**Connections:**

We need to design the following connections:

**Joist Hangers**

<table>
<thead>
<tr>
<th>Joist: 2x12 @ 16”</th>
<th>#2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downward: 430 lbf</td>
<td>CD=1.25</td>
</tr>
<tr>
<td>Upward: 290 lbf</td>
<td>CD=1.6</td>
</tr>
</tbody>
</table>

**Beam-Column connection**

<table>
<thead>
<tr>
<th>Beam: 6¾” x 34¾”</th>
<th>24F-1.8E SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column: 6¾” x 6¾”</td>
<td>#47 N2M14 SYP</td>
</tr>
<tr>
<td>Upward: 9,600 lbf</td>
<td>CD=1.6</td>
</tr>
</tbody>
</table>

**Column-Foundation connection**

<table>
<thead>
<tr>
<th>Column: 6¾” x 6¾”</th>
<th>#47 N2M14 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upward: 9,600 lbf</td>
<td>CD=1.6</td>
</tr>
<tr>
<td>16,000 lb for ACI 318 App D</td>
<td></td>
</tr>
</tbody>
</table>

**Stud Connection (top and bottom)**

<table>
<thead>
<tr>
<th>Stud: 2x10 @ 16”</th>
<th>#2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear: 380 lbf/stud</td>
<td>CD=1.6</td>
</tr>
</tbody>
</table>

**Sill Plate Anchorage**

<table>
<thead>
<tr>
<th>Sill Plate: 2x10 #2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear parallel to grain: 770 plf</td>
</tr>
<tr>
<td>Shear perpendicular to grain: 290 plf</td>
</tr>
<tr>
<td>1,280 plf for ACI 318 App D</td>
</tr>
<tr>
<td>483 plf for ACI 318 App D</td>
</tr>
</tbody>
</table>

**Diaphragm chord splice**

<table>
<thead>
<tr>
<th>Top Plate: DBL 2x10</th>
<th>#2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: 14,400 lbf</td>
<td>CD=1.6</td>
</tr>
</tbody>
</table>

**Shear wall drag strut splice**

<table>
<thead>
<tr>
<th>Top Plate: DBL 2x10</th>
<th>#2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: 4,920 lbf</td>
<td>CD=1.6</td>
</tr>
</tbody>
</table>

**Shear wall panel chord**

<table>
<thead>
<tr>
<th>Chord: DBL 2x10</th>
<th>#2 SYP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension: 12,300 lbf</td>
<td>CD=1.6</td>
</tr>
<tr>
<td>20,500 lb for ACI 318 App D</td>
<td></td>
</tr>
</tbody>
</table>
Nail Pullout
8d common nail in 5/8" thick OSB
Roof: 90 psf upward CD=1.6
Wall: 45 psf outward CD=1.6

For a single sheet of plywood:

\[ \text{Uplift } = 90 \text{ psf} \cdot 4 \text{ ft} \cdot 8 \text{ ft} = 2880 \text{ lbf} \]

\[ \text{Nail Pullout } = \frac{\text{Uplift}}{33 \text{ nail}} = \frac{2880}{33 \text{ nail}} = 87.273 \frac{\text{lbf}}{\text{nail}} \]

Therefore:

We will only cover bolts and nails:

Bolts: Double Top Plate Splice
Shear Wall Chord Anchorage
Sill Plate Anchorage
Beam and Column Connection and Anchorage

Nails: Plywood Pullout
Joists
Studs
**Splice in Double Top Plate:**

\[ C_D = 1.6 \quad T = 14400 \text{ lbs} \quad ***\text{Members are 2 - 2x10} \]

Choose Bolt Size (Southern Pine):

For: \[ D = \frac{3}{4} \text{ in} \]

Geometry Factor (C_D):

\[ 1 = 1.5 \quad \frac{1}{D} = 2 < 6 \]

(a) \[ \text{Edge}_{\text{min}} = 1.5 \cdot D = 1.125 \text{ in} \quad \text{[Table: 11.5.1A]} \]

