Connections:

We need to design the following connections:

Joist Hangers

Joist:	2x12 @ 16"	#2 SYP	
Downward:	430 lbf	CD=1.25	
Upward:	290 lbf	CD=1.6	

Beam-Column connection

Beam:	6¾" x 34⅞"	24F-1.8E SYP	
Column:	6¾" x 6⅞"	#47 N2M14 SYP	
Upward:	9,600 lbf	CD=1.6	

Column-Foundation connection

Column:	6¾" x 6⅞"	#47 N2M14 SYP	
Upward:	9,600 lbf	CD=1.6	16,000 lb for ACI 318 App D

Stud Connection (top and bottom)

Stud:	2x10 @ 16"	#2 SYP
Shear:	380 lbf/stud	CD=1.6

Sill Plate Anchorage

Sill Plate:	2x10 #2 SYP		
Shear parallel to grain:	770 plf	CD=1.6	1,280 plf for ACI 318 App D
Shear perpendicular to grain:	290 plf	CD=1.6	483 plf for ACI 318 App D

Diaphragm chord splice

Top Plate:	DBL 2x10	#2 SYP
Tension:	14,400 lbf	CD=1.6

Shear wall drag strut splice

Top Plate:	DBL 2x10	#2 SYP
Tension:	4,920 lbf	CD=1.6

Shear wall panel chord

Chord:	DBL 2x10	#2 SYP	
Tension:	12,300 lbf	CD=1.6	20,500 lb for ACI 318 App D

Nail Pullout

8d common nail in 5/8" thick OSB

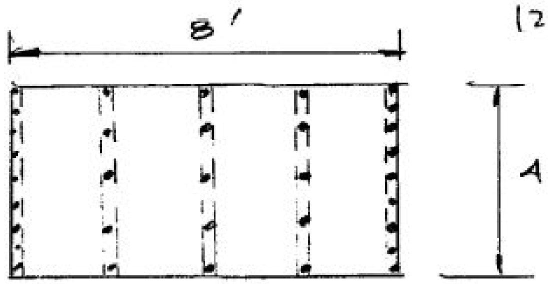
Roof:	90 psf upward	CD=1.6
Wall:	45 psf outward	CD=1.6

For a single sheet of plywood:

$$\text{Uplift} := 90\text{psf}\cdot 4\cdot \text{ft}\cdot 8\cdot \text{ft} = 2880\cdot \text{lbf}$$

NAILING SCHEME:

6" OC EDGES
12" OC INTERMEDIATE



33 NAILS PER SHEET

Therefore:

$$\text{Nail}_{\text{Pullout}} := \frac{\text{Uplift}}{33\cdot \text{nail}} = 87.273\cdot \frac{\text{lbf}}{\text{nail}}$$

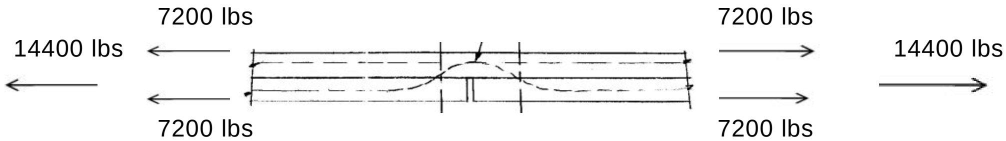
We will only cover bolts and nails:

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

Nails: Plywood Pullout
Joists
Studs

- **<u>Splice in Double Top Plate:</u>**

$C_D := 1.6$ $T := 14400 \text{ lbs}$ ***Members are 2 - 2x10



$t_m := 1.5$ in $t_s := 1.5$ in

Choose Bolt Size (Southern Pine):   2005 NDS, Table 11A (p.80)

For : $D := \frac{3}{4}$ in $Z_{11} := 800 \text{ lbs}$

Geometry Factor (C_Δ) :   2005 NDS, Section 11.5.1 (p.76)

$l := 1.5$ $\frac{l}{D} = 2 < 6$

(a) $Edge_{min} := 1.5 \cdot D = 1.125$ in   [Table: 11.5.1A]

