.28</td>
<td>OK</td>
</tr>
<tr>
<td>Side Blowout Resistance</td>
<td>0.00</td>
<td>OK</td>
</tr>
<tr>
<td><strong>Shear</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stud Shear Resistance</td>
<td>0.26</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Shear Breakout Resistance</td>
<td>0.50</td>
<td>OK</td>
</tr>
<tr>
<td>Conc. Pryout Shear Resistance</td>
<td>0.27</td>
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<tr>
<td><strong>Tension Shear Interaction</strong></td>
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<tr>
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<td>OK</td>
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<td><strong>Ductility</strong></td>
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<td></td>
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<tr>
<td>Tension</td>
<td>Non-ductile</td>
<td></td>
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<td>Shear</td>
<td>Non-ductile</td>
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<td><strong>Seismic Design Requirement</strong></td>
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<td>SDC&lt; C, ACI318-08 D.3.3 ductility requirement is NOT required</td>
<td>OK</td>
<td>D.3.3.4</td>
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CALCULATION

Anchor Stud Tensile Force

<table>
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<tr>
<th>Single bolt tensile force</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>T&lt;sub&gt;1&lt;/sub&gt; = 6.22 [kips]</td>
<td>No of bolt for T&lt;sub&gt;1&lt;/sub&gt; n&lt;sub&gt;T1&lt;/sub&gt; = 3</td>
</tr>
<tr>
<td>T&lt;sub&gt;2&lt;/sub&gt; = 2.48 [kips]</td>
<td>No of bolt for T&lt;sub&gt;2&lt;/sub&gt; n&lt;sub&gt;T2&lt;/sub&gt; = 2</td>
</tr>
<tr>
<td>T&lt;sub&gt;3&lt;/sub&gt; = 0.00 [kips]</td>
<td>No of bolt for T&lt;sub&gt;3&lt;/sub&gt; n&lt;sub&gt;T3&lt;/sub&gt; = 0</td>
</tr>
</tbody>
</table>

Sum of bolt tensile force

\[ N_u = \sum n_{T_i} T_i = 23.6 \text{ [kips]} \]

Tensile bolts outer distance \( s_{ob} \)

\[ s_{ob} = 10.5 \text{ [in]} \]

Eccentricity \( e'_N \) -- distance between resultant of tensile load and centroid of anchors

Loaded in tension

\[ e'_N = 2.00 \text{ [in]} \]

Fig. RD.5.2.4 (b)

Eccentricity modification factor

\[ \Psi_{ec,N} = \frac{1}{1 + \frac{2 s_{ob}}{3 h_{ef}}} = 0.87 \]

\[ \text{D.5.2.4 (D-9)} \]

Stud Tensile Resistance

\[ \phi_{ts} N_{sa} = \phi_{ts} A_{se} f_{ula} = 29.3 \text{ [kips]} \]

\[ \text{ratio} = 0.21 > T_1 \text{ OK} \]

Conc. Tensile Breakout Resistance

\[ N_b = 24 \frac{h_{ef}^{1.5}}{h_{ef}^{0.5}} \text{ if } h_{ef} < 11^* \text{ or } h_{ef} > 25^* = 43.5 \text{ [kips]} \]

\[ 16 \frac{h_{ef}^{1.5}}{h_{ef}^{0.5}} \text{ if } 11^* \leq h_{ef} \leq 25^* \]

\[ \text{D.5.2.2 (D-7)} \]

\[ \text{D.5.2.2 (D-8)} \]

Projected conc failure area

\[ 1.5h_{ef} = 13.50 \text{ [in]} \]

\[ A_{nc} = [s_{ib} + \min(c_1, 1.5h_{ef}) + \min(c_3, 1.5h_{ef})]x = 1215.0 \text{ [in}^2] \]

\[ s_{ib} + \min(c_2, 1.5h_{ef}) + \min(c_4, 1.5h_{ef})] \]

\[ A_{ncvo} = 9 h_{ef}^2 = 729.0 \text{ [in}^2] \]

\[ A_{nc} = \min( A_{ncvo}, n_t A_{ndvo}) = 1215.0 \text{ [in}^2] \]

\[ \text{D.5.2.1} \]

\[ \text{D.5.2.1} \]

Min edge distance

\[ c_{min} = \min(c_1, c_2, c_3, c_4) = 6.0 \text{ [in]} \]

\[ \text{Eccentricity effects} \]

\[ \Psi_{ec,N} = 0.87 \]

\[ \text{D.5.2.4 (D-9)} \]

\[ \text{Edge effects} \]

\[ \Psi_{ed,N} = \min(0.7 + 0.3c_{min}/1.5h_{ef}, 1.0) = 0.83 \]

\[ \text{D.5.2.5} \]

\[ \text{Concrete cracking} \]

\[ \Psi_{c,N} = 1.0 \text{ for cracked concrete} \]

\[ \text{D.5.2.6} \]

\[ \text{Concrete splitting} \]

\[ \Psi_{cp,N} = 1.0 \text{ for cast-in anchor} \]

\[ \text{D.5.2.7} \]

Concrete breakout resistance

\[ \phi_{ts} N_{cbg} = \phi_{ts} A_{bw} A_{se,N} \Psi_{ws,N} \Psi_{es,N} \Psi_{ps,N} N_b = 36.8 \text{ [kips]} \]

\[ \text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 36.8 \text{ [kips]} \]

\[ \text{ratio} = 0.64 > N_u \text{ OK} \]

Stud Pullout Resistance

Single bolt pullout resistance

\[ N_p = 8 A_{wog} f_{cu} = 31.8 \text{ [kips]} \]

\[ \phi_{tc} N_{pn} = \phi_{tc} \Psi_{w,p} N_p = 22.3 \text{ [kips]} \]

\[ \text{Seismic design strength reduction} = x 1.0 \text{ not applicable} = 22.3 \text{ [kips]} \]

\[ \text{ratio} = 0.28 > T_1 \text{ OK} \]

\[ \Psi_{w,p} = 1 \text{ for cracked conc} \]

\[ \phi_{tc} = 0.70 \text{ pullout strength is always Condition B} \]

\[ \text{D.5.3.6} \]

\[ \text{D.4.4(c)} \]
Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout along pedestal width edge

\[ N_{bw} = n T_1 \]

where

\[ T_1 = 18.7 \text{ [kips]} \]

RD.5.4.2

Check if side blowout applicable

\[ h_{bw} = 9.0 \text{ [in]} \]

< 2.5c side bowout is NOT applicable D.5.4.2

Check if edge anchors work as a group or work individually

\[ s < 2.5c \text{ side bowout is NOT applicable} \]

< 6c side bowout is NOT applicable D.5.4.1

Single anchor SB resistance

\[ \phi_{t,c} N_{sb} = \phi_{t,c} \left( 160 c \sqrt{A_{avg}} \right) \lambda \sqrt{t_2} \]

= 0.0 [kips] D.5.4.1 (D-17)

Multiple anchors SB resistance

\[ \phi_{t,c} N_{sbg,w} = \]

work as a group - not applicable

\[ = (1+s/6c) \times \phi_{t,c} N_{sb} \]

= 0.0 [kips] D.5.4.2 (D-18)

work individually - not applicable

\[ = n_{bw} \times \phi_{t,c} N_{sb} \times \left( 1+c_2 or c_4 \right) / 4 \]

= 0.0 [kips] D.5.4.1

Seismic design strength reduction

\[ \text{ratio} = x 1.0 \text{ not applicable} \]

\[ \leq N_{bw} \text{ OK} \]

Group side blowout resistance

\[ \phi_{t,c} N_{sbg} = \phi_{t,c} N_{avg} \times \frac{n_t}{n_1} \]

= 0.0 [kips]

Govern Tensile Resistance

\[ N_r = \min \left[ \phi_{t,s} n_1 N_{sb}, \phi_{t,c} (N_{sbg}, n_1 N_{pbc}, N_{sbw}) \right] = 36.8 \text{ [kips]} \]

Stud Shear Resistance

\[ \phi_{v,s} V_{sa} = \phi_{v,s} n_s A_{w} f_{uta} = 76.2 \text{ [kips]} \]

D.6.1.2 (a) (D-19)

Reduction due to built-up grout pads

\[ \text{ratio} = x 1.0, \text{ not applicable} \]

\[ \geq V_u \text{ OK} \]

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment Fig. RD.6.2.1(b) notes

This applies to welded stud case so only Mode 2 is considered for shear checking in Case 2

Mode 2 Failure cone at back anchors
Bolt edge distance
\( c_{a1} = 27.0 \) [in]  
Limiting \( c_{a1} \) when anchors are influenced by 3 or more edges  
\( \text{No} \)  
\( c_{a1} \) needs NOT to be adjusted  
\( 27.0 \) [in]  
\( c_2 = 6.0 \) [in]  
\( 1.5 c_{a1} = 40.5 \) [in]  
\( A_{vc} = [\min(c_2,1.5c_{a1}) + s_2 + \min(c_4,1.5c_{a1})] x \min(1.5c_{a1}, h_a) \)  
\( 1215.0 \) [in²]  
\( A_{vco} = 4.5c_{a1}^2 \)  
\( 3280.5 \) [in²]  
\( l_s = \min(8d_a, h_{ef}) \)  
\( 7.0 \) [in]  
\( V_b = \left( \frac{1}{\sqrt{d_b}} \right)^{0.2} \sqrt{d_s} \left( \frac{1}{\sqrt{c^{1.5}_{a1}}} \right) \)  
\( 106.7 \) [kips]  
Eccentricity effects  
\( \Psi_{ec,v} = 1.0 \) shear acts through center of group  
\( 0.74 \)  
Edge effects  
\( \Psi_{ed,v} = \min[(0.7+0.3c_2/1.5c_{a1}), 1.0] \)  
\( 1.20 \)  
Concrete cracking  
\( \Psi_{c,v} = \)  
\( 1.50 \)  
Member thickness  
\( \Psi_{h,v} = \max[(\sqrt{1.5c_{a1}/h_a}), 1.0] \)  
\( 1.50 \)  
Conc shear breakout resistance  
\( V_{dcp2} = \phi_{v,c} A_{vco} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} \Psi_{h,v} V_b \)  
\( 39.7 \) [kips]  
Seismic design strength reduction  
\( \times 1.0 \) not applicable  
\( 39.7 \) [kips]  
ratio = 0.50  
\( > V_u \)  
OK  
Conc. Pryout Shear Resistance  
\( k_{cp} = 2.0 \)  
Factored shear pryout resistance  
\( \phi_{v,c} V_{dcp} = \phi_{v,c} k_{cp} N_{dcp} \)  
\( 73.6 \) [kips]  
\( \phi_{v,c} = 0.70 \) pryout strength is always Condition B  
\( 8.4.4(c) \)  
Seismic design strength reduction  
\( \times 1.0 \) not applicable  
\( 73.6 \) [kips]  
ratio = 0.27  
\( > V_u \)  
OK  
Govern Shear Resistance  
\( V_r = \min \{ \phi_{v,c} V_{sa}, \phi_{v,c} (V_{dcp}, V_{dcp}) \} \)  
\( 39.7 \) [kips]  
Tension Shear Interaction  
Check if \( N_s > 0.2 \phi_s N_t \) and \( V_u > 0.2 \phi_s V_n \)  
\( \text{Yes} \)  
\( N_s/N_t + V_u/V_n \)  
\( 1.14 \)  
\( 0.95 \)  
\( < 1.2 \)  
OK  
Ductility Tension  
\( \phi_{v,s} N_{sa} = 29.3 \) [kips]  
\( \times \phi_{v,s} \min(N_{dcp}, N_{dcp}, N_{dcp}) \)  
\( 22.3 \) [kips]  
Ductility Shear  
\( \phi_{v,s} V_{sa} = 76.2 \) [kips]  
\( \times \phi_{v,s} \min(V_{dcp}, V_{dcp}) \)  
\( 39.7 \) [kips]  
Non-ductile
Example 34: Welded Stud + No Anchor Reinft + Tension Shear & Moment + CSA A23.3-04 Code

This example taken from Example 10 on page 82 of ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D

\[ M_u = 40.7 \text{ kNm} \quad N_u = 0 \text{ kN}, \quad V_u = 89 \text{ kN}, \quad f_{c'} = 31 \text{ MPa} \]

Anchor stud \( d_a = 7/8 \text{ in} \) \( h_{ef} = 229 \text{ mm} \) \( h_d = 457 \text{ mm} \)

Supplementary reinforcement
Tension \( \rightarrow \) Condition B
Shear \( \rightarrow \) Condition A \( \Psi_{c,V} = 1.2 \)

Provide built-up grout pad
Seismic is not a consideration

Field welded plate washers to base plate at each anchor

Notes:

There are two locations in this calculation which are different from calculation in ACI 355.3R-11 Example 10

1. Concrete tension breakout \( A_{nc} = 1215 \text{ in}^2 \), different from \( A_{nc} = 1519 \text{ in}^2 \), value in ACI 355.3R-11 page 86.
   We assume the moment may apply in both directions. When moment causes tensile anchors being close to the edge side, the \( A_{nc} \) value is consequently reduced.