(b) \[ \text{End}_{\text{min}} = 7 \cdot D = 5.25 \text{ in} \quad \text{[Table: 11.5.1B]} \]

(c) \[ \text{Spacing}_{\text{min}} = 1.5 \cdot D = 1.125 \text{ in} \quad \text{[Table: 11.5.1C]} \]

\[ \text{Spacing}_{\text{max}} = 5 \text{ in} \]

All spacing requirements in (a), (b), and (c) can be met, therefore:

\[ C_D = 1.0 \]

For Group Action Factor (C_g):

\[ A_m = 1.5 \cdot 10 = 15 \text{ in}^2 \]

\[ A_S = 1.5 \cdot 10 = 15 \text{ in}^2 \]

\[ \frac{A_m}{A_s} = 1 \quad \text{And Number of Fasteners per row is 3, therefore:} \]

\[ C_g = 0.99 \]

\[ C_M = 1.0 \]

\[ C_t = 1.0 \]

\[ Z_{11} = Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_D = 1267.2 \text{ lbs} \]
Max Capacity For Six Bolts:

\[
\text{Max Capacity} = 6 \cdot Z'_{11} = 7603.2 \text{ lbs} > \frac{T}{2} = 7200 \text{ lbs}
\]

Final Top Chord Splice Design (Use 6 3/4" Diameter Bolts):

Shear Wall Chord Anchorage:

Use bolts and a metal bracket

12,300 lb

Edge of opening 2
For Double Studs And Metal Bracket:

\[ T = 12300 \text{ lbs} \quad \text{(studs are 2 - 2x10 in)} \]

**Choose Bolt Size (Southern Pine):**

2005 NDS, Table 11B (p.82)

Interpolate b/t 2.5" and 3" main member thickness:

For : \[ D = 1 \text{ in} \]

\[ Z_{11} = \frac{2480}{2} + 1830 = 2155 \text{ lbs} \]

\[ n = \frac{T}{(C_D Z_{11})} = 3.567 \]

**Try 4 - 1" bolts, 2 rows of 2.**

**Geometry Factor \((C_\Delta)\):**

2005 NDS, Section 11.5.1 (p.76)

\[ \frac{1}{D} = 3 < 6 \]

(a) \( \text{Edge}_{\text{min}} = 1.5\cdot D = 1.5 \text{ in} \) [Table :11.5.1A]

(b) \( \text{End}_{\text{min}} = 7\cdot D = 7 \text{ in} \) [Table :11.5.1B]

(c) \( \text{Spacing}_{\text{min}} = 1.5\cdot D = 1.5 \text{ in} \) [Table :11.5.1C]

\( \text{Spacing}_{\text{max}} = 5 \text{ in} \)

All spacing requirements in (a), (b), and (c) can be met.

\[ C_\Delta = 1.0 \]

**For Group Action Factor \((C_g)\):**

2005 NDS, Table 10.3.6C (p.63)

\[ A_m = 1.5\cdot 10 = 15 \text{ in}^2 \]

\[ A_s = .25\cdot 8 = 2 \text{ in}^2 \]

\[ \frac{A_m}{A_s} = 7.5 \]

And Number of Fasteners per row is 2, therefore:

\[ C_g = 0.99 \]

\[ C_M = 1.0 \]

\[ C_t = 1.0 \]

\[ Z'_{11} = Z_{11}\cdot C_D\cdot C_M\cdot C_t\cdot C_g\cdot C_\Delta = 3413.52 \text{ lbs} \]

**Max Capacity For Four Bolts:**

\[ \text{Max Capacity} = 4\cdot Z'_{11} = 13654.08 \text{ lbs} > T = 12300 \text{ lbs} \]
Foundation Anchorage (ACI 318-05 Appendix D):

\[ T = 20500 \text{ lbs} \]

Check Steel Strength: 2005 AISC Manual, Table 7-2 (p.7-23)

Try 1" diameter bolt, A307 steel bolts used for conservative design:

\[ N_u = T = 20500 \text{ lbs} \]

\[ \phi = 0.75 \quad A_s = 0.785 \text{ in} \quad \phi F_{nt} = 33800 \text{ psi} \]

\[ A_s \cdot \phi F_{nt} = 26533 \text{ lbs} > N_u = 20500 \text{ lbs} \]