(b) $End_{min} := 7 \cdot D = 5.25$ in   [Table: 11.5.1B]

(c) $Spacing_{min} := 1.5D = 1.125$ in   [Table: 11.5.1C]

$Spacing_{max} := 5$ in

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

$C_\Delta := 1.0$

For Group Action Factor (C_g):   2005 NDS, Table 10.3.6A (p.62)

$A_m := 1.5 \cdot 10 = 15 \text{ in}^2$

$A_s := 1.5 \cdot 10 = 15 \text{ in}^2$

$\frac{A_m}{A_s} = 1$ And Number of Fasteners per row is 3, therefore:

$C_g := 0.99$

$C_M := 1.0$

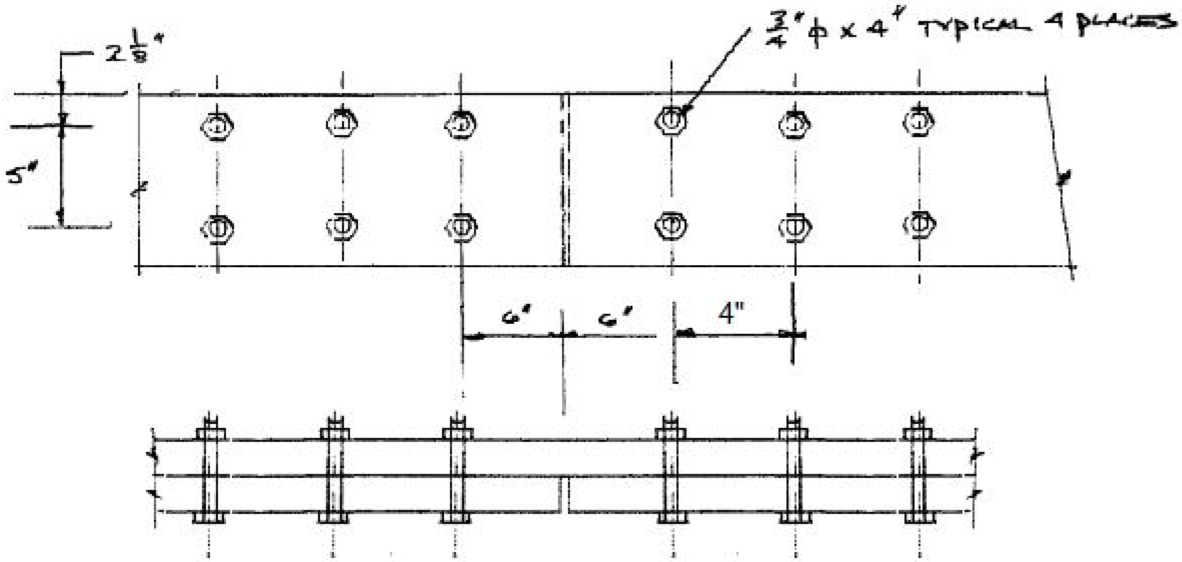
$C_t := 1.0$

$Z'_{11} := Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 1267.2 \text{ lbs}$