2. Concrete shear breakout \( c_a \) reduction from 27\" to 12\" in ACI 355.3R-11 page 90 is not correct. It doesn't comply with both edge distances \( c_{a2,1} < 1.5c_a \) and \( c_{a2,2} < 1.5c_a \). Refer to ACI 318-11 Fig. RD.6.2.4 for more details.
STUD ANCHOR DESIGN  Combined Tension, Shear and Moment

Anchor bolt design based on
CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
PIP STE05121 Anchor Bolt Design Guide-2006

Assumptions
1. Concrete is cracked
2. Condition B for tension - no supplementary reinforcement provided
3. Shear load acts through center of bolt group $\Psi_{\text{ec,V}} = 1.0$
4. For anchor group subject to moment, the anchor tensile load is designed using elastic analysis and there is no redistribution of the forces between highly stressed and less stressed anchors
5. For anchor tensile force calc in anchor group subject to moment, assume the compression resultant is at the outside edge of the compression flange and base plate exhibits rigid-body rotation. This simplified approach yields conservative output

Anchor Stud Data

| Factored moment     | $M_u = 40.7$ [kNm] | $= 30.0$ [kip-ft] |
| Factored tension /compression | $N_u = 0.0$ [kN] | $= 0.0$ [kips] |
| Factored shear       | $V_u = 89.0$ [kN] | $= 20.0$ [kips] |

No of bolt line for resisting moment = 3 Bolt Line

No of bolt along outermost bolt line = 3
Outermost stud line spacing $s_1 = 533$ [mm] 89

Outermost stud line spacing $s_2 = 533$ [mm] 89

Internal stud line spacing $s_{b1} = 267$ [mm] 89

Internal stud line spacing $s_{b2} = 0$ [mm] 89

Column depth $d = 353$ [mm]

Concrete strength $f'_c = 31$ [MPa] = 4.5 [ksi]

Anchor bolt material = AWS D1.1 Grade B

Anchor tensile strength $f_{uta} = 65$ [ksi] = 448 [MPa] A23.3-04 (R2010) Stud is ductile steel element D.2

Stud diameter $d_a = 0.875$ [in] = 22.2 [mm]

Stud shank area $A_{so} = 0.60$ [in$^2$] = 388 [mm$^2$]

Stud head bearing area $A_{brg} = 0.88$ [in$^2$] = 570 [mm$^2$]

Anchor bolt embedment depth $h_{ef} = 229$ [mm] 267 Warn Page A -1 Table 1

Concrete thickness $h_a = 457$ [mm] 305 OK

Stud edge distance $c_1 = 152$ [mm] 115 OK Page A -1 Table 1

Stud edge distance $c_2 = 152$ [mm] 115 OK

Stud edge distance $c_3 = 2540$ [mm] 115 OK

Stud edge distance $c_4 = 2540$ [mm] 115 OK A23.3-04 (R2010)

$c_i > 1.5h_{ef}$ for at least two edges to avoid reducing of $h_{ef}$ when $N_i > 0$ Yes D.6.2.3

Adjusted $h_{ef}$ for design $h_{ef} = 229$ [mm] 267 Warn D.6.2.3
No of stud at bolt line 1 \( n_1 = 3 \)
No of stud at bolt line 2 \( n_2 = 3 \)
Total no of welded stud \( n = 8 \)
No of stud carrying tension \( n_t = 5 \)
No of stud carrying shear \( n_s = 3 \)
Seismic region where \( I_{EFa}S_a(0.2) \geq 0.35 \) ? \( \text{No} \)

**Code Reference**

A23.3-04 (R2010)

<table>
<thead>
<tr>
<th>Supplementary reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>For tension ( \psi_{c,V} = 1.2 )</td>
</tr>
<tr>
<td>For shear ( \psi_{c,V} = 1.2 )</td>
</tr>
<tr>
<td>Provide built-up grout pad ?</td>
</tr>
</tbody>
</table>

**Strength reduction factors**

| Anchor reinforcement factor | \( \psi_{as} = 0.75 \) | D.7.2.9 |
| Steel anchor resistance factor | \( \psi_s = 0.85 \) | 8.4.3 (a) |
| Concrete resistance factor | \( \psi_c = 0.65 \) | 8.4.2 |

**Resistance modification factors**

| Anchor rod - ductile steel | \( R_{c,s} = 0.80 \) | \( R_{as,s} = 0.75 \) | D.5.4(a) |
| Concrete | \( R_{c,c} = 1.00 \) | \( R_{as,c} = 1.15 \) | Cdn-B | Cdn-A | D.5.4(c) |

**CONCLUSION**

**Anchor Rod Embedment, Spacing and Edge Distance**

| Overall | ratio = 1.00 | NG |
| Tension |
| Stud Tensile Resistance | ratio = 0.23 | OK |
| Conc. Tensile Breakout Resistance | ratio = 0.69 | OK |
| Stud Pullout Resistance | ratio = 0.30 | OK |
| Side Blowout Resistance | ratio = 0.00 | OK |
| Shear |
| Stud Shear Resistance | ratio = 0.27 | OK |
| Conc. Shear Breakout Resistance | ratio = 0.51 | OK |
| Conc. Pryout Shear Resistance | ratio = 0.29 | OK |
| Stud on Conc Bearing | ratio = 0.27 | OK |
| Tension Shear Interaction |
| Tension Shear Interaction | ratio = 1.00 | NG |
| Ductility |
| Tension | Non-ductile |
| Shear | Non-ductile |
| Seismic Design Requirement |
| IeFaSa(0.2)<0.35, A23.3-04 D.4.3.3 ductility requirement is NOT required | OK | D.4.3.6 |
CALCULATION

Anchor Tensile Force

Single stud tensile force
- \( T_1 = 27.7 \) [kN] No of stud for \( T_1 n_{T1} = 3 \)
- \( T_2 = 11.0 \) [kN] No of stud for \( T_2 n_{T2} = 2 \)
- \( T_3 = 0.0 \) [kN] No of stud for \( T_3 n_{T3} = 0 \)

Sum of stud tensile force
- \( N_u = \sum n_i T_i = 105.1 \) [kN]

Tensile studs outer distance \( s_{ib} \)
- \( s_{ib} = 267 \) [mm]

Eccentricity \( e'N \) -- distance between resultant of tensile load and centroid of studs
- loaded in tension \( e'_N = 51 \) [mm] Figure D.8 (b)

Eccentricity modification factor
- \( \Psi_{ec,N} = \frac{1}{1 + \frac{2e'_N}{3h_{ud}}} = 0.87 \) D.6.2.4 (D-9)

Stud Tensile Resistance
- \( N_{sf} = A_{se} f_{lsd} R_{la} = 118.2 \) [kN] D.6.1.2 (D-3)
- ratio = 0.23 > \( T_1 \) OK

Conc. Tensile Breakout Resistance
- \( N_{cb} = 10 \sqrt{\frac{3}{5}} h_{cf}^{3/5} R_{la} = 125.4 \) [kN] D.6.2.2 (D-7)
- Projected conc failure area
  - \( 1.5h_{ef} = 344 \) [mm]
  - \( A_{nc0} = \left[ s_b + \min(c_1, 1.5h_{ud}) + \min(c_3, 1.5h_{ud}) \right] \times [s_e + \min(c_2, 1.5h_{ud}) + \min(c_4, 1.5h_{ud})] = 7.8E+05 \) [mm²]
  - \( A_{nc0} = 9 h_{ef}^2 = 4.7E+05 \) [mm²] D.6.2.1 (D-6)
  - \( A_{nc0} = \min \left( A_{nc0}, n_t A_{nc0} \right) = 7.8E+05 \) [mm²] D.6.2.1
- Min edge distance
  - \( c_{min} = \min(c_1, c_2, c_3, c_4) = 152 \) [mm]
- Eccentricity effects
  - \( \Psi_{ec,N} = 0.87 \) D.6.2.4 (D-9)
- Edge effects
  - \( \Psi_{ed,N} = \min \left( 0.7 + 0.3c_{min}/1.5h_{ud}, 1.0 \right) = 0.83 \) D.6.2.5
- Concrete cracking
  - \( \Psi_{c,N} = 1.0 \) for cracked concrete D.6.2.6
- Concrete splitting
  - \( \Psi_{cp,N} = 1.0 \) for cast-in anchor D.6.2.7
- Concrete breakout resistance
  - \( N_{cb} = A_{nc0} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_{cb} = 151.2 \) [kN] D.6.2.1 (D-5)

Seismic design strength reduction
- \( x = 1.0 \) not applicable = 151.2 [kN] D.4.3.5
- ratio = 0.69 > \( N_u \) OK

Stud Pullout Resistance

Single bolt pullout resistance
- \( N_{pf} = 8 A_{pb} f_{plc} R_{zc} = 91.9 \) [kN] D.6.3.4 (D-16)
- \( N_{cf} = \Psi_{cp} N_{pf} = 91.9 \) [kN] D.6.3.1 (D-15)
- Seismic design strength reduction
  - \( x = 1.0 \) not applicable = 91.9 [kN] D.4.3.5
  - ratio = 0.30 > \( T_1 \) OK
- \( \Psi_{cp} = 1 \) for cracked conc D.6.3.6
- \( R_{zc} = 1.00 \) pullout strength is always Condition B D.5.4(c)
Side Blowout Resistance

Failure Along Pedestal Width Edge

Tensile load carried by anchors close to edge which may cause side-face blowout

\[ N_{bw} = n_1 T_1 \]
\[ c = \min (c_1, c_3) \]

- along pedestal width edge
- \( c = 152 \) [mm]

Check if side blowout applicable

- \( h_w = 229 \) [mm]
- side blowout is NOT applicable

Check if edge anchors work as a group or work individually

- \( s_{22} = 0 \) [mm] \( s = s_2 = 0 \) [mm]
- side blowout is NOT applicable

Single anchor SB resistance

\[ N_{sbr,w} = 13.3 \sqrt{A_{avg}} \phi_s \sqrt{f_{c,t}} R_{1,c} \]
\[ c = \min (c_1, c_3) \]

Multiple anchors SB resistance

- work as a group - not applicable
- work individually - not applicable

Seismic design strength reduction

\[ \text{ratio} = 0.00 \]

Group side blowout resistance

\[ N_{sbg,w} = \frac{N_{sbr,w}}{n_1} \]

Govern Tensile Resistance

\[ N_t = \min (n_t N_{sbr}, N_{nr}, n_t N_{cpr}, N_{sbg,w}) = 151.2 \] [kN]