Check Breakout Strength: ACI 318-08, Appendix D .5.2 (p.420)

Tension load on ductile steel element:

\[ \phi = 0.70 \quad \text{ACI 318-08, Appendix D .4.4 (p.418)} \]

Distance from slab edge is 4.625":

\[ c_{al} = 4.625 \text{ in} \quad c_{amin} = c_{al} \]

Embedment depth is 15":

\[ h_{ef} = 15 \text{ in} \]

\[ A_{Ne} = (c_{al} + h_{ef}) \left(2 \cdot 1.5 \cdot h_{ef}\right) = 883.125 \text{ in}^2 \]

\[ A_{Neo} = 9 \cdot h_{ef}^2 = 2025 \text{ in}^2 \]

Modification factor for edge effects:

\[ \frac{c_{amin}}{1.5 \cdot h_{ef}} < 1.5 \cdot h_{ef} \quad \text{ACI 318-08, Appendix D .5.2.5 (p.423)} \]

\[ \Psi_{ed \_N} = 0.7 + 0.3 \left( \frac{c_{amin}}{1.5 \cdot h_{ef}} \right) = 0.762 \]
Modification factor for cracked concrete at service loads:

\[ \Psi_{c, N} = 1.0 \]  

ACI 318-08, Appendix D.5.2.6 (p.424)

Modification factor for post-installed anchors:

\[ \Psi_{cp, N} = 1.0 \]  

For cast in place anchor.  

ACI 318-08, Appendix D.5.2.7 (p.425)

Basic concrete breakout strength of a single anchor in cracked concrete:

\[ k_c = 24 \] (for bolts)  

\[ f_c' = 4000 \]

\[ N_b = k_c \cdot f_c' \cdot h_{ef}^{1.5} = 88181.631 \text{ lbs} \]

\[ N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ed, N} \cdot \Psi_{c, N} \cdot \Psi_{cp, N} \cdot N_b = 29291.407 \text{ lbs} \]

\[ \phi \cdot N_{cb} = 20503.985 \text{ lbs} > N_u = 20500 \text{ lbs} \]

Check Pullout Strength:

\[ \phi = 0.70 \]  

\[ \Psi_{c, P} = 1.0 \] (for cracking)  

\[ A_{brg} = \frac{\pi \cdot 1.625^2}{4} = 1.289 \]

\[ N_p = 8 \cdot A_{brg} \cdot f_c' = 41233.404 \]

\[ \phi \cdot N_p = 28863.383 \text{ lb} \]

Check Side-Face Blowout Strength:

2.5\(a_1\) < \(h_{ef}\), therefore no blowout calculation is not needed

Final Shear Wall Chord Anchorage Design:
Bottom Sill Plate Anchorage:

Shear parallel to grain: \( F_{11} = 770 \text{ plf} \quad C_D = 1.6 \)

Shear perpendicular to grain: \( F_{\text{per}} = 290 \text{ plf} \quad C_D = 1.6 \)

Sill Plate Size: 2 x 10 On Concrete \( t_s = 1.5 \text{ in} \)

Choose Bolt Size:

Try 1" bolt so that the same bolt is used everywhere:

\( D = 1 \text{ in} \)

Check Shear Perpendicular To Grain:

\[
Z_{\text{per}} = 2250 \text{ lbs / bolt}
\]

\[
C_\Delta = 1.0
\]

\[
C_g = 1.0
\]

\[
C_M = 1.0
\]

\[
C_t = 1.0
\]

\[
Z'_{\text{per}} = Z_{\text{per}} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 3600 \text{ lbs / bolt}
\]

Check Shear Parallel To Grain:

\[
Z_{11} = 1020 \frac{\text{lbs}}{\text{bolt}}
\]

\[
C_\Delta = 1.0
\]

\[
C_g = 1.0
\]

\[
C_M = 1.0
\]

\[
C_t = 1.0
\]

\[
Z'_{11} = Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 1632 \frac{\text{lbs}}{\text{bolt}}
\]