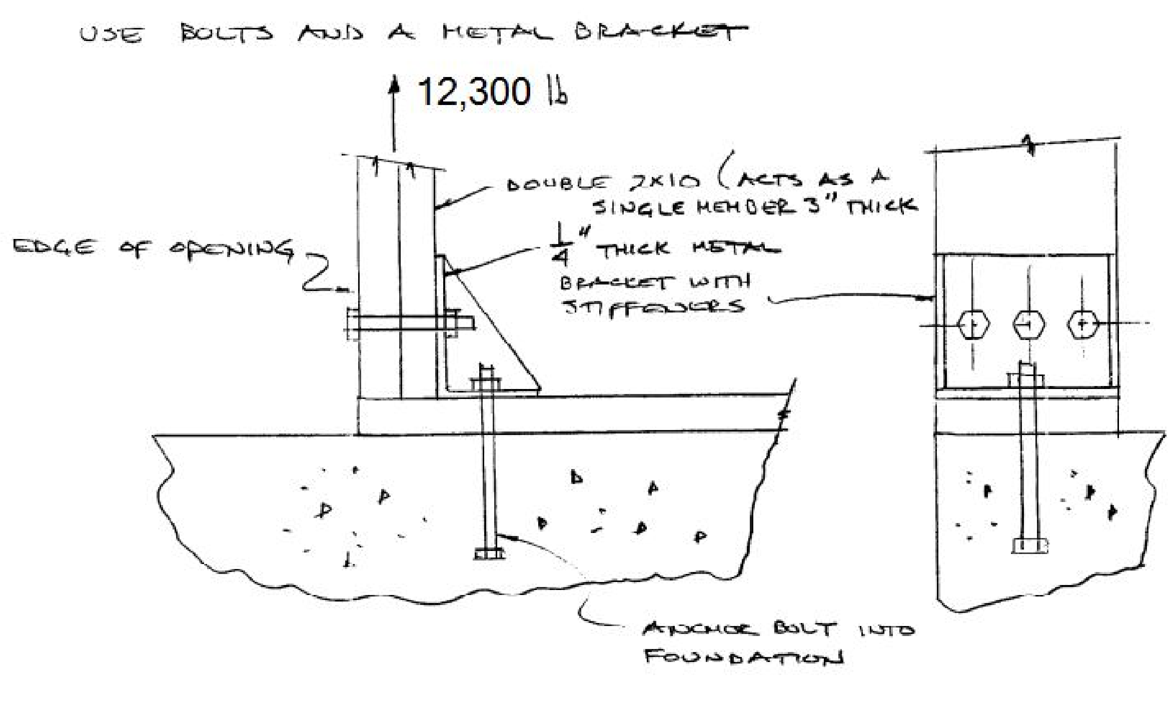
Max Capacity For Six Bolts:

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:**



For Double Studs And Metal Bracket:

$T := 12300 \text{ lbs}$ (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 : $D := 1 \text{ in}$ $Z_{11} := \frac{2480 - 1830}{2} + 1830 = 2155 \text{ lbs}$

$n := \frac{T}{(C_D \cdot Z_{11})} = 3.567$ **Try 4 - 1" bolts, 2 rows of 2.**

Geometry Factor (C_Δ) : 2005 NDS, Section 11.5.1 (p.76)

$l := 3$ $\frac{l}{D} = 3 < 6$

(a) $Edge_{min} := 1.5 \cdot D = 1.5 \text{ in}$ [Table : 11. 5.1A]

(b) $End_{min} := 7 \cdot D = 7 \text{ in}$ [Table : 11. 5.1B]

(c) $Spacing_{min} := 1.5D = 1.5 \text{ in}$ [Table : 11. 5.1C]

$Spacing_{max} := 5 \text{ in}$

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:

$C_g := 0.99$

$C_M := 1.0$

$C_t := 1.0$

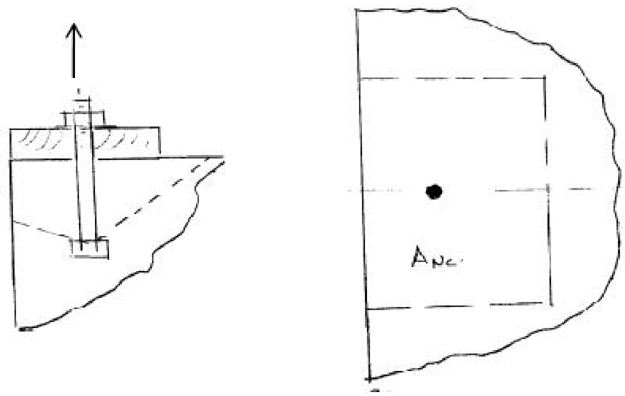
$Z'_{11} := Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 3413.52 \text{ lbs}$

Max Capacity For Four Bolts:

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:

$N_u := T = 20500 \quad \text{lbs}$

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

$A_s \cdot \phi F_{nt} = 26533 \quad \text{lbs} \qquad > \qquad N_u = 20500 \quad \text{lbs}$