Stud Shear Resistance

\[ V_{sr} = n_s A_{se} f_u R_{v,s} \]
\[ = 332.5 \] [kN]

Reduction due to built-up grout pads

\[ \text{ratio} = 0.27 \]

Conc. Shear Breakout Resistance

Only Case 2 needs to be considered when anchors are rigidly connected to the attachment

This applies to welded stud case so only Mode 2 is considered for shear checking

**Mode 2** Failure cone at back anchors

- Bolt edge distance
- \( c_{a1} = c_1 + s_1 = 685 \) [mm]
- Limiting \( c_{a1} \) when anchors are influenced by 3 or more edges
- \( = \text{No} \)
- Bolt edge distance - adjusted
- \( c_{a1} = c_{a1} \) needs NOT to be adjusted
- \( = 685 \) [mm]
### Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

**Dongxiao Wu P. Eng.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_2 )</td>
<td>152</td>
<td>[mm]</td>
<td>A23.3-04 (R2010)</td>
</tr>
<tr>
<td>( 1.5c_{a1} )</td>
<td>1028</td>
<td>[mm]</td>
<td>D.7.2.1</td>
</tr>
<tr>
<td>( A_{vc} )</td>
<td>7.8E+05</td>
<td>[mm²]</td>
<td>D.7.2.1</td>
</tr>
<tr>
<td>( A_{vc0} )</td>
<td>2.1E+06</td>
<td>[mm²]</td>
<td>D.7.2.1 (D-24)</td>
</tr>
<tr>
<td>( l_o )</td>
<td>178</td>
<td>[mm]</td>
<td>D.3</td>
</tr>
<tr>
<td>( V_{cr} )</td>
<td>352.2</td>
<td>[kN]</td>
<td>D.7.2.3 (D-26)</td>
</tr>
<tr>
<td>Eccentricity effects</td>
<td>( \Psi_{ec,v} = 1.0 )</td>
<td></td>
<td>D.7.2.5</td>
</tr>
<tr>
<td>Edge effects</td>
<td>( \Psi_{ed,v} = \min \left( \left( 0.7+0.3c_2/1.5c_{a1} \right), 1.0 \right) )</td>
<td>0.74</td>
<td>D.7.2.6</td>
</tr>
<tr>
<td>Concrete cracking</td>
<td>( \Psi_{c,v} = \max \left( \sqrt{1.5c_{a1} / h_a} , 1.0 \right) )</td>
<td>1.20</td>
<td>D.7.2.7</td>
</tr>
<tr>
<td>Member thickness</td>
<td>( \Psi_{h,v} = \max \left( \sqrt{1.5c_{a1} / h_a} , 1.0 \right) )</td>
<td>1.50</td>
<td>D.7.2.8</td>
</tr>
<tr>
<td>Conc shear breakout resistance</td>
<td>( V_{cbgr} = 174.8 )</td>
<td>[kN]</td>
<td>D.7.2.1 (D-23)</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>( x = 1.0 )</td>
<td></td>
<td>D.4.3.5</td>
</tr>
<tr>
<td>ratio</td>
<td>0.51</td>
<td></td>
<td>OK</td>
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</table>

**Conc. Pryout Shear Resistance**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_{cp} )</td>
<td>2.0</td>
<td></td>
<td>D.7.3</td>
</tr>
<tr>
<td>Factored shear pryout resistance</td>
<td>( V_{cpgr} = k_{cp} N_{cbgr} )</td>
<td>302.4</td>
<td>D.7.3 (D-32)</td>
</tr>
<tr>
<td>( R_{v,c} )</td>
<td>1.00</td>
<td></td>
<td>D.5.4(c)</td>
</tr>
<tr>
<td>Seismic design strength reduction</td>
<td>( x = 1.0 )</td>
<td></td>
<td>D.4.3.5</td>
</tr>
<tr>
<td>ratio</td>
<td>0.29</td>
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<td>OK</td>
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**Stud on Conc Bearing**

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<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( B_r )</td>
<td>334.4</td>
<td>[kN]</td>
<td>CSA S16-09</td>
</tr>
<tr>
<td>ratio</td>
<td>0.27</td>
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<td>OK</td>
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**Govern Shear Resistance**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_r )</td>
<td>174.8</td>
<td>[kN]</td>
<td>A23.3-04 (R2010)</td>
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</tbody>
</table>

**Tension Shear Interaction**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Check if ( N_s &gt; 0.2 N_t ) and ( V_s &gt; 0.2 V_r )</td>
<td>Yes</td>
<td></td>
<td>D.8.2 &amp; D.8.3</td>
</tr>
<tr>
<td>( N_s/N_t + V_s/V_r )</td>
<td>1.20</td>
<td></td>
<td>D.8.4 (D-35)</td>
</tr>
<tr>
<td>ratio</td>
<td>1.00</td>
<td></td>
<td>NG</td>
</tr>
</tbody>
</table>

**Ductility Tension**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( N_{sr} )</td>
<td>118.2</td>
<td>[kN]</td>
<td></td>
</tr>
</tbody>
</table>

**Ductility Shear**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_{sr} )</td>
<td>332.5</td>
<td>[kN]</td>
<td></td>
</tr>
</tbody>
</table>

---

**Page 126 of 155**
Example 41: Shear Lug Design ACI 349-06 Code

SHEAR LUG / SHEAR KEY DESIGN

Shear Lug / Shear Key design based on

ACI 349-06 Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary
ACI 349-06

AISC Design Guide 1: Base Plate and Anchor Rod Design - 2nd Edition
AISC Design Guide 1

AISC 360-05 Specification for Structural Steel Buildings
AISC 360-05

INPUT DATA

Code Abbreviation
ACI 349-06
AISC Design Guide 1
AISC 360-05

Factored shear along strong axis \( V_{ux} = 75.0 \) [kips]
Factored shear along weak axis \( V_{uy} = 50.0 \) [kips] applicable for W Shape only

Pedestal width \( b_c = 26.0 \) [in]
Pedestal depth \( d_c = 26.0 \) [in]
Pedestal height \( h_a = 30.0 \) [in]
Grout thickness \( g = 2.0 \) [in]
Shear key type = W_Shape
Shear key width Shape \( w = 8.07 \) [in] Applicable
Shear key width used for design \( w = 8.07 \) [in]
Shear key embed depth \( d = 8.0 \) [in]

Concrete strength \( f'_c = 4.5 \) [ksi] \( 4 = 31.0 \) [MPa] A36 A992
Shear key steel strength \( F_y = 50 \) [ksi] 36 50 = 344.8 [MPa]
\( F_u = 65 \) [ksi] 58 65 = 448.2 [MPa]
Weld electrode = E70XX AISC 360-05
Electrode ultimate tensile \( F_{EXX} = 70 \) [ksi] 70 = 482.7 [MPa]
Fillet weld leg size \( A_m = 5 \) [1/16 in] 5/16 = 7.9 [mm] Table J2.4

![Shear Lug Diagram](image)
## Conclusion

<table>
<thead>
<tr>
<th>Section</th>
<th>Ratio</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Bearing</td>
<td>0.94</td>
<td>ACI 349-06</td>
</tr>
<tr>
<td>Shear Toward Free Edge</td>
<td>0.81</td>
<td>D.4.6.2</td>
</tr>
<tr>
<td>Shear Key Section Flexure &amp; Shear Check</td>
<td>0.94</td>
<td>D.11.2</td>
</tr>
<tr>
<td>Shear Key To Base Plate Fillet Weld</td>
<td>0.69</td>
<td></td>
</tr>
</tbody>
</table>

### Calculation

#### Concrete Bearing

<table>
<thead>
<tr>
<th>Formula</th>
<th>Value</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A_b = w \cdot d_u = w \cdot (d-g) )</td>
<td>48.42</td>
<td></td>
</tr>
<tr>
<td>( V_b = 1.3 \cdot f'_c \cdot A_b )</td>
<td>184.1</td>
<td>D.4.6.2</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.41</td>
<td>&gt; ( V_{ux} )</td>
</tr>
<tr>
<td>( \phi = 0.65 ) for anchor controlled by concrete bearing</td>
<td></td>
<td>D.4.4 (d)</td>
</tr>
</tbody>
</table>

#### Shear Toward Free Edge

<table>
<thead>
<tr>
<th>Formula</th>
<th>Value</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( e = 0.5 \cdot (d_c - t) )</td>
<td>12.38</td>
<td></td>
</tr>
<tr>
<td>( e = \min(e, h_u) )</td>
<td>12.38</td>
<td></td>
</tr>
<tr>
<td>( A_{eff} = \left[ e + (d-g) \right] \cdot b_c - w \cdot (d-g) )</td>
<td>429.3</td>
<td></td>
</tr>
<tr>
<td>( \phi V_n = 4 \cdot \sqrt{f'<em>c} \cdot A</em>{eff} )</td>
<td>92.2</td>
<td>D.11.2</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.81</td>
<td>&gt; ( V_u )</td>
</tr>
<tr>
<td>( \phi = 0.80 )</td>
<td></td>
<td>D.4.4 (f)</td>
</tr>
</tbody>
</table>

#### Shear Key Section Flexure & Shear Check

**Shear Key Plate Sect**

- This case does not apply
- \( M_{ux} = V_{ux} \times \left[ 0.5 \cdot (d-g) + g \right] \) = 375.0 [kip-in]
- \( Z = \frac{w \times t^2}{4} \) = 3.15 [in³]
- Flexure: \( \phi M_n = 0.9 \times Z \times F_y \) = 141.9 [kip-in]
- Shear: \( \phi V_n = 0.9 \times A_w \times 0.6 F_y \) = 272.4 [kips]
- Ratio: \( \phi \) = 0.00, > \( V_{ux} \) OK

**Shear Key Pipe Sect**

- This case does not apply
- \( M_{ux} = V_{ux} \times \left[ 0.5 \cdot (d-g) + g \right] \) = 375.0 [kip-in]
- \( Z = \frac{w \times t^2}{4} \) = 0.00 [in³]
- Flexure: \( \phi M_n = 0.9 \times Z \times F_y \) = 0.0 [kip-in]
- Shear: \( A_w = 0.000 \) [in³]
- \( \phi V_n = 0.9 \times A_w \times 0.6 F_y \) = 0.0 [kips]
- Ratio: \( \phi \) = 0.00, < \( V_{ux} \) OK
Shear Key HSS Sect

This case does not apply

\[ M_{ux} = V_{ux} \times \left[ 0.5(x-y) + g \right] \]
\[ Z = \]
\[ \phi M_{ux} = 0.9 \times Z \times F_y \]
\[ \text{ratio} = 0.00 \leq 0.00 \]

Shear

\[ A_w = \]
\[ \phi V_n = 0.9 \times A_w \times 0.6 F_y \]
\[ \text{ratio} = 0.00 \leq 0.00 \]

Shear Key W Sect

This case applies

Flexure strong axis

\[ M_{ux} = V_{ux} \times \left[ 0.5(x-y) + g \right] \]
\[ Z_x = \]
\[ \phi M_{ux} = 0.9 \times Z_x \times F_y \]
\[ \text{ratio} = 0.21 > 0.00 \]

\[ M_{uy} = V_{uy} \times \left[ 0.5(x-y) + g \right] \]
\[ Z_y = \]
\[ \phi M_{uy} = 0.9 \times Z_y \times F_y \]
\[ \text{ratio} = 0.30 > 0.00 \]

Shear

\[ A_w = tw \times d \]
\[ \phi V_n = 0.9 \times A_w \times 0.6 F_y \]
\[ \text{ratio} = 0.94 > 0.00 \]

[Shear Key To Base Plate Fillet Weld]

Resultant angle

\[ \theta = 90 \, \text{[deg]} \]

Nominal fillet weld strength

\[ F_w = 0.6 F_{EHH} (1.0 + 0.5 \sin^{1.5} \theta) \]
\[ \phi = 0.75 \]