Design Edge Distance:

\[
\text{Edge}_{\text{min}} = 4 \cdot D = 4 \text{ in} \quad < \quad 4.625 \text{ in}
\]

Design Lateral Spacing:

\[
\text{Spacing}_{\text{per}} = \frac{Z'_{\text{per}}}{F_{\text{per}}} = 12.414 \text{ ft}
\]

Therefore, use 12’ o.c.
Spacing \( \frac{Z_{11}}{F_{11}} = 2.119 \text{ ft} \)

**Max load in 12’ section:**

\[
12F_{11} = 9240 \text{ lbs} \quad >> \quad Z'_{\text{per}} = 3600 \frac{\text{lbs}}{\text{bolt}}
\]

Therefore, we must make the spacing alot closer; try 4’ o.c.:

\[
4F_{11} = 3080 \text{ lbs} < Z'_{\text{per}} = 3600 \frac{\text{lbs}}{\text{bolt}}
\]

**Check Bolt Anchorage for Shear (ACI 318-05 Appendix D):**

\[
\frac{9.25}{2} = 4.625" \quad \sqrt{\psi} = 290 \quad \sqrt{\psi_{\text{act}}} = 290(4) = 1160 \text{ lbs}
\]

\[
\sqrt{\alpha} = 1.6(1160) = 1856 \text{ lbs}
\]

\[
4\ell(15) = 6.9" \quad 1" \phi
\]
**Check Steel Strength:**  

Try 1" diameter bolt, A307 steel bolts used for conservative design:

\[ V_u : = 1856 \text{ lbs} \]

\[ \phi : = 0.75 \quad A_s : = 0.785 \quad \text{in} \quad \phi F_{nv} : = 18000 \quad \text{psi} \]

\[ A_s \cdot \phi F_{nv} = 14130 \quad \text{lbs} \quad > \quad V_u = 1856 \quad \text{lbs} \]

**Check Breakout Strength:**  

Tension load on ductile steel element:

\[ \phi : = 0.70 \]

Distance from slab edge is 4.625":

\[ c_{al} : = 4.625 \quad \text{in} \]

Embedment depth is 11":

\[ h_{ef} : = 11 \quad \text{in} \]

\[ A_{Vc} : = (1.5 \cdot c_{al} + 1.5 \cdot c_{al}) \cdot 1.5 \cdot c_{al} = 96.258 \quad \text{in}^2 \]

\[ A_{Vco} : = 4.5 \cdot c_{al}^2 = 96.258 \quad \text{in}^2 \]

Modification factor for edge effects:

\[ \Psi_{ed\_V} : = 1.0 \quad \text{Only one edge.} \]

Modification factor for cracked concrete at service loads:

\[ \Psi_{c\_V} : = 1.0 \]

Basic concrete breakout strength in shear of a single anchor in cracked concrete:

\[ d_a : = 1 \quad \text{in} \quad 8 \cdot d_a = 8 \]

\[ l_e : = 8 \cdot d_a \quad f'_c : = 4000 \]

\[ V_b : = 7 \cdot \left( \frac{l_e}{d_a} \right)^{0.2} \cdot \sqrt{d_a} \cdot \sqrt{f'_c \cdot c_{al}}^{1.5} = 6674.422 \quad \text{lbs} \]

\[ V_{cb} : = \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot V_b = 6674.422 \quad \text{lbs} \]

\[ \phi \cdot V_{cb} = 4672.095 \quad \text{lbs} \quad > \quad V_u = 1856 \quad \text{lbs} \]
Beam to Column Connection:

\[ T = 9600 \text{ lbs} \quad C_D = 1.6 \]

Estimate Number of Bolts Needed:

Try 3/4\" diameter bolts: \[ D = \frac{3}{4} \text{ in} \]

\[ Z_{11} = 3480 \text{ lbs} \quad Z_{per} = 2000 \text{ lbs} \]

For Beam: \[ n = \frac{T}{C_D Z_{per}} = 3 \]