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

Tension load on ductile steel element:

$\phi := .70$ ACI 318-08, Appendix D.4.4 (p.418)

Distance from slab edge is 4.625":

$c_{a1} := 4.625 \text{ in} \qquad c_{amin} := c_{a1}$

Embedment depth is 15":

$h_{ef} := 15 \text{ in}$

$A_{Nc} := (c_{a1} + h_{ef}) \cdot (2 \cdot 1.5 \cdot h_{ef}) = 883.125 \quad \text{in}^2$

$A_{Nco} := 9 \cdot h_{ef}^2 = 2025 \quad \text{in}^2$

Modification factor for edge effects: $c_{amin} < 1.5 \cdot h_{ef}$

$\Psi_{ed\_N} := 0.7 + 0.3 \cdot \left(\frac{c_{amin}}{1.5 \cdot h_{ef}} \right) = 0.762$ 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) $f'_c := 4000$

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

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b = 29291.407 \ \text{lbs}$$

$\phi \cdot N_{cb} = 20503.985$ lbs > $N_u = 20500$ lbs

Check Pullout Strength: ACI 318-08, Appendix D.5.3 (p.426)

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

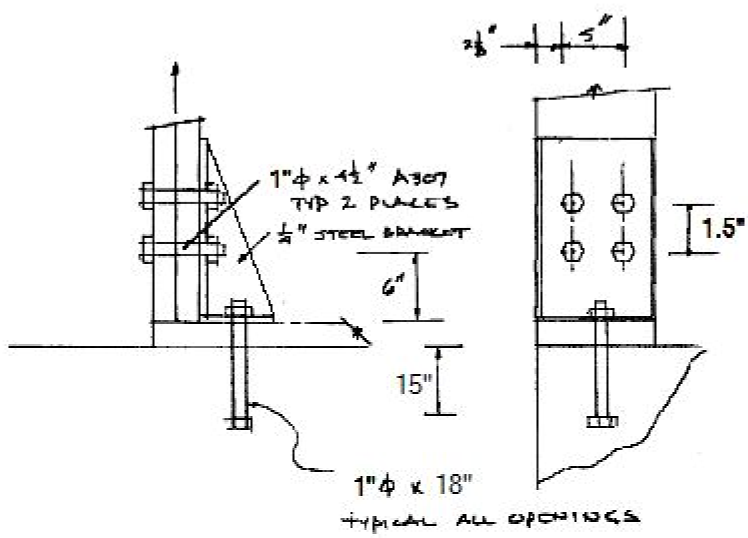
$$N_p := 8 \cdot A_{brg} \cdot f'_c = 41233.404$$

$\phi \cdot N_p = 28863.383$ lb

Check Side-Face Blowout Strength:

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

Final Shear Wall Chord Anchorage Design:



- **Bottom Sill Plate Anchorage:**

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

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

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: 2005 NDS, Table 11E (p.85)

$Z_{per} := 2250$ lbs / bolt

$C_\Delta := 1.0$

$C_g := 1.0$

$C_M := 1.0$

$C_t := 1.0$

$$Z'_{per} := Z_{per} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 3600\ \frac{\text{lbs}}{\text{bolt}}$$

Check Shear Parallel To Grain: 2005 NDS, Table 11E (p.85)

$Z_{11} := 1020\ \frac{\text{lbs}}{\text{bolt}}$

$C_\Delta := 1.0$

$C_g := 1.0$

$C_M := 1.0$

$C_t := 1.0$

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

Design Edge Distance:

$Edge_{min} := 4 \cdot D = 4\ \text{in}$ < $4.625\ \text{in}$

Design Lateral Spacing:

$$Spacing_{per} := \frac{Z'_{per}}{F_{per}} = 12.414\ \text{ft}$$ Therefore, use 12' o.c.