Weld metal shear strength

\[ \phi r_{n1} = \phi (0.707 \times A_{nm}) \times F_w \]
\[ \phi r_{n2} = \min \left[ 1.0(0.6F_y t), 0.75(0.6F_u t) \right] \]

For PLATE shear key only

\[ \phi r_n = \min (\phi r_{n1}, \phi r_{n2}) \]

Nominal base metal thickness

\[ t = 0.000 \, \text{[in]} \]

Base metal shear strength

\[ \phi r_{n2} = \min \left[ 1.0(0.6F_y t), 0.75(0.6F_u t) \right] \]

Shear strength used for design

\[ \phi r_n = \min (\phi r_{n1}, \phi r_{n2}) \]

\[ \phi r_n = 10.44 \, \text{[kips/in]} \]
Factored moment to base plate

\[ M_{ux} = V_{ux} \times [0.5x(d-g) + g] = 375.0 \text{ [kip-in]} \]
\[ M_{uy} = V_{uy} \times [0.5x(d-g) + g] = 250.0 \text{ [kip-in]} \]

**Shear Key Plate**

This case does not apply

\[ s = t + (1/3)A_x \times 2 = 1.458 \text{ [in]} \]
\[ f_r = V_{ux} / (w \times 2) = 0.00 \text{ [kips/in]} \]
\[ f_t = \sqrt{f_r^2 + f_v^2} = 0.00 \text{ [kips/in]} \]
\[ \text{ratio} = 0.00 < \phi \, f_n \quad \text{OK} \]

**Shear Key Pipe Sect**

This case does not apply

\[ D = = 8.07 \text{ [in]} \]
\[ f_r = V_{ux} / (\pi D^2 / 4) = 0.00 \text{ [kips/in]} \]
\[ f_t = V_{ux} / (\pi D \times 1) = 0.00 \text{ [kips/in]} \]
\[ f_t = \sqrt{f_r^2 + f_v^2} = 0.00 \text{ [kips/in]} \]
\[ \text{ratio} = 0.00 < \phi \, f_n \quad \text{OK} \]

**Shear Key HSS Sect**

This case does not apply

\[ b = 8.07 \text{ [in]} \]
\[ d = 0.00 \text{ [in]} \]
\[ f_r = \frac{M_{ux}}{(bd + d^2/3)} = 0.00 \text{ [kips/in]} \]
\[ f_t = \frac{V_{ux}}{(2xd)} = 0.00 \text{ [kips/in]} \]
\[ f_t = \sqrt{f_r^2 + f_v^2} = 0.00 \text{ [kips/in]} \]
\[ \text{ratio} = 0.00 < \phi \, f_n \quad \text{OK} \]

**Shear Key W Sect**

This case applies

\[ b = 8.07 \text{ [in]} \]
\[ d = 8.25 \text{ [in]} \]

**Strong Axis**

\[ f_r = M_{ux} / (bxd) = 5.63 \text{ [kips/in]} \]
\[ f_t = V_{ux} / (2xd) = 4.55 \text{ [kips/in]} \]
\[ f_t = \sqrt{f_r^2 + f_v^2} = 7.24 \text{ [kips/in]} \]
\[ \text{ratio} = 0.69 < \phi \, f_n \quad \text{OK} \]

**Weak Axis**

\[ f_r = M_{uy} / [(1xb^2/6) \times 4] = 5.76 \text{ [kips/in]} \]
\[ f_t = V_{uy} / (4xb) = 1.55 \text{ [kips/in]} \]
\[ f_t = \sqrt{f_r^2 + f_v^2} = 5.96 \text{ [kips/in]} \]
\[ \text{ratio} = 0.57 < \phi \, f_n \quad \text{OK} \]
Example 42: Shear Lug Design ACI 349M-06 Code

### SHEAR LUG / SHEAR KEY DESIGN
Shear Lug / Shear Key design based on ACI 349M-06 Metric Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary

<table>
<thead>
<tr>
<th>Code Abbreviation</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 349M-06</td>
<td>ACI 349M-06 Metric Code Requirements for Nuclear Safety-Related Concrete Structures &amp; Commentary</td>
</tr>
<tr>
<td>CSA S16-09 Design of Steel Structures</td>
<td>CSA S16-09</td>
</tr>
</tbody>
</table>

### INPUT DATA

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored shear along strong axis</td>
<td>$V_{ux} = 333.6$ [kN]</td>
</tr>
<tr>
<td>Factored shear along weak axis</td>
<td>$V_{uy} = 222.4$ [kN] applicable for W Shape only</td>
</tr>
<tr>
<td>Pedestal width</td>
<td>$b_c = 660$ [mm]</td>
</tr>
<tr>
<td>Pedestal depth</td>
<td>$d_c = 660$ [mm]</td>
</tr>
<tr>
<td>Pedestal height</td>
<td>$h_a = 762$ [mm]</td>
</tr>
<tr>
<td>Grout thickness</td>
<td>$g = 51$ [mm]</td>
</tr>
<tr>
<td>Shear key type</td>
<td>W_Shape</td>
</tr>
<tr>
<td>Shear key width Shape</td>
<td>$w = 205$ [mm] Applicable</td>
</tr>
<tr>
<td>Shear key width used for design</td>
<td>$w = 205$ [mm]</td>
</tr>
<tr>
<td>Shear key embed depth</td>
<td>$d = 203$ [mm]</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>$f'_c = 31$ [MPa] 30 = 4.5 [ksi]</td>
</tr>
<tr>
<td>Shear key steel strength</td>
<td>$F_y = 345$ [MPa] 300 = 50.0 [ksi]</td>
</tr>
<tr>
<td>Shear key steel strength</td>
<td>$F_u = 448$ [MPa] 450 = 65.0 [ksi]</td>
</tr>
<tr>
<td>Weld electrode</td>
<td>E49XX</td>
</tr>
<tr>
<td>Fillet weld leg size</td>
<td>$D = 8$ [mm]</td>
</tr>
</tbody>
</table>

**Shear Key Design Equation**

$$e = 0.5(d_c - t)$$

**Concrete Moment Capacity**

$$M = V_d(h_a - pedestal height)$$

**Concrete Shear Resistance**

$$V_d = rac{f'_c}{f_{shear}} (V_{ux} + V_{uy} - V_d)$$

**Shear Key Embedment Depth**

$$d = 203$$ [mm]
CONCLUSION

OVERALL ratio = 0.94 OK

Concrete Bearing ratio = 0.41 OK D.4.6.2
Shear Toward Free Edge ratio = 0.81 OK D.11.2

Shear Key Section Flexure & Shear Check ratio = 0.94 OK
Shear Key To Base Plate Fillet Weld ratio = 0.79 OK

CALCULATION

Concrete Bearing

\[ A_b = w \Delta_d = w (d-g) = 31242 \text{ [mm}^2] \]
\[ V_b = 1.3 \phi_f' \frac{f'c}{A_b} = 818.4 \text{ [kN]} \text{ D.4.6.2} \]
\[ \text{ratio} = 0.41 > V_{ux} \text{ OK} \]
\[ \phi = 0.65 \text{ for anchor controlled by concrete bearing D.4.4 (d)} \]

Shear Toward Free Edge

\[ e = 0.5x(d_e - t) = 314 \text{ [mm]} \]
\[ e = \min(e, h_u) = 314 \text{ [mm]} \]
\[ A_{eff} = [e + (d-g)] x b_e - wx(d-g) = 2.8E+05 \text{ [mm}^2] \]
\[ \phi[V_n = 4\phi\sqrt{\frac{f'c}{A}} A_{eff} = 409.6 \text{ [kN]} \text{ D.11.2} \]
\[ \text{ratio} = 0.81 > V_{u} \text{ OK} \]
\[ \phi = 0.80 \text{ D.4.4 (f)} \]

Shear Key Section Flexure & Shear Check

Shear Key Plate Sect This case does not apply
\[ M_{ux} = V_{ux} x [0.5x(d-g) + g] = 42.4 \text{ [kNm]} \]
\[ Z = w x t^2 / 4 = 52.5 \text{ [x10^3mm}^3] \]
\[ \phi M_n = 0.9 x Z x F_y = 16.3 \text{ [kNm]} \text{ OK} \]
\[ \text{ratio} = 0.00 < M_{ux} \text{ OK} \]
\[ \phi V_n = 0.9 x A_w x 0.6F_y = 1222.1 \text{ [kN]} \text{ OK} \]
\[ \text{ratio} = 0.00 > V_{ux} \text{ OK} \]

Shear Key Pipe Sect This case does not apply
\[ M_{ux} = V_{ux} x [0.5x(d-g) + g] = 42.4 \text{ [kNm]} \]
\[ Z = 0.0 \text{ [x10^3mm}^3] \]
\[ \phi M_n = 0.9 x Z x F_y = 0.0 \text{ [kNm]} \text{ OK} \]
\[ \text{ratio} = 0.00 < M_{ux} \text{ OK} \]
\[ \phi V_n = 0.9 x A_w x 0.6F_y = 0.0 \text{ [kN]} \text{ OK} \]
\[ \text{ratio} = 0.00 < V_{ux} \text{ OK} \]
### Shear Key HSS Sect

This case does not apply

<table>
<thead>
<tr>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{ux}$</td>
<td>42.4 [kNm]</td>
</tr>
<tr>
<td>$Z$</td>
<td>0.0 [$x10^3$mm$^3$]</td>
</tr>
</tbody>
</table>

#### Flexure

- $\phi M_{ux} = 0.9 \times Z \times F_y$ = 0.0 [kNm]
- ratio = 0.00
- $\phi V_n = 0.9 \times A_w \times 0.6F_y$ = 0.0 [kN]
- ratio = 0.00

OK

### Shear Key W Sect

This case applies

Flexure strong axis

- $M_{ux} = V_{ux} \times [0.5x(d-g) + g]$ = 42.4 [kNm]
- $Z_x = 653$ [$x10^3$mm$^3$]
- $\phi M_{nx} = 0.9 \times Z_x \times F_y$ = 202.8 [kNm]
- ratio = 0.21

OK

Flexure weak axis

- $M_{uy} = V_{uy} \times [0.5x(d-g) + g]$ = 28.2 [kNm]
- $Z_y = 303$ [$x10^3$mm$^3$]
- $\phi M_{ny} = 0.9 \times Z_y \times F_y$ = 94.1 [kNm]
- ratio = 0.30

OK

### Shear Key To Base Plate Fillet Weld

<table>
<thead>
<tr>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$bf = 205.0$ [mm]</td>
<td>$d = 210.0$ [mm]</td>
</tr>
<tr>
<td>$tw = 9.1$ [mm]</td>
<td>$tf = 14.2$ [mm]</td>
</tr>
</tbody>
</table>

Shear strong axis

- $A_w = t_w \times d = 1911$ [mm$^2$]
- $\phi V_{nx} = 0.9 \times A_w \times 0.6F_y$ = 356.0 [kN]
- ratio = 0.94

OK

Shear weak axis

- $A_w = 2 \times tf \times bf = 5822$ [mm$^2$]
- $\phi V_{ny} = 0.9 \times A_w \times 0.6F_y$ = 1084.6 [kN]
- ratio = 0.21

OK

### Base metal resistance

<table>
<thead>
<tr>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_m = D \times 1mm$</td>
<td>8.00 [mm$^2$]</td>
</tr>
<tr>
<td>$v_m = 0.67 \phi_w A_m F_u$</td>
<td>1.61 [kN/mm] 13.13.2.2</td>
</tr>
<tr>
<td>$\phi_w = 0.67$</td>
<td></td>
</tr>
</tbody>
</table>

### Weld metal resistance

<table>
<thead>
<tr>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_w = 0.707 \times D \times 1mm$</td>
<td>5.66 [mm$^2$]</td>
</tr>
</tbody>
</table>