This is misleading. Therefore, use 4 bolts

For Column: \[ n = \frac{T}{C_D Z_{11}} = 1.724 \]

Therefore, use 2 Bolts

Determine Plate Size (try 3\" x 1/4\" thick plate, A36 Steel):

\[ F_y = 36000 \text{ psi} \]

\[ F_t = 0.6 \cdot F_y = 21600 \text{ psi} \]

\[ A_{req} = \frac{T}{F_t} = 0.444 \text{ in}^2 \]

\[ A_{net} = \left( 3 \cdot \frac{13}{16} \right) \cdot 0.25 = 0.547 \text{ in}^2 > A_{req} = 0.444 \text{ in}^2 \]
For Group Action Factor, $C_g$:

\[
A_m = 6.75 \cdot 6.875 = 46.406 \text{ in}^2
\]
\[
A_s = 2 \cdot 25 \cdot 3 = 1.5 \text{ in}^2
\]
\[\frac{A_m}{A_s} = 30.938\]

For 2 bolts:

\[C_{g2} = 1.0\]

For 4 bolts:

\[C_{g4} = 0.96\]

Geometry Factor ($C_\Delta$):

2005 NDS, Section 11.5.1 (p.76)

(a) \(\text{Edge}_{\text{min}} = 1.5 \cdot D = 1.125\text{ in}\)  
\[\text{[Table :11.5.1A]}\]

(b) \(\text{End}_{\text{min}} = 7 \cdot D = 5.25\text{ in}\)  
\[\text{[Table :11.5.1B]}\]

(c) \(\text{Spacing}_{\text{min}} = 4 \cdot D = 3\text{ in}\)  
\[\text{[Table :11.5.1C]}\]

\(\text{Spacing}_{\text{max}} = 5\text{ in}\)

All spacing requirements in (a), (b), and (c) can be met, therefore:

\[C_\Delta = 1.0\]
\[C_M = 1.0\]
\[C_t = 1.0\]

Capacity For Column:

\[Z'_{\text{column}} = Z_{11} C_D C_M C_t C_{g2} C_\Delta = 5568 \text{ lbs}\]

For 2 Bolts:

\[2 \cdot Z'_{\text{column}} = 11136 \text{ lbs} > T = 9600 \text{ lbs}\]

Capacity For Beam:

\[Z'_{\text{beam}} = Z_{\text{per}} C_D C_M C_t C_{g4} C_\Delta = 3072 \text{ lbs}\]

For 4 Bolts:

\[4 \cdot Z'_{\text{beam}} = 12288 \text{ lbs} > T = 9600 \text{ lbs}\]
Final Column-Beam Connection Design:

Column to Foundation Anchorage:

USE BOLTS AND A METAL BRACKET

For Double Studs And Metal Bracket:

\[ T = 9600 \text{ lbs} \]
Choose Bolt Size (Southern Pine):  
2005 NDS, Table 11B (p.82)

Main member size is 6.875", therefore interpolate b/t 7.5" and 5.5":

For: \( D = 1 \) in  
\[ Z_{11} = 2980 \text{ lbs} \]

\[ n = \frac{T}{(C_D Z_{11})} = 2.013 \]

Try 2 - 1" bolts, I know we should round up, but I'm using my engineering judgement to say it's close enough.

Geometry Factor \((C_\Delta)\):  
2005 NDS, Section 11.5.1 (p.76)

\[ l = 6.875 \quad \frac{1}{D} = 6.875 < 6 \]

(a) \( \text{Edge}_{\text{min}} = 1.5 \cdot D = 1.5 \) in  

[b] [Table: 11.5.1A]

(b) \( \text{End}_{\text{min}} = 7 \cdot D = 7 \) in  

[b] [Table: 11.5.1B]

(c) \( \text{Spacing}_{\text{min}} = 1.5 \cdot D = 1.5 \) in  

[b] [Table: 11.5.1C]

\[ \text{Spacing}_{\text{max}} = 5 \] in

All spacing requirements in (a), (b), and (c) can be met.

\[ C_\Delta = 1.0 \]