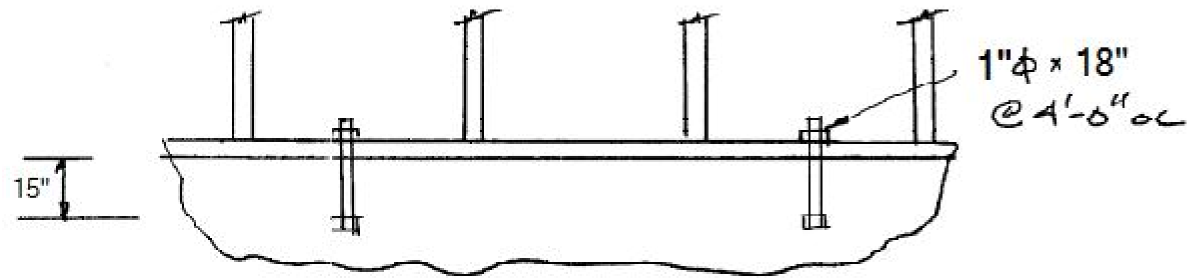
$$\text{Spacing}_{11} := \frac{Z'_{11}}{F_{11}} = 2.119 \quad \text{ft}$$

Max load in 12' section:

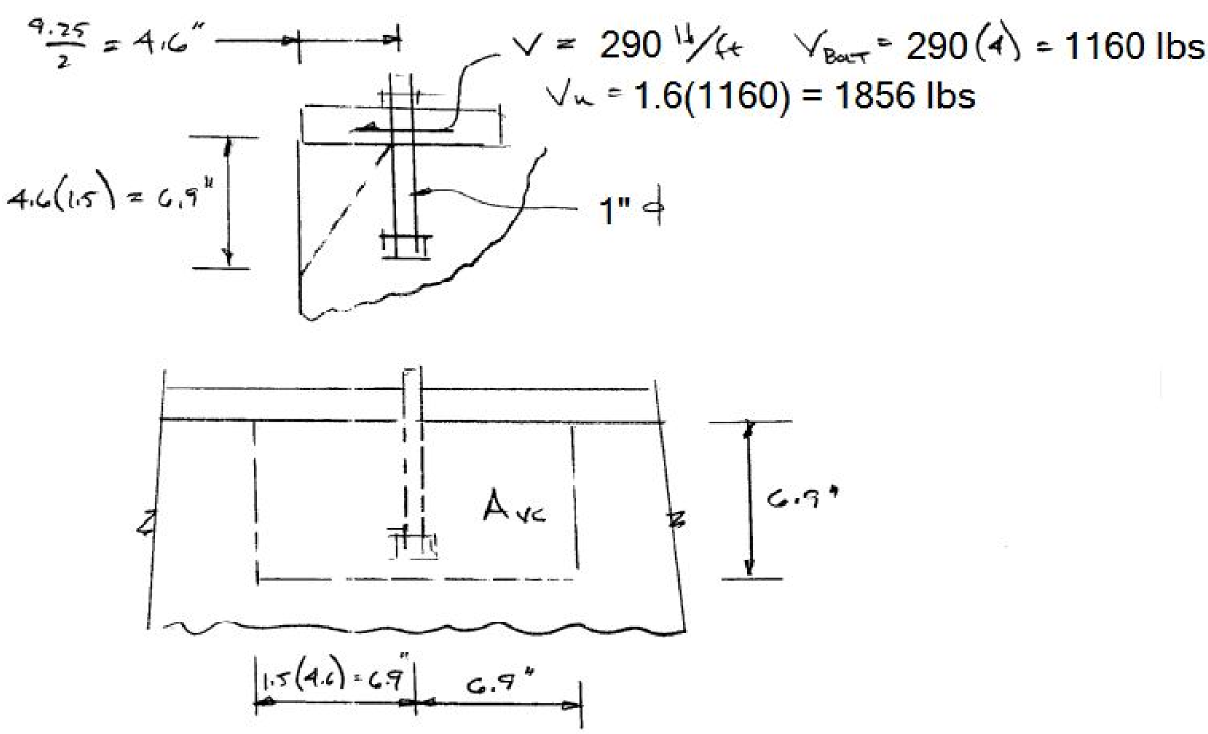
$12 \cdot F_{11} = 9240$ lbs >> $Z'_{per} = 3600 \ \frac{\text{lbs}}{\text{bolt}}$