### Fillet weld resistance - shear

<table>
<thead>
<tr>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta = 90$</td>
<td></td>
</tr>
<tr>
<td>$v_w = 0.67 \phi_w A_w X_u (1 + 0.5 \sin \theta)^{1.5}$</td>
<td>1.87 [kN/mm] 13.13.2.2</td>
</tr>
<tr>
<td>$v_r = \min(v_m, v_w)$</td>
<td>1.61 [kN/mm]</td>
</tr>
</tbody>
</table>
### Code Reference

Factored moment to base plate

\[ M_{ux} = V_{ux} \times [0.5x(d-g) + g] \]

\[ M_{uy} = V_{uy} \times [0.5x(d-g) + g] \]

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>4 of 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Key Plate</td>
<td>This case does not apply</td>
</tr>
<tr>
<td>( s = t + (1/3)D \times 2 )</td>
<td>37.3 [mm]</td>
</tr>
<tr>
<td>( f_t = M_{ux} / (s \times w) )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>( f_v = V_{ux} / (w \times 2) )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>( f_r = \sqrt{f_t^2 + f_v^2} )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>ratio</td>
<td>0.00</td>
</tr>
<tr>
<td>( \phi )</td>
<td>( r_n )</td>
</tr>
<tr>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

Shear Key Pipe Sect

This case does not apply

Weld ring diameter

\[ D = 205.0 \text{ [mm]} \]

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>4 of 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Key HSS Sect</td>
<td>This case does not apply</td>
</tr>
<tr>
<td>Weld box width/depth</td>
<td></td>
</tr>
<tr>
<td>( b = 205.0 \text{ [in]} )</td>
<td>( d = 205.0 \text{ [mm]} )</td>
</tr>
<tr>
<td>( f_t = M_{ux} / (bd + d^2/3) )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>( f_v = V_{ux} / (2xd) )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>( f_r = \sqrt{f_t^2 + f_v^2} )</td>
<td>0.00 [kN/mm]</td>
</tr>
<tr>
<td>ratio</td>
<td>0.00</td>
</tr>
<tr>
<td>( \phi )</td>
<td>( r_n )</td>
</tr>
<tr>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

Shear Key W Sect

This case applies

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>4 of 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strong Axis</td>
<td></td>
</tr>
<tr>
<td>( b = 205.0 \text{ [in]} )</td>
<td>( d = 210.0 \text{ [mm]} )</td>
</tr>
<tr>
<td>( f_t = M_{ux} / (bxd) )</td>
<td>0.98 [kN/mm]</td>
</tr>
<tr>
<td>( f_v = V_{ux} / (2xd) )</td>
<td>0.79 [kN/mm]</td>
</tr>
<tr>
<td>( f_r = \sqrt{f_t^2 + f_v^2} )</td>
<td>1.26 [kN/mm]</td>
</tr>
<tr>
<td>ratio</td>
<td>0.79</td>
</tr>
<tr>
<td>( \phi )</td>
<td>( f_n )</td>
</tr>
<tr>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

Weak Axis

\[ f_t = M_{uy} / [(1xb^2/6) \times 4] \]

\[ f_v = V_{uy} / (4xb) \]

\[ f_r = \sqrt{f_t^2 + f_v^2} \]

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>4 of 4</th>
</tr>
</thead>
</table>

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Example 51: Base Plate (LRFD) & Anchor Bolt (ACI 318-08) Design With Anchor Reinforcement

**BASE PLATE & ANCHOR BOLT DESIGN - MOMENT CONNECTION**

**Base Plate Data**

- Column section type: W_Shape
- Column size: W14X53
- Depth: \( d = 13.900 \) [in]
- Flange thickness: \( t_f = 0.660 \) [in]
- Flange width: \( b_f = 8.060 \) [in]
- Web thickness: \( t_w = 0.370 \) [in]
- Base plate anchor bolt pattern: 4 or 6-Bolt MC WF
- Base plate anchor bolt location: Bolt Outside Flange Only

**Base Plate Width & Depth**

- Base plate width: \( B = 22.0 \) [in] (suggest)
- Base plate depth: \( N = 22.0 \) [in] (suggest)
- Base plate thickness: \( t_p = 2.00 \) [in]
- Anchor bolt spacing \( s_2 = C = 18.0 \) [in]
- Anchor bolt spacing \( s_1 = D = 18.0 \) [in]

**Base Plate Geometric**

- Bolt to column center dist.: \( f = 9.0 \) [in]
- Bolt to column web center dist.: \( f_1 = 9.0 \) [in]
- Suggested plate thickness for rigidity: \( t_p = \text{max of } m/4 \) and \( n/4 \)

**Factored Column Load**

<table>
<thead>
<tr>
<th>LCB</th>
<th>Cases</th>
<th>( P_u ) [kips]</th>
<th>( V_u ) [kips]</th>
<th>( M_u ) [kip-ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCB1</td>
<td>Axial Comp.</td>
<td>100.0</td>
<td>15.0</td>
<td>0.0</td>
</tr>
<tr>
<td>LCB2</td>
<td>Axial Comp. + M</td>
<td>0.0</td>
<td>20.0</td>
<td>30.0</td>
</tr>
<tr>
<td>LCB3</td>
<td>Axial Comp. + M</td>
<td>15.0</td>
<td>20.0</td>
<td>30.0</td>
</tr>
<tr>
<td>LCB4</td>
<td>Axial Tensile</td>
<td>10.0</td>
<td>35.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
### Anchor Bolt Data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of bolt line for resisting moment</td>
<td>3 Bolt Line</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No of bolt along outermost bolt line</td>
<td>3</td>
<td></td>
<td>min required</td>
</tr>
<tr>
<td>Outermost bolt line spacing ( s_1 )</td>
<td>18.0</td>
<td>[in]</td>
<td>OK</td>
</tr>
<tr>
<td>Outermost bolt line spacing ( s_2 )</td>
<td>18.0</td>
<td>[in]</td>
<td>OK</td>
</tr>
<tr>
<td>Internal bolt line spacing ( s_{b1} )</td>
<td>9.0</td>
<td>[in]</td>
<td>OK</td>
</tr>
<tr>
<td>Internal bolt line spacing ( s_{b2} )</td>
<td>0.0</td>
<td>[in]</td>
<td>OK</td>
</tr>
<tr>
<td>Anchor bolt material</td>
<td>F1554 Grade 55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anchor tensile strength ( f_{ut} )</td>
<td>75.0</td>
<td>[ksi]</td>
<td>517 [MPa]</td>
</tr>
<tr>
<td>Anchor bolt diameter ( d_a )</td>
<td>0.875</td>
<td>[in]</td>
<td>max 1.5 in</td>
</tr>
<tr>
<td>Bolt sleeve diameter ( d_s )</td>
<td>2.0</td>
<td>[in]</td>
<td></td>
</tr>
<tr>
<td>Bolt sleeve height ( h_s )</td>
<td>7.0</td>
<td>[in]</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt embedment depth ( h_{ef} )</td>
<td>20.0</td>
<td>[in]</td>
<td>10.5</td>
</tr>
<tr>
<td>Pedestal height ( h_p )</td>
<td>23.0</td>
<td>[in]</td>
<td>23.0</td>
</tr>
<tr>
<td>Pedestal width ( b_c )</td>
<td>124.0</td>
<td>[in]</td>
<td></td>
</tr>
<tr>
<td>Pedestal depth ( d_c )</td>
<td>124.0</td>
<td>[in]</td>
<td></td>
</tr>
</tbody>
</table>

**Code Reference**

- Anchor is ductile steel element D.1
- Anchor bolt diameter Requirement: \( d_a \) = 0.875 in max 1.5 in
- Bolt sleeve diameter Requirement: \( d_s \) = 2.0 in
- Anchor bolt embedment depth Requirement: \( h_{ef} \) = 20.0 in
- Pedestal height Requirement: \( h_p \) = 23.0 in
- Pedestal width Requirement: \( b_c \) = 124.0 in
- Pedestal depth Requirement: \( d_c \) = 124.0 in
Bolt edge distance \( c_1 \)  
\[ c_1 = 6.0 \text{ [in]} \] 
OK

Bolt edge distance \( c_2 \)  
\[ c_2 = 6.0 \text{ [in]} \] 
OK

Bolt edge distance \( c_3 \)  
\[ c_3 = 100.0 \text{ [in]} \] 
OK

Bolt edge distance \( c_4 \)  
\[ c_4 = 100.0 \text{ [in]} \] 
OK

To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within \( 0.5h_{hf} \) from the outmost anchor's centerline. In this design \( 0.5h_{hf} \) value is limited to 8 in.

\[ 0.5h_{hf} = 8.0 \text{ [in]} \]

No of ver. rebar that are effective for resisting anchor tension

\[ n_v = 6 \]

Ver. bar size No.  
8 \( \downarrow \) 1.000 [in] dia single bar area \( A_v = 0.79 \text{ [in}^2\text{]} \]

To be considered effective for resisting anchor shear, hor. reinft shall be located within \( \min(0.5c_1, 0.3c_2) \) from the outmost anchor's centerline

\[ \min(0.5c_1, 0.3c_2) = 1.8 \text{ [in]} \]

No of tie leg that are effective to resist anchor shear

\[ n_{leg} = 2 \]

No of tie layer that are effective to resist anchor shear

\[ n_{lay} = 2 \]

Hor. tie bar size No.  
4 \( \downarrow \) 0.500 [in] dia single bar area \( A_t = 0.20 \text{ [in}^2\text{]} \]

For anchor reinft shear breakout strength calc  
100% hor. tie bars develop full yield strength

ACI 318-08
Concrete strength $f'_{c} = 4.5$ [ksi] 4
Rebar yield strength $f_{y} = 60.0$ [ksi] 60
Base plate yield strength $F_{y} = 36.0$ [ksi] 36

Total no of anchor bolt $n = 8$
No of anchor bolt carrying shear $n_{s} = 8$
For side-face blowout check use No of bolt along width edge $n_{bw} = 3$
No of bolt along depth edge $n_{bd} = 3$
Anchor head type = ?
Anchor effective cross sect area $A_{se} = 0.462$ [in$^2$]
Bearing area of one head $A_{avg} = 1.188$ [in$^2$]

CONCLUSION

<table>
<thead>
<tr>
<th>OVERALL</th>
<th>ratio = 0.97</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>BASE PLATE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Plate Size and Anchor Bolt Tensile</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Base Plate Thickness</td>
<td>ratio = 0.52</td>
<td>OK</td>
</tr>
<tr>
<td>ANCHOR BOLT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LCB1 Axial Compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abhor Rod Embedment, Spacing and Edge Distance</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Min Rquired Anchor Reinft. Development Length</td>
<td>ratio = 0.97</td>
<td>OK</td>
</tr>
<tr>
<td>Overall Ratio</td>
<td>ratio = 0.42</td>
<td>OK</td>
</tr>
<tr>
<td>LCB2 Axial Compression + Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abhor Rod Embedment, Spacing and Edge Distance</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Min Rquired Anchor Reinft. Development Length</td>
<td>ratio = 0.97</td>
<td>OK</td>
</tr>
<tr>
<td>Overall Ratio</td>
<td>ratio = 0.83</td>
<td>OK</td>
</tr>
<tr>
<td>LCB3 Axial Compression + Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abhor Rod Embedment, Spacing and Edge Distance</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Min Rquired Anchor Reinft. Development Length</td>
<td>ratio = 0.97</td>
<td>OK</td>
</tr>
<tr>
<td>Overall Ratio</td>
<td>ratio = 0.72</td>
<td>OK</td>
</tr>
<tr>
<td>LCB4 Axial Tensile</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abhor Rod Embedment, Spacing and Edge Distance</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>Min Rquired Anchor Reinft. Development Length</td>
<td>ratio = 0.97</td>
<td>OK</td>
</tr>
<tr>
<td>Overall Ratio</td>
<td>ratio = 0.97</td>
<td>OK</td>
</tr>
</tbody>
</table>
BASE PLATE DESIGN
Base plate design based on
AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition
ACI 318-08 Building Code Requirements for Structural Concrete and Commentary