For Group Action Factor \((C_g)\):  
2005 NDS, Table 10.3.6C (p.63)

\[ A_m = 1.5 \cdot 10 = 15 \text{ in}^2 \]

\[ A_s = .25 \cdot 8 = 2 \text{ in}^2 \]

\[ \frac{A_m}{A_s} = 7.5 \]

And Number of Fasteners per row is 2, therefore:

\[ C_g = 0.99 \]

\[ C_M = 1.0 \]

\[ C_t = 1.0 \]

\[ Z'_{11} = Z_{11} C_D C_M C_t C_g C_\Delta = 4720.32 \text{ lbs} \]

Max Capacity For Four Bolts:

\[ \text{Max Capacity} = 4 \cdot Z'_{11} = 18881.28 \text{ lbs} \]

\[ T = 9600 \text{ lbs} \]
Foundation Anchorage (ACI 318-05 Appendix D):

\[ T : = 16000 \text{ lbs} \]

Check Steel Strength: 2005 AISC Manual, Table 7-2 (p.7-23)

Try 1" diameter bolt, A307 steel bolts used for conservative design:

\[ N_u : = T = 16000 \text{ lbs} \]

\[ \phi : = 0.75 \quad A_s : = 0.785 \text{ in} \quad \phi F_{nt} : = 33800 \text{ psi} \]

\[ A_s \cdot \phi F_{nt} = 26533 \text{ lbs} \quad > \quad N_u = 16000 \text{ lbs} \]

Check Breakout Strength: ACI 318-08, Appendix D.5.2 (p.420)

Tension load on ductile steel element:

\[ \phi : = .70 \]

Distance from slab edge is 4.625":

\[ c_{al} : = 4.625 \text{ in} \quad c_{amin} : = c_{al} \]

Embedment depth is 15":

\[ h_{ef} : = 15 \text{ in} \]

\[ A_{Ne} : = (c_{al} + h_{ef}) \left( 2 \cdot 1.5 \cdot h_{ef} \right) = 883.125 \text{ in}^2 \]

\[ A_{Neo} : = 9 \cdot h_{ef}^2 = 2025 \text{ in}^2 \]

Modification factor for edge effects:

\[ c_{amin} < 1.5 \cdot h_{ef} \]

\[ \Psi_{ed_N} : = 0.7 + 0.3 \left( \frac{c_{amin}}{1.5 \cdot h_{ef}} \right) = 0.762 \]

ACI 318-08, Appendix D.5.2.5 (p.423)
Modification factor for cracked concrete at service loads:
$$\Psi_{c_N} = 1.0$$  
ACI 318-08, Appendix D.5.2.6 (p.424)

Modification factor for post-installed anchors:
$$\Psi_{cp_N} = 1.0$$  For cast in place anchor.  
ACI 318-08, Appendix D.5.2.7 (p.425)

Basic concrete breakout strength of a single anchor in cracked concrete:

$$k_c = 24$$  (for bolts)  \quad f'_c = 4000$$

$$N_b = k_c \cdot \sqrt{f'_c \cdot h_{ef}}^{1.5} = 88181.631 \text{ lbs}$$

$$N_{cb} = \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ed_N} \cdot \Psi_{c_N} \cdot \Psi_{cp_N} \cdot N_b = 29291.407 \text{ lbs}$$

$$\phi \cdot N_{cb} = 20503.985 \text{ lbs} > N_u = 16000 \text{ lbs}$$

**Check Pullout Strength:**

$$\phi = .70$$  \quad $$\Psi_{c_P} = 1.0$$  (for cracking)  
$$A_{brg} = \frac{\pi \cdot 1.625^2}{4} = \frac{\pi \cdot 1^2}{4} = 1.289$$

$$N_p = 8 \cdot A_{brg} \cdot f'_c = 41233.404$$

$$\phi \cdot N_p = 28863.383 \text{ lb}$$

**Check Side-Face Blowout Strength:**

$$2.5c_{a1} < h_{ef}$$, therefore no blowout calculation is not needed

**Final Shear Wall Chord Anchorage Design:**