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

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



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



Check Steel Strength: 2005 AISC Manual, Table 7-1 (p. 7-22)

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

$V_u := 1856\ lbs$

$\phi := 0.75 \qquad A_s := 0.785\ in \qquad \phi F_{nv} := 18000\ psi$

$A_s \cdot \phi F_{nv} = 14130\ lbs \qquad > \qquad V_u = 1856\ lbs$

Check Breakout Strength: ACI 318-08, Appendix D.6.2 (p.429)

Tension load on ductile steel element:

$\phi := .70$ ACI 318-08, Appendix D.4.4 (p.418)

Distance from slab edge is 4.625":

$c_{a1} := 4.625\ in$

Embedment depth is 11":

$h_{ef} := 11\ in$

$A_{Vc} := (1.5 \cdot c_{a1} + 1.5 \cdot c_{a1}) \cdot 1.5 \cdot c_{a1} = 96.258\ in^2$

$A_{Vco} := 4.5 \cdot c_{a1}^{\ 2} = 96.258\ in^2$

Modification factor for edge effects:

$\Psi_{ed\_V} := 1.0$ Only one edge. ACI 318-08, Appendix D.6.2.6 (p.433)

Modification factor for cracked concrete at service loads:

$\Psi_{c\_V} := 1.0$ ACI 318-08, Appendix D.6.2.7 (p.434)

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

$d_a := 1\ in \qquad 8 \cdot d_a = 8$

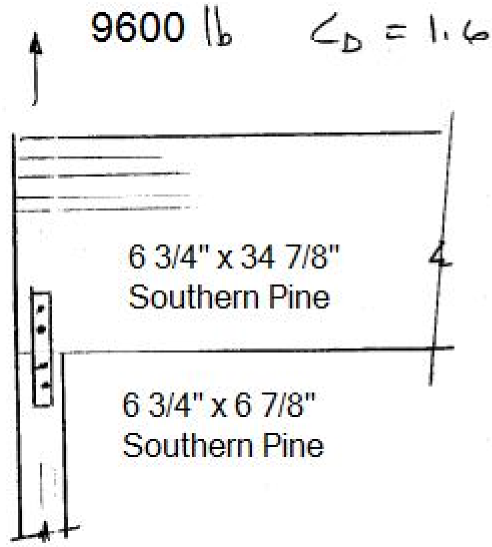
$l_e := 8 \cdot d_a \qquad f'_c := 4000$

$V_b := 7 \cdot \left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{d_a} \cdot \sqrt{f'_c} \cdot c_{a1}^{\ 1.5} = 6674.422\ lbs$ ACI 318-08, Appendix D.6.2.2 (p.431)

$V_{cb} := \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot V_b = 6674.422\ lbs$

$\phi \cdot V_{cb} = 4672.095\ lbs \qquad > \qquad V_u = 1856\ lbs$

- **Beam to Column Connection:**



$T := 9600$ lbs $\quad C_D := 1.6$

Estimate Number of Bolts Needed: 2005 NDS, Table 11I (p.90)

Try 3/4" diameter bolts: $D := \frac{3}{4}$ in

$Z_{11} := 3480$ lbs $\quad Z_{per} := 2000$ lbs

For Beam: $n := \frac{T}{C_D \cdot Z_{per}} = 3$ This is misleading. Therefore, use 4 bolts

For Column: $n := \frac{T}{C_D \cdot Z_{11}} = 1.724$ Therefore, use 2 Bolts

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

$F_y := 36000$ psi

$F_t := 0.6 \cdot F_y = 21600$ psi

$A_{req} := \frac{T}{F_t} = 0.444$ in^2

$A_{net} := \left(3 - \frac{13}{16}\right) \cdot .25 = 0.547$ in^2 $>$ $A_{req} = 0.444$ in^2