DESIGN DATA
Column section type         W_Shape
Column size                W14X53
Depth                      d = 13.900 [in]  Flange thickness tf = 0.660 [in]
Flange width               b_f = 8.060 [in]  Web thickness t_w = 0.370 [in]
Base plate anchor bolt pattern 4 or 6-Bolt MC WF  base plate is moment connection

![Base plate design diagram]

2-BOLT PIN        4-BOLT PIN        4 or 6-Bolt MC WF        4 or 6-Bolt MC HS

Base plate width   B = 22.0 [in]  15.0
Base plate depth   N = 22.0 [in]  21.0
Base plate thickness  t_p = 2.000 [in]  1.8
Anchor bolt spacing  C = 18.0 [in]  11.0
Anchor bolt spacing  D = 18.0 [in]  17.0
Anchor bolt diameter  d = 0.875 [in]  max 1.5 in

![Base plate geometric and subject to tensile load diagrams]

Bolt to column center dist.   f = 9.0 [in]  9 in
Bolt to column web center dist.  f_1 = 9.0 [in]  9 in
Pedestal width   b_c = 124.0 [in]  >= 28.5 in
Pedestal depth    d_c = 124.0 [in]  >= 28.5 in
Factored column load

<table>
<thead>
<tr>
<th>LCB</th>
<th>Cases</th>
<th>( P_c ) [kips]</th>
<th>( M_u ) [kip-ft]</th>
<th>( t_p ) (in)</th>
<th>Base Plate Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCB1</td>
<td>Axial Compressive</td>
<td>100.0</td>
<td>0.0</td>
<td>0.88</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB2</td>
<td>Compression + M</td>
<td>0.0</td>
<td>30.0</td>
<td>0.89</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB3</td>
<td>Compression + M</td>
<td>15.0</td>
<td>30.0</td>
<td>1.04</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB4</td>
<td>Axial Tensile</td>
<td>10.0</td>
<td>0.0</td>
<td>0.28</td>
<td>Anchor Bolt Tensile OK</td>
</tr>
</tbody>
</table>

Min required plate thickness \( t_p \) = 1.04 in

Suggested plate thickness for rigidity: \( t_p = \text{max. of } m/4 \text{ and } n/4 \) = No

For base plate subject to tensile force only
- Total No of anchor bolt \( n = 8 \)
- Bolt pattern Bolt Outside Flange Only

For base plate subject to large moment
- No of bolt resisting tensile force \( n_t = 5 \)
- Anchor rod material F1554 Grade 55
- Anchor rod tensile strength \( f_{uta} = 75.0 \) [ksi]
- Bolt 1/8” (3mm) corrosion allowance No
- Anchor rod effective area \( A_{se} = 0.462 \) [in²]
- Concrete strength \( f_c = 4.5 \) [ksi]
- Base plate yield strength \( F_y = 36.0 \) [ksi]

Strength reduction factor
- Bearing on concrete \( \phi_b = 0.65 \)
- Base plate bending \( \phi_b = 0.90 \)

\[ \text{ACI 318-08} \]

CONCLUSION

[Base Plate Size and Anchor Bolt Tensile Is Adequate] OK

[The Base Plate Thickness Is Adequate] ratio= 0.52
### DESIGN CHECK

**For base plate subject to large moment**

<table>
<thead>
<tr>
<th>Anchor rod tensile resistance</th>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_r = \phi_{t,s} n_t A_{se} f_{uta}$</td>
<td><strong>ACI 318-08</strong></td>
</tr>
<tr>
<td>$\phi_{t,s} = 0.75$ for ductile steel element</td>
<td><strong>D.5.1.2 (D-3)</strong></td>
</tr>
<tr>
<td>$\phi_{t,s} = 0.75$ for ductile steel element</td>
<td><strong>D.4.4 (a)</strong></td>
</tr>
</tbody>
</table>

**W Shapes**

| $m = (N - 0.95d) / 2$ | $4.40$ [in] |
| $n = (B - 0.8b) / 2$ | $7.78$ [in] |

**HSS Rectangle Shapes**

| $m = (N - 0.95d) / 2$ | $4.40$ [in] |
| $n = (B - 0.8b) / 2$ | $7.17$ [in] |

**HSS Round Shapes**

| $m = (N - 0.8d) / 2$ | $5.44$ [in] |
| $n = (B - 0.8bf) / 2$ | $5.44$ [in] |

**m value used for design**

$m = 4.40$ [in]

**n value used for design**

$n = 7.78$ [in]

**Suggested plate thickness for rigidity:** $t_p = \text{max. of } \frac{m}{4}$ and $\frac{n}{4}$

$t_p = 1.94$ [in]

**Base plate area**

$A_1 = B \times N$

$484.0$ [in$^2$]

**Pedestal area**

$A_2 = b_c x d_c$

$15376.0$ [in$^2$]

$k = \min \left\{ \sqrt{\frac{A_2}{A_1}}, 2 \right\}$

$2.000$ [in]

$\phi_{cPn} = \phi_{c} 0.85 f'c A_1 k$

$2406.7$ [kips]

$\phi_{cPn} > P_u$

**LCB1: Axial Compressive**

$X = \frac{4db_{d}P}{(d + b_{d})^{2}}$

$0.039$ [3.1.2 on Page 16]

$\lambda = \min \left( \frac{2\sqrt{X}}{1 + \sqrt{1 - X}}, 1 \right)$

$0.2$

$\lambda n' = \lambda \sqrt{d b_{d}} / 4$

$0.53$ [in]

**For W shape**

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

$7.78$ [in]

**For HSS and Pipe**

$L = \max (m, n)$

$7.78$ [in]

**L value used for design**

$L = 7.78$ [in]

$T_p = L \left( \frac{2 P_{u}}{A_{se} f_{u} B N} \right)$

$0.88$ [in]

**Base Plate $B \times N$ OK**
**LCB2: Axial Compression + Moment**

### Code Reference

- **Pu** = 0.1 [kips]  
  **Mu** = 30.0 [kip-ft]  
  **e** = \( M_u / Pu \) = 3600.00 [in]  
  **f_p(max)** = \( f_p \times 0.85 \times f'_c \times k \) = 4.97 [ksi]  
  **q_{max}** = \( f_p(max) \times B \) = 109.40 [kips/in]  
  **e_{crit}** = \( N/2 - Pu / (2q_{max}) \) = 11.00 [in]  
  **e** > **e_{crit}** Large moment case applied

### Small moment case

- This case does not apply
  - **Bearing length**
    - \( Y = N - 2e \) = 0.00 [in]
  - **Verify linear bearing pressure**
    - \( q = Pu / Y \) = 0.00 [kips/in]
  - \( f_p = Pu / BY \) = 0.00 [ksi]
  - \( m = \max(m, n) \) = 7.78 [in]
  - If \( Y=m \)
    - \( t_{req1} = 1.49m \sqrt{f_p / F_y} \) = 0.00 [in]  
    - Eq. 3.3.14a-1
  - If \( Y<m \)
    - \( t_{req2} = 2.11 \left( \frac{f_p Y (m - \frac{Y}{2})}{F_y} \right) \) = 0.00 [in]  
    - Eq. 3.3.15a-1
  - \( t_{min} = \max(t_{req1}, t_{req2}) \) = 0.00 [in]

### Large moment case

- This case applies
  - **Check if real solution of Y exist**
    - \( \text{var}_1 = \left( f + N/2 \right)^2 \) = 400 [in^2]
    - \( \text{var}_2 = 2Pu (e+f) / q_{max} \) = 7 [in^2]
    - \( \text{var}_1 > \text{var}_2 \) OK
  - **Bearing length**
    - \( Y = \left( f + N/2 \right) \pm \sqrt{ \left( f + N/2 \right)^2 - 2Pu (e+f) / q_{max} } \) = 0.17 [in]  
    - Eq. 3.4.3
  - **Anchor rod tension force**
    - \( T_u = q_{max} Y - Pu \) = 18.0 [kips]  
    - ratio = 0.14 < \( T_r \) OK
  - **At anchor rod tension interface**
    - \( x = f - d/2 + t_f / 2 \) = 2.38 [in]  
    - Eq. 3.4.6
    - \( t_{req1} = 2.11 \left( \frac{T_u x}{BF_y} \right) \) = 0.49 [in]  
    - Eq. 3.4.7a
  - **At conc. bearing interface**
    - \( m = \max(m, n) \) = 7.78 [in]
    - If \( Y=m \)
      - \( t_{req-b} = 1.49m \sqrt{f_p(max) / F_y} \) = 0.00 [in]  
      - Eq. 3.3.14a-2
    - If \( Y<m \)
      - \( t_{req-b} = 2.11 \left( \frac{f_p(max) Y (m - \frac{Y}{2})}{F_y} \right) \) = 0.89 [in]  
      - Eq. 3.3.15a-2
    - \( t_{min} = \max(t_{req-b}, t_{req-b}) \) = 0.89 [in]

### Base Plate B x N Ok
**LCB3: Axial Compression + Moment**

### Code Reference

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_u$</td>
<td>15.0  [kips]</td>
</tr>
<tr>
<td>$M_u$</td>
<td>30.0  [kip-ft]</td>
</tr>
<tr>
<td>$e$</td>
<td>$M_u / P_u$</td>
</tr>
<tr>
<td>$f_{(max)}$</td>
<td>$\phi_c 0.85 f'_c k$</td>
</tr>
<tr>
<td>$q_{max}$</td>
<td>$f_{(max)} X B$</td>
</tr>
<tr>
<td>$e_{crit}$</td>
<td>$N/2 - P_u / (2q_{max})$</td>
</tr>
</tbody>
</table>

**Small moment case**

This case does not apply

Bearing length $Y = N - 2e = 0.00$ [in]
Verify linear bearing pressure $q = P_u / Y = 0.00$ [kips/in]

If $Y=m$

$t_{eq1} = 1.49m \sqrt{f_p / F_y} = 0.00$ [in] Eq. 3.3.14a-1

If $Y<m$

$t_{eq2} = 2.11 \left[ \frac{f_p Y (m - Y)}{2} \right] = 0.00$ [in] Eq. 3.3.15a-1

$t_{min} = \max(t_{eq1}, t_{eq2}) = 0.00$ [in]

**Large moment case**

This case applies

Check if real solution of $Y$ exist

$\text{var}_1 = (f + N/2)^2 = 400$ [in$^2$]
$\text{var}_2 = 2P_u (e+f) / q_{max} = 9$ [in$^2$]
$\text{var}_1 > \text{var}_2$ OK

Bearing length $Y = \left( f + \frac{N}{2} \right) \pm \left( f + \frac{N}{2} \right)^2 - 2P_u (e+f) / q_{max} = 0.23$ [in] Eq. 3.4.3

Anchor rod tension force $T_u = q_{max} Y - P_u = 9.9$ [kips]
ratio $= 0.08 < T_r$ OK

At anchor rod tension interface

$x = f - d/2 + t_f / 2 = 2.38$ [in] Eq. 3.4.6

$t_{eq1} = 2.11 \left[ \frac{T_u X}{BF_y} \right] = 0.36$ [in] Eq. 3.4.7a

At conc. bearing interface

$m = \max(m, n) = 7.78$ [in]

If $Y=m$

$t_{eq1} = 1.49m \sqrt{f_{(max)} / F_y} = 0.00$ [in] Eq. 3.3.14a-2

If $Y<m$

$t_{eq2} = 2.11 \left[ \frac{f_{(max)} Y (m - Y)}{2} \right] = 1.04$ [in] Eq. 3.3.15a-2

$t_{min} = \max(t_{eq1}, t_{eq2}) = 1.04$ [in]

Base Plate B x N OK
LCB4: Axial Tensile

Factored tensile load

\[ P_u = 10.0 \text{ [kips]} \]

For base plate subject to tensile force only

\[ T_r = \phi_{ts} n A_{se} f_{ut} = 207.9 \text{ [kips]} \]

\[ \phi_{ts} = 0.75 \text{ for ductile steel element} \]

ratio = 0.05 > Pu

OK

Bolt pattern Bolt Outside Flange Only

Total No of anchor bolt

n = 8

Bolt to column center dist. \( f = 9.0 \text{ [in]} \)

Bolt to column web center dist. \( f_1 = 9.0 \text{ [in]} \)

Each bolt factored tensile load \( T_u = 1.3 \text{ [kips]} \)

Bending to Column Flange

Moment lever arm \( a = 2.38 \text{ [in]} \)

Moment to column flange \( M_u = 0.25 \text{ [kip-ft]} \)

Effective plate width \( b_{eff} = 2 \times a \)

\[ b_{eff} = 4.76 \text{ [in]} \]

Base plate required thickness \( t_{p1} = \frac{4 M_u}{b_{eff} \phi_y F_y} \)

\[ t_{p1} = 0.28 \text{ [in]} \]

Bending to Column Web

Moment lever arm \( a = 8.82 \text{ [in]} \)

Moment to column flange \( M_u = 0.92 \text{ [kip-ft]} \)

Effective plate width \( b_{eff} = 2 \times a \)

\[ b_{eff} = 17.63 \text{ [in]} \]

Base plate required thickness \( t_{p2} = \frac{4 M_u}{b_{eff} \phi_y F_y} \)

\[ t_{p2} = 0.00 \text{ [in]} \]

\[ t_{min} = \max ( t_{p1}, t_{p2} ) \]

\[ t_{min} = 0.28 \text{ [in]} \]

Anchor Bolt Tensile OK
Example 52: Base Plate (S16-09) & Anchor Bolt (CSA A23.3-04) Design With Anchor Reinforcement

**BASE PLATE & ANCHOR BOLT DESIGN - MOMENT CONNECTION**

**Base Plate Data**
- Column section type: W_Shape
- Column size: W360x79
- Depth: \( d = 354.0 \) [mm]
- Flange thickness: \( t_f = 16.8 \) [mm]
- Flange width: \( b_f = 205.0 \) [mm]
- Web thickness: \( t_w = 9.4 \) [mm]
- Base plate anchor bolt pattern: 4 or 6-Bolt MC WF
- Base plate anchor bolt location: Bolt Outside Flange Only

**Base Plate Dimensions**
- Base plate width: \( B = 559 \) [mm]
- Base plate depth: \( N = 559 \) [mm]
- Base plate thickness: \( t_p = 51 \) [mm]
- Anchor bolt spacing \( s_2 = C \): \( C = 457 \) [mm]
- Anchor bolt spacing \( s_1 = D \): \( D = 457 \) [mm]

**Bolt to Column Dimensions**
- Bolt to column center dist. \( f = 229 \) [mm]
- Bolt to column web center dist. \( f_1 = 229 \) [mm]

**Suggested Plate Thickness**
- Suggested plate thickness for rigidity: \( t_p = \text{max. of } \frac{m}{4} \text{ and } \frac{n}{4} \)

**Factored Column Load**

<table>
<thead>
<tr>
<th>LCB</th>
<th>Cases</th>
<th>( P_u ) [kN]</th>
<th>( V_u ) [kN]</th>
<th>( M_u ) [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCB1</td>
<td>Axial Comp.</td>
<td>444.8</td>
<td>66.7</td>
<td>0.0</td>
</tr>
<tr>
<td>LCB2</td>
<td>Axial Comp. + M</td>
<td>0.0</td>
<td>89.0</td>
<td>40.7</td>
</tr>
<tr>
<td>LCB3</td>
<td>Axial Comp. + M</td>
<td>66.7</td>
<td>89.0</td>
<td>40.7</td>
</tr>
<tr>
<td>LCB4</td>
<td>Axial Tensile</td>
<td>44.5</td>
<td>155.7</td>
<td>0.0</td>
</tr>
</tbody>
</table>
### Anchor Bolt Data

<table>
<thead>
<tr>
<th>Anchor Bolt Measurement</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of bolt line for resisting moment</td>
<td>3 Bolt Line</td>
<td></td>
</tr>
<tr>
<td>No of bolt along outermost bolt line</td>
<td>3</td>
<td>min required</td>
</tr>
<tr>
<td>Outermost bolt line spacing $s_1$</td>
<td>457 [mm] 89</td>
<td>OK</td>
</tr>
<tr>
<td>Outermost bolt line spacing $s_2$</td>
<td>457 [mm] 89</td>
<td>OK</td>
</tr>
<tr>
<td>Internal bolt line spacing $s_{b1}$</td>
<td>229 [mm] 89</td>
<td>OK</td>
</tr>
<tr>
<td>Internal bolt line spacing $s_{b2}$</td>
<td>0 [mm] 89</td>
<td>OK</td>
</tr>
<tr>
<td>Anchor bolt material</td>
<td>F1554 Grade 55</td>
<td>D.2</td>
</tr>
<tr>
<td>Anchor tensile strength $f_{atu}$</td>
<td>75.0 [ksi]</td>
<td>517 [MPa]</td>
</tr>
<tr>
<td>Anchor bolt diameter $d_a$</td>
<td>0.875 [in] max 1.5 in</td>
<td>22.2 [mm]</td>
</tr>
<tr>
<td>Bolt sleeve diameter $d_s$</td>
<td>51 [mm]</td>
<td>Page A-1 Table 1</td>
</tr>
<tr>
<td>Bolt sleeve height $h_s$</td>
<td>178 [mm]</td>
<td></td>
</tr>
<tr>
<td>Anchor bolt embedment depth $h_{ef}$</td>
<td>508 [mm] 267</td>
<td>OK</td>
</tr>
<tr>
<td>Pedestal height $h_a$</td>
<td>584 [mm] 584</td>
<td>OK</td>
</tr>
<tr>
<td>Pedestal width $b_c$</td>
<td>3150 [mm]</td>
<td></td>
</tr>
<tr>
<td>Pedestal depth $d_c$</td>
<td>3150 [mm]</td>
<td></td>
</tr>
</tbody>
</table>
To be considered effective for resisting anchor tension, ver reinforcing bars shall be located within 0.5$h_{pf}$ from the outmost anchor's centerline. In this design $0.5h_{pf}$ value is limited to 200 mm.

\[ 0.5h_{pf} = 200 \text{ [mm]} \]

No. of ver. rebar that are effective for resisting anchor tension:
\[ n_v = 6 \]
Ver. bar size:
\[ d_b = 25 \text{ [mm] } \]
Single bar area:
\[ A_s = 500 \text{ [mm}^2\text{]} \]

ACI318 M-08

To be considered effective for resisting anchor shear, hor. reinf shall be located within \( \min(0.5c_1, 0.3c_2) \) from the outmost anchor's centerline.

\[ \min(0.5c_1, 0.3c_2) = 46 \text{ [mm]} \]

No. of tie leg that are effective to resist anchor shear:
\[ n_{leg} = 2 \]
No. of tie layer that are effective to resist anchor shear:
\[ n_{lay} = 2 \]
Tie bar size:
\[ d_b = 15 \text{ [mm] } \]
Single bar area:
\[ A_s = 200 \text{ [mm}^2\text{]} \]

For anchor reinf shear breakout strength calc:
\[ 100\% \text{ hor. tie bars develop full yield strength} \]
### Design of Anchorage to Concrete Using ACI 318-08 & CSA-A23.3-04 Code

**Concrete strength** \( f'_c = 31 \text{ [MPa]} \) 30

**Rebar yield strength** \( f_y = 414 \text{ [MPa]} \) 400

**Base plate yield strength** \( F_y = 248 \text{ [MPa]} \) 300

Total no of anchor bolt \( n = 8 \)

No of anchor bolt carrying shear \( n_s = 8 \)

For side-face blowout check use

No of bolt along width edge \( n_{bw} = 3 \)

No of bolt along depth edge \( n_{bd} = 3 \)

Anchor head type = Heavy Hex

Anchor effective cross sect area \( A_{se} = 0.462 \text{ [in}^2\text{]} = 298 \text{ [mm}^2\text{]} \)

Bearing area of head \( A_{org} = 1.188 \text{ [in}^2\text{]} = 766 \text{ [mm}^2\text{]} \)

Bolt 1/8” (3mm) corrosion allowance = No

Provide shear key? = No

Seismic region where \( I_{F_a} S_d(0.2) \geq 0.35 \) = No

Provide built-up grout pad? = Yes

### Code Reference

- A23.3-04 (R2010)
- D.4.3.5
- D.7.1.3

### CONCLUSION

**OVERALL**

**BASE PLATE**

Base Plate Size and Anchor Bolt Tensile OK

Base Plate Thickness ratio = 0.52 OK

**ANCHOR BOLT**

**LCB1 Axial Compression**

Anchor Rod Embedment, Spacing and Edge Distance OK

Min Required Anchor Reinft. Development Length ratio = 0.94 OK

Overall Ratio ratio = 0.29 OK

**LCB2 Axial Compression + Moment**

Anchor Rod Embedment, Spacing and Edge Distance OK

Min Required Anchor Reinft. Development Length ratio = 0.94 OK

Overall Ratio ratio = 0.57 OK

**LCB3 Axial Compression + Moment**

Anchor Rod Embedment, Spacing and Edge Distance OK

Min Required Anchor Reinft. Development Length ratio = 0.94 OK

Overall Ratio ratio = 0.49 OK

**LCB4 Axial Tensile**

Anchor Rod Embedment, Spacing and Edge Distance OK

Min Required Anchor Reinft. Development Length ratio = 0.94 OK

Overall Ratio ratio = 0.68 OK
BASE PLATE DESIGN

Base plate design based on

- AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition
- CSA-A23.3-04 (R2010) Design of Concrete Structures Annex D
- ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary

DESIGN DATA

<table>
<thead>
<tr>
<th>Column section type</th>
<th>W_Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column size</td>
<td>W360x79</td>
</tr>
<tr>
<td>Depth</td>
<td>d = 354.0 [mm]</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>t_f = 16.8 [mm]</td>
</tr>
<tr>
<td>Flange width</td>
<td>b_f = 205.0 [mm]</td>
</tr>
<tr>
<td>Web thickness</td>
<td>t_w = 9.4 [mm]</td>
</tr>
</tbody>
</table>

Base plate anchor bolt pattern: 4 or 6-Bolt MC WF (base plate is moment connection)

| Base plate width | B = 559 [mm] |
| Base plate depth | N = 559 [mm] |
| Base plate thickness | t_p = 51 [mm] |
| Anchor bolt spacing | C = 457 [mm] |
| Anchor bolt spacing | D = 457 [mm] |
| Anchor bolt diameter | d = 0.875 [in] |

Bolt to column center dist. | f = 229 [mm] |
Bolt to column web center dist. | f_1 = 229 [mm] |
Pedestal width | b_c = 3150 [mm] |
Pedestal depth | d_c = 3150 [mm] |
Factored column load

<table>
<thead>
<tr>
<th>LCB</th>
<th>Cases</th>
<th>P_c [kN]</th>
<th>M_u [kNm]</th>
<th>t_p (mm)</th>
<th>Base Plate Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCB1</td>
<td>Axial Compressive</td>
<td>444.8</td>
<td>0.0</td>
<td>22.3</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB2</td>
<td>Compression + M</td>
<td>0.0</td>
<td>40.7</td>
<td>22.5</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB3</td>
<td>Compression + M</td>
<td>66.7</td>
<td>40.7</td>
<td>26.3</td>
<td>Base Plate B x N OK</td>
</tr>
<tr>
<td>LCB4</td>
<td>Axial Tensile</td>
<td>44.5</td>
<td>0.0</td>
<td>7.1</td>
<td>Anchor Bolt Tensile OK</td>
</tr>
</tbody>
</table>