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

$A_m := 6.75 \cdot 6.875 = 46.406\ in^2$

$A_s := 2 \cdot .25 \cdot 3 = 1.5\ in^2$

$\frac{A_m}{A_s} = 30.938$

For 2 bolts: For 4 bolts:

$C_{g2} := 1.0$ $C_{g4} := 0.96$

Geometry Factor (C_Δ) : 2005 NDS, Section 11.5.1 (p.76)

(a) $Edge_{min} := 1.5 \cdot D = 1.125\ in$ [Table : 11. 5.1A]

(b) $End_{min} := 7 \cdot D = 5.25\ in$ [Table : 11. 5.1B]

(c) $Spacing_{min} := 4 \cdot D = 3\ in$ [Table : 11. 5.1C]

$Spacing_{max} := 5\ in$

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:

$Z'_{column} := Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_{g2} \cdot C_\Delta = 5568\ lbs$

For 2 Bolts:

$2 \cdot Z'_{column} = 11136\ lbs$ > $T = 9600\ lbs$

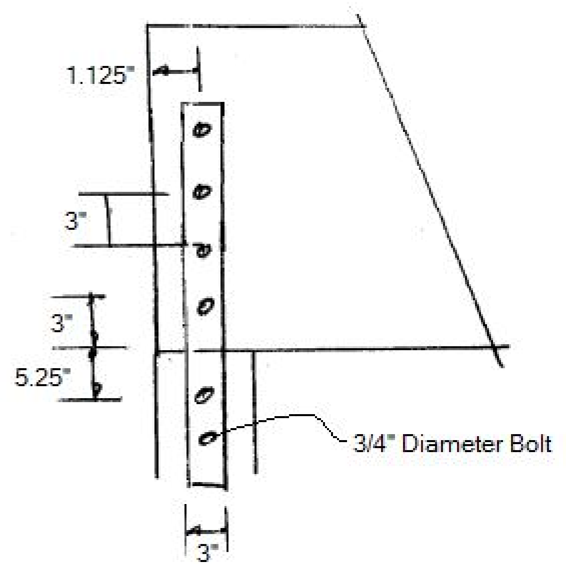
Capacity For Beam:

$Z'_{beam} := Z_{per} \cdot C_D \cdot C_M \cdot C_t \cdot C_{g4} \cdot C_\Delta = 3072\ lbs$

For 4 Bolts:

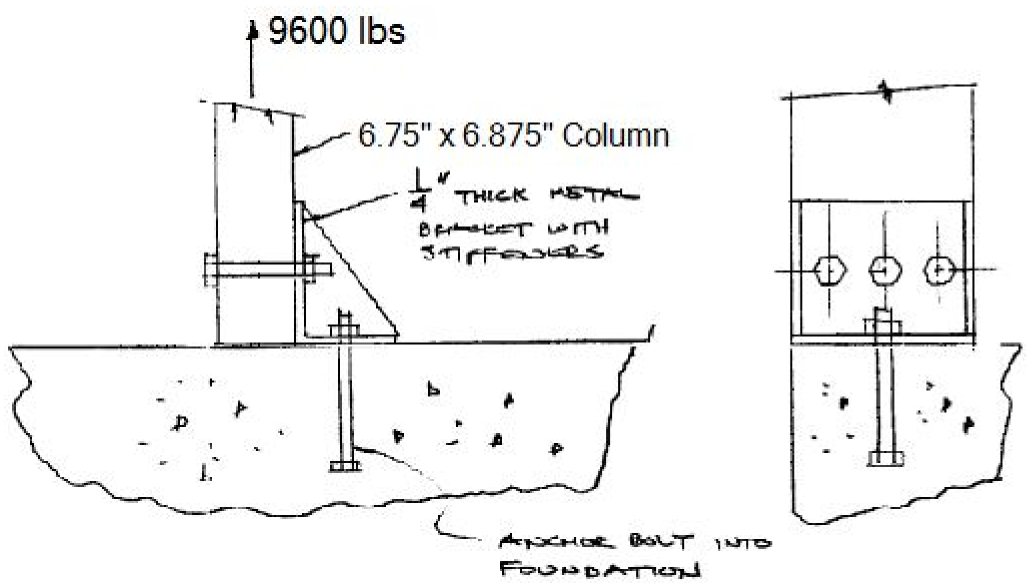
$4 \cdot Z'_{beam} = 12288\ lbs$ > $T = 9600\ lbs$