Min required plate thickness = 26.3 mm

Suggest max plate thickness = 45 mm

For base plate subject to tensile force only

Total No of anchor bolt = 8
Bolt pattern = Bolt Outside Flange Only

For base plate subject to large moment

No of bolt resisting tensile force = 5

Anchor rod material = F1554 Grade 55
Anchor rod tensile strength = 75.0 [ksi] = 517 [MPa]

Bolt 1/8” (3mm) corrosion allowance = No
Anchor rod effective area = 0.462 [in^2] = 298 [mm^2]

Concrete strength = 31 [MPa]
Base plate yield strength = 248 [MPa]

Code Reference
A23.3-04 (R2010)

Strength reduction factor
Bearing on concrete = 0.65
Steel anchor resistance factor = 0.85
Base plate bending = 0.90

CONCLUSION

[Base Plate Size and Anchor Bolt Tensile Is Adequate] OK

[The Base Plate Thickness Is Adequate] ratio = 0.52
DESIGN CHECK

For base plate subject to large moment
Anchor rod tensile resistance

\[ T_r = n_t A_{se} f_u t_{se} \]
\[ R_{t,s} = 0.80 \text{ for ductile steel in tension} \]

**Code Reference**

A23.3-04 (R2010)

D.6.1.2 (D-3)

D.5.4(a)

**W Shapes**

\[ m = \frac{(N - 0.95d)}{2} = 111.3 \text{ [mm]} \]

\[ n = \frac{(B - 0.8bf)}{2} = 197.4 \text{ [mm]} \]

**HSS Rectangle Shapes**

\[ m = \frac{(N - 0.95d)}{2} = 111.3 \text{ [mm]} \]

\[ n = \frac{(B - 0.9bf)}{2} = 182.0 \text{ [mm]} \]

**HSS Round Shapes**

\[ m = \frac{(N - 0.8d)}{2} = 137.8 \text{ [mm]} \]

\[ n = \frac{(B - 0.8bf)}{2} = 137.8 \text{ [mm]} \]

**Base plate area**

\[ A_1 = B \times N \]

\[ A_2 = b_c \times d_c \]

**Pedestal area**

\[ k = \min \left( \sqrt{\frac{A_2}{A_1}}, 2 \right) = 2.00 \]

\[ f_{c'} A_1 k = 10696.4 \text{ [kN]} \]

\[ > P_u \quad \text{OK} \]

**LCB1: Axial Compressive**

\[ X = \frac{4db_y}{(d + b_y)^2} \]

\[ \lambda = \min\left( \frac{2\sqrt{X}}{1 + \sqrt{1 - X}}, 1 \right) = 0.2 \]

\[ \lambda n' = \lambda \sqrt{\frac{2}{d+2b_y}} / 4 = 13.4 \text{ [mm]} \]

**Base Plate B x N OK**
LCB2: Axial Compression + Moment

<table>
<thead>
<tr>
<th>Code Reference</th>
<th>P_u = 0.1 [kN]</th>
<th>M_u = 40.7 [kNm]</th>
<th>e = M_u / P_u = 407000 [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f_p(max) = \phi_p 0.85 f'_c k = 34.3 [MPa]</td>
<td>q_{max} = f_p(max) \times B = 19142 [N/mm]</td>
<td>e_{crit} = N/2 - P_u / (2q_{max}) = 279.4 [mm]</td>
</tr>
<tr>
<td></td>
<td>e &gt; e_{crit} Large moment case applied</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Small moment case**

This case does not apply

- Verify linear bearing pressure
  - q = P_u / Y = 0 [N/mm] < q_{max} OK

- If Y=m
  - t_{eq1} = 1.49m \sqrt{f_p / f_y} = 0.0 [mm] Eq. 3.3.14a-1

- If Y<m
  - t_{eq2} = 2.11 \frac{f_p Y (m - Y/2)}{f_y} = 0.0 [mm] Eq. 3.3.15a-1

- t_{eq} = max (t_{eq1}, t_{eq2}) = 0.0 [mm]

**Large moment case**

This case applies

- Check if real solution of Y exist
  - var_1 = (f + N/2)^2 = 258064 [mm^2]
  - var_2 = 2P_u (e+f) / q_{max} = 4255 [mm^2]
  - var_1 > var_2 OK

- Bearing length
  - Y = \left( f + \frac{N}{2} \right)^{\pm} \sqrt{\left( f + \frac{N}{2} \right)^{2} - 2P_u (e + f) \frac{Y}{q_{max}}} = 4.2 [mm] Eq. 3.4.3

- Anchor rod tension force
  - T_u = q_{max} Y - P_u = 80.4 [kN] Eq. 3.4.2
  - ratio = 0.15 < T_u OK

- At anchor rod tension interface
  - x = f - d/2 + t_f / 2 = 60.0 [mm] Eq. 3.4.6
  - t_{eq1} = 2.11 \frac{T_u X}{BF_y} = 12.4 [mm] Eq. 3.4.7a

- At conc. bearing interface
  - m = max(m, n) = 197.4 [mm]
  - If Y\geq m
    - t_{eq-b} = 1.49m \sqrt{f_p(max) / f_y} = 0.0 [mm] Eq. 3.3.14a-2
  - If Y<m
    - t_{eq-b} = 2.11 \frac{f_p(max) Y (m - Y/2)}{f_y} = 22.5 [mm] Eq. 3.3.15a-2

- t_{min} = max (t_{eq-b}, t_{eq-b}) = 22.5 [mm]

Base Plate B x N OK
LCB3: Axial Compression + Moment

<table>
<thead>
<tr>
<th>Code Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>P_u = 66.7 [kN]</td>
</tr>
<tr>
<td>e = M_u / P_u = 610 [mm]</td>
</tr>
<tr>
<td>f_u(\text{max}) = \phi_x 0.85 f'_c k = 34.3 [MPa]</td>
</tr>
<tr>
<td>q_{\text{max}} = f_u(\text{max}) \times B = 19142 [N/mm]</td>
</tr>
<tr>
<td>e_{\text{crit}} = N/2 - P_u / (2q_{\text{max}}) = 277.7 [mm]</td>
</tr>
</tbody>
</table>

**Large moment case**  \[ e > e_{\text{crit}} \] Large moment case applied

**Small moment case**  \[ e < e_{\text{crit}} \] This case does not apply

**AISC Design Guide 1**

### Small moment case
- **Bearing length**
  \[ Y = N - 2e = 0.0 \ [mm] \]
- **Verify linear bearing pressure**
  \[ q = P_u / Y = 0 \ [N/mm] \]
  \[ < q_{\text{max}} \] **OK**

  \[ f_p = P_u / (BY) = 0.0 \ [MPa] \]
  \[ m = \max(m, n) = 197.4 \ [mm] \]

  - **If** \[ Y = m \]
    \[ t_{\text{req1}} = 1.49 m \sqrt{f_p / f_y} = 0.00 \ [mm] \ Eq. 3.3.14a-1
  
  - **If** \[ Y < m \]
    \[ t_{\text{req2}} = 2.11 \left( f_p Y \left( m - Y^2 \right) / f_y \right) = 0.00 \ [mm] \ Eq. 3.3.15a-1

  \[ t_{\text{min}} = \max(t_{\text{req1}}, t_{\text{req2}}) = 0.0 \ [mm] \]

### Large moment case
- **Check if real solution of Y exist**
  \[ \text{var}_1 = (f + N/2)^2 / 2 = 258064 \ [mm^2] \]
  \[ \text{var}_2 = 2P_u (e+f) / q_{\text{max}} = 5846 \ [mm^2] \]
  \[ \text{var}_1 > \text{var}_2 \] **OK**

- **Bearing length**
  \[ Y = \left( f + \frac{N}{2} \right) / \left( \sqrt{\frac{(f+N/2)^2}{2} - 2P_u (e+f) / q_{\text{max}}} \right) = 5.8 \ [mm] \ Eq. 3.4.3

- **Anchor rod tension force**
  \[ T_u = q_{\text{max}} Y - P_u = 44.1 \ [MPa] \ Eq. 3.4.2
  
  \[ \text{ratio} = 0.08 < T_r \] **OK**

- **At anchor rod tension interface**
  \[ x = f - d/2 + t / 2 = 60.0 \ [mm] \ Eq. 3.4.6
  
  \[ t_{\text{req1}} = 2.11 \frac{F_x}{Bf_y} = 9.2 \ [mm] \ Eq. 3.4.7a

- **At conc. bearing interface**
  \[ m = \max(m, n) = 197.4 \ [mm] \]

  - **If** \[ Y = m \]
    \[ t_{\text{req}} = 1.49 m \sqrt{f_p / f_y} = 0.00 \ [mm] \ Eq. 3.3.14a-2
  
  - **If** \[ Y < m \]
    \[ t_{\text{req}} = 2.11 \left( f_p Y \left( m - Y^2 \right) / f_y \right) = 26.3 \ [mm] \ Eq. 3.3.15a-2

  \[ t_{\text{min}} = \max(t_{\text{req}}, t_{\text{req}}) = 26.3 \ [mm] \]
LCB4: Axial Tensile

Factored tensile load \( P_u = 44.5 \) [kN]

For base plate subject to tensile force only

Anchor rod tensile resistance\( T_r = n A_w f_{w} \) [kN]
\( R_{ts} = 0.80 \) for ductile steel in tension
\( \frac{R_{ts}}{t} = 0.05 > P_u \)

Anchor rod tensile resistance \( T_r = n A_w f_{w} \) [kN] \( R_{ts} = 0.80 \) for ductile steel in tension \( \frac{R_{ts}}{t} = 0.05 > P_u \)

Bolt pattern Bolt Outside Flange Only

Total No of anchor bolt \( n = 8 \)

Bolt to column center dist. \( f = 229 \) [mm]

Bolt to column web center dist. \( f_1 = 229 \) [mm]

Each bolt factored tensile load \( T_u = 5.6 \) [kN]

Bending to Column Flange

Moment lever arm \( a = 60 \) [mm]

Moment to column flange \( M_u = 0.3 \) [kNm]

Effective plate width \( b_{eff} = 2 \times a \) \( = 120 \) [mm]

Base plate required thickness \( t_{p1} = \frac{4 M_u}{b_{eff} f_{y}} \) \( = 7.1 \) [mm]

Bending to Column Web

Moment lever arm \( a = 224 \) [mm]

Moment to column flange \( M_u = 1.2 \) [kNm]

Effective plate width \( b_{eff} = 2 \times a \) \( = 448 \) [mm]

Base plate required thickness \( t_{p2} = \frac{4 M_u}{b_{eff} f_{y}} \) \( = 0.0 \) [mm]

\( t_{min} = \max ( t_{p1}, t_{p2} ) \) \( = 7.1 \) [mm]

Anchor Bolt Tensile OK
3.0 REFERENCES

1. ACI 318-08 Building Code Requirements for Structural Concrete and Commentary
2. ACI 318M-08 Metric Building Code Requirements for Structural Concrete and Commentary
3. ACI 349-06 Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary
4. ACI 349.2R-07 Guide to the Concrete Capacity Design (CCD) Method - Embedment Design Examples
5. ACI 355.3R-11 Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D
6. Design of Anchor Reinforcement in Concrete Pedestals by Widianto, Chandu Patel, and Jerry Owen
7. CSA A23.3-04 (R2010) - Design of Concrete Structures
8. AISC Design Guide 1: Base Plate and Anchor Rod Design 2nd Edition