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		$Z_{11} := 2980$ lbs

$n := \frac{T}{(C_D \cdot Z_{11})} = 2.013$		**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_Δ) : 2005 NDS, Section 11.5.1 (p.76)

$l := 6.875$	$\frac{l}{D} = 6.875 <$ 6

(a) $Edge_{min} := 1.5 \cdot D = 1.5$ in		[Table : 11. 5.1A]

(b) $End_{min} := 7 \cdot D = 7$ in		[Table : 11. 5.1B]

(c) $Spacing_{min} := 1.5D = 1.5$ in		[Table : 11. 5.1C]

$Spacing_{max} := 5$ 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)

$A_m := 1.5 \cdot 10 = 15$ in^2

$A_s := .25 \cdot 8 = 2$ in^2

$\frac{A_m}{A_s} = 7.5$	And Number of Fasteners per row is 2, therefore:

$C_g := 0.99$

$C_M := 1.0$

$C_t := 1.0$

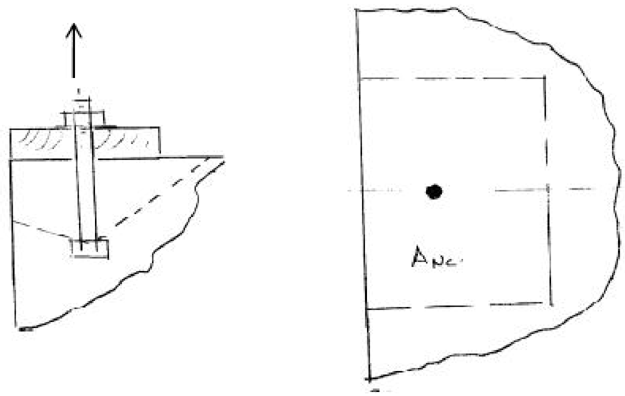
$Z'_{11} := Z_{11} \cdot C_D \cdot C_M \cdot C_t \cdot C_g \cdot C_\Delta = 4720.32$ lbs

Max Capacity For Four Bolts:

Max Capacity = $4 \cdot Z'_{11} = 18881.28$ lbs	>	$T = 9600$ 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:

$N_u := T = 16000 \quad \text{lbs}$

$\phi := 0.75 \qquad A_s := 0.785 \ \text{in} \qquad \phi F_{nt} := 33800 \ \text{psi}$

$A_s \cdot \phi F_{nt} = 26533 \ \text{lbs} \qquad > \qquad N_u = 16000 \ \text{lbs}$

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

Tension load on ductile steel element:

$\phi := .70$ ACI 318-08, Appendix D.4.4 (p.418)

Distance from slab edge is 4.625":

$c_{a1} := 4.625 \ \text{in} \qquad c_{amin} := c_{a1}$

Embedment depth is 15":

$h_{ef} := 15 \ \text{in}$

$A_{Nc} := \left(c_{a1} + h_{ef}\right) \cdot \left(2 \cdot 1.5 \cdot h_{ef}\right) = 883.125 \quad \text{in}^2$

$A_{Nco} := 9 \cdot h_{ef}^{\ 2} = 2025 \ \text{in}^2$

Modification factor for edge effects: $c_{amin} < 1.5 \cdot h_{ef}$

$\Psi_{ed\_N} := 0.7 + 0.3 \cdot \left(\dfrac{c_{amin}}{1.5 \cdot h_{ef}}\right) = 0.762$ 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) $f'_c := 4000$

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

$$N_{cb} := \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b = 29291.407 \text{ lbs}$$

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

Check Pullout Strength: ACI 318-08, Appendix D.5.3 (p.426)

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

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

$\phi \cdot N_p = 28863.383$ lb

Check Side-Face Blowout Strength:

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

Final Shear Wall Chord Anchorage Design: