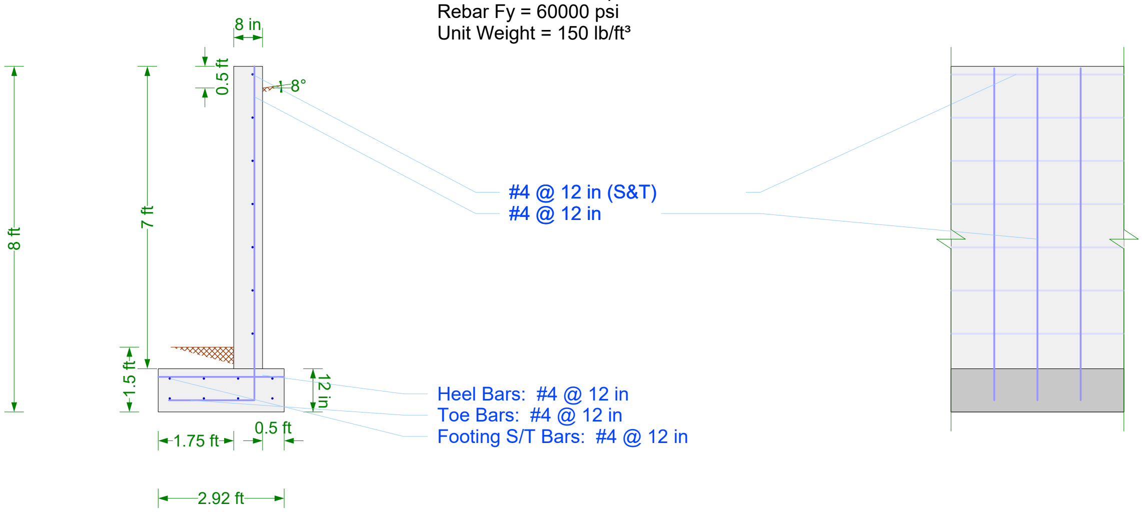
JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 6' Tall

Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



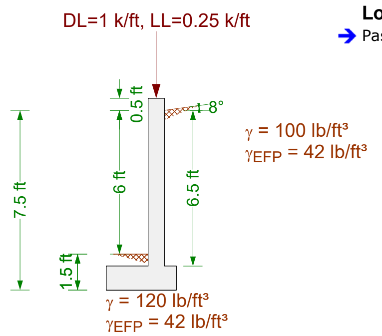
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.882	Overturning	1.70	1.50	1.0D + 1.0L + 1.0H
✓ 0.807	Bearing Pressure	2500 psf	2017 psf	1.0D + 1.0L + 1.0H
✓ 0.361	Bearing Eccentricity	6.32 in	17.5 in	1.0D + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.215	Shear	8.63 k/ft	1.85 k/ft	1.4D
✓ 0.342	Moment	7.7 ft·k/ft	2.63 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.074	Min Strain	0.0539	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.059	Shear	9.61 k/ft	0.56 k/ft	1.4D
✓ 0.014	Moment	8.6 ft·k/ft	0.12 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.066	Min Strain	0.0604	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.444	Development	27 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.615	Moment	5 ft·k/ft	3.08 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.250	Shear	5.67 k/ft	1.42 k/ft	1.2D + 1.6L + 1.6H
✓ 0.116	Max Steel	0.0344	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²/in	0 in²/in	1.2D + 1.6L + 1.6H
✓ 0.667	Base Development	9 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2018
Concrete Load Combs	IBC 2018 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)
1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H
1.2D + 1.6H
1.2D + 0.9H
0.9D + 1.6H
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	3.08	5	1.42	5.67	6	9	0.0040	0.0344	0
1.2D + 1.6L + 0.9H	1.73	5	0.8	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 1.6H	3.08	5	1.42	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 0.9H	1.73	5	0.8	5.67	6	9	0.0040	0.0344	0
1.2D + 1.6H	3.08	5	1.42	5.67	6	9	0.0040	0.0344	0
1.2D + 0.9H	1.73	5	0.8	5.67	6	9	0.0040	0.0344	0
0.9D + 1.6H	3.08	5	1.42	5.67	6	9	0.0040	0.0344	0
0.9D + 0.9H	1.73	5	0.8	5.67	6	9	0.0040	0.0344	0
1.4D	0	0	0	0	6	9	0.0040	0.0344	0

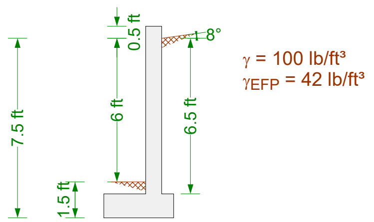
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.02	0.12	8.6	0.48	9.61	2.63	7.7	1.83	8.63
1.2D + 1.6L + 0.9H	0.02	0.12	8.6	0.48	9.61	2.63	7.7	1.83	8.63
1.2D + 0.5L + 1.6H	0.02	0.12	8.6	0.48	9.61	2.4	7.7	1.66	8.63
1.2D + 0.5L + 0.9H	0.02	0.12	8.6	0.48	9.61	2.4	7.7	1.66	8.63
1.2D + 1.6H	0.02	0.12	8.6	0.48	9.61	2.29	7.7	1.59	8.63
1.2D + 0.9H	0.02	0.12	8.6	0.48	9.61	2.29	7.7	1.59	8.63
0.9D + 1.6H	0.02	0.09	8.6	0.36	9.61	1.72	7.7	1.19	8.63
0.9D + 0.9H	0.02	0.09	8.6	0.36	9.61	1.72	7.7	1.19	8.63
1.4D	0.02	0.14	8.6	0.56	9.61	2.67	7.7	1.85	8.63

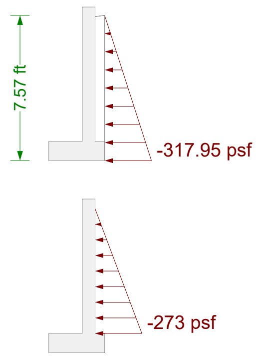
Stability Check Results Summary

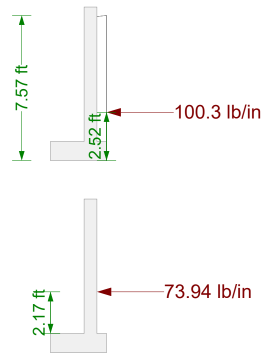
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H	3.04	5.17	1.701	1.500	1.200	100.3	86.17	0.859
1.0D + 1.0H	3.04	5.17	1.701	1.500	1.200	100.3	78.87	0.786

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H	1.500	1.200	2017	2500	6.32	17.5	0.36
1.0D + 1.0H	1.500	1.200	1838	2500	6.32	17.5	0.36

Backfill Pressure







Lateral Earth Pressure

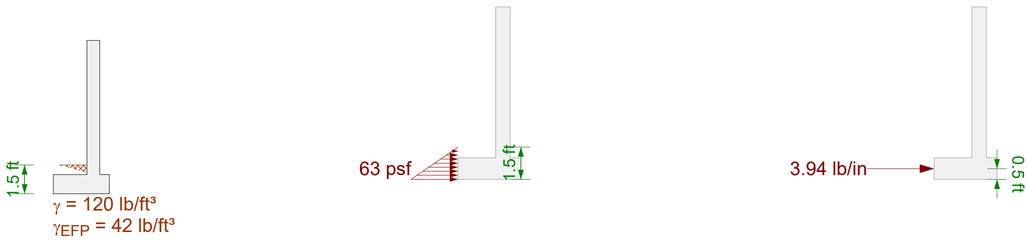
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (7.57\text{ ft})(42\text{ lb / ft}^3) = 318\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (6.5\text{ ft})(42\text{ lb / ft}^3) = 273\text{ psf}$

Passive Pressure



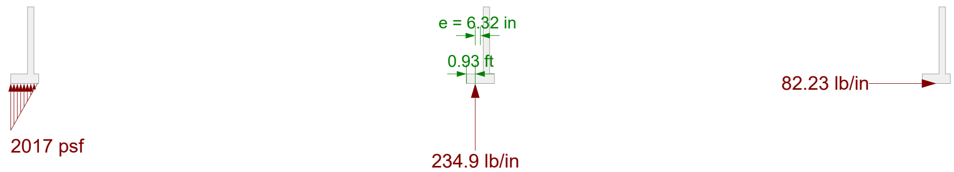
Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ R = (0.350)(234.9 lb / in) = 82.23 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-100.29 lb/in	2.52 ft	36443 in·lb/ft
Axial Dead Load	-83.33 lb/in	2.08 ft	0 lb/in	-	-25000 in·lb/ft
Axial Live Load	-20.83 lb/in	2.08 ft	0 lb/in	-	-6250 in·lb/ft
Footing Weight	-36.46 lb/in	1.46 ft	0 lb/in	-	-7656.25 in·lb/ft
Stem Weight	-58.33 lb/in	2.08 ft	0 lb/in	-	-17500 in·lb/ft
Backfill Weight	-27.08 lb/in	2.67 ft	0 lb/in	-	-10400 in·lb/ft
Backfill Weight	-0.15 lb/in	2.75 ft	0 lb/in	-	-57.97 in·lb/ft
Soil over toe Weight	-8.75 lb/in	0.88 ft	0 lb/in	-	-1102.5 in·lb/ft
	-234.94 lb/in				-31523.78 in·lb/ft

$$\frac{-\,31523.78\ \text{in·lb / ft}}{-\,234.94\ \text{lb / in}} = 0.93\ \text{ft}$$

Stability Checks [1.0D + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	100.3 lb/in	2.52 ft	36443 in·lb/ft
		Total:	36443 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	2.08 ft	25000 in·lb/ft
Footing Weight	-36.46 lb/in	1.46 ft	7656 in·lb/ft
Stem Weight	-58.33 lb/in	2.08 ft	17500 in·lb/ft
Backfill Weight	-27.08 lb/in	2.67 ft	10400 in·lb/ft
Backfill Weight	-0.15 lb/in	2.75 ft	57.97 in·lb/ft
Soil over toe Weight	-8.75 lb/in	0.88 ft	1103 in·lb/ft
		Total:	62000 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{62000\ in\cdot lb/ft}{36443\ in\cdot lb/ft} = 1.701 > 1.50\ (OK)$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

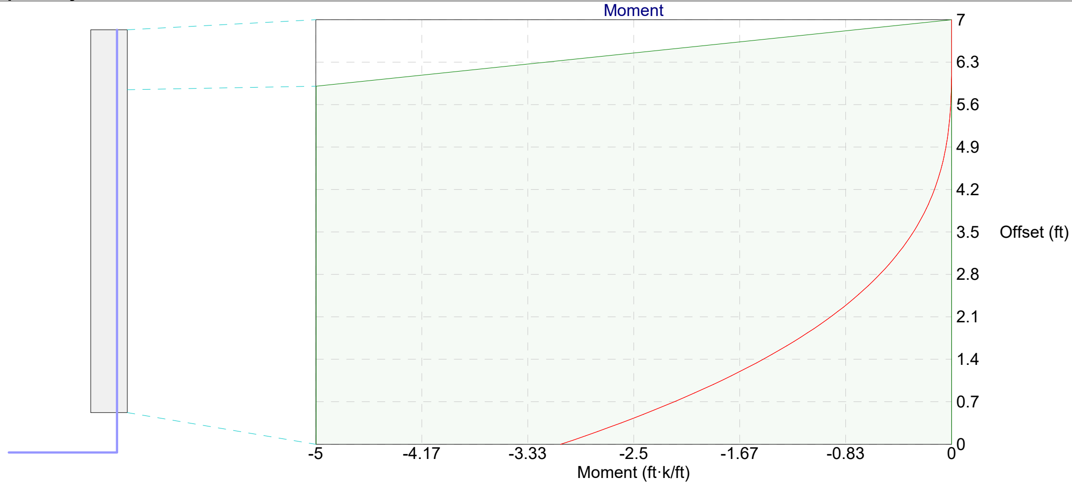
Bearing pressure < allowable (2017 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (6.32 in < 17.5 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.019 in
Deflection due to rotation from settlement	0.336 in
Total deflection at top of wall (positive towards toe)	0.355 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(5.75 \text{ in}) - (0.39 \text{ in}) / 2] = 5 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 5.9 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(5.75 \text{ in}) - (0.39 \text{ in}) / 2] = 5 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 7 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(5.75 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Stem Shear Capacity

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000 \text{ psi}}\,(5.75 \text{ in}) \;=\; 7.56 \text{ k / ft}$

$\phi V_n = \phi\, V_c \;=\; (0.750)\,(7.56 \text{ k / ft}) \;=\; 5.67 \text{ k / ft}$

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 7 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000 \text{ psi}}\,(5.75 \text{ in}) \;=\; 7.56 \text{ k / ft}$

$\phi V_n = \phi\, V_c \;=\; (0.750)\,(7.56 \text{ k / ft}) \;=\; 5.67 \text{ k / ft}$

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50(1.0)\sqrt{3000\ psi}}\right](0.5\ in) = 7.67\ in$$

$8\ d_b = 8(0.5\ in) = 4.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

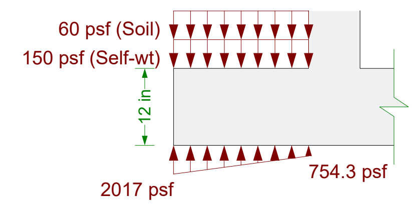
K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\ in) + (0.0)}{(0.5\ in)} = 4.50$$

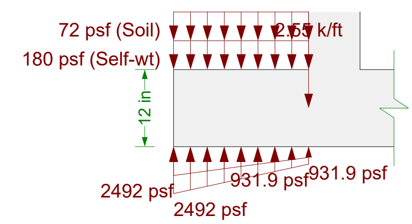
$$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\frac{(60000\ psi)}{(1.0)\sqrt{3000\ psi}}\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5\ in) = 13.15\ in$$

Toe Checks [1.2D + 1.6L + 1.6H]

Toe Unfactored Loads



Toe Factored Loads



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 2.63 ft·k / ft < M_{stem} = 3.08 ft·k / ft

M_u = 2.63 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(8.75 \text{ in}) - (0.39 \text{ in})/2] = 7.7 \text{ ft·k/ft}$$

ϕM_n = 7.7 ft·k / ft ≥ M_u = 2.63 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}\,(8.75 \text{ in}) = 11.5 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.5 \text{ k/ft}) = 8.63 \text{ k/ft}$$

ϕV_n = 8.63 k / ft ≥ V_u = 1.83 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(8.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0539$$

ε_t = 0.0539 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 7.7 ft·k / ft ≥ $(4/3) M_u$ = [4 / 3] (2.63 ft·k / ft) = 3.51 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(2.63 \text{ ft·k/ft})}{(7.7 \text{ ft·k/ft})} = 0.3421$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r}{50\, \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000 \text{ psi}}}\right](0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.3421) per 25.4.10: l_{dh} = 2.62 in

$8\, d_b$ = 8 (0.5 in) = 4.0 (minimum limit, does not control)

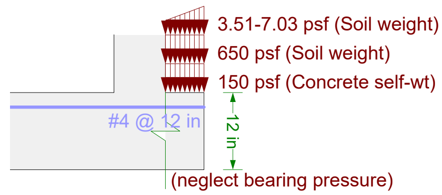
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

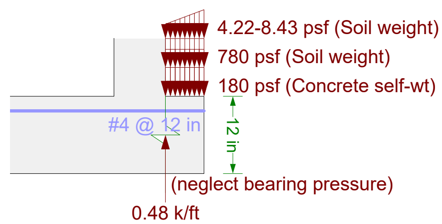
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.12 ft·k / ft < M_{stem} = 3.08 ft·k / ft

M_u = 0.12 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in})/2] = 8.6 \text{ ft·k/ft}$

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.12 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\,(9.75 \text{ in}) = 12.82 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k/ft}) = 9.61 \text{ k/ft}$

ϕV_n = 9.61 k / ft ≥ V_u = 0.48 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0604$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.12 ft·k / ft) = 0.16 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0.12 \text{ ft·k/ft})}{(8.6 \text{ ft·k/ft})} = 0.0141$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right] (0.5 \text{ in}) = 13.15 \text{ in}$

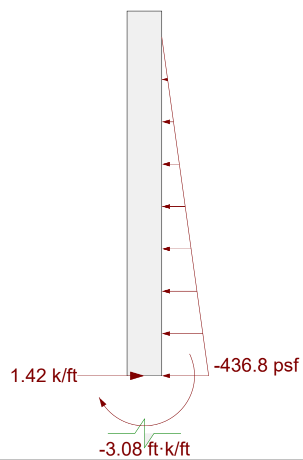
Factoring l_d by the excess reinforcement ratio (0.0141) per 25.4.10: l_d = 0.18 in

12 inch minimum controls

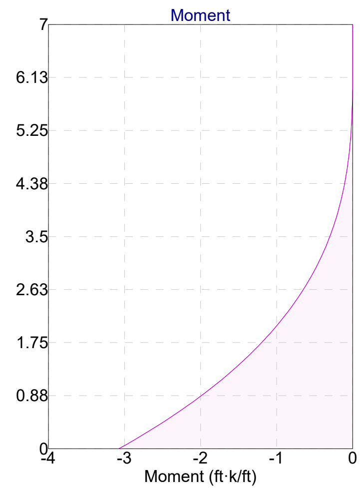
l_{d_prov} = 27 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

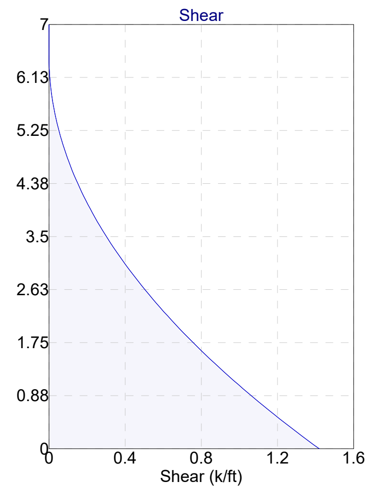
Stem Internal Forces



Stem Internal Forces



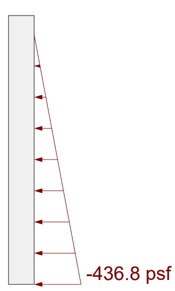
Stem Internal Forces



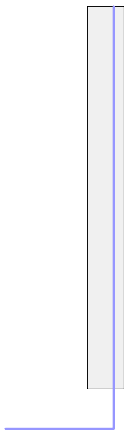
Stem Joint Force Transfer

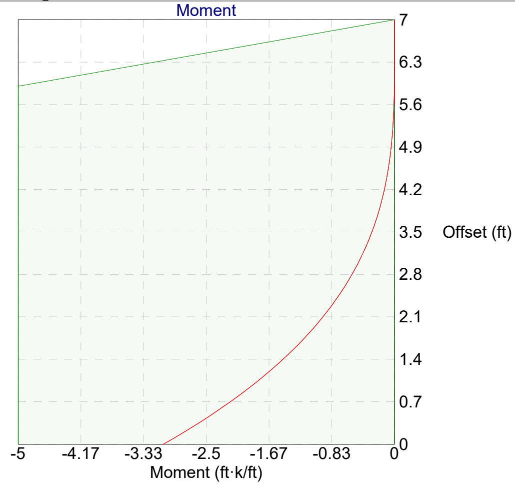
Location	Force
@ stem base	1.42 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]





Check (ACI 318-14 11.5.5.1b)   @ 0 ft from base

ϕM_n = 5 ft·k / ft ≥ M_u = 3.08 ft·k / ft ✓

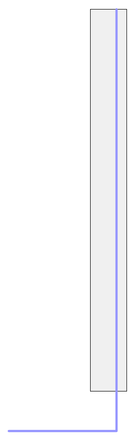
Check (ACI 318-14 11.5.5.1b)   @ 5.9 ft from base

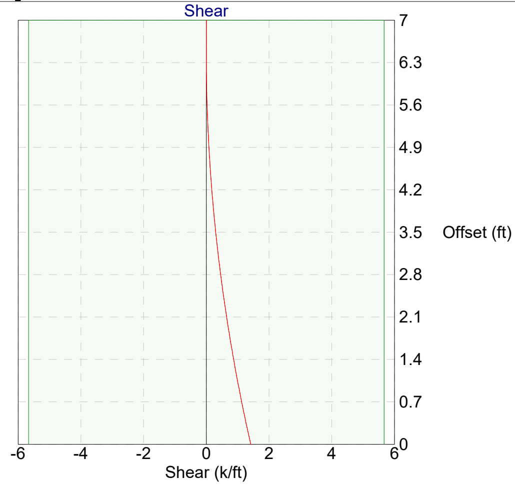
ϕM_n = 5 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b)   @ 5.94 ft from base

ϕM_n = 4.84 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.67 k / ft ≥ V_u = 1.42 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 5 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (3.08 ft·k / ft) = 4.1 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 7 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02\ in^2 / in)(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.75\ in)}{(0.39\ in) / (0.850)} - 1\right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02\ in^2 / in)(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.75\ in)}{(0.39\ in) / (0.850)} - 1\right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2\ in^2) / (12\ in)}{(8\ in)} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(3.08\ ft \cdot k / ft)}{(5\ ft \cdot k / ft)} = 0.6153$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r}{50\,\lambda\,\sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000\ psi}}\right](0.5\ in) = 7.67\ in$$

Factoring l_{dh} by the excess reinforcement ratio (0.6153) per 25.4.10: l_{dh} = 4.72 in

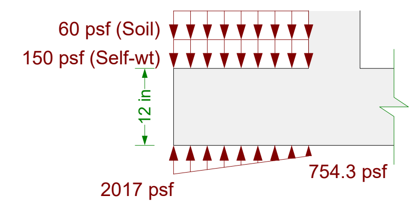
8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

6 inch minimum controls

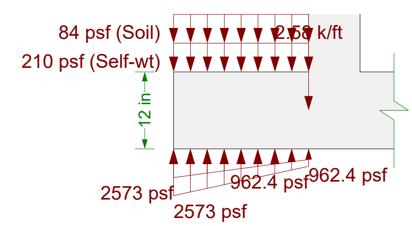
l_{dh_prov} = 9 in ≥ l_{dh} = 6 in ✓

Toe Checks [1.4D]

Toe Unfactored Loads



Toe Factored Loads



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 2.67 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.75 \text{ in}) - (0.39 \text{ in}) / 2] = 7.7 \text{ ft·k / ft}$$

ϕM_n = 7.7 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (8.75 \text{ in}) = 11.5 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.5 \text{ k / ft}) = 8.63 \text{ k / ft}$$

ϕV_n = 8.63 k / ft ≥ V_u = 1.85 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(8.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1\right] = 0.0539$$

ε_t = 0.0539 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 7.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(7.7 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right] (0.5 \text{ in}) = 7.67 \text{ in}$$

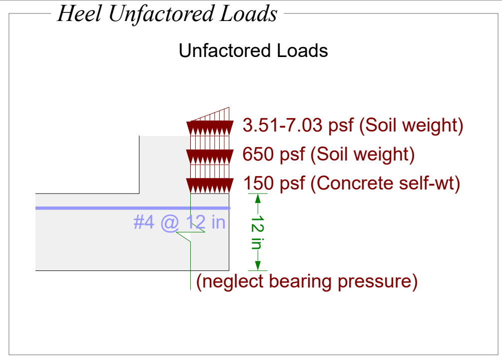
Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

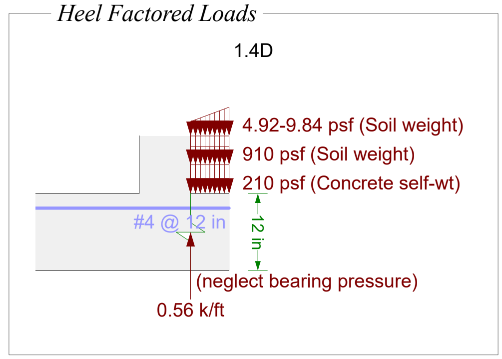
8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]





Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.14 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)} = 0.39\ in$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02\ in^2/in)(60000\ psi)[(9.75\ in) - (0.39\ in)/2] = 8.6\ ft·k/ft$$

ϕM_n = 8.6 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000\ psi}\,(9.75\ in) = 12.82\ k/ft$$

$$\phi V_n = \phi V_c = (0.750)(12.82\ k/ft) = 9.61\ k/ft$$

ϕV_n = 9.61 k / ft ≥ V_u = 0.56 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)} = 0.39\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.75\ in)}{(0.39\ in)/(0.850)} - 1\right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4\ in^2/in)}{(12\ in)(12\ in)} = 0.0028$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4\ in^2/in)}{(12\ in)(12\ in)} = 0.0028$$

$$\frac{0.0018\ (60000)}{f_y} = \frac{0.0018\ (60000)}{(60000\ psi)} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(-0\ ft·k/ft)}{(8.6\ ft·k/ft)} = -0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\ in) + (0.0)}{(0.5\ in)} = 4.50$$

$$l_d = \left(\frac{3.}{40}\ \frac{f_y}{\lambda\sqrt{F'_c}}\ \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\ \frac{(60000\ psi)}{(1.0)\sqrt{3000\ psi}}\ \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5\ in) = 13.15\ in$$

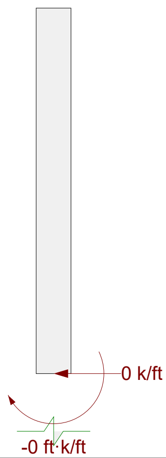
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

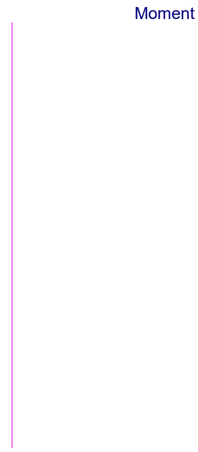
l_{d_prov} = 27 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

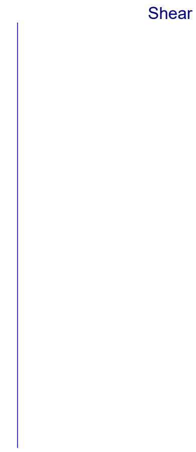
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

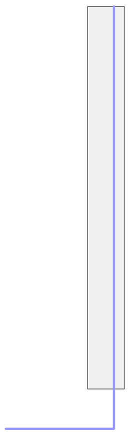


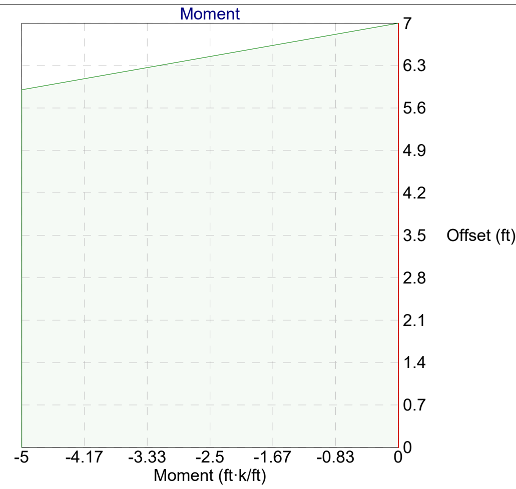
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

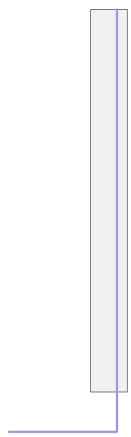
Stem Internal Forces

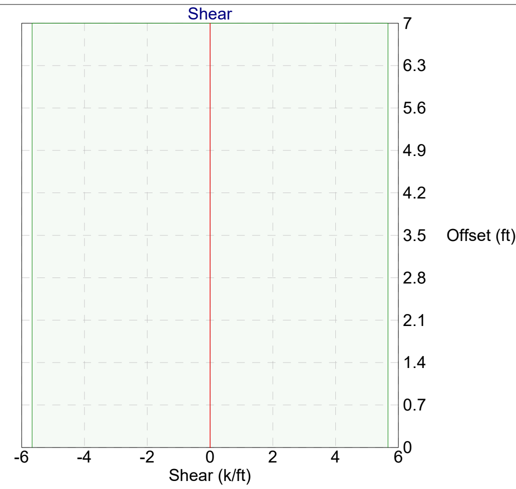
Stem Moment Checks [1.4D]





Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 5 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 7 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1\right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1\right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: l_{dh} = 0 in

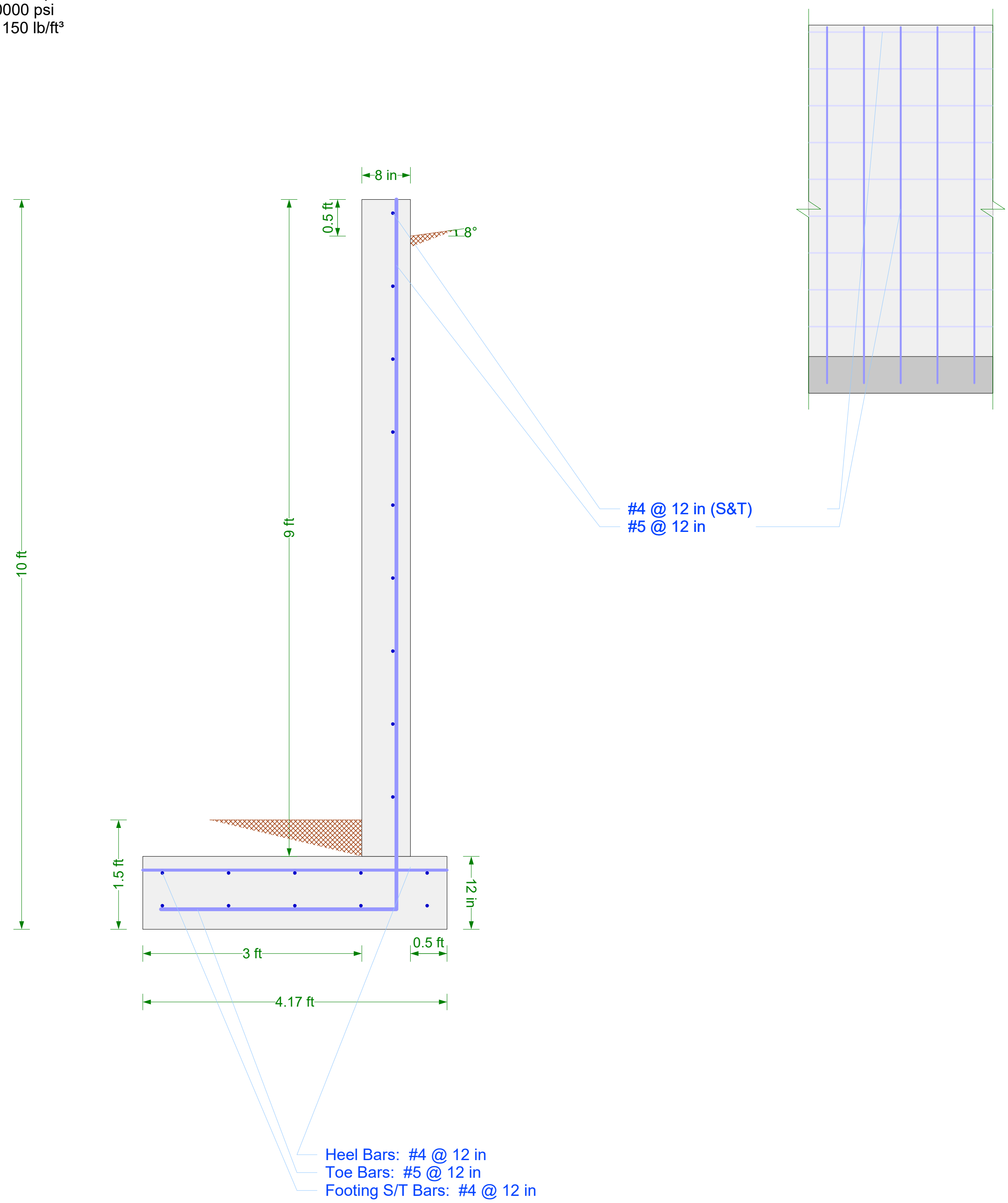
8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 9 in ≥ l_{dh} = 6 in ✓

Design Detail

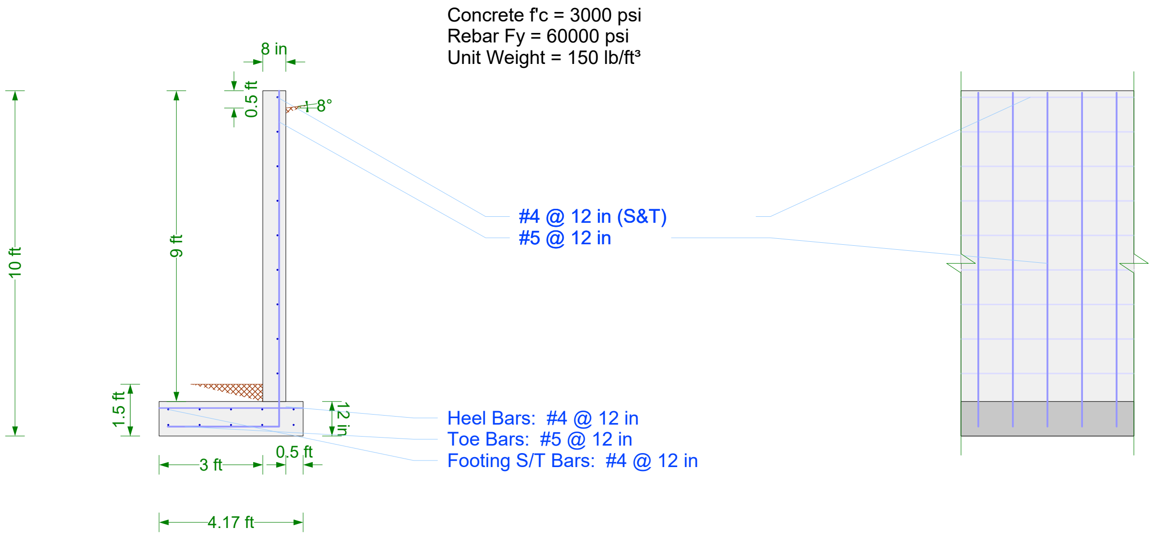
Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Design Detail



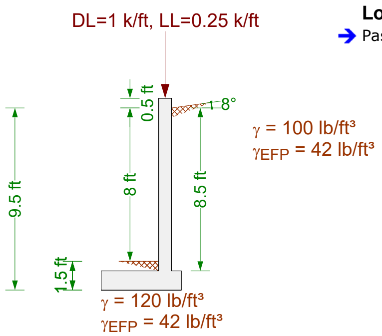
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.959	Overturning	1.56	1.50	1.0D + 1.0L + 1.0H
✓ 0.713	Bearing Pressure	2500 psf	1784 psf	1.0D + 1.0L + 1.0H
✓ 0.393	Bearing Eccentricity	9.83 in	25 in	1.0D + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.352	Shear	8.57 k/ft	3.01 k/ft	1.4D
✓ 0.524	Moment	11.7 ft·k/ft	6.13 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.120	Min Strain	0.0334	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.03 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.073	Shear	9.61 k/ft	0.7 k/ft	1.4D
✓ 0.018	Moment	8.6 ft·k/ft	0.15 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.066	Min Strain	0.0604	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.286	Development	42 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.916	Moment	7.51 ft·k/ft	6.88 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.433	Shear	5.61 k/ft	2.43 k/ft	1.2D + 1.6L + 1.6H
✓ 0.192	Max Steel	0.0209	0.0040	1.2D + 1.6L + 1.6H
✓ 0.734	Min Steel	0.03 in²/in	0.02 in²/in	1.2D + 1.6L + 1.6H
✓ 0.975	Base Development	9 in	8.78 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2018
Concrete Load Combs	IBC 2018 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H
1.2D + 1.6H
1.2D + 0.9H
0.9D + 1.6H
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	6.88	7.51	2.43	5.61	8.78	9	0.0040	0.0209	0.02
1.2D + 1.6L + 0.9H	3.87	7.51	1.37	5.61	6	9	0.0040	0.0209	0
1.2D + 0.5L + 1.6H	6.88	7.51	2.43	5.61	8.78	9	0.0040	0.0209	0.02
1.2D + 0.5L + 0.9H	3.87	7.51	1.37	5.61	6	9	0.0040	0.0209	0
1.2D + 1.6H	6.88	7.51	2.43	5.61	8.78	9	0.0040	0.0209	0.02
1.2D + 0.9H	3.87	7.51	1.37	5.61	6	9	0.0040	0.0209	0
0.9D + 1.6H	6.88	7.51	2.43	5.61	8.78	9	0.0040	0.0209	0.02
0.9D + 0.9H	3.87	7.51	1.37	5.61	6	9	0.0040	0.0209	0
1.4D	0	0	0	0	6	9	0.0040	0.0209	0

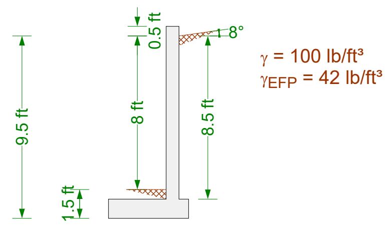
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.03	0.15	8.6	0.6	9.61	6.13	11.7	2.92	8.57
1.2D + 1.6L + 0.9H	0.03	0.15	8.6	0.6	9.61	6.13	11.7	2.92	8.57
1.2D + 0.5L + 1.6H	0.03	0.15	8.6	0.6	9.61	5.65	11.7	2.69	8.57
1.2D + 0.5L + 0.9H	0.03	0.15	8.6	0.6	9.61	5.65	11.7	2.69	8.57
1.2D + 1.6H	0.03	0.15	8.6	0.6	9.61	5.43	11.7	2.58	8.57
1.2D + 0.9H	0.03	0.15	8.6	0.6	9.61	5.43	11.7	2.58	8.57
0.9D + 1.6H	0.03	0.11	8.6	0.45	9.61	4.07	11.7	1.94	8.57
0.9D + 0.9H	0.03	0.11	8.6	0.45	9.61	4.07	11.7	1.94	8.57
1.4D	0.03	0.18	8.6	0.7	9.61	6.34	11.7	3.01	8.57

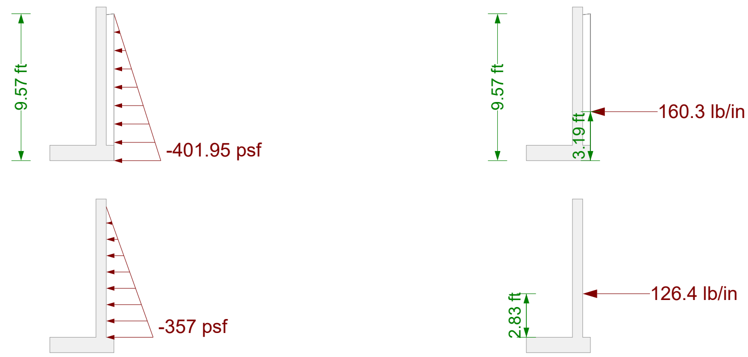
Stability Check Results Summary

Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H	6.14	9.6	1.565	1.500	1.200	160.3	102.6	0.640
1.0D + 1.0H	6.14	9.6	1.565	1.500	1.200	160.3	95.28	0.594

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H	1.500	1.200	1784	2500	9.83	25	0.34
1.0D + 1.0H	1.500	1.200	1652	2500	9.83	25	0.34

Backfill Pressure





Lateral Earth Pressure

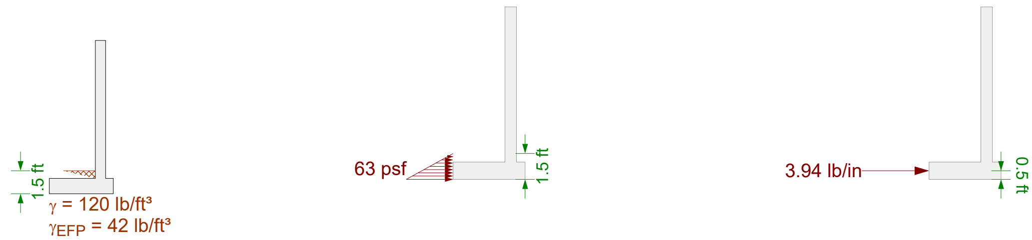
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (9.57\text{ ft})(42\text{ lb / ft}^3) = 402\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (8.5\text{ ft})(42\text{ lb / ft}^3) = 357\text{ psf}$

Passive Pressure

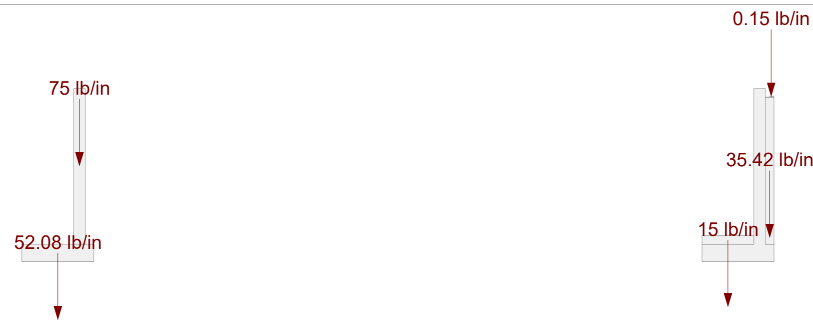


Lateral Earth Pressure

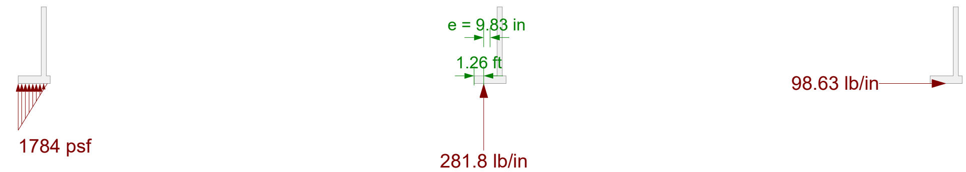
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Wall/Soil Weights



Bearing Pressure



Friction

F = μ R = (0.350)(281.8 lb / in) = 98.63 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-160.28 lb/in	3.19 ft	73630 in·lb/ft
Axial Dead Load	-83.33 lb/in	3.33 ft	0 lb/in	-	-40000 in·lb/ft
Axial Live Load	-20.83 lb/in	3.33 ft	0 lb/in	-	-10000 in·lb/ft
Footing Weight	-52.08 lb/in	2.08 ft	0 lb/in	-	-15625 in·lb/ft
Stem Weight	-75 lb/in	3.33 ft	0 lb/in	-	-36000 in·lb/ft
Backfill Weight	-35.42 lb/in	3.92 ft	0 lb/in	-	-19975 in·lb/ft
Backfill Weight	-0.15 lb/in	4 ft	0 lb/in	-	-84.32 in·lb/ft
Soil over toe Weight	-15 lb/in	1.5 ft	0 lb/in	-	-3240 in·lb/ft
	-281.81 lb/in				-51294.81 in·lb/ft

$$\frac{-51294.81 \text{ in·lb / ft}}{-281.81 \text{ lb / in}} = 1.26 \text{ ft}$$

Stability Checks [1.0D + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	160.3 lb/in	3.19 ft	73630 in·lb/ft
		Total:	73630 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	3.33 ft	40000 in·lb/ft
Footing Weight	-52.08 lb/in	2.08 ft	15625 in·lb/ft
Stem Weight	-75 lb/in	3.33 ft	36000 in·lb/ft
Backfill Weight	-35.42 lb/in	3.92 ft	19975 in·lb/ft
Backfill Weight	-0.15 lb/in	4 ft	84.32 in·lb/ft
Soil over toe Weight	-15 lb/in	1.5 ft	3240 in·lb/ft
		Total:	115208 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{115208 \text{ in·lb / ft}}{73630 \text{ in·lb / ft}} = 1.565 > 1.50 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

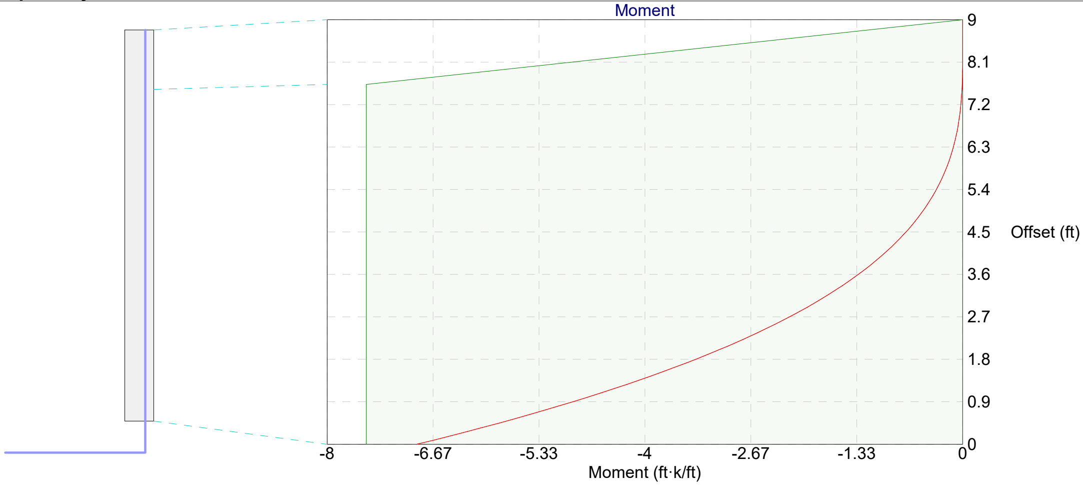
Bearing pressure < allowable (1784 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (9.83 in < 25 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.072 in
Deflection due to rotation from settlement	0.268 in
Total deflection at top of wall (positive towards toe)	0.34 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0.61 \text{ in}) / 2] = 7.51 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 7.63 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0.61 \text{ in}) / 2] = 7.51 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 9 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Stem Shear Capacity

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(5.69 \text{ in}) = 7.48$ k / ft

$\phi V_n = \phi V_c = (0.750)\,(7.48 \text{ k / ft}) = 5.61$ k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 9 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(5.69 \text{ in}) = 7.48$ k / ft

$\phi V_n = \phi V_c = (0.750)\,(7.48 \text{ k / ft}) = 5.61$ k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \, \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

$8\,d_b = 8\,(0.63 \text{ in}) = 5.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.63 in) / 2 = 2.31 in

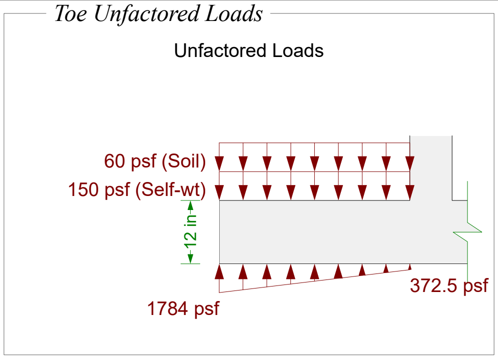
c_b = 2.31 in (lesser of half spacing, ctr to surface)

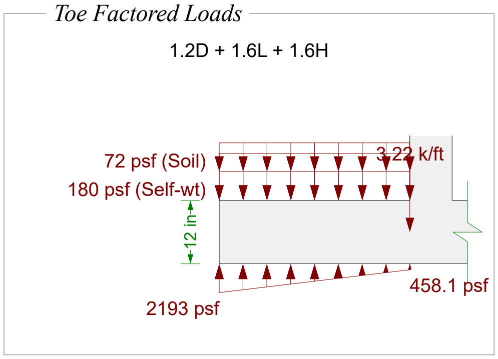
K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.63 \text{ in}) = 16.43 \text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]





Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 6.13 ft·k / ft < M_{stem} = 6.88 ft·k / ft

M_u = 6.13 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)}$ = 0.61 in

$\phi M_n = \phi A_s f_y (d - a/2)$ = (0.90)(0.03 in² / in)(60000 psi)[(8.69 in) - (0.61 in) / 2] = 11.7 ft·k / ft

ϕM_n = 11.7 ft·k / ft ≥ M_u = 6.13 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000\ psi}\,(8.69\ in)$ = 11.42 k / ft

$\phi V_n = \phi V_c$ = (0.750)(11.42 k / ft) = 8.57 k / ft

ϕV_n = 8.57 k / ft ≥ V_u = 2.92 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)}$ = 0.61 in

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(8.69\ in)}{(0.61\ in)/(0.850)} - 1\right]$ = 0.0334

ε_t = 0.0334 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 11.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](6.13 ft·k / ft) = 8.18 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4\ in^2/in)}{(12\ in)(12\ in)}$ = 0.0028

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4\ in^2/in)}{(12\ in)(12\ in)}$ = 0.0028

$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000\ psi)}$ = 0.0018

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(6.13\ ft \cdot k/ft)}{(11.7\ ft \cdot k/ft)}$ = 0.5244 (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50\ (1.0)\sqrt{3000\ psi}}\right](0.63\ in)$ = 9.59 in

Factoring l_{dh} by the excess reinforcement ratio (0.5244) per 25.4.10: l_{dh} = 5.03 in

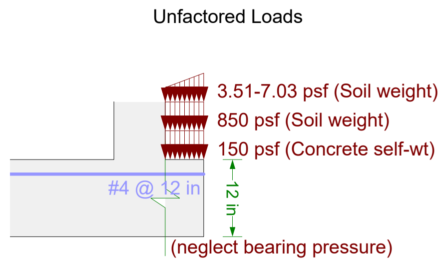
8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

6 inch minimum controls

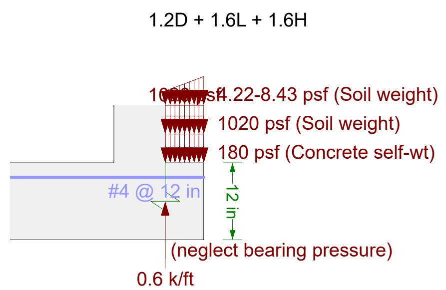
l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.15 ft·k / ft < M_{stem} = 6.88 ft·k / ft

M_u = 0.15 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in})/2] = 8.6 \text{ ft·k/ft}$

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.15 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\,(9.75 \text{ in}) = 12.82 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k/ft}) = 9.61 \text{ k/ft}$

ϕV_n = 9.61 k / ft ≥ V_u = 0.6 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \leq 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0604$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.15 ft·k / ft) = 0.2 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0.15 \text{ ft·k/ft})}{(8.6 \text{ ft·k/ft})} = 0.0175$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

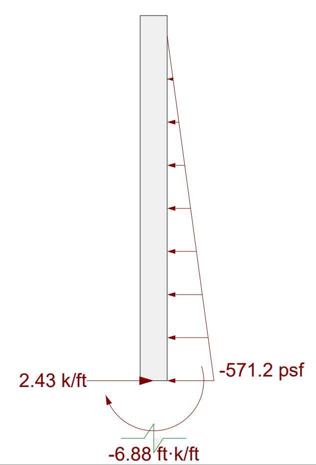
Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

12 inch minimum controls

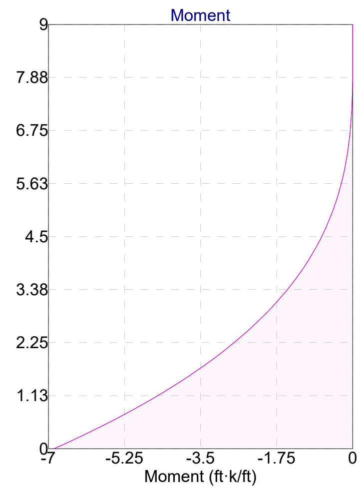
l_{d_prov} = 42 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

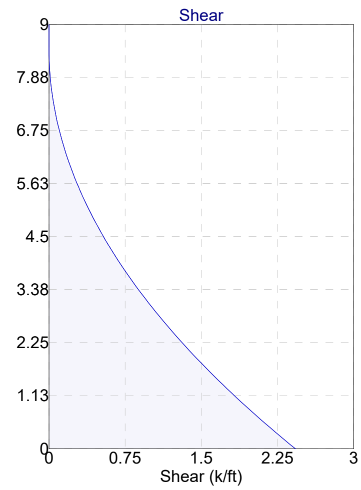
Stem Internal Forces



Stem Internal Forces



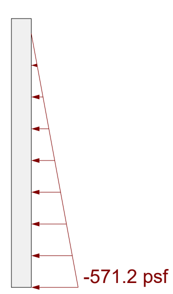
Stem Internal Forces



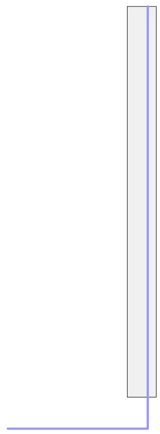
Stem Joint Force Transfer

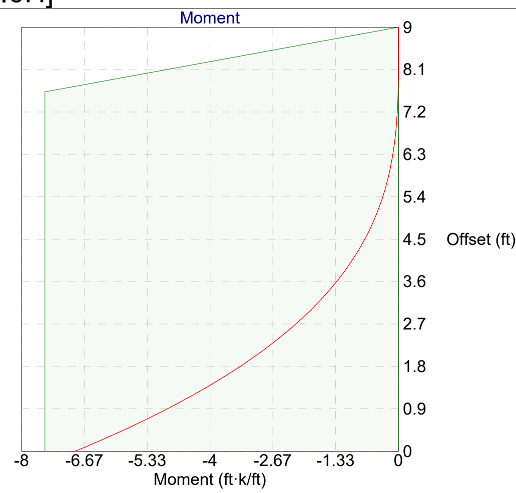
Location	Force
@ stem base	2.43 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]





Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 7.51 ft·k / ft ≥ M_u = 6.88 ft·k / ft ✓

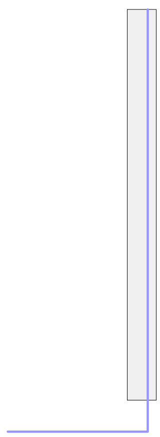
Check (ACI 318-14 11.5.5.1b) @ 7.63 ft from base

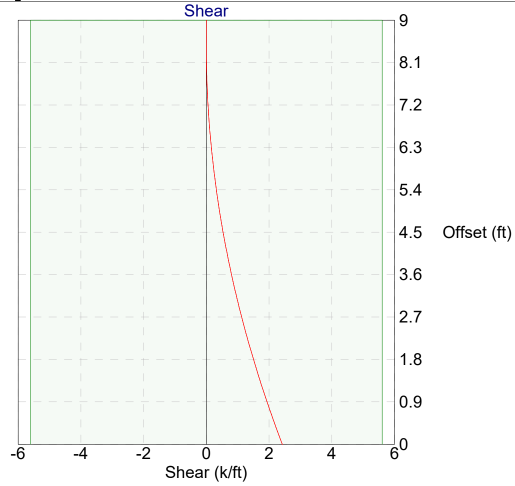
ϕM_n = 7.51 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 7.64 ft from base

ϕM_n = 7.48 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.61 k / ft ≥ V_u = 2.43 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 7.51 ft·k / ft < (4 / 3) M_u = [4 / 3] (6.88 ft·k / ft) = 9.17 ft·k / ft

$A_{s\_min} = \frac{3\sqrt{F'_c}}{f_y} d = \frac{3\sqrt{3000\text{ psi}}}{(60000\text{ psi})}(5.69\text{ in}) = 0.02\text{ in}^2/\text{in}$

200 d / f_y = 200 (5.69 in) / (60000 psi) = 0.02 in² / in

A_{s_min} = 0.02 in² / in

A_s = 0.03 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 9 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\text{ in}^2/\text{in})(60000\text{ psi})}{0.85(3000\text{ psi})} = 0.61\text{ in}$

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69\text{ in})}{(0.61\text{ in})/(0.850)} - 1\right] = 0.0209$

ε_t = 0.0209 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\text{ in}^2/\text{in})(60000\text{ psi})}{0.85(3000\text{ psi})} = 0.61\text{ in}$

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69\text{ in})}{(0.61\text{ in})/(0.850)} - 1\right] = 0.0209$

ε_t = 0.0209 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$\rho_t = \frac{A_{s\_horz}/s_{horz}}{t} = \frac{(0.2\text{ in}^2)/(12\text{ in})}{(8\text{ in})} = 0.0021$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(6.88\text{ ft·k/ft})}{(7.51\text{ ft·k/ft})} = 0.9159$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{F'_c}}\right) d_b = \left[\frac{(60000\text{ psi})(1.0)(0.70)(1.0)}{50(1.0)\sqrt{3000\text{ psi}}}\right](0.63\text{ in}) = 9.59\text{ in}$

Factoring l_{dh} by the excess reinforcement ratio (0.9159) per 25.4.10: l_{dh} = 8.78 in

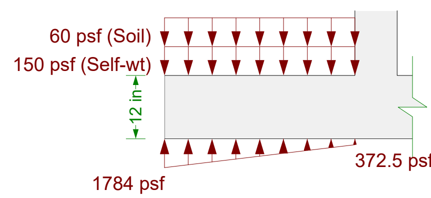
8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

l_{dh_prov} = 9 in ≥ l_{dh} = 8.78 in ✓

Toe Checks [1.4D]

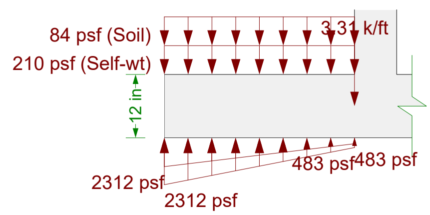
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 6.34 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.61 \text{ in}) / 2] = 11.7 \text{ ft·k / ft}$

ϕM_n = 11.7 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (8.69 \text{ in}) = 11.42 \text{ k / ft}$

$\phi V_n = \phi V_c = (0.750)(11.42 \text{ k / ft}) = 8.57 \text{ k / ft}$

ϕV_n = 8.57 k / ft ≥ V_u = 3.01 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(8.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1\right] = 0.0334$

ε_t = 0.0334 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 11.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(-0 \text{ ft·k / ft})}{(11.7 \text{ ft·k / ft})} = -0.0$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right] (0.63 \text{ in}) = 9.59 \text{ in}$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

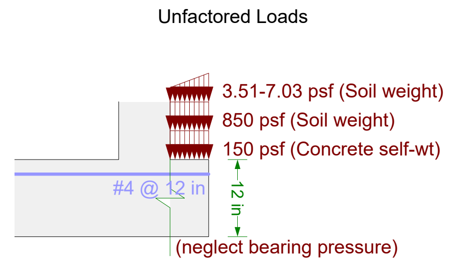
$8 d_b = 8 (0.63 \text{ in}) = 5.0$ (minimum limit, does not control)

6 inch minimum controls

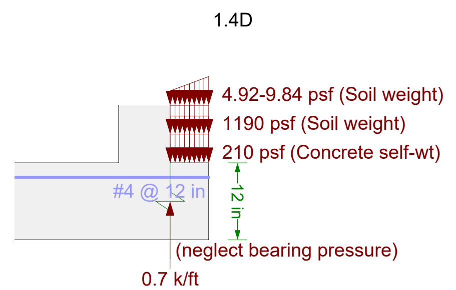
l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.18 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in})/2] = 8.6 \text{ ft·k/ft}$

ϕM_n = 8.6 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (9.75 \text{ in}) = 12.82 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k/ft}) = 9.61 \text{ k/ft}$

ϕV_n = 9.61 k / ft ≥ V_u = 0.7 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \leq 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0604$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ $(4/3) M_u$ = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(-0 \text{ ft·k/ft})}{(8.6 \text{ ft·k/ft})} = -0.0$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right] (0.5 \text{ in}) = 13.15 \text{ in}$

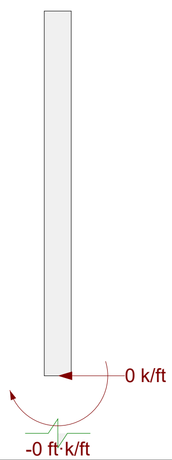
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

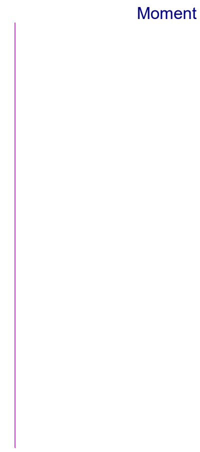
l_{d_prov} = 42 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

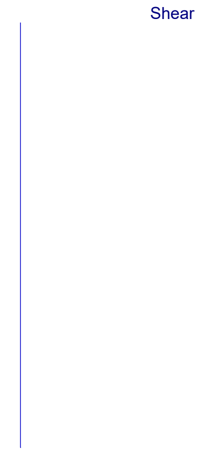
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

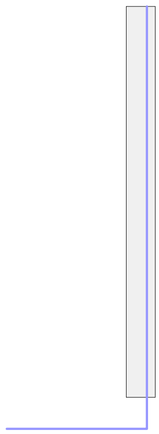


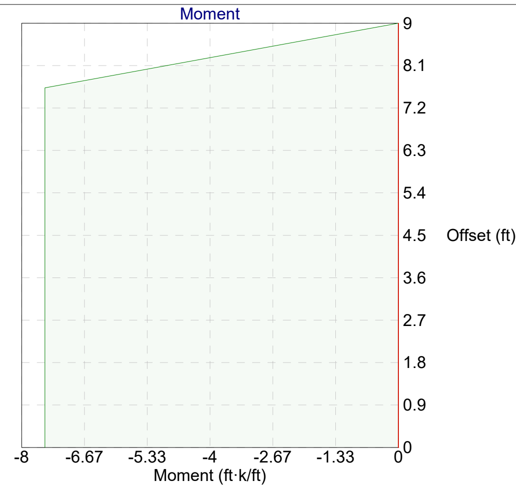
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

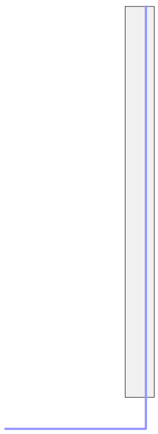
Stem Internal Forces

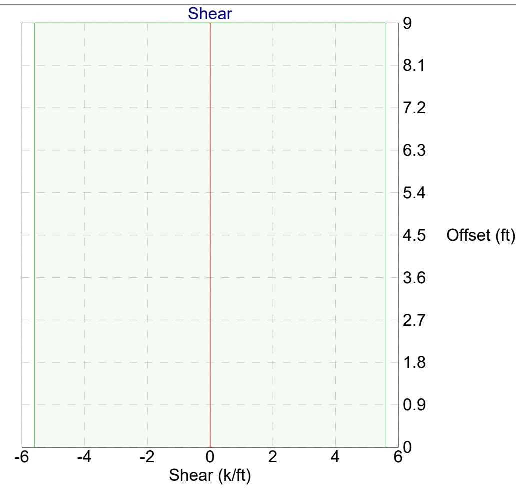
Stem Moment Checks [1.4D]





Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 7.51 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 9 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\ in^2 / in)(60000\ psi)}{0.85\ (3000\ psi)} = 0.61\ in$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.69\ in)}{(0.61\ in) / (0.850)} - 1\right] = 0.0209$$

ε_t = 0.0209 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03\ in^2 / in)(60000\ psi)}{0.85\ (3000\ psi)} = 0.61\ in$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.69\ in)}{(0.61\ in) / (0.850)} - 1\right] = 0.0209$$

ε_t = 0.0209 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2\ in^2) / (12\ in)}{(8\ in)} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0\ ft \cdot k / ft)}{(7.51\ ft \cdot k / ft)} = 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50\ (1.0) \sqrt{3000\ psi}}\right] (0.63\ in) = 9.59\ in$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: l_{dh} = 0 in

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

6 inch minimum controls

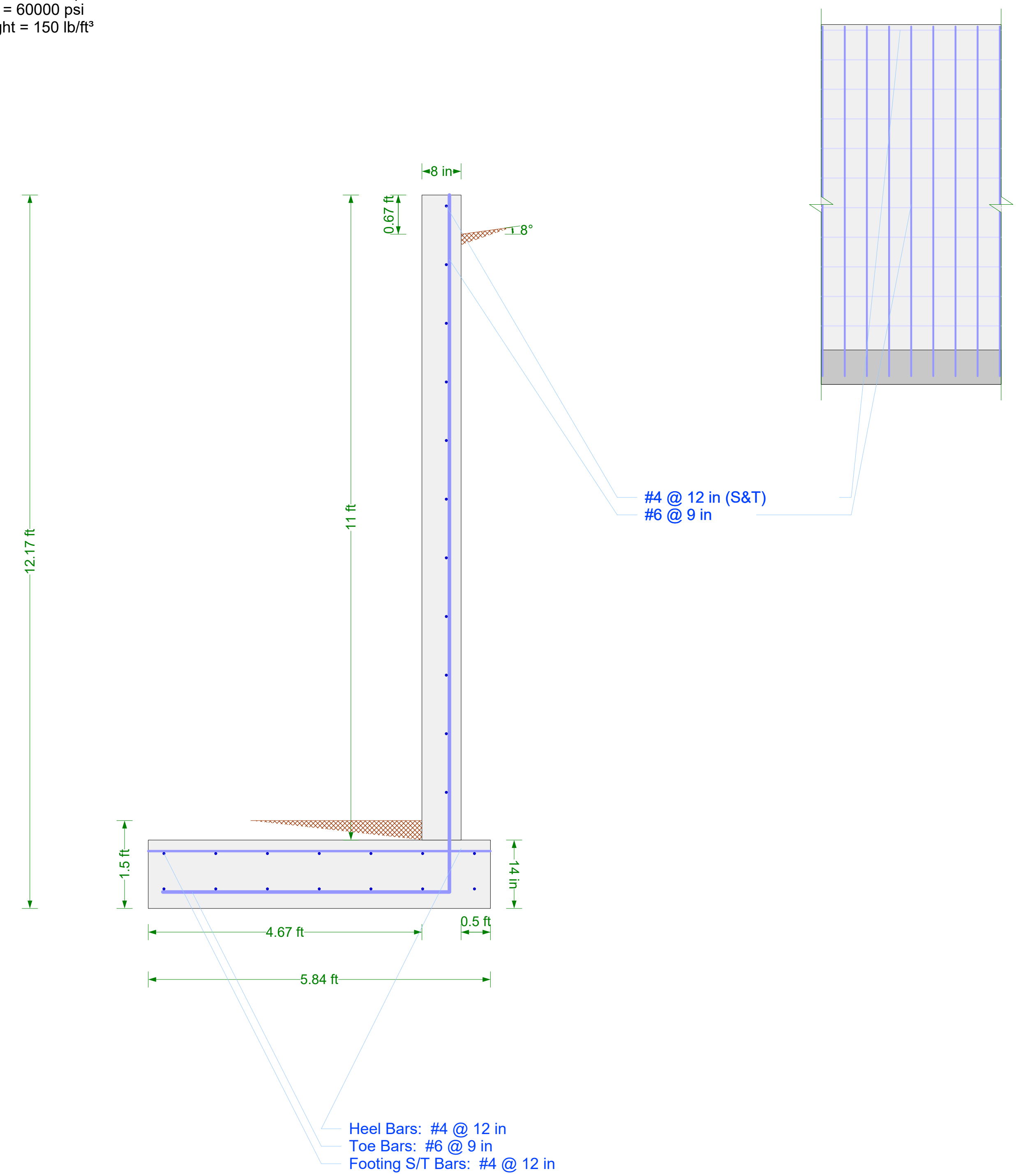
l_{dh_prov} = 9 in ≥ l_{dh} = 6 in ✓

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

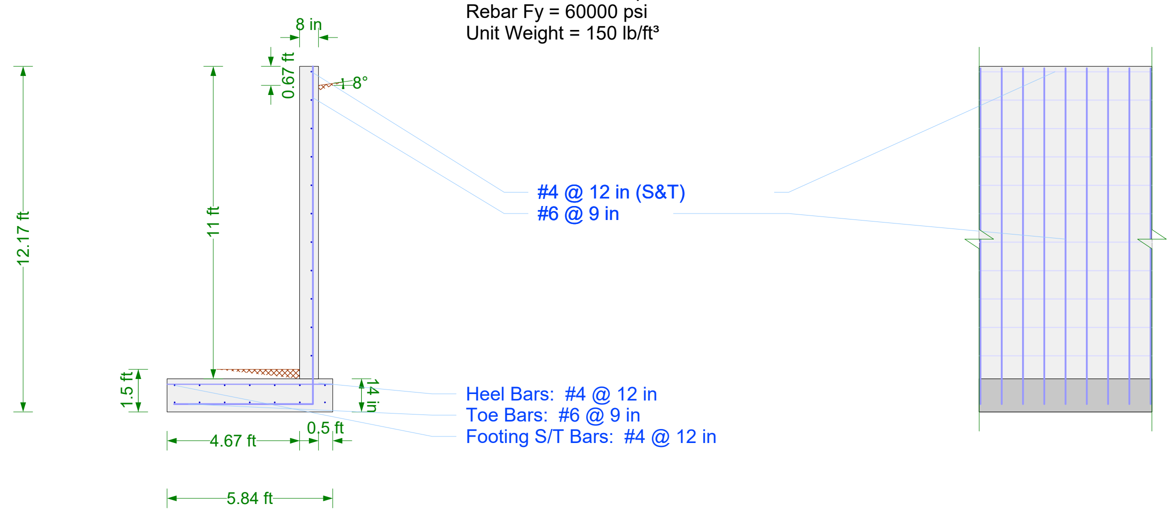
Cantilever Wall 10' Tall

Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



Design Detail



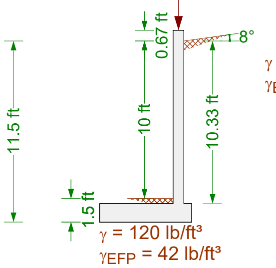
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.966	Overturning	1.55	1.50	1.0D + 1.0L + 1.0H
✓ 0.613	Bearing Pressure	2500 psf	1533 psf	1.0D + 1.0L + 1.0H
✓ 0.392	Bearing Eccentricity	13.74 in	35.02 in	1.0D + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.360	Shear	10.48 k/ft	3.77 k/ft	1.4D
✓ 0.440	Moment	26.53 ft·k/ft	11.66 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.195	Min Strain	0.0206	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.05 in²	0 in²	1.2D + 1.6L + 1.6H
✗ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.073	Shear	11.58 k/ft	0.85 k/ft	1.4D
✓ 0.018	Moment	10.4 ft·k/ft	0.18 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.054	Min Strain	0.0734	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.193	Development	62.04 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.927	Moment	13.33 ft·k/ft	12.36 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.647	Shear	5.55 k/ft	3.59 k/ft	1.2D + 1.6L + 1.6H
✓ 0.422	Max Steel	0.0095	0.0040	1.2D + 1.6L + 1.6H
✓ 0.384	Min Steel	0.05 in²/in	0.02 in²/in	1.2D + 1.6L + 1.6H
✓ 0.969	Base Development	11 in	10.66 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2018
Concrete Load Combs	IBC 2018 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)
1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H
1.2D + 1.6H
1.2D + 0.9H
0.9D + 1.6H
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	12.36	13.33	3.59	5.55	10.66	11	0.0040	0.0095	0.02
1.2D + 1.6L + 0.9H	6.95	13.33	2.02	5.55	6	11	0.0040	0.0095	0
1.2D + 0.5L + 1.6H	12.36	13.33	3.59	5.55	10.66	11	0.0040	0.0095	0.02
1.2D + 0.5L + 0.9H	6.95	13.33	2.02	5.55	6	11	0.0040	0.0095	0
1.2D + 1.6H	12.36	13.33	3.59	5.55	10.66	11	0.0040	0.0095	0.02
1.2D + 0.9H	6.95	13.33	2.02	5.55	6	11	0.0040	0.0095	0
0.9D + 1.6H	12.36	13.33	3.59	5.55	10.66	11	0.0040	0.0095	0.02
0.9D + 0.9H	6.95	13.33	2.02	5.55	6	11	0.0040	0.0095	0
1.4D	0	0	0	0	6	11	0.0040	0.0095	0

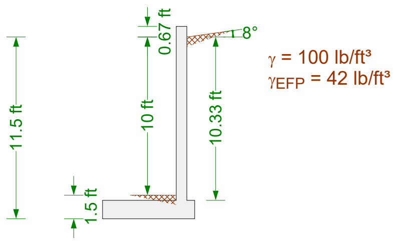
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.05	0.18	10.4	0.73	11.58	11.66	26.53	3.6	10.48
1.2D + 1.6L + 0.9H	0.05	0.18	10.4	0.73	11.58	11.66	26.53	3.6	10.48
1.2D + 0.5L + 1.6H	0.05	0.18	10.4	0.73	11.58	10.87	26.53	3.34	10.48
1.2D + 0.5L + 0.9H	0.05	0.18	10.4	0.73	11.58	10.87	26.53	3.34	10.48
1.2D + 1.6H	0.05	0.18	10.4	0.73	11.58	10.5	26.53	3.23	10.48
1.2D + 0.9H	0.05	0.18	10.4	0.73	11.58	10.5	26.53	3.23	10.48
0.9D + 1.6H	0.05	0.14	10.4	0.55	11.58	7.88	26.53	2.42	10.48
0.9D + 0.9H	0.05	0.14	10.4	0.55	11.58	7.88	26.53	2.42	10.48
1.4D	0.05	0.21	10.4	0.85	11.58	12.25	26.53	3.77	10.48

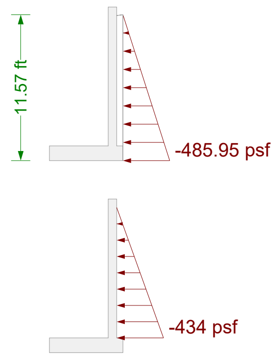
Stability Check Results Summary

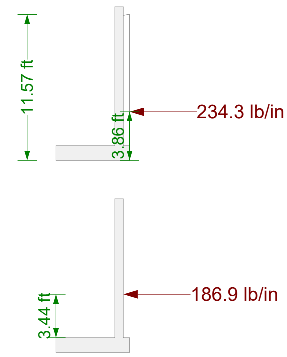
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H	10.84	16.84	1.554	1.500	1.200	234.3	122.8	0.524
1.0D + 1.0H	10.84	16.84	1.554	1.500	1.200	234.3	115.5	0.493

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H	1.500	1.200	1533	2500	13.74	35.02	0.39
1.0D + 1.0H	1.500	1.200	1439	2500	13.74	35.02	0.39

Backfill Pressure







Lateral Earth Pressure

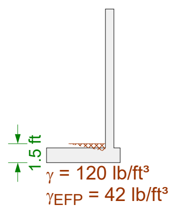
Equivalent Fluid Pressure

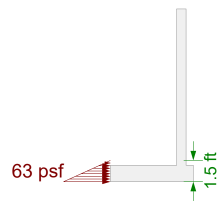
$\sigma_h = H\,\gamma_{fluid} = (11.57\text{ ft})(42\text{ lb / ft}^3) = 486\text{ psf}$

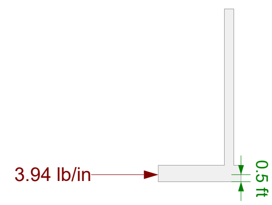
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (10.33\text{ ft})(42\text{ lb / ft}^3) = 434\text{ psf}$

Passive Pressure





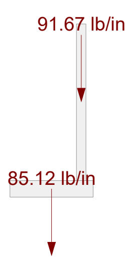


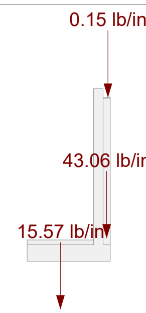
Lateral Earth Pressure

Equivalent Fluid Pressure

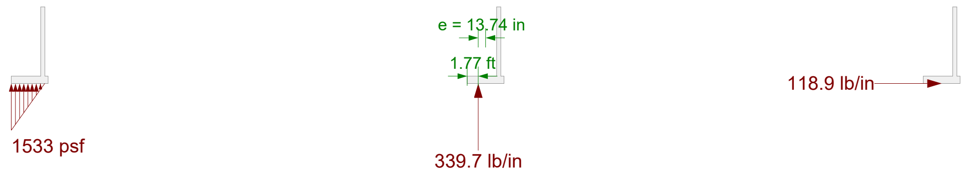
$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Wall/Soil Weights





Bearing Pressure



Friction

F = μ R = (0.350)(339.7 lb / in) = 118.9 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-234.27 lb/in	3.86 ft	130110 in·lb/ft
Axial Dead Load	-83.33 lb/in	5 ft	0 lb/in	-	-60040 in·lb/ft
Axial Live Load	-20.83 lb/in	5 ft	0 lb/in	-	-15010 in·lb/ft
Footing Weight	-85.12 lb/in	2.92 ft	0 lb/in	-	-35770.01 in·lb/ft
Stem Weight	-91.67 lb/in	5 ft	0 lb/in	-	-66044 in·lb/ft
Backfill Weight	-43.06 lb/in	5.59 ft	0 lb/in	-	-34637.33 in·lb/ft
Backfill Weight	-0.15 lb/in	5.67 ft	0 lb/in	-	-119.53 in·lb/ft
Soil over toe Weight	-15.57 lb/in	2.34 ft	0 lb/in	-	-5234.14 in·lb/ft
	-339.72 lb/in				-86745.27 in·lb/ft

$$\frac{-\ 86745.27\ \text{in·lb / ft}}{-\ 339.72\ \text{lb / in}} = 1.77\ \text{ft}$$

Stability Checks [1.0D + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	234.3 lb/in	3.86 ft	130110 in·lb/ft
		Total:	130110 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	5 ft	60040 in·lb/ft
Footing Weight	-85.12 lb/in	2.92 ft	35770 in·lb/ft
Stem Weight	-91.67 lb/in	5 ft	66044 in·lb/ft
Backfill Weight	-43.06 lb/in	5.59 ft	34637 in·lb/ft
Backfill Weight	-0.15 lb/in	5.67 ft	119.5 in·lb/ft
Soil over toe Weight	-15.57 lb/in	2.34 ft	5234 in·lb/ft
		Total:	202129 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{202129 \text{ in·lb / ft}}{130110 \text{ in·lb / ft}} = 1.554 > 1.50 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

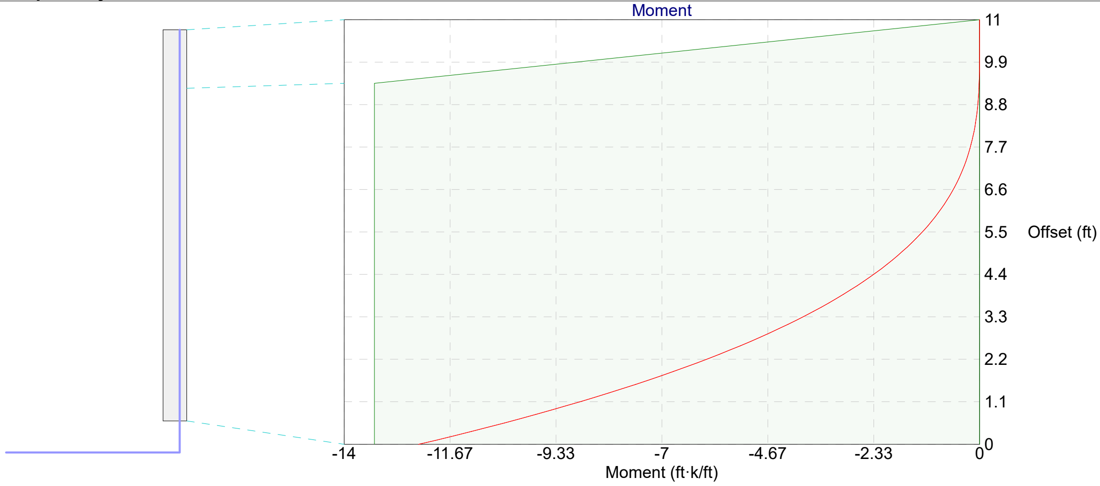
Bearing pressure < allowable (1533 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (13.74 in < 35.02 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.193 in
Deflection due to rotation from settlement	0.201 in
Total deflection at top of wall (positive towards toe)	0.393 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (1.15 \text{ in}) / 2] = 13.33 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 9.36 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

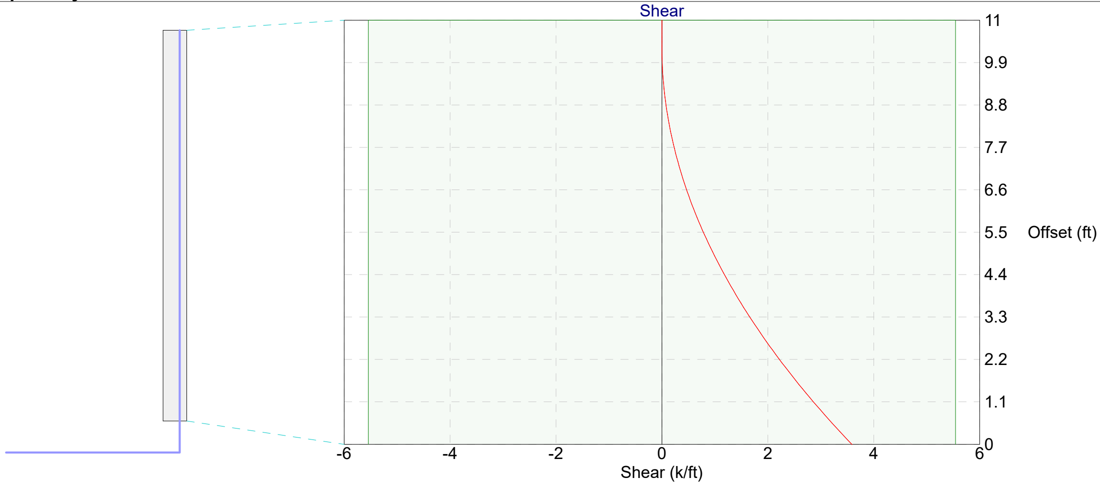
$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (1.15 \text{ in}) / 2] = 13.33 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 11 ft from base

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.63\text{ in}) = 7.39\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(7.39\text{ k / ft}) = 5.55\text{ k / ft}$

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 11 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.63\text{ in}) = 7.39\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(7.39\text{ k / ft}) = 5.55\text{ k / ft}$

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\ \psi_e\ \psi_c\ \psi_r}{50\ \lambda\ \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\text{ psi})(1.0)(0.70)(1.0)}{50\ (1.0)\sqrt{3000\text{ psi}}}\right](0.75\text{ in}) = 11.5\text{ in}$$

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (2 in) + (0.75 in) / 2 = 2.38 in

c_b = 2.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

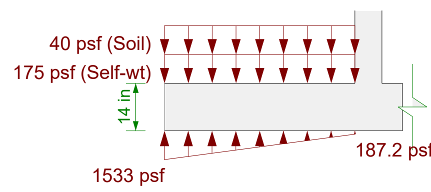
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.38\text{ in}) + (0.0)}{(0.75\text{ in})} = 3.1667$$

$$l_d = \left(\frac{3.}{40}\ \frac{f_y}{\lambda\sqrt{F'_c}}\ \frac{\psi_t\ \psi_e\ \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\ \frac{(60000\text{ psi})}{(1.0)\sqrt{3000\text{ psi}}}\ \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.75\text{ in}) = 19.72\text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

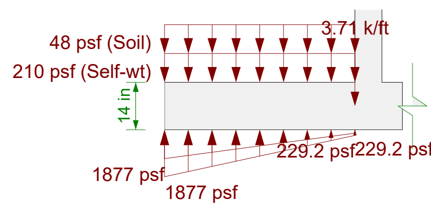
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 11.66 ft·k / ft < M_{stem} = 12.36 ft·k / ft

M_u = 11.66 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})[(10.63 \text{ in}) - (1.15 \text{ in}) / 2] = 26.53 \text{ ft·k / ft}$

ϕM_n = 26.53 ft·k / ft ≥ M_u = 11.66 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (10.63 \text{ in}) = 13.97 \text{ k / ft}$

$\phi V_n = \phi V_c = (0.750)(13.97 \text{ k / ft}) = 10.48 \text{ k / ft}$

ϕV_n = 10.48 k / ft ≥ V_u = 3.6 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \leq 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(10.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1\right] = 0.0206$

ε_t = 0.0206 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.53 ft·k / ft ≥ $(4 / 3) M_u = [4 / 3](11.66 \text{ ft·k / ft}) = 15.55$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(11.66 \text{ ft·k / ft})}{(26.53 \text{ ft·k / ft})} = 0.4396$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right](0.75 \text{ in}) = 11.5 \text{ in}$

Factoring l_{dh} by the excess reinforcement ratio (0.4396) per 25.4.10: l_{dh} = 5.06 in

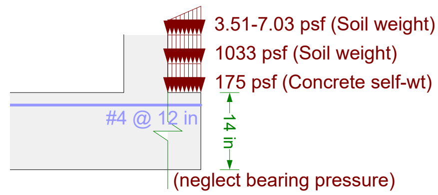
$8 d_b = 8 (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

6 inch minimum controls

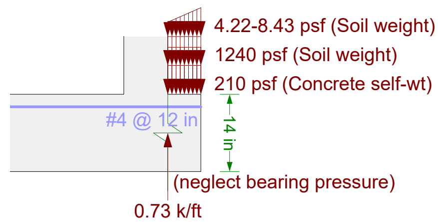
l_{dh_prov} = 6 in < l_{dh} = 6 in ✗

Heel Checks [1.2D + 1.6L + 1.6H]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.18 ft·k / ft < M_{stem} = 12.36 ft·k / ft

M_u = 0.18 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in})/2] = 10.4 \text{ ft·k/ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.18 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(11.75 \text{ in}) = 15.45 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k/ft}) = 11.58 \text{ k/ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.73 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(11.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.18 ft·k / ft) = 0.24 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.18 \text{ ft·k/ft})}{(10.4 \text{ ft·k/ft})} = 0.0175$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda \sqrt{F'_c}}\,\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$$

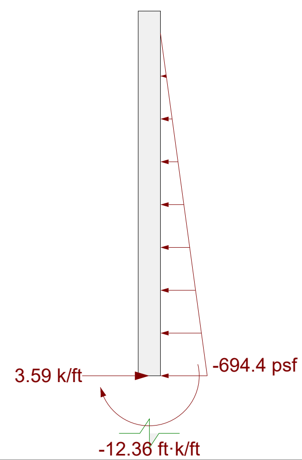
Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

12 inch minimum controls

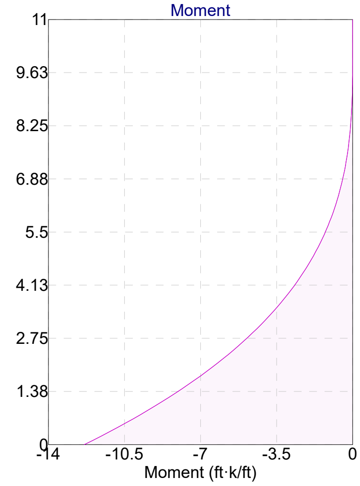
l_{d_prov} = 62.04 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

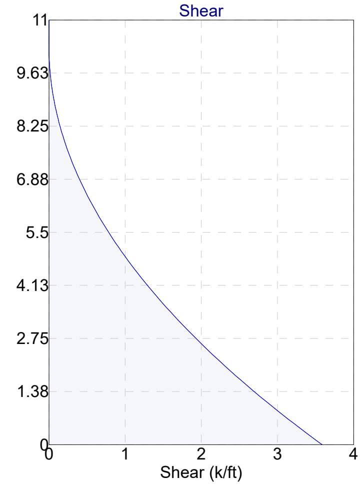
Stem Internal Forces



Stem Internal Forces



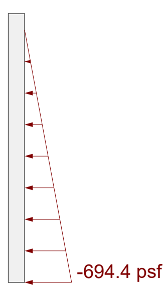
Stem Internal Forces



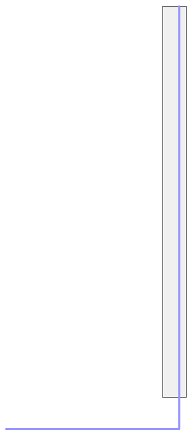
Stem Joint Force Transfer

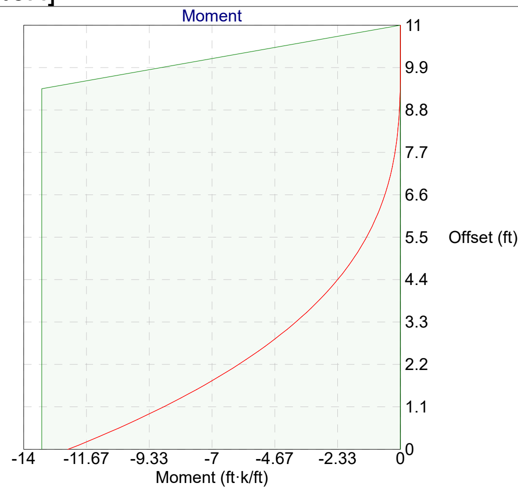
Location	Force
@ stem base	3.59 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]





Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 13.33 ft·k / ft ≥ M_u = 12.36 ft·k / ft ✓

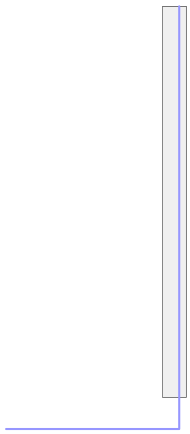
Check (ACI 318-14 11.5.5.1b) @ 9.36 ft from base

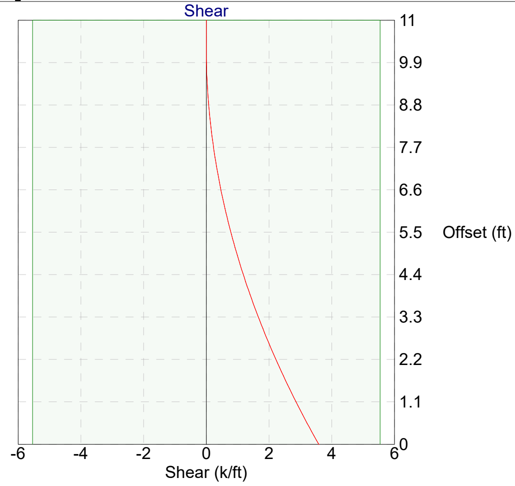
ϕM_n = 13.33 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 9.44 ft from base

ϕM_n = 12.62 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.55 k / ft ≥ V_u = 3.59 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 13.33 ft·k / ft < (4 / 3) M_u = [4 / 3] (12.36 ft·k / ft) = 16.48 ft·k / ft

$$A_{s\_min} = \frac{3\sqrt{F'_c}}{f_y} d = \frac{3\sqrt{3000\ psi}}{(60000\ psi)}(5.63\ in) = 0.02\ in^2 / in$$

200 d / f_y = 200 (5.63 in) / (60000 psi) = 0.02 in² / in

A_{s_min} = 0.02 in² / in

A_s = 0.05 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05\ in^2 / in)(60000\ psi)}{0.85 (3000\ psi)} = 1.15\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.63\ in)}{(1.15\ in) / (0.850)} - 1\right] = 0.0095$$

ε_t = 0.0095 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05\ in^2 / in)(60000\ psi)}{0.85 (3000\ psi)} = 1.15\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.63\ in)}{(1.15\ in) / (0.850)} - 1\right] = 0.0095$$

ε_t = 0.0095 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2\ in^2) / (12\ in)}{(8\ in)} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(12.36\ ft·k / ft)}{(13.33\ ft·k / ft)} = 0.9270$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50 (1.0)\sqrt{3000\ psi}}\right](0.75\ in) = 11.5\ in$$

Factoring l_{dh} by the excess reinforcement ratio (0.9270) per 25.4.10: l_{dh} = 10.66 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

l_{dh_prov} = 11 in ≥ l_{dh} = 10.66 in ✓

Toe Checks [1.4D]

Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 12.25 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.15 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})[(10.63 \text{ in}) - (1.15 \text{ in})/2] = 26.53 \text{ ft·k/ft}$$

ϕM_n = 26.53 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}(10.63 \text{ in}) = 13.97 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(13.97 \text{ k/ft}) = 10.48 \text{ k/ft}$$

ϕV_n = 10.48 k / ft ≥ V_u = 3.77 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(10.63 \text{ in})}{(1.15 \text{ in})/(0.850)} - 1\right] = 0.0206$$

ε_t = 0.0206 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.53 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018(60000)}{f_y} = \frac{0.0018(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(-0 \text{ ft·k/ft})}{(26.53 \text{ ft·k/ft})} = -0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50(1.0)\sqrt{3000 \text{ psi}}}\right](0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

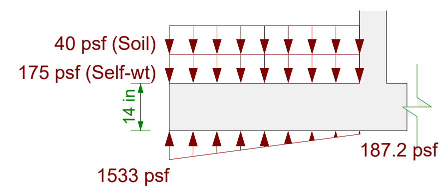
8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 6 in < l_{dh} = 6 in ✗

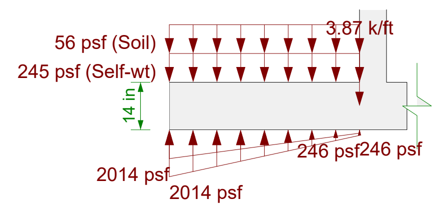
Toe Unfactored Loads

Unfactored Loads



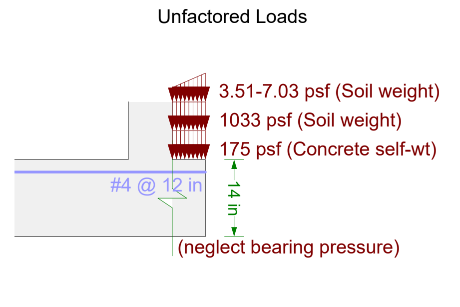
Toe Factored Loads

1.4D

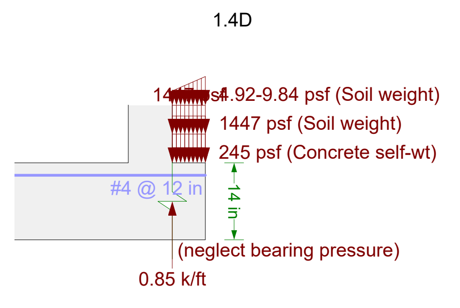


Heel Checks [1.4D]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.21 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in})/2] = 10.4 \text{ ft·k/ft}$

ϕM_n = 10.4 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(11.75 \text{ in}) = 15.45 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k/ft}) = 11.58 \text{ k/ft}$

ϕV_n = 11.58 k / ft ≥ V_u = 0.85 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(11.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0734$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ $(4/3) M_u$ = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(-0 \text{ ft·k/ft})}{(10.4 \text{ ft·k/ft})} = -0.0$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\sqrt{F'_c}}\,\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

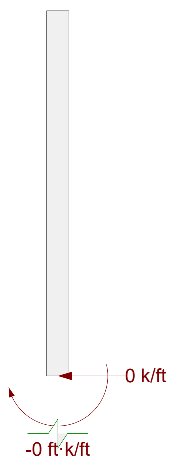
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

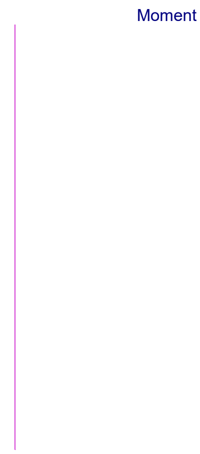
l_{d_prov} = 62.04 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

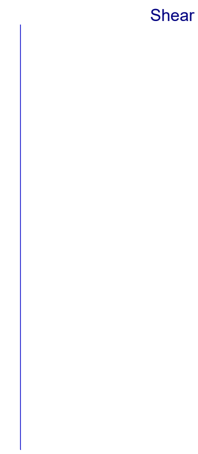
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

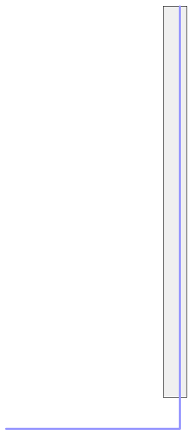


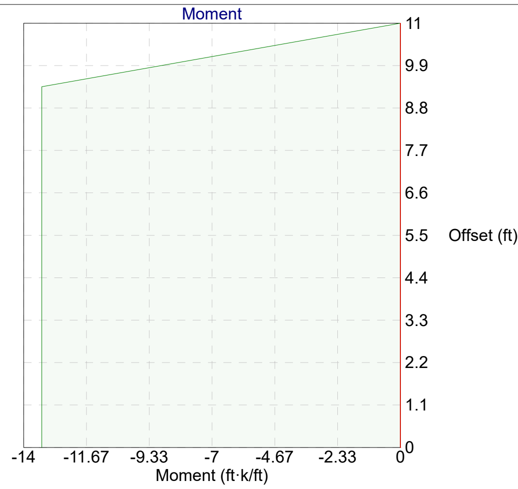
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

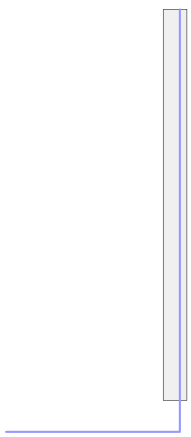
Stem Internal Forces

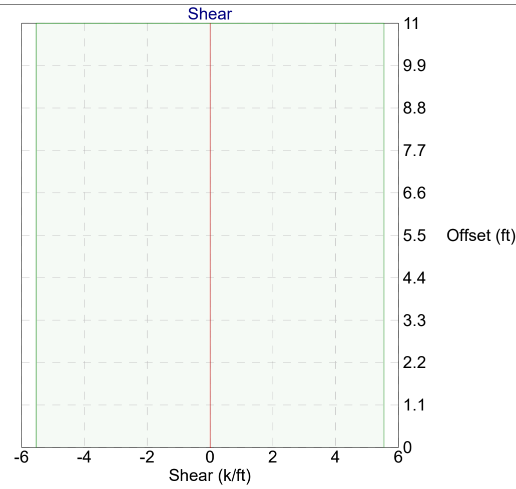
Stem Moment Checks [1.4D]





Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 13.33 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1\right] = 0.0095$$

ε_t = 0.0095 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1\right] = 0.0095$$

ε_t = 0.0095 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(13.33 \text{ ft·k / ft})} = 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: l_{dh} = 0 in

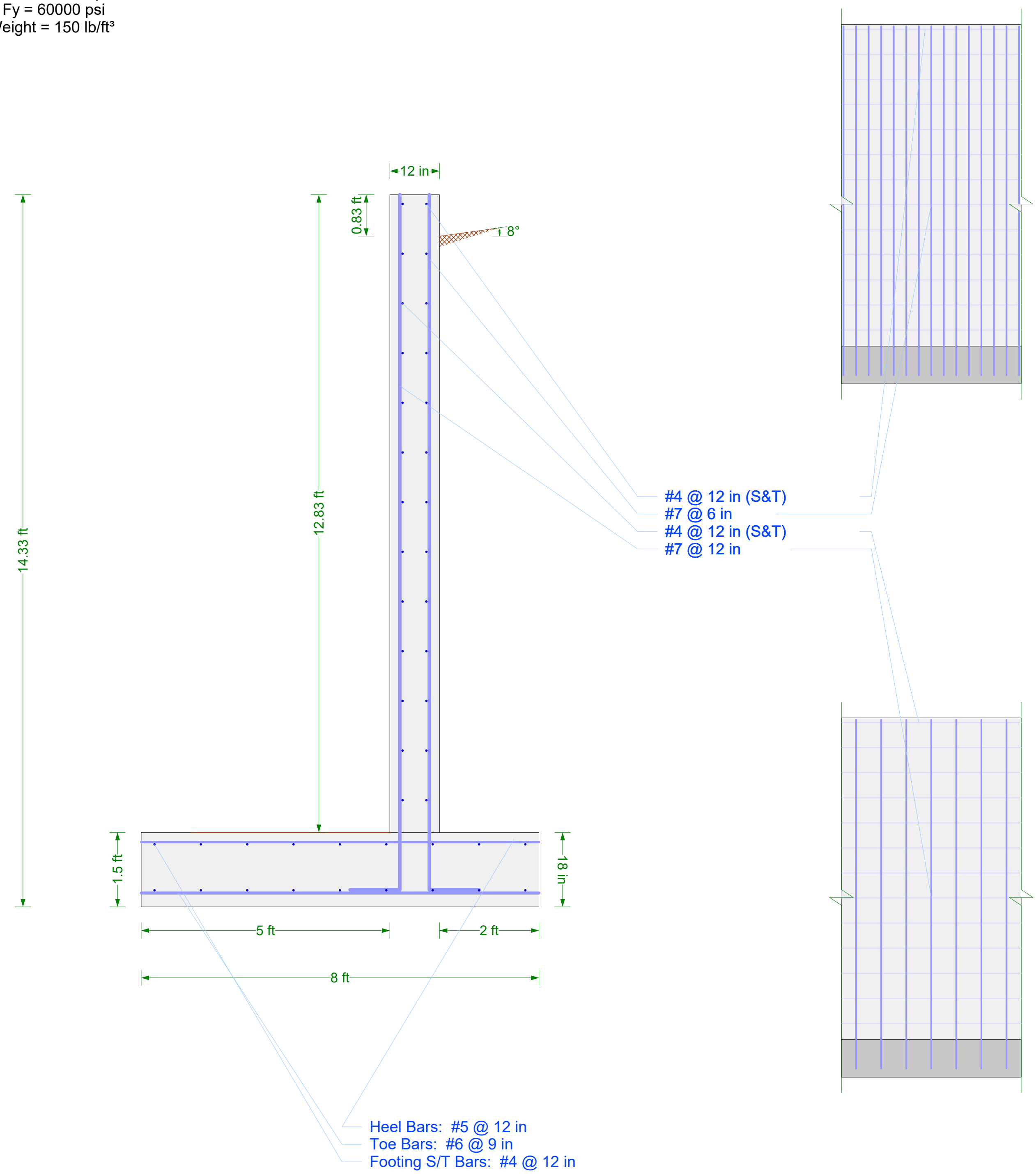
8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 11 in ≥ l_{dh} = 6 in ✓

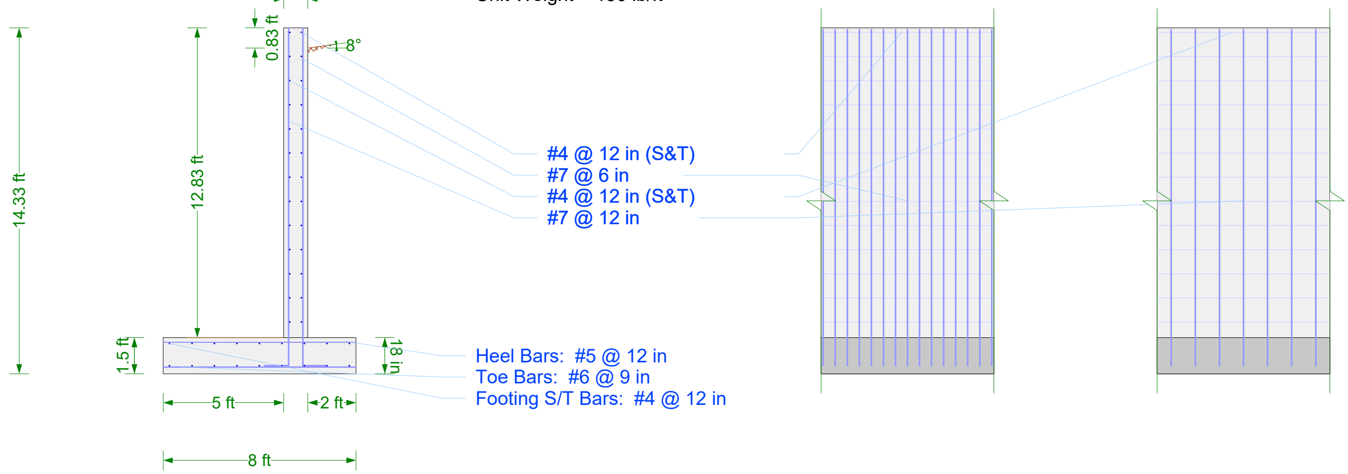
Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.374	Overturning	4.01	1.50	1.0D + 1.0L + 1.0H
✓ 0.768	Bearing Pressure	2500 psf	1919 psf	1.0D + 1.0L + 1.0H
✓ 0.022	Bearing Eccentricity	1.05 in	48 in	1.0D + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.590	Shear	14.42 k/ft	8.51 k/ft	1.4D
✓ 0.669	Moment	37.09 ft·k/ft	24.83 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.136	Min Strain	0.0294	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.05 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.400	Development	33 in	13.2 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.972	S&T Min Rho	0.0019	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.262	Shear	15.47 k/ft	4.05 k/ft	1.4D
✓ 0.189	Moment	21.46 ft·k/ft	4.06 ft·k/ft	1.4D
✓ 0.064	Min Strain	0.0628	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.03 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.171	Development	70 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.972	S&T Min Rho	0.0019	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.575	Moment	45.28 ft·k/ft	26.02 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.513	Shear	9.43 k/ft	4.84 k/ft	1.2D + 1.6L + 1.6H
✓ 0.543	Max Steel	0.0074	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.1 in²/in	0 in²/in	1.2D + 1.6L + 1.6H
✓ 0.514	Base Development	15 in	7.71 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

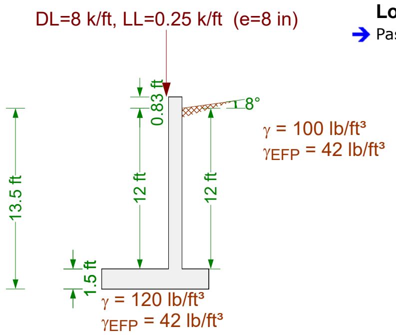
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2018
Concrete Load Combs	IBC 2018 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H
1.2D + 1.6H
1.2D + 0.9H
0.9D + 1.6H
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	26.02	45.28	4.84	9.43	7.71	15	0.0040	0.0074	0
1.2D + 1.6L + 0.9H	17.55	45.28	2.72	9.43	7	15	0.0040	0.0074	0
1.2D + 0.5L + 1.6H	25.84	45.28	4.84	9.43	7.66	15	0.0040	0.0074	0
1.2D + 0.5L + 0.9H	17.37	45.28	2.72	9.43	7	15	0.0040	0.0074	0
1.2D + 1.6H	25.75	45.28	4.84	9.43	7.63	15	0.0040	0.0074	0
1.2D + 0.9H	17.29	45.28	2.72	9.43	7	15	0.0040	0.0074	0
0.9D + 1.6H	24.15	45.28	4.84	9.43	7.16	15	0.0040	0.0074	0
0.9D + 0.9H	15.69	45.28	2.72	9.43	7	15	0.0040	0.0074	0
1.4D	7.47	44.09	0	0	7	15	0.0040	0.0074	0

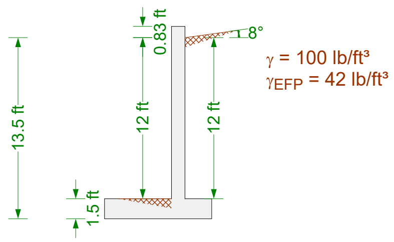
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.1	3.48	21.46	3.47	15.47	24.83	37.09	7.49	14.42
1.2D + 1.6L + 0.9H	0.1	3.48	21.46	3.47	15.47	24.83	37.09	7.49	14.42
1.2D + 0.5L + 1.6H	0.1	3.48	21.46	3.47	15.47	24.38	37.09	7.36	14.42
1.2D + 0.5L + 0.9H	0.1	3.48	21.46	3.47	15.47	24.38	37.09	7.36	14.42
1.2D + 1.6H	0.1	3.48	21.46	3.47	15.47	24.18	37.09	7.3	14.42
1.2D + 0.9H	0.1	3.48	21.46	3.47	15.47	24.18	37.09	7.3	14.42
0.9D + 1.6H	0.1	2.61	21.46	2.6	15.47	18.14	37.09	5.47	14.42
0.9D + 0.9H	0.1	2.61	21.46	2.6	15.47	18.14	37.09	5.47	14.42
1.4D	0.1	4.06	21.46	4.05	15.47	28.21	37.09	8.51	14.42

Stability Check Results Summary

Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H	18.32	73.48	4.011	1.500	1.200	332.4	424	1.276
1.0D + 1.0H	18.32	73.48	4.011	1.500	1.200	332.4	416.7	1.254

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H	1.500	1.200	1919	2500	1.05	48	0.29
1.0D + 1.0H	1.500	1.200	1886	2500	1.05	48	0.29

Backfill Pressure



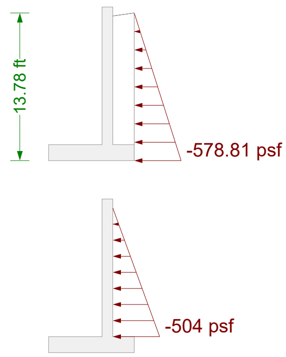
Lateral Earth Pressure

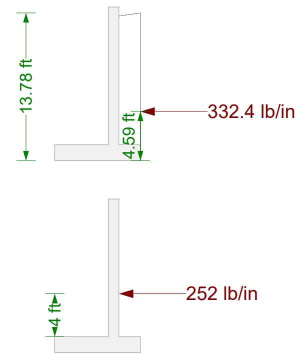
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid}$ = (13.78 ft) (42 lb / ft³) = 578.8 psf

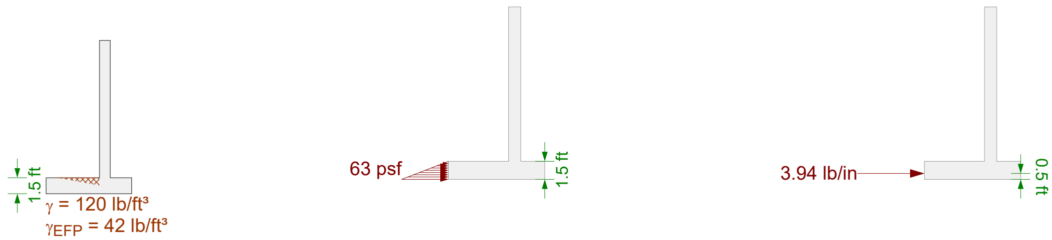
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (12 ft) (42 lb / ft³) = 504 psf





Passive Pressure

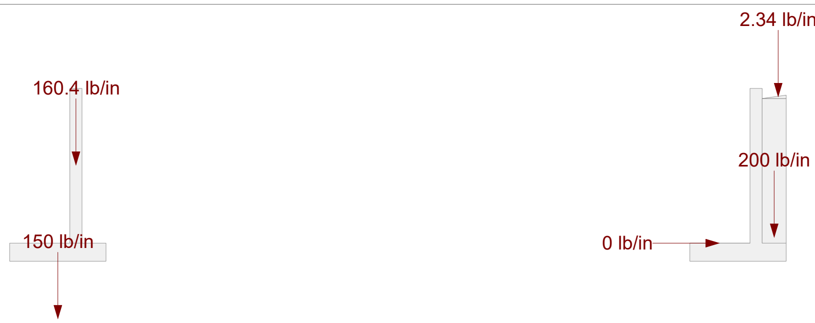


Lateral Earth Pressure

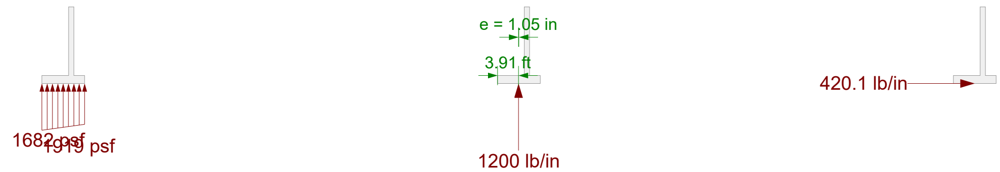
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Wall/Soil Weights



Bearing Pressure



Friction

F = μ R = (0.350)(1200 lb / in) = 420.1 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-332.36 lb/in	4.59 ft	219851 in·lb/ft
Axial Dead Load	-666.67 lb/in	4.83 ft	0 lb/in	-	-464000 in·lb/ft
Axial Live Load	-20.83 lb/in	4.83 ft	0 lb/in	-	-14500 in·lb/ft
Footing Weight	-150 lb/in	4 ft	0 lb/in	-	-86400 in·lb/ft
Stem Weight	-160.42 lb/in	5.5 ft	0 lb/in	-	-127050 in·lb/ft
Backfill Weight	-200 lb/in	7 ft	0 lb/in	-	-201600 in·lb/ft
Backfill Weight	-2.34 lb/in	7.33 ft	0 lb/in	-	-2473.52 in·lb/ft
Soil over toe Weight	-0 lb/in	-	0 lb/in	-	-0 in·lb/ft
	-1200.26 lb/in				-676172.13 in·lb/ft

$$\frac{-676172.13 \text{ in·lb / ft}}{-1200.26 \text{ lb / in}} = 3.91 \text{ ft}$$

Stability Checks [1.0D + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	332.4 lb/in	4.59 ft	219851 in·lb/ft
		Total:	219851 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-666.67 lb/in	4.83 ft	464000 in·lb/ft
Footing Weight	-150 lb/in	4 ft	86400 in·lb/ft
Stem Weight	-160.42 lb/in	5.5 ft	127050 in·lb/ft
Backfill Weight	-200 lb/in	7 ft	201600 in·lb/ft
Backfill Weight	-2.34 lb/in	7.33 ft	2474 in·lb/ft
Soil over toe Weight	-0 lb/in	2.5 ft	0 in·lb/ft
		Total:	881807 in·lb/ft

$$\text{F.S.} = \frac{\text{RM}}{\text{OTM}} = \frac{881807 \text{ in·lb / ft}}{219851 \text{ in·lb / ft}} = 4.011 > 1.50 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

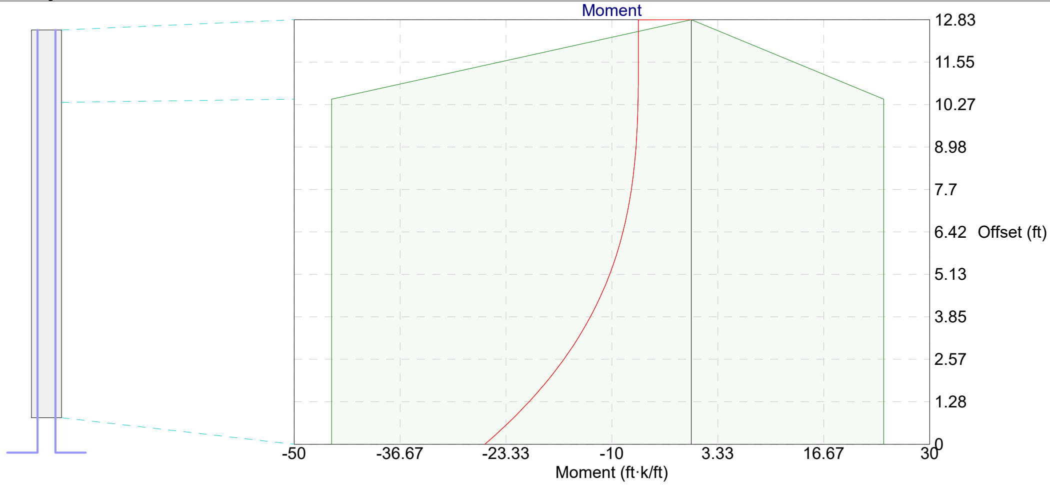
Bearing pressure < allowable (1919 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (1.05 in < 48 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.266 in
Deflection due to rotation from settlement	0.026 in
Total deflection at top of wall (positive towards toe)	0.293 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.1 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 2.35 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.1 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (2.35 \text{ in}) / 2] = 45.28 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.18 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.05 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (1.18 \text{ in}) / 2] = 24.23 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 10.44 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.1 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 2.35 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.1 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (2.35 \text{ in}) / 2] = 45.28 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 10.44 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.18 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0.05 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (1.18 \text{ in}) / 2] = 24.23 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 12.83 ft from base [Negative bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$

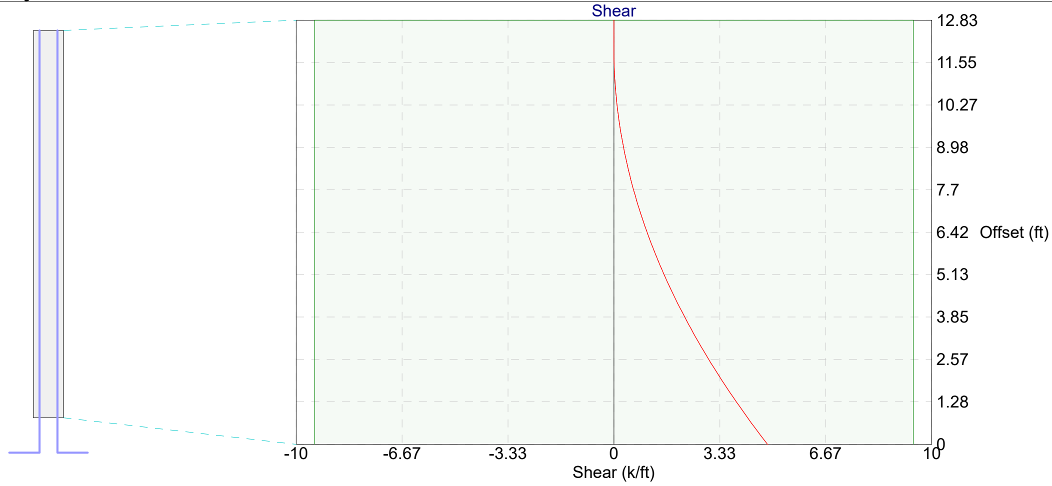
$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 12.83 ft from base [Positive bending]

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90) (0 \text{ in}^2 / \text{in}) (60000 \text{ psi}) [(9.56 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base [Positive shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(9.56\text{ in}) \;=\; 12.57\text{ k / ft}$

$\phi V_n = \phi\,V_c \;=\; (0.750)\,(12.57\text{ k / ft}) \;=\; 9.43\text{ k / ft}$

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base [Negative shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(9.56\text{ in}) \;=\; 12.57\text{ k / ft}$

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Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 12.83 ft from base [Positive shear]

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$\phi V_n = \phi\,V_c \;=\; (0.750)\,(12.57\text{ k / ft}) \;=\; 9.43\text{ k / ft}$

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\ \psi_e\ \psi_c\ \psi_r}{50\ \lambda\ \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\text{ psi})\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000\text{ psi}}}\right](0.88\text{ in}) = 13.42\text{ in}$$

$8\ d_b = 8\,(0.88\text{ in}) = 7.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 1.0 (bars are #7 or larger)

λ = 1.0 (normal weight concrete)

s / 2 = (6 in) / 2 = 3 in

cover + d_b / 2 = (2 in) + (0.88 in) / 2 = 2.44 in

c_b = 2.44 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.44\text{ in}) + (0.0)}{(0.88\text{ in})} = 2.7857$$

$$l_d = \left(\frac{3.}{40}\ \frac{f_y}{\lambda\ \sqrt{F'_c}}\ \frac{\psi_t\ \psi_e\ \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\ \frac{(60000\text{ psi})}{(1.0)\,\sqrt{3000\text{ psi}}}\ \frac{(1.0)\,(1.0)\,(1.0)}{2.5}\right](0.88\text{ in}) = 28.76\text{ in}$$

2nd curtain vertical bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 1.0 (bars are #7 or larger)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.88 in) / 2 = 2.44 in

c_b = 2.44 in (lesser of half spacing, ctr to surface)

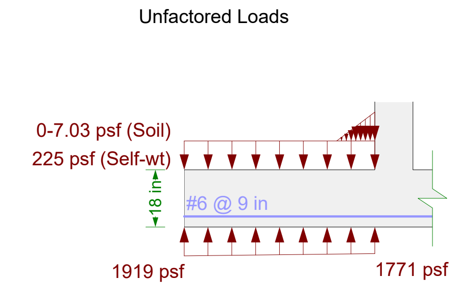
K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.44\text{ in}) + (0.0)}{(0.88\text{ in})} = 2.7857$$

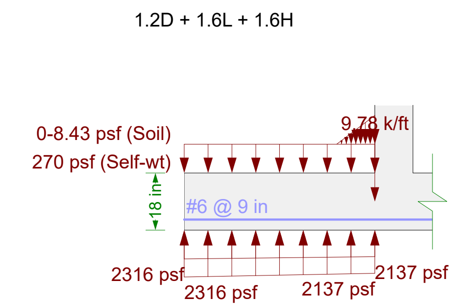
$$l_d = \left(\frac{3.}{40}\ \frac{f_y}{\lambda\ \sqrt{F'_c}}\ \frac{\psi_t\ \psi_e\ \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\ \frac{(60000\text{ psi})}{(1.0)\,\sqrt{3000\text{ psi}}}\ \frac{(1.0)\,(1.0)\,(1.0)}{2.5}\right](0.88\text{ in}) = 28.76\text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

Toe Unfactored Loads



Toe Factored Loads



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 24.83 ft·k / ft < M_{stem} = 26.02 ft·k / ft

M_u = 24.83 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.15 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2)$ = (0.90)(0.05 in² / in)(60000 psi)[(14.63 in) - (1.15 in) / 2] = 37.09 ft·k / ft

ϕM_n = 37.09 ft·k / ft ≥ M_u = 24.83 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2\lambda\sqrt{F'_c}\,d = 2(1.0)\sqrt{3000 \text{ psi}}(14.63 \text{ in}) = 19.23 \text{ k/ft}$

$\phi V_n = \phi V_c$ = (0.750)(19.23 k / ft) = 14.42 k / ft

ϕV_n = 14.42 k / ft ≥ V_u = 7.49 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le$ 4000 psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.15 \text{ in}$

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(14.63 \text{ in})}{(1.15 \text{ in})/(0.850)} - 1\right] = 0.0294$

ε_t = 0.0294 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 37.09 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](24.83 ft·k / ft) = 33.11 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\,s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\,s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\frac{0.0018(60000)}{f_y} = \frac{0.0018(60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0019 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(24.83 \text{ ft·k/ft})}{(37.09 \text{ ft·k/ft})} = 0.6694$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (3 in) + (0.75 in) / 2 = 3.38 in

c_b = 3.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(3.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t\psi_e\psi_s}{2.5}\right)d_b = \left[\frac{3.}{40}\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.75 \text{ in}) = 19.72 \text{ in}$

Factoring l_d by the excess reinforcement ratio (0.6694) per 25.4.10: l_d = 13.2 in

l_{d_prov} = 33 in ≥ l_d = 13.2 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 3.48 ft·k / ft < M_{stem} = 26.02 ft·k / ft

M_u = 3.48 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61$ in

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})[(15.69 \text{ in}) - (0.61 \text{ in})/2] = 21.46$ ft·k / ft

ϕM_n = 21.46 ft·k / ft ≥ M_u = 3.48 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}(15.69 \text{ in}) = 20.62$ k / ft

$\phi V_n = \phi V_c = (0.750)(20.62 \text{ k/ft}) = 15.47$ k / ft

ϕV_n = 15.47 k / ft ≥ V_u = 3.47 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61$ in

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(15.69 \text{ in})}{(0.61 \text{ in})/(0.850)} - 1\right] = 0.0628$

ε_t = 0.0628 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 21.46 ft·k / ft ≥ $(4/3) M_u$ = [4 / 3](3.48 ft·k / ft) = 4.63 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0019 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(3.48 \text{ ft·k/ft})}{(21.46 \text{ ft·k/ft})} = 0.1620$ (ratio to represent excess reinforcement)

ψ_t = 1.30 (more than 12 inches cast below - 15.38 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.63 in) / 2 = 2.31 in

c_b = 2.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 3.70$

$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\frac{(1.30)(1.0)(0.80)}{2.5}\right](0.63 \text{ in}) = 21.36$ in

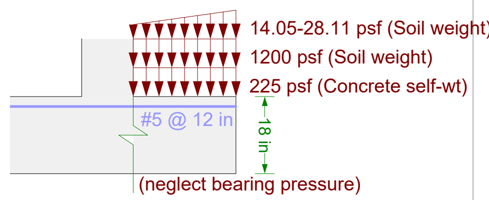
Factoring l_d by the excess reinforcement ratio (0.1620) per 25.4.10: l_d = 3.46 in

12 inch minimum controls

l_{d_prov} = 70 in ≥ l_d = 12 in ✓

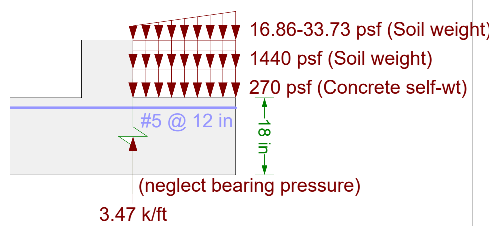
Heel Unfactored Loads

Unfactored Loads



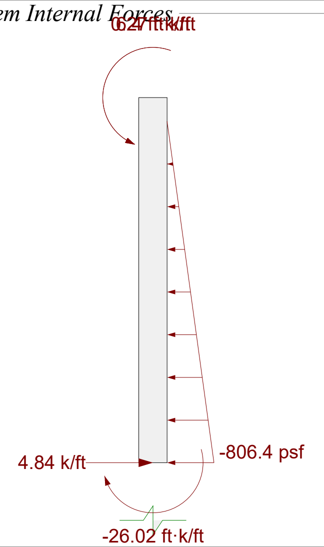
Heel Factored Loads

1.2D + 1.6L + 1.6H

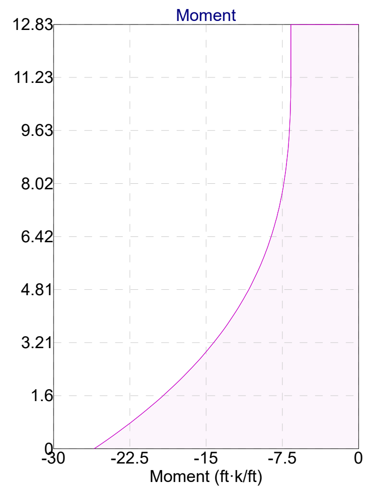


Stem Forces [1.2D + 1.6L + 1.6H]

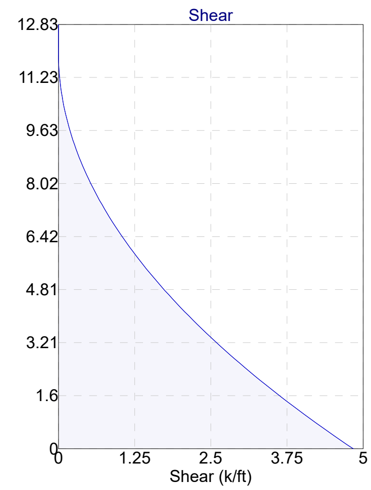
Stem Internal Forces



Stem Internal Forces



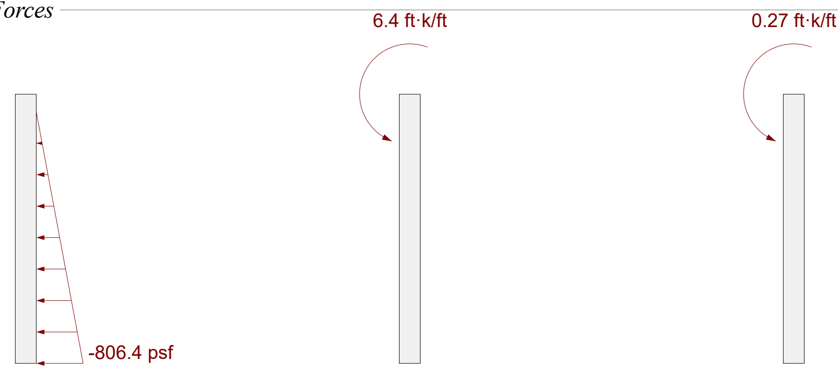
Stem Internal Forces



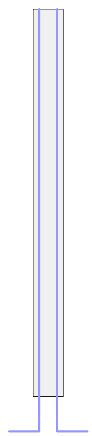
Stem Joint Force Transfer

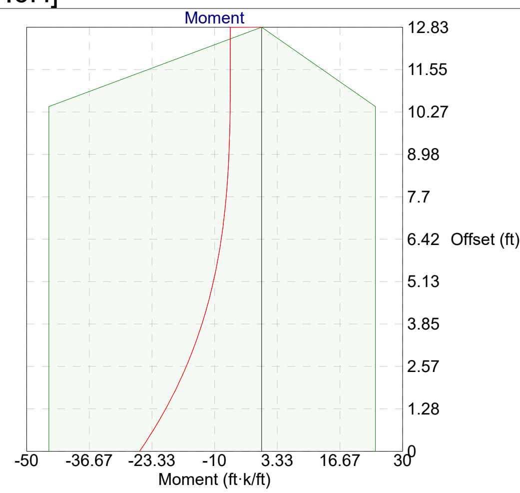
Location	Force
@ stem base	4.84 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]





Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 45.28 ft·k / ft ≥ M_u = 26.02 ft·k / ft ✓

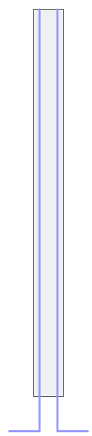
Check (ACI 318-14 11.5.5.1b) @ 10.44 ft from base

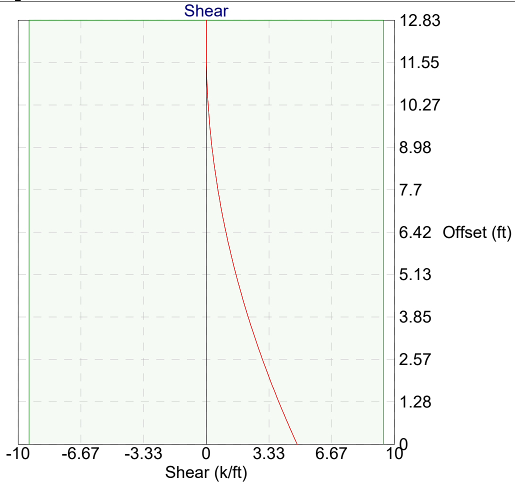
ϕM_n = 45.28 ft·k / ft ≥ M_u = 6.7 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 10.5 ft from base

ϕM_n = 44.09 ft·k / ft ≥ M_u = 6.7 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 9.43 k / ft ≥ V_u = 4.84 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 45.28$ ft·k / ft $\geq$ $(4/3)\,M_u = [4/3]\,(26.02$ ft·k / ft$) = 34.69$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 12.83 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq$ $(4/3)\,M_u = [4/3]\,(0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.1 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 2.35 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.56 \text{ in})}{(2.35 \text{ in})/(0.850)} - 1\right] = 0.0074$$

$\varepsilon_t = 0.0074 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 12.83 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.1 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 2.35 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.56 \text{ in})}{(2.35 \text{ in})/(0.850)} - 1\right] = 0.0074$$

$\varepsilon_t = 0.0074 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz}/s_{horz}}{t} = \frac{(0.4 \text{ in}^2)/(12 \text{ in})}{(12 \text{ in})} = 0.0028$$

$\rho_{t\_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0028 \geq \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(12 \text{ in}) = 36$ in

18 inch limit governs

$s_{horz} = 12 \text{ in} \leq s_{horz\_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(26.02 \text{ ft·k/ft})}{(45.28 \text{ ft·k/ft})} = 0.5746$$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r}{50\, \lambda\, \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000 \text{ psi}}}\right](0.88 \text{ in}) = 13.42 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.5746) per 25.4.10: $l_{dh} = 7.71$ in

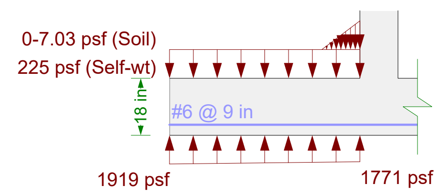
$8\,d_b = 8\,(0.88 \text{ in}) = 7.0$ (minimum limit, does not control)

$l_{dh\_prov} = 15 \text{ in} \geq l_{dh} = 7.71$ in ✓

Toe Checks [1.4D]

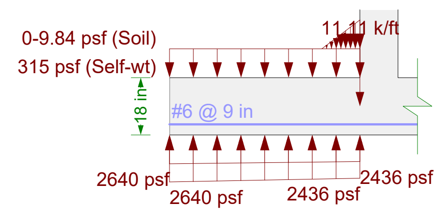
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 28.21 ft·k / ft ≥ M_{stem} = 7.47 ft·k / ft

M_u = 7.47 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})[(14.63 \text{ in}) - (1.15 \text{ in})/2] = 37.09 \text{ ft·k/ft}$

ϕM_n = 37.09 ft·k / ft ≥ M_u = 7.47 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}(14.63 \text{ in}) = 19.23 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(19.23 \text{ k/ft}) = 14.42 \text{ k/ft}$

ϕV_n = 14.42 k / ft ≥ V_u = 8.51 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$

$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(14.63 \text{ in})}{(1.15 \text{ in})/(0.850)} - 1\right] = 0.0294$

ε_t = 0.0294 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 37.09 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (7.47 ft·k / ft) = 9.96 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$

$\frac{0.0018(60000)}{f_y} = \frac{0.0018(60000)}{(60000 \text{ psi})} = 0.0018$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0019 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(7.47 \text{ ft·k/ft})}{(37.09 \text{ ft·k/ft})} = 0.2013$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (3 in) + (0.75 in) / 2 = 3.38 in

c_b = 3.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$\frac{c_b + K_{tr}}{d_b} = \frac{(3.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40}\frac{f_y}{\lambda\sqrt{F'_c}}\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.75 \text{ in}) = 19.72 \text{ in}$

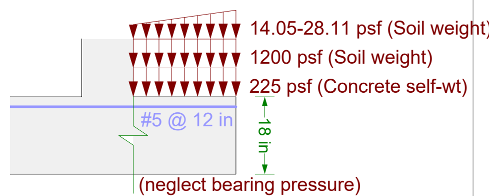
Factoring l_d by the excess reinforcement ratio (0.2013) per 25.4.10: l_d = 3.97 in

12 inch minimum controls

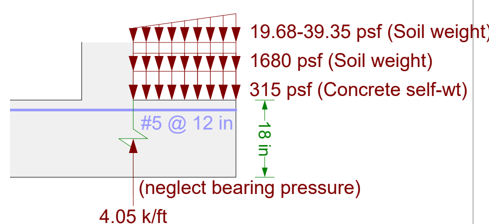
l_{d_prov} = 33 in ≥ l_d = 12 in ✓

Heel Checks [1.4D]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 4.06 ft·k / ft < M_{stem} = 7.47 ft·k / ft

M_u = 4.06 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.61 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})[(15.69 \text{ in}) - (0.61 \text{ in})/2] = 21.46 \text{ ft·k/ft}$$

ϕM_n = 21.46 ft·k / ft ≥ M_u = 4.06 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(15.69 \text{ in}) = 20.62 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(20.62 \text{ k/ft}) = 15.47 \text{ k/ft}$$

ϕV_n = 15.47 k / ft ≥ V_u = 4.05 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(15.69 \text{ in})}{(0.61 \text{ in})/(0.850)} - 1\right] = 0.0628$$

ε_t = 0.0628 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 21.46 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (4.06 ft·k / ft) = 5.41 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(18 \text{ in})(12 \text{ in})} = 0.0019$$

$$\frac{0.0018\,(60000)}{f_y} = \frac{0.0018\,(60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0019 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(4.06 \text{ ft·k/ft})}{(21.46 \text{ ft·k/ft})} = 0.1890$$ (ratio to represent excess reinforcement)

ψ_t = 1.30 (more than 12 inches cast below - 15.38 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.63 in) / 2 = 2.31 in

c_b = 2.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.30)(1.0)(0.80)}{2.5}\right](0.63 \text{ in}) = 21.36 \text{ in}$$

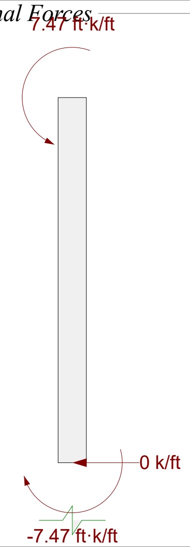
Factoring l_d by the excess reinforcement ratio (0.1890) per 25.4.10: l_d = 4.04 in

12 inch minimum controls

l_{d_prov} = 70 in ≥ l_d = 12 in ✓

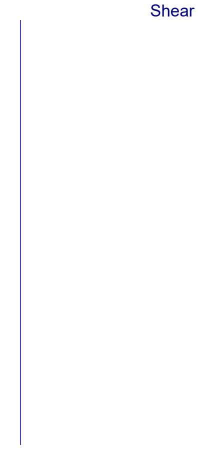
Stem Forces [1.4D]

Stem Internal Forces



Stem Internal Forces

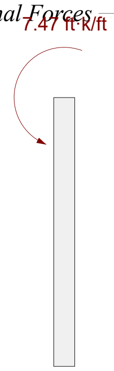
Stem Internal Forces



Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

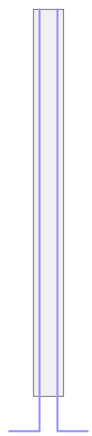
Stem Internal Forces

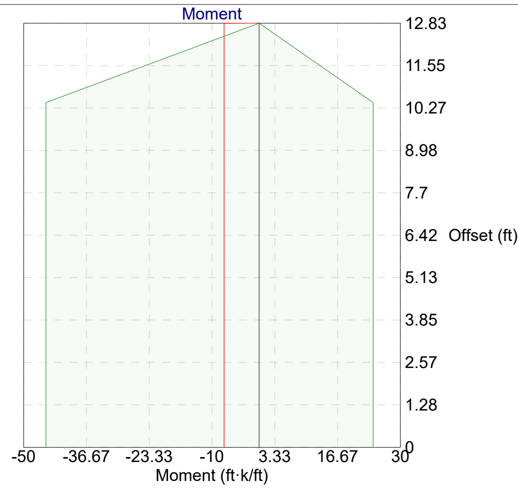


JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Stem Moment Checks [1.4D]





Check (ACI 318-14 11.5.5.1b)   @ 0 ft from base

ϕM_n = 45.28 ft·k / ft   ≥ M_u = 7.47 ft·k / ft ✓

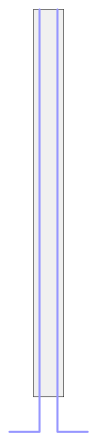
Check (ACI 318-14 11.5.5.1b)   @ 10.44 ft from base

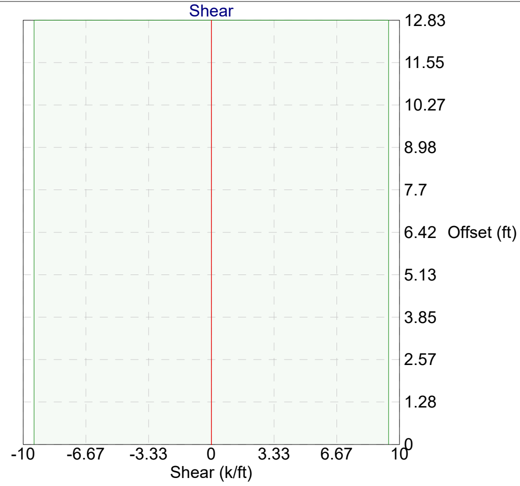
ϕM_n = 45.28 ft·k / ft   ≥ M_u = 7.47 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b)   @ 10.5 ft from base

ϕM_n = 44.09 ft·k / ft   ≥ M_u = 7.47 ft·k / ft ✓

Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 45.28 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (7.47 ft·k / ft) = 9.96 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 12.83 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.1\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)} = 2.35\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.56\ in)}{(2.35\ in)/(0.850)} - 1\right] = 0.0074$$

ε_t = 0.0074 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 12.83 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \leq$ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.1\ in^2/in)(60000\ psi)}{0.85\ (3000\ psi)} = 2.35\ in$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(9.56\ in)}{(2.35\ in)/(0.850)} - 1\right] = 0.0074$$

ε_t = 0.0074 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.4\ in^2)/(12\ in)}{(12\ in)} = 0.0028$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0028 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (12 in) = 36 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(7.47\ ft \cdot k/ft)}{(45.28\ ft \cdot k/ft)} = 0.1649$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50\ (1.0)\sqrt{3000\ psi}}\right](0.88\ in) = 13.42\ in$$

Factoring l_{dh} by the excess reinforcement ratio (0.1649) per 25.4.10: l_{dh} = 2.21 in

8 d_b = 8 (0.88 in) = 7.0

8d_b minimum controls

l_{dh_prov} = 15 in ≥ l_{dh} = 7 in ✓

Tension Straps for Cross-Laminated Timber (CLT) Applications

Value-Engineered Design Options for CLT

SIMPSON Strong-Tie®

Simpson Strong-Tie® load-rated tension straps and fasteners to minimize cost and maximize speed and precision — while providing the strength, versatility and reliability you depend on.

Straps are designed to carry tension loads in a wide variety of applications.

MDCST48 — A 14-gauge high-capacity strap specifically designed to carry tension forces across a CLT floor or wall panel joint. The MDCST48 installs with Simpson Strong-Tie Strong-Drive® SDS Heavy-Duty Connector screws.

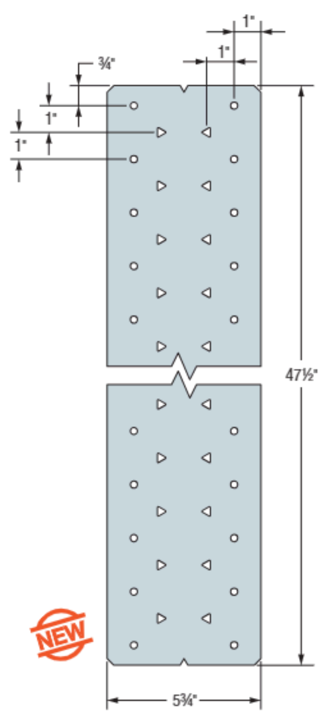
MSTC — A high-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

HRS — A 12-gauge strap with a nailing pattern designed for installation on the edge of 2x members, but also suitable for many CLT applications. The HRS416Z installs with Simpson Strong-Tie Strong-Drive SDS Heavy-Duty Connector screws.

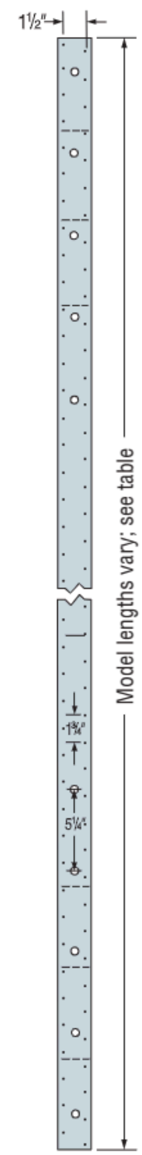
MST — A high capacity strap that uses nails and is suitable for a variety of CLT applications. Suitable for double 2x member connections or greater, and also for a variety of CLT applications.

Finish: Galvanized G90. Some products are available in ZMAX coating; see Corrosion Information at strongtie.com.

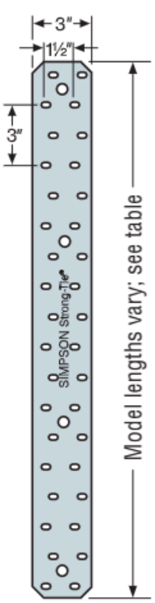
Installation: Use all specified fasteners. See General Notes.



MDCST48



MST



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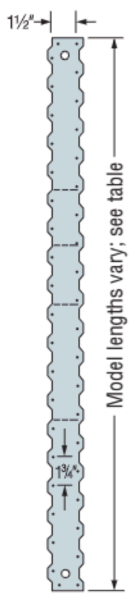

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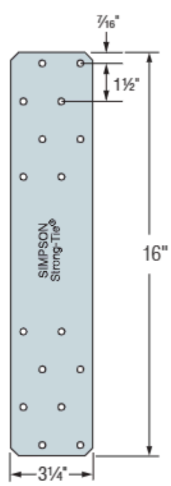


HRS416Z

SIMPSON Strong-Tie

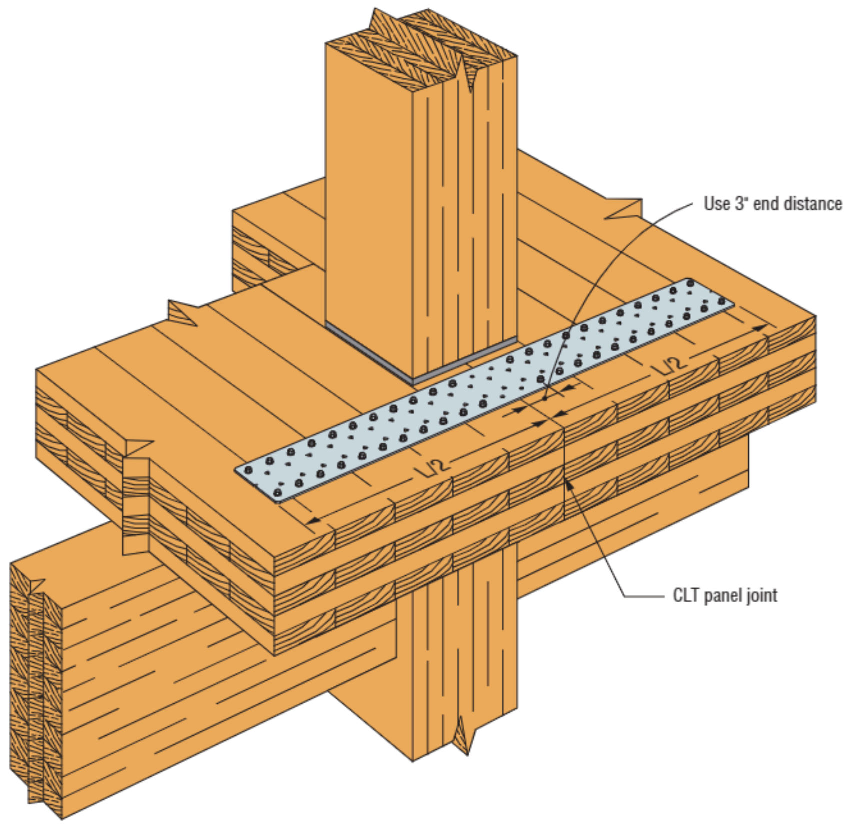
Value-Engineered Design Options for CLT

Factored Tensile Resistance Table

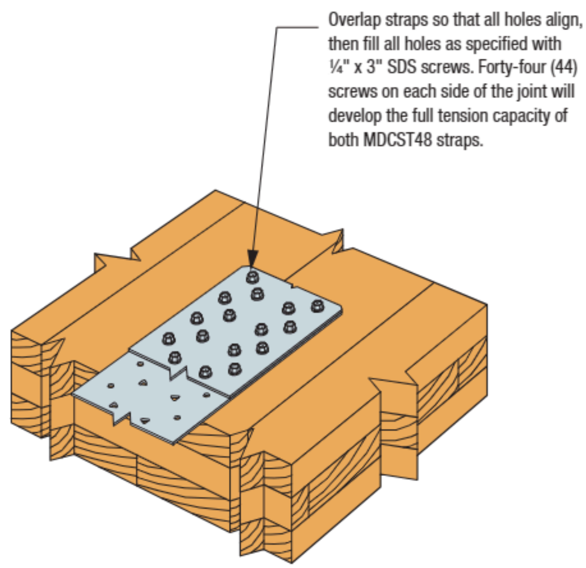
Model No.	Ga.	Dimensions (in.)		Fasteners (Total)	Factored Tensile Resistance							
					D.Fir-L				S-P-F			
					K_D = 1.00		K_D = 1.15		K_D = 1.00		K_D = 1.15	
		W	L		lb.	kN	lb.	kN	lb.	kN	lb.	kN
ST6215	16	2 1/16	16 5/16	(16) 8d x 2½"	1365	6.07	1565	6.98	1265	5.64	1455	6.49
ST6224		2 1/16	23 5/16	(24) 8d x 2½"	2245	10.00	2580	11.50	2065	9.20	2375	10.58
MSTC28		3	28¼	(32) 10d x 2½"	3850	17.14	4425	19.71	3430	15.28	3945	17.57
MSTC40		3	40¼	(48) 10d x 2½"	5770	25.71	6640	29.57	5145	22.92	5915	26.36
MSTC52		3	52¼	(54) 10d x 2½"	6495	28.93	6940	30.87	5790	25.79	6655	29.65
MSTC66	14	3	65¾	(66) 10d x 2½"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
MSTC78		3	77¾	(66) 10d x 2½"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
ST6236		2 1/16	33 13/16	(36) 8d x 2½"	3550	15.82	4085	18.19	3105	13.84	3575	15.91
MDCST48		5¾	47½	(44) ¼" x 3" SDS	12350	55.00	14200	63.26	11320	50.43	13020	58.00
MDCST48 (Double/ Overlapped)		5¾	47½	(88) ¼" x 3" SDS	34670	154.42	35505	157.94	31110	138.57	35505	157.94
HRS416Z	12	3¼	16	(16) ¼" x 1½" SDS	3095	13.80	3560	15.87	2920	13.00	3355	14.95
MST27		2 1/16	27	(26) 8d x 2½"	2555	11.38	2940	13.09	2235	9.97	2575	11.46
MST37		2 1/16	37½	(38) 8d x 2½"	3735	16.63	4295	19.13	3270	14.57	3760	16.75
MST48		2 1/16	48	(50) 8d x 2½"	4915	21.90	5655	25.18	4305	19.17	4950	22.05
MST60	10	2 1/16	60	(64) 8d x 2½"	6290	28.03	7235	32.23	5510	24.54	6335	28.22
MST72		2 1/16	72	(78) 8d x 2½"	7670	34.16	8450	37.60	6715	29.91	7720	34.39

1. Factored resistances have been increased 15% for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. Install fasteners as specified by Designer.
3. Use half of required fasteners in each member being connected to achieve the listed resistances.
4. When using the MDCST48 as a single strap, fill only round holes. When using the MDCST48 as a double/overlapped strap, fill round and triangle holes.
5. Nails: 8d = 0.131" x 2½" long, 10d = 0.148" x 2½" long. Screws: ¼" x 1½" long SDS, ¼" x 3" long SDS. All nails shown are common wire.

Value-Engineered Design Options for CLT



Typical MDCST48 Installation for Diaphragm Chord Tension Across a Five-Ply CLT Panel Joint
(three-ply and seven-ply CLT similar; other strap options similar)



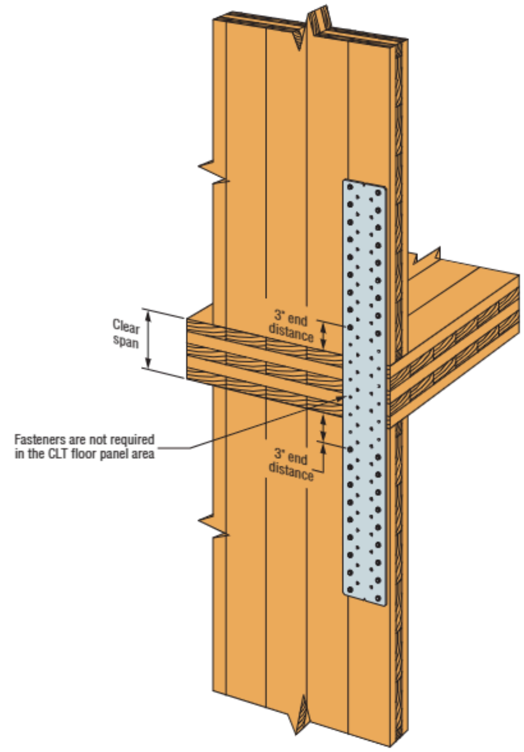
Typical Installation of Double/Overlapped MDCST48

SIMPSON Strong-Tie

Value-Engineered Design Options for CLT

Floor-to-Floor Clear Span Table

Model	CLT Plys	Clear Span (in.)	Fasteners (Total)	Factored Tensile Resistance							
				D.Fir-L				S-P-F			
				K_D = 1.00		K_D = 1.15		K_D = 1.00		K_D = 1.15	
				lb.	kN	lb.	kN	lb.	kN	lb.	kN
MSTC40	3	4.125	(44) 10d x 2½"	5290	23.53	6085	27.07	4715	20.97	5425	24.13
	5	6.875	(40) 10d x 2½"	4810	21.40	5530	24.60	4290	19.08	4930	21.93
	7	9.625	(36) 10d x 2½"	4330	19.26	4980	22.15	3860	17.17	4440	19.75
	9	12.375	(32) 10d x 2½"	3850	17.13	4425	19.68	3430	15.26	3945	17.55
MSTC52	3	4.125	(60) 10d x 2½"	6940	30.87	6940	30.87	6430	28.60	6940	30.87
	5	6.875	(56) 10d x 2½"	6735	29.96	6940	30.87	6005	26.71	6905	30.71
	7	9.625	(52) 10d x 2½"	6255	27.82	6940	30.87	5575	24.80	6410	28.51
	9	12.375	(48) 10d x 2½"	5770	25.67	6640	29.54	5145	22.89	5915	26.31
MSTC66	3	4.125	(76) 10d x 2½"	8570	38.12	8570	38.12	8145	36.23	8570	38.12
	5	6.875	(72) 10d x 2½"	8570	38.12	8570	38.12	7720	34.34	8570	38.12
	7	9.625	(68) 10d x 2½"	8330	37.05	8570	38.12	7290	32.43	8385	37.30
	9	12.375	(64) 10d x 2½"	7840	34.87	8570	38.12	6860	30.51	7890	35.10
MSTC78	3	4.125	(92) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	5	6.875	(88) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	7	9.625	(84) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	9	12.375	(80) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
MST48	3	4.125	(48) 8d x 2½"	4720	21.00	5425	24.13	4130	18.37	4750	21.13
	5	6.875	(44) 8d x 2½"	4325	19.24	4975	22.13	3790	16.86	4355	19.37
	7	9.625	(40) 8d x 2½"	3930	17.48	4520	20.11	3445	15.32	3960	17.61
	9	12.375	(36) 8d x 2½"	3540	15.75	4070	18.10	3100	13.79	3565	15.86
MST60	3	4.125	(60) 8d x 2½"	5900	26.24	6785	30.18	5165	22.98	5940	26.42
	5	6.875	(56) 8d x 2½"	5505	24.49	6330	28.16	4820	21.44	5545	24.67
	7	9.625	(52) 8d x 2½"	5110	22.73	5880	26.16	4475	19.91	5150	22.91
	9	12.375	(52) 8d x 2½"	5110	22.73	5880	26.16	4475	19.91	5150	22.91
MST72	3	4.125	(72) 8d x 2½"	7080	31.49	8070	35.90	6200	27.58	7125	31.69
	5	6.875	(72) 8d x 2½"	7080	31.49	7895	35.12	6200	27.58	7125	31.69
	7	9.625	(68) 8d x 2½"	6685	29.74	7550	33.58	5855	26.04	6730	29.94
	9	12.375	(64) 8d x 2½"	6290	27.98	7235	32.18	5510	24.51	6335	28.18
MDCST48	3	4.125	(36) ¼" x 3" SDS	10105	44.95	11620	51.69	9265	41.21	10655	47.40
	5	6.875	(32) ¼" x 3" SDS	8980	39.95	10330	45.95	8235	36.63	9470	42.12
	7	9.625	(32) ¼" x 3" SDS	8980	39.95	10330	45.95	8235	36.63	9470	42.12
	9	12.375	(28) ¼" x 3" SDS	7860	34.96	9035	40.19	7205	32.05	8285	36.85
MDCST48 (Double/ Overlapped)	3	4.125	(72) ¼" x 3" SDS	28365	126.17	32620	145.10	25455	113.23	29270	130.20
	5	6.875	(64) ¼" x 3" SDS	25215	112.16	28995	128.98	22625	100.64	26020	115.74
	7	9.625	(60) ¼" x 3" SDS	23635	105.13	27185	120.92	21210	94.35	24390	108.49
	9	12.375	(56) ¼" x 3" SDS	22060	98.13	25370	112.85	19795	88.05	22765	101.26



Typical Floor-to-Floor Tie Installation with MDCST48 and SDS Heavy-Duty Connector Screws at Three-Ply CLT Walls and Five-Ply CLT Floor

(other strap options similar; other CLT ply combinations similar)

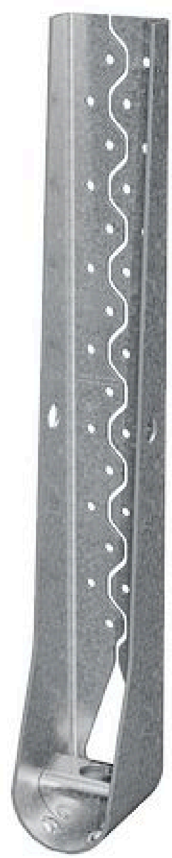
1. Factored resistances have been increased 15% for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. Install fasteners as specified by Designer.
3. Use half of required fasteners in each member being connected to achieve the listed resistances.
4. When nailing the strap over OSB/plywood, use a minimum 2-½" long nail.
5. When using the MDCST48 as a single strap, fill only round holes. When using the MDCST48 as a double/ overlapped strap, fill round and triangle holes.
6. Nails: 8d = 0.131" x 2½" long, 10d = 0.148" x 2½" long. Screws: ¼" x 3" long SDS. All nails shown are common wire.

F-C-CLTNSTCAN19 7/19 exp. 6/24

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

HDU Holdown

Select Media



HDU11-SDS2.5

SIMPSON Strong-Tie

Product Details

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

Load Tables

The HDU is designed for use in shearwalls and braced-wall panels, as well as other lateral applications in wood construction. The HDU uses Strong-Drive® SDS Heavy-Duty Connector screws which install easily, reduce fastener slip and provide a greater net section than bolts.

Key Features

- Strong-Drive SDS Heavy-Duty Connector screws are supplied with the holdown to ensure proper fasteners are used
- Predeflected seat eliminating deflection under load
- No stud bolts to countersink at openings

Material

- See table

Finish

- Galvanized

Installation

- See General Notes for Holdowns and Tension Ties.
- The HDU requires no additional washer.
- Strong-Drive SDS Heavy-Duty Connector screws install best with a low-speed, high-torque drill equipped with a 3/8" hex-head driver.
- Fasteners and crescent washer are included with the holdowns. To order replacements, order part no. SDS25212-HDU_. (Fill in the size needed, e.g., HDU2.)

Related Links

- Wood Construction Connectors Technical and Installation Notes
- General Notes for Holdowns and Tension Ties
- Holdown Selector Software

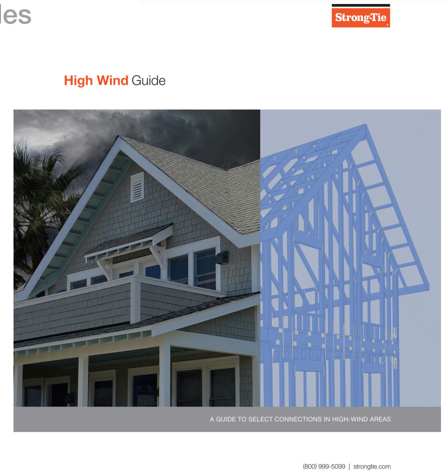
Catalog Pages

- C-C-2024 (Wood Construction Connectors), pages 54–55

SIMPSON Strong-Tie

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

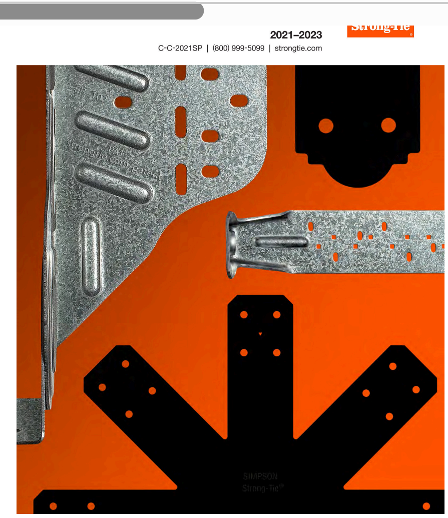
Load Tables



High Wind Guide

F-C-HWG23 — *Product Guide*

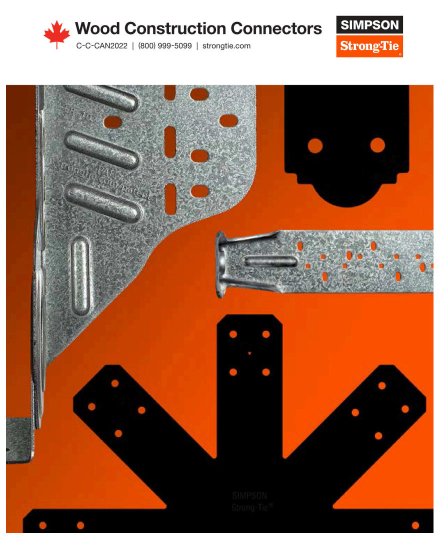
A comprehensive guide to help designers select the most appropriate connections for high-wind regions.



Conectores para la construcción con madera 2021–2023

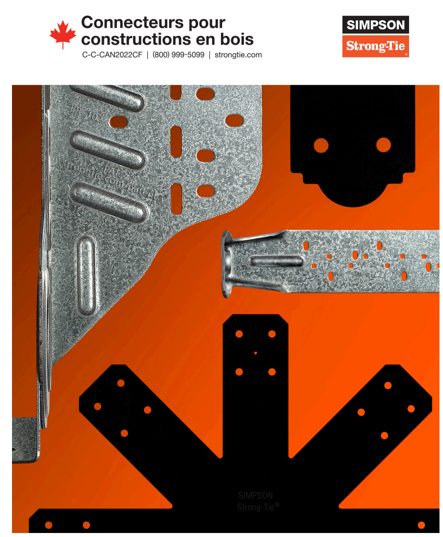
C-C-2021SP — *Catalog*

El catálogo incluye información sobre nuevos productos, así como especificaciones e instrucciones de instalación aplicables para conectores estructurales de madera a madera y de madera a concreto.



Wood Construction Connectors – Canadian Limit States Design

C-C-CAN2022 — *Catalog*



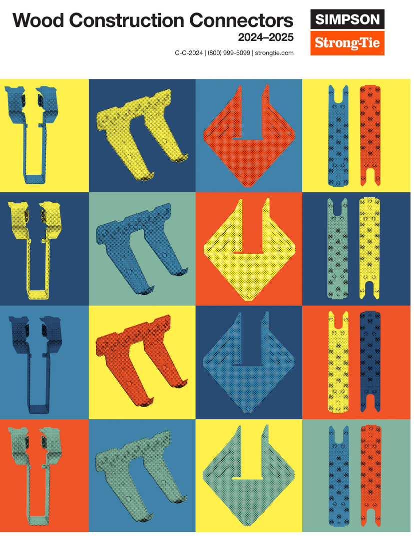
Connecteurs pour constructions en bois — Conception aux états limites canadiens

C-C-CAN2022CF — *Catalog*

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

A complete product and application catalogue including load tables and installation infor... wood-to-concrete and wood-to-masonry connectors.

Un catalogue complet sur les produits et les applications, y compris de l'information sur ... les connecteurs bois-à-bois, bois-à-béton et bois-à-maçonnerie.



Wood Construction Connectors 2024-2025

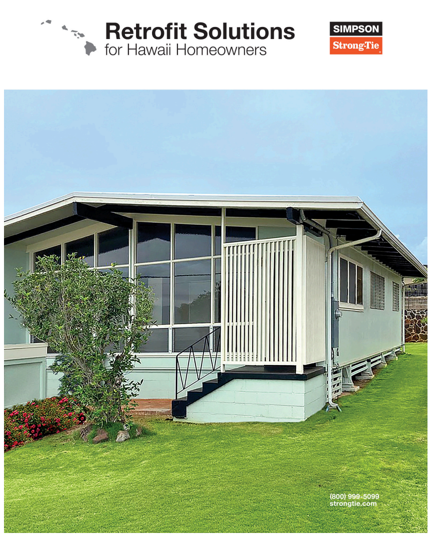
C-C-2024 — *Catalog*

A catalog including new product information as well as any applicable specification and installation instructions for wood-to-wood and wood-to-concrete structural connectors.

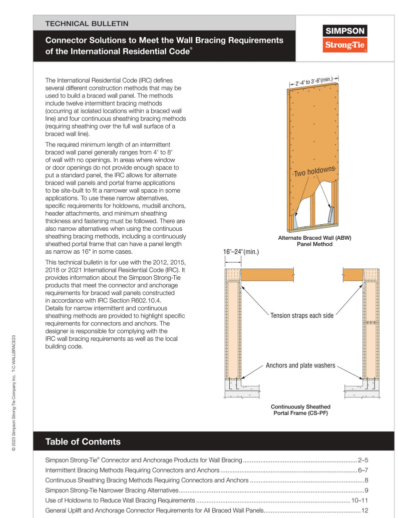
Installer's Pocket Guide

S-C-INSTALL22 — *Product Guide*

A bilingual pocket installation guide for connectors.



Retrofit Solutions for Hawaii Homeowners


TECHNICAL BULLETIN

Connector Solutions to Meet the Wall Bracing Requirements of the International Residential Code®

SIMPSON Strong-Tie

The International Residential Code (IRC) defines several different construction methods that may be used to build a braced wall panel. The methods include twelve intermittent bracing methods (occurring at isolated locations within a braced wall line) and four continuous sheathing bracing methods (requiring sheathing over the full wall surface of a braced wall line).

The required minimum length of an intermittent braced wall panel generally ranges from 4' to 8' of wall with no openings. In areas where window or door openings do not provide enough space to put a standard panel, the IRC allows for alternate braced wall panels and portal frame applications to be site-built to fit a narrower wall space in some applications. To use these narrow alternatives, specific requirements for holdowns, mudsill anchors, header attachments, and minimum sheathing thickness and fastening must be followed. There are also narrow alternatives when using the continuous sheathing bracing methods, including a continuously sheathed portal frame that can have a panel length as narrow as 16" in some cases.

This technical bulletin is for use with the 2012, 2015, 2018 or 2021 International Residential Code (IRC). It provides information about the Simpson Strong-Tie products that meet the connector and anchorage requirements for braced wall panels constructed in accordance with IRC Section R602.10.4. Details for narrow intermittent and continuous sheathing methods are provided to highlight specific requirements for connectors and anchors. The designer is responsible for complying with the IRC wall bracing requirements as well as the local building code.

2'-4" to 3'-6" (min.)
Two holdowns
Alternate Braced Wall (ABW) Panel Method

16"–24" (min.)
Tension straps each side
Anchors and plate washers
Continuously Sheathed Portal Frame (CS-PF)

Table of Contents

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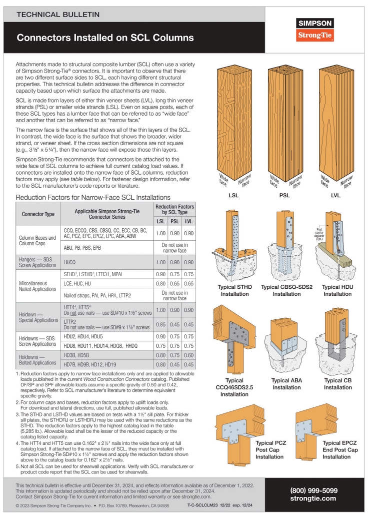
T-?-TWHRG24 — *Flier*

A flier providing recommendations on reducing the risk of seismic-, hurricane- and high-wind-induced damage to existing homes in Hawaii.

Connector Solutions to Meet the Wall Bracing Requirements of the International Residential Code

T-C-WALLBRACE23 — *Technical Bulletin*

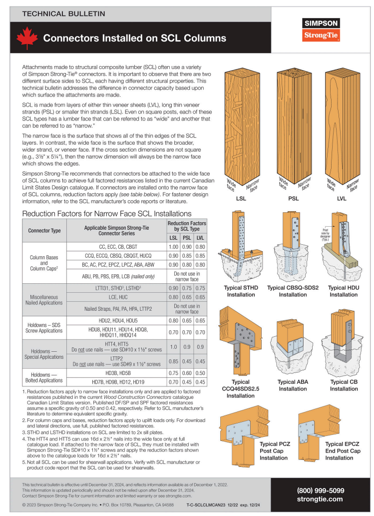
A technical bulletin providing technical information about wall bracing methods and code sections that require connectors (e.g., holdowns or straps) and anchors.



Connectors Installed on SCL Columns

T-C-SCLCLM23 — *Technical Bulletin*

A technical bulletin discussing the structural properties of the two different surface sides of SCL columns, with connector solutions (PCZ and EPC) added to the column bases and caps section.



Connectors Installed on SCL Columns

T-C-SCLCLMCAN23 — *Technical Bulletin*

A Canadian Limit States technical bulletin addressing typical installations of Simpson Strong-Tie® connectors on SCL columns as well as reduction factors for narrow-face SCL installations.

SIMPSON Strong-Tie

strongtie.com

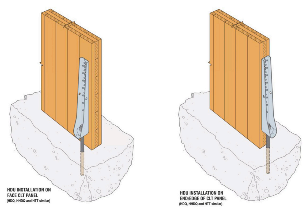
Simpson Strong-Tie Company, Inc. | 5956 West Las Positas Boulevard | Pleasanton, CA 94588 | (925) 560-9000

January 29, 2024

Re: Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT Holdowns in Cross-Laminated Timber Panel (CLT) applications installed with SD and SDS screws. The following illustrations and tables illustrate face and end/edge holdown installations on CLT and provide the Allowable Tension Loads and Deflection at maximum Allowable Load for face and end/edge installations. For holdowns installed on face of the CLT panel, full catalog loads apply. Based on testing completed at the Simpson Strong-Tie IAS accredited test lab, holdowns installed on the end/edge of 3-ply CLT panels require a reduction factor of 0.67 be applied to the catalog loads as shown in tables on page 2. Higher loads for end/edge installations require the use of ¼" x 6" long SDS screws (not supplied with the holdown). See footnote 2 in following tables for more information.

Testing was performed on 3-ply CLT panels where each layer was 1.375" thick. 3-ply CLT panels with 0.67" thick minor (tangential) layer were also evaluated with similar results. The published allowable tension loads in tables shown on page 2 are applicable to 3, 5, 7 and 9-ply CLT panels. For holdown installed on end/edge of 3-ply CLT panel, the holdown shall be centered on the CLT panel. For holdown installed on end/edge of 5-ply or greater thickness CLT panel or on face of CLT panel, minimum edge distance from edge of CLT panel to center of holdown shall be 1 ¾".



HDU INSTALLATION ON FACE CLT PANEL (HDQ, HHDQ and HTT similar)

HDU INSTALLATION ON END/EDGE OF CLT PANEL (HDQ, HHDQ and HTT similar)

Page 1 of 3

L-C-HDCLT24

Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

L-C-HDCLT24 — *Engineering Letter*

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT holdowns in cross-laminated timber (CLT) panel applications installed with SD and SDS screws.

Product Information Table

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material	Product Includes	Packaging Qty.
HDU11-SDS2.5	10	3	22 1/4	3 1/2	1 3/8	1 1/2	Zinc Galvanized, G90	(30) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU11-SDS2.5HDG	10	3	22 1/4	3 1/2	1 3/8	1 1/2	Hot-Dip Galvanized	(30) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU14-SDS2.5	7	3	25 11/16	3 1/2	1 9/16	1 9/16	Zinc Galvanized, G90	(36) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws, (1) crescent washer and (1) heavy-hex anchor nut	1
HDU14-SDS2.5HDG	7	3	25 11/16	3 1/2	1 9/16	1 9/16	Hot-Dip Galvanized	(36) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws, (1) crescent washer and (1) heavy-hex anchor nut	1
HDU2-SDS2.5	14	3	8 11/16	3 1/4	1 5/16	1 3/8	Zinc Galvanized, G90	(6) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material	Product Includes	Packaging Qty.
HDU2-SDS2.5HDG	14	3	8 11/16	3 1/4	1 5/16	1 3/8	Hot-Dip Galvanized	(6) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU4-SDS2.5	14	3	10 15/16	3 1/4	1 5/16	1 3/8	Zinc Galvanized, G90	(10) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU4-SDS2.5HDG	14	3	10 15/16	3 1/4	1 5/16	1 3/8	Hot-Dip Galvanized	(10) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
								(14) 1/4 in. x 2-1/2 in. Strong-	

Load Tables

These products are available with additional corrosion protection.

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Loa (in.)
HDU2-SDS2.5	14	3	8 11/16	3 1/4	1 5/16	1 3/8	5/8	(6) 1/4 x 2 1/2 SDS	3 x 3 1/2	3,075	2,215	0.088
HDU4-SDS2.5	14	3	10 15/16	3 1/4	1 5/16	1 3/8	5/8	(10) 1/4 x 2 1/2 SDS	3 x 3 1/2	4,565	3,285	0.114
HDU5-SDS2.5	14	3	13 3/16	3 1/4	1 5/16	1 3/8	5/8	(14) 1/4 x 2 1/2 SDS	3 x 3 1/2	5,645	4,340	0.115
HDU8-SDS2.5	10	3	16 5/8	3 1/2	1 3/8	1 1/2	7/8	(20) 1/4 x 2 1/2 SDS	3 x 3 1/2	6,765	5,820	0.11
									3 1/2 x 3 1/2	6,970	5,995	0.116
									3 1/2 x 4 1/2	7,870	6,580	0.113
HDU11-SDS2.5	10	3	22 1/4	3 1/2	1 3/8	1 1/2	1	(30) 1/4 x 2 1/2 SDS	3 1/2 x 5 1/2	9,535	8,030	0.137
									3 1/2 x 7 1/4	11,175	9,610	0.137
HDU14-SDS2.5	7	3	25 11/16	3 1/2	1 9/16	1 9/16	1	(36) 1/4 x 2 1/2 SDS	3 1/2 x 5 1/2	10,770	9,260	0.122
									3 1/2 x 7 1/4	14,390	12,375	0.177
									5 1/2 x 5 1/2	14,445	12,425	0.172

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. Fasteners: Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie® Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

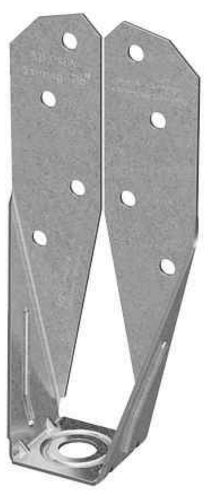
Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
HDU	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2330
	Florida Building Code	FL10441

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



DTT™
Deck Tension Tie

Strong-Drive® SDS HEAVY-DUTY CONNECTOR Screw

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

Social Contact

Load Tables

Product Use & Corrosion Info

Blogs

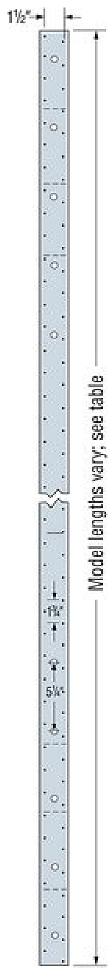
Subscribe to Our Newsletter

By checking this box, I agree to receive communications from Simpson Strong-Tie in accordance with their privacy policy.

Connect With Us

MST Medium Strap Tie

Select Media



MST

Straight Straps

This product's information may differ depending on the category of use. You are currently viewing details related to **Straight Straps**. You can also view product information related to the category: [Lateral Connectors, Ties and Straps for Cold-Formed Steel Construction]

SIMPSON Strong-Tie

Product Details

Load Tables

The MST is designed to resist tension loads in floor-to-floor and other applications. This high-capacity strap features a dense nailing pattern and can be installed with either nails or bolts. The MST is a load-rated, precut strap designed for quick installation and optimal performance. It is suitable for connecting double 2x members or greater.

Key Features

Can be used in conjunction with embedded purlin anchor (PA) straps for additional length
Designed to resist wind and seismic forces

Material

See table

Finish

Galvanized. Some products are available in stainless steel, ZMAX® coating or black powder coat (add PC to SKU); see Corrosion Information.

Installation

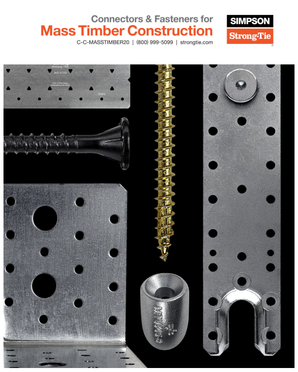
Use all specified fasteners; see General Notes

Related Links

Wood Construction Connectors Technical and Installation Notes
Cold-Formed Steel Connectors Technical and Installation Notes
General Notes for Wood Construction Connectors
General Notes for Cold-Formed Steel
Corrosion Information

Catalog Pages

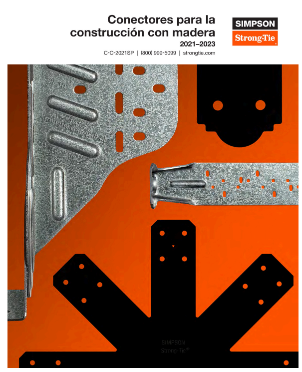
C-C-2024 (Wood Construction Connectors), pages 278–279 and page 280
C-CF-2023 (Connectors for Cold-Formed Steel Construction), pages 231–233



Connectors and Fasteners for Mass Timber Construction Catalog

C-C-MASSTIMBER20 — *Catalog*

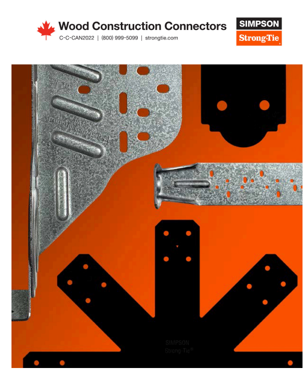
A catalog featuring connectors and fasteners for mass timber and CLT construction solutions.



Conectores para la construcción con madera 2021–2023

C-C-2021SP — *Catalog*

El catálogo incluye información sobre nuevos productos, así como especificaciones e instrucciones de instalación aplicables para conectores estructurales de madera a madera y de madera a concreto.



Wood Construction Connectors – Canadian Limit States Design

C-C-CAN2022 — *Catalog*

A complete product and application catalogue including load tables and installation information for wood-to-wood, wood-to-concrete and wood-to-masonry connectors.

Connecteurs pour constructions en bois — Conception aux états limites canadiens

C-C-CAN2022CF — *Catalog*

Un catalogue complet sur les produits et les applications, y compris de l'information sur l'installation et les tableaux de charge pour les connecteurs bois-à-bois, bois-à-béton et bois-à-maçonnerie.

Wood Construction Connectors 2024-2025

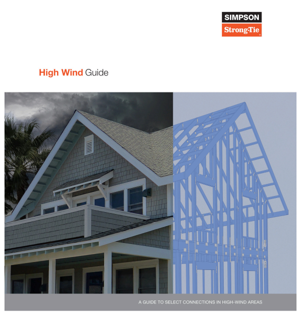
C-C-2024 — *Catalog*

A catalog including new product information as well as any applicable specification and installation instructions for wood-to-wood and wood-to-concrete structural connectors.

Connectors for Cold-Formed Steel Construction Catalog

C-CF-2023 — *Catalog*

A catalog for Connectors for Cold-Formed Steel Construction products and solutions.



High Wind Guide

F-C-HWG23 — *Product Guide*

A comprehensive guide to help designers select the most appropriate connections for high-wind regions.

Installer's Pocket Guide

S-C-INSTALL22 — *Product Guide*

A bilingual pocket installation guide for connectors.

OEM Product Line Card

S-OM-LCARD23 — *Flier*

A flier featuring the wide range of product categories that Simpson Strong-Tie offers as possible solutions within the OEM space.

Retrofit Solutions for Hawaii Homeowners

S-C-12TWHRG24 — *Flier*

A flier providing recommendations on reducing the risk of seismic-, hurricane-

SIMPSON Strong-Tie ...high-wind-induced damage to existing ...s in Hawaii.

Load Tables

Product Information Table

Model No.	Thickness (mil)	Ga.	W (in.)	L (in.)	Coating/Material	Packaging Qty.
MST27	97	12	2 1/16	27	Zinc Galvanized, G90	1
MST27HDG	97	12	2 1/16	27	Hot-Dip Galvanized	1
MST37	97	12	2 1/16	37	Zinc Galvanized, G90	1
MST37HDG	97	12	2 1/16	37	Hot-Dip Galvanized	1
MST48	97	12	2 1/16	48	Zinc Galvanized, G90	1
MST48HDG	97	12	2 1/16	48	Hot-Dip Galvanized	1
MST60	118	10	2 1/16	60	Zinc Galvanized, G90	1
MST60HDG	118	10	2 1/16	60	Hot-Dip Galvanized	1
MST72	118	10	2 1/16	72	Zinc Galvanized, G90	1

Load Tables

Floor-to-Floor Span Table

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws.

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	DF/SP Allowable Tension Loads (160)	SPF/HF Allowable Tension Loads (160)
MST37	24	(14) 0.162 x 2½	1,720	1,500
	18	(20) 0.162 x 2½	2,460	2,140
	16	(22) 0.162 x 2½	2,705	2,355
MST48	24	(26) 0.162 x 2½	3,210	2,780
	18	(32) 0.162 x 2½	3,950	3,425
	16	(34) 0.162 x 2½	4,200	3,640
MST60	30	(34) 0.162 x 2½	4,605	3,995
	24	(40) 0.162 x 2½	5,240	4,700
	18	(46) 0.162 x 2½	6,235	5,405
MST72	30	(48) 0.162 x 2½	6,505	5,640
	24	(54) 0.162 x 2½	6,730	6,345
	18	(62) 0.162 x 2½	6,730	6,475

notes below.

Load Tables

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)			DF/SP Allowable Tension Loads		SPF/HF Allowable Tension Loads	
		W	L	Nails (in.)	Bolts		Nails	Bolts	Nails	Bolts
					Qty.	Dia.	(160)	(160)	(160)	(160)
MST27	12	2 1/16	27	(30) 0.162 x 2½	4	½	3,700	2,165	3,210	2,000
MST37	12	2 1/16	37½	(42) 0.162 x 2½	6	½	5,070	3,030	4,495	2,800
MST48	12	2 1/16	48	(50) 0.162 x 2½	8	½	5,310	3,675	5,190	3,395
MST60	10	2 1/16	60	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150
MST72	10	2 1/16	72	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150

1. See General Notes for Straps and Ties.
2. Install bolts or nails as specified by designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and minimum member thickness: MST — 2 1/2".
4. Splitting may be a problem with installations on lumber smaller than 3 1/2"; either fill every nail hole with 0.148" x 1 1/2" nails or fill every other hole with 0.162" x 2 1/2" nails. Reduce the allowable load based on the size and quantity of fasteners used.
5. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Load Values with Strong-Drive® SD Connector Screws

Model No.	Fasteners (Total)	(DF/SP) Allowable Tension Loads	(SPF/HF) Allowable Tension Loads
		(160)	(160)
MST27	30-SD10112	4150	3310
MST37	40-SD10112	5070	4415
MST48	52-SD10112	5310	5035
MST60	68-SD10112	6765	6375
MST72	70-SD10112	6765	6375

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
MST	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-3096
	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2105
	Florida Building Code	FL10456

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

SIMPSON Strong-Tie

Related Products

Load Tables

HRS
Strap Tie

MSTA
Medium Strap Tie

MSTC
Medium Strap Tie with Countersunk Nail Slots

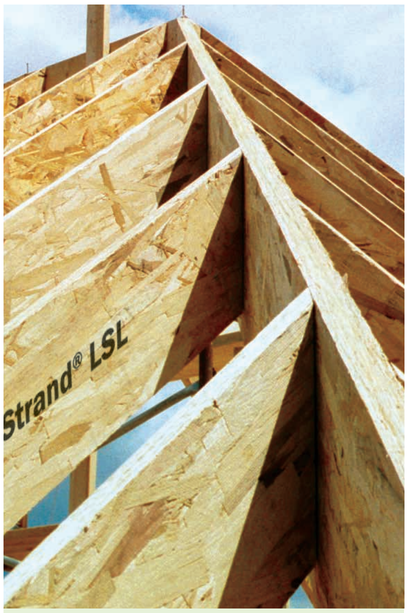
Trus Joist™ Weyerhaeuser

#TJ-9005 | SPECIFIER'S GUIDE

ROOF SYSTEM DESIGN GUIDE

Featuring Trus Joist® TimberStrand® LSL, Microllam® LVL, and Parallam® PSL

- Long-Length Rafters with Consistent Strength and Stability
- Engineered Solutions for Complex Roof Systems
- Quality Products that Speed Up Construction
- Promotes Flat Roofs, Crisp Ridge Lines, and Smooth Cathedral Ceilings
- Limited Product Warranty



The products in this guide are readily available through our nationwide network of distributors and dealers. For more information on other applications or other Trus Joist® products, contact your Weyerhaeuser representative.

Code Evaluations:
See ICC-ES ESR-1387

TABLE OF CONTENTS

SUSTAINABLE FORESTRY INITIATIVE

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SFI-00008

Why Choose Trus Joist® Roof Components?

- Long lengths allow more versatile roof design
- Engineered for strength, consistency, and durability
- Backed by limited product warranties

Many of today's homes have complex roof lines, with open vaults and varied slopes that cannot be built using plated trusses. Designs like these require structural components that are strong, long, and straight enough to give you flat planes and crisp ridge lines.

Weyerhaeuser's Trus Joist® roof system components provide the edge you need when framing complex roof designs. Our TimberStrand® LSL, Microllam® LVL, and Parallam® PSL engineered lumber products are peak performers—they're strong, durable, come in long lengths, and work together.

Get an edge on your next roof project with top-quality engineered lumber products from Weyerhaeuser—and let your craftsmanship show for years to come.

This guide features Trus Joist® roof components in the following sizes:

TimberStrand® LSL

1.5E TimberStrand® LSL:
Width: 1½"
Depths: 7¼", 9½", and 11⅞"

1.55E TimberStrand® LSL:
Width: 1¾"
Depths: 9½", 11⅞", and 14"

1.3E TimberStrand® LSL:
Width: 1½"
Depth: 5½"
Columns and posts: 2x4 2x6

Microllam® LVL

2.0E Microllam® LVL:
Width: 1¾"
Depths: 9¼", 11¼", 14", 16", and 18"

Parallam® PSL

1.8E Parallam® PSL (columns and posts):
3½" x 3½"   3½" x 5¼"   5¼" x 5¼"

This specifier's guide provides information for using Trus Joist® products in the design of a complete roof system. Individual components may be specified using this guide provided that the remaining components are properly sized and connected.

GENERAL ASSUMPTIONS

- Technical information in this guide is based on the following assumptions:
  – Roof slopes from 4:12 to 12:12.
  – Lateral design and uplift connections by others or per code.
  – Rafter and ceiling joist spacing of 24" on-center, maximum.
  – Fully sheathed roof areas.
  – Proper detailing for ventilation is the responsibility of others.
- Connections are based on NDS®, using a specific gravity of 0.5.
- A code-allowed repetitive member increase of 4% in bending moment has been included.
- 12d (0.128" x 3¼") nails may be substituted for the 16d (0.131" x 3¼") nails specified throughout this guide.
- Ceiling joists must be properly installed and connected to the heel end of the rafters to resist thrust. Contact Weyerhaeuser if any of the following conditions exist:
  – Ceiling joists are raised above the bearing plate and fastened within the span of the rafter.
  – Raised or cathedral ceiling area exceeds 15% of the total floor area or is located in a room larger than 320 sq. ft. (16' x 20').

WARNING: *This product can expose you to chemicals including wood dust which are known to the State of California to cause cancer, and methanol, which are known to the State of California to cause birth defects or other reproductive harm. Drilling, sawing, sanding or machining wood products can expose you to wood dust. Avoid inhaling wood dust or use a dust mask or other safeguards for personal protection. For more information go to www.P65Warnings.ca.gov and www.P65Warnings.ca.gov/wood.*

DESIGN PROPERTIES

Design Stresses(1) (Beam Orientation, 100% Load Duration)

			1.3E TimberStrand® LSL	1.5E TimberStrand® LSL	1.55E TimberStrand® LSL	2.0E Microllam® LVL	1.8E Parallam® PSL
Shear modulus of elasticity	G	=	81,250 psi	93,750 psi	96,875 psi	125,000 psi	112,500 psi
Modulus of elasticity(2)	E	=	1.3×10^6 psi	1.5×10^6 psi	1.55×10^6 psi	2.0×10^6 psi	1.8×10^6 psi
Adjusted modulus of elasticity(3)	E_{min}	=	660,750 psi	762,400 psi	787,815 psi	1,016,535 psi	914,880 psi
Flexural stress(4)	F_b	=	1,700 psi	2,250 psi	2,325 psi	2,600 psi	2,400 psi(10)
Tension stress(5)	F_t	=	1,300 psi	1,815 psi	1,290 psi(9)	1,895 psi	1,995 psi
Compression perpendicular to grain(6)	$F_{c\perp}$	=	710 psi	860 psi	900 psi	750 psi	545 psi(10)
Compression parallel to grain	$F_{c\parallel}$	=	1,835 psi	2,105 psi	2,170 psi	2,510 psi	2,500 psi
Horizontal shear parallel to grain	F_v	=	425 psi	505 psi	310 psi(9)	285 psi	190 psi(10)
Equivalent specific gravity(7)	SG		0.50(8)	0.50(8)	0.50(8)	0.50	0.50

(1) Unless otherwise noted, adjustment to the design stresses for duration of load are permitted in accordance with the applicable code.

(2) To properly calculate deflections for the full range of typical SCL span and loading applications, bending and shear deflection must be considered. Use the following equation for simple span, uniformly loaded beams:

$$\Delta = \frac{270\,wL^4}{Ebd^3} + \frac{28.8\,wL^2}{Ebd}$$

Where: Δ = deflection (in.) w = uniform load (plf)
L = span (feet) b = beam thickness (in.)
d = beam depth (in.) E = modulus of elasticity (psi)

For other span and loading conditions, use engineering mechanics to account for both bending and shear deflection or use ForteWEB® software.

(3) Reference modulus of elasticity for beam and column stability calculations, per NDS®.

(4) For 12" depth. For other depths, multiply f_b by the appropriate factor as follows:
– TimberStrand® LSL $\left[\frac{12}{d}\right]^{0.092}$ – Microllam® LVL $\left[\frac{12}{d}\right]^{0.136}$ – Parallam® PSL $\left[\frac{12}{d}\right]^{0.111}$.

(5) Referenced tension design values are based on a standard 4 foot length. For lengths longer than 4 foot, multiply F_t by the following adjustment (where L is length in feet):
– TimberStrand® LSL $(4/L)^{0.083}$ – Parallam® PSL $(4/L)^{0.056}$ – Microllam® LVL $(4/L)^{0.085}$.

(6) $F_{c\perp}$ shall not be increased for duration of load.

(7) For lateral connection design only.

(8) Specific gravity of 0.58 may be used for bolts installed perpendicular to face and loaded perpendicular to grain.

(9) Value accounts for large hole capabilities. See **Allowable Holes** on page 11.

(10) Value shown is for plank orientation.

Allowable Design Properties (100% Load Duration)

1½" Rafters, Ceiling Joists, and Hip and Valley Members

Design Property	1.3E TimberStrand® LSL	1.5E TimberStrand® LSL		
	Member Depth	Member Depth		
	5½"	7¼"	9½"	11⅞"
Moment (ft-lbs)	1,150	2,580	4,320	6,615
Shear (lbs)	2,340	3,660	4,795	5,995
Moment of Inertia (in.⁴)	21	48	107	209
Weight (plf)	2.4	3.3	4.4	5.4

1¾" Hip and Valley Members

Design Property	1.55E TimberStrand® LSL			2.0E Microllam® LVL				
	Member Depth			Member Depth				
	9½"	11⅞"	14"	9¼"	11¼"	14"	16"	18"
Moment (ft-lbs)	5,210	7,975	10,920	5,600	8,070	12,130	15,555	19,375
Shear (lbs)	3,435	4,295	5,065	3,075	3,740	4,655	5,320	5,985
Moment of Inertia (in.⁴)	125	244	400	115	208	400	597	851
Weight (plf)	5.2	6.5	7.7	4.7	5.7	7.1	8.2	9.2

PRODUCT STORAGE

Protect products from sun and water

CAUTION: Wrap is slippery when wet or icy

Align stickers (2x3 or larger) directly over support blocks

Use support blocks (6x6 or larger) at 10' on-center to keep bundles out of mud and water

TimberStrand® LSL, Microllam® LVL, and untreated Parallam® PSL are intended for dry-use applications

CEILING JOIST SPAN TABLE

1½" TimberStrand® LSL Ceiling Joists

On-Center Spacing	Joist Depth	Design Load			
		10 LL / 5 DL	20 LL / 10 DL	30 LL / 10 DL	40 LL / 10 DL
		Maximum Ceiling Joist Clear Span			
12"	5½"	15'-8"	12'-4"	10'-9"	9'-9"
	7¼"	21'-8"	17'-2"	14'-11"	13'-6"
	9½"	28'-6"	22'-6"	19'-7"	17'-9"
	11⅞"	35'-8"	28'-2"	24'-7"	22'-3"
16"	5½"	14'-2"	11'-2"	9'-9"	8'-10"
	7¼"	19'-8"	15'-7"	13'-6"	12'-3"
	9½"	25'-10"	20'-5"	17'-9"	16'-1"
	11⅞"	32'-4"	25'-7"	22'-3"	20'-2"
19.2"	5½"	13'-4"	10'-6"	9'-2"	8'-3"
	7¼"	18'-6"	14'-7"	12'-8"	11'-6"
	9½"	24'-4"	19'-2"	16'-8"	15'-1"
	11⅞"	30'-5"	24'-0"	20'-11"	18'-11"
24"	5½"	12'-4"	9'-9"	8'-5"	7'-8"
	7¼"	17'-2"	13'-6"	11'-9"	10'-8"
	9½"	22'-6"	17'-9"	15'-6"	14'-0"
	11⅞"	28'-2"	22'-3"	19'-4"	17'-6"

How to Use This Table

1. Determine the live load and dead load condition.
2. Determine the joist **On-Center Spacing**.
3. Scan down the appropriate **Design Load** column until you find a cell (within your on-center spacing) that meets or exceeds the span of your application.
4. Select the TimberStrand® LSL joist depth.

General Notes

- Table is based on:
  - Deflection criteria of L/360 live load and L/240 total load.
  - Uniform loads.
  - 100% load duration.
  - Minimum ceiling joist bearing length of 2", assuming a top plate $F_{c\perp}$ of 425 psi.
- Lateral support required at bearing and along ceiling joist compression edge at 48" on-center (maximum).
- Connect to rafter per **Rafter Span** tables (see pages 4–7).

No Directly Applied Ceiling

On-Center Spacing	Rafter Depth	Span/ Nailing	1½" TimberStrand® LSL Rafters														
			Roof Snow Load (PSF)														
			20 LL + 10 DL			25 LL + 10 DL			30 LL + 10 DL			35 LL + 10 DL			40 LL + 10 DL		
			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope		
			4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12
12"	5½"	Span	13'-9"			13'-1"			12'-6"			11'-11"			11'-5"		
		Nail Qty.(1)	6	4	3	7	4	3	7	4	3	8	4	3	8	5	3
	7¼"	Span	17'-8"			17'-1"			16'-6"			15'-10"			15'-4"		
		Nail Qty.(1)	8	4	3	9	5	4	10	5	4	10	6	4	11	6	4
	9½"	Span	21'-7"			20'-10"			20'-2"			19'-5"			18'-9"		
		Nail Qty.(1)	10	5	4	11	6	4	12	6	5	13	7	5	14	7	5
	11⅞"	Span	26'-11"			25'-9"			24'-9"			23'-10"			23'-1"		
		Nail Qty.(1)	12	7	5	13	7	5	14	8	6	*	8	6	*	9	6
16"	5½"	Span	12'-6"			11'-11"			11'-5"			10'-10"			10'-4"		
		Nail Qty.(1)	7	4	3	8	5	3	9	5	4	9	5	4	10	5	4
	7¼"	Span	16'-5"			15'-10"			15'-4"			14'-9"			14'-3"		
		Nail Qty.(1)	10	5	4	11	6	4	12	6	5	13	7	5	14	7	5
	9½"	Span	20'-1"			19'-5"			18'-9"			18'-0"			17'-5"		
		Nail Qty.(1)	12	6	5	13	7	5	15	8	6	*	8	6	*	9	6
	11⅞"	Span	24'-6"			23'-4"			22'-5"			21'-8"			21'-1"		
		Nail Qty.(1)	14	8	6	*	9	6	*	9	7	*	10	7	*	11	8
19.2"	5½"	Span	11'-9"			11'-3"			10'-9"			10'-2"			9'-9"		
		Nail Qty.(1)	8	5	3	9	5	4	10	5	4	11	6	4	11	6	4
	7¼"	Span	15'-8"			15'-2"			14'-8"			14'-0"			13'-5"		
		Nail Qty.(1)	11	6	4	12	7	5	14	7	5	15	8	6	15	8	6
	9½"	Span	19'-2"			18'-6"			17'-11"			17'-3"			16'-8"		
		Nail Qty.(1)	13	7	5	15	8	6	*	9	6	*	9	7	*	10	7
	11⅞"	Span	23'-0"			22'-0"			21'-2"			20'-8"			19'-10"		
		Nail Qty.(1)	*	9	6	*	10	7	*	10	7	*	11	8	*	12	8
24"	5½"	Span	10'-11"			10'-5"			10'-0"			9'-6"			9'-1"		
		Nail Qty.(1)	10	5	4	11	6	4	11	6	4	12	6	5	13	7	5
	7¼"	Span	14'-10"			14'-4"			13'-8"			13'-0"			12'-5"		
		Nail Qty.(1)	13	7	5	15	8	6	*	8	6	*	9	6	*	9	7
	9½"	Span	18'-2"			17'-6"			16'-11"			16'-3"			15'-9"		
		Nail Qty.(1)	*	9	6	*	9	7	*	10	7	*	11	8	*	12	8
	11⅞"	Span	21'-5"			20'-8"			20'-1"			19'-3"			18'-7"		
		Nail Qty.(1)	*	10	7	*	11	8	*	12	9	*	13	9	*	14	10

(1) Nail Qty. indicates required number of 16d (0.131" x 3¼") nails for heel/lap connection.
* Contact your Weyerhaeuser representative for appropriate connection information.

How to Use These Tables

1. Determine the roof snow load in pounds per square foot (psf).
2. Determine the rafter **On-Center Spacing**.
3. Scan down the appropriate **Roof Snow Load** column until you find a cell (within your on-center spacing) that meets or exceeds the span of your application.
4. Select the TimberStrand® LSL **Rafter Depth** and note the number of 16d (0.131" x 3¼") nails required at the heel and ceiling joist lap connection for your roof slope.
5. Size ceiling joists. See page 3.

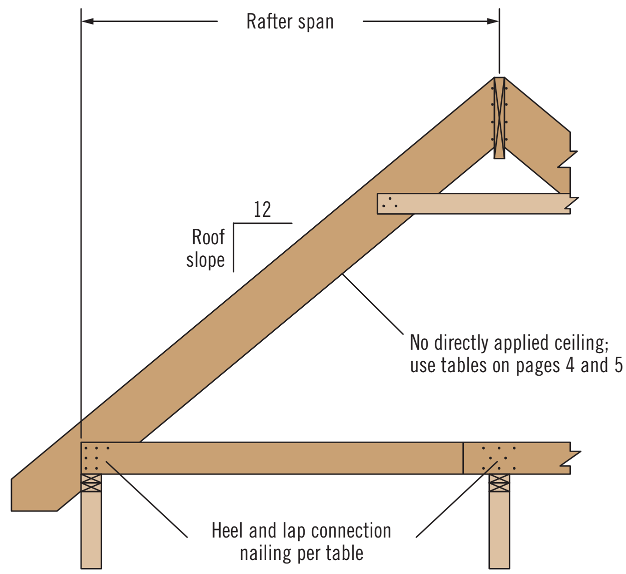
General Notes

- Tables are based on:
  - Deflection criteria as follows:
    1. For sloped rafter lengths ≤ 20': Total load is L/120
       Live load is L/180
    2. For sloped rafter lengths > 20': Total load is the greater of 2" or L/180
       Live load is the greater of 1.33" or L/240
  - Minimum rafter bearing length of 3½", assuming a top plate $F_{c\perp}$ of 425 psi.
  - Uniform loads.
  - 115% load duration.
- ***Bold italic*** values require 2x6 exterior bearing wall.
- Spans shown are the maximum horizontal distance between supports.
- Purlins may be installed (per section R802.5.1 of the 2015 IRC and section R802.4.5 of the 2018 IRC) to reduce rafter spans.
- Interpolation to determine nail quantity for other slopes is permitted.

No Directly Applied Ceiling *continued*

On-Center Spacing	Rafter Depth	Span/ Nailing	1½" TimberStrand® LSL Rafters																	
			Roof Snow Load (PSF)																	
			45 LL + 10 DL			50 LL + 10 DL			55 LL + 10 DL			60 LL + 10 DL			70 LL + 10 DL			80 LL + 10 DL		
			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope		
			4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12
12"	5½"	Span	10'-11"			10'-7"			10'-3"			10'-0"			9'-6"			9'-1"		
		Nail Qty.(1)	9	5	3	9	5	4	10	5	4	10	5	4	11	6	4	11	6	4
	7¼"	Span	14'-11"			14'-6"			14'-1"			13'-8"			13'-0"			12'-5"		
		Nail Qty.(1)	12	6	5	13	7	5	13	7	5	14	7	5	15	8	6	*	8	6
	9½"	Span	18'-2"			17'-9"			17'-4"			16'-11"			16'-3"			15'-9"		
		Nail Qty.(1)	14	8	5	15	8	6	*	9	6	*	9	6	*	10	7	*	11	7
	11⅞"	Span	22'-4"			21'-6"			20'-10"			20'-3"			19'-3"			18'-7"		
		Nail Qty.(1)	*	9	7	*	10	7	*	10	7	*	11	7	*	11	8	*	12	9
16"	5½"	Span	10'-0"			9'-7"			9'-4"			9'-1"			8'-7"			8'-3"		
		Nail Qty.(1)	10	6	4	11	6	4	11	6	4	12	6	4	13	7	5	14	7	5
	7¼"	Span	13'-8"			13'-3"			12'-10"			12'-5"			11'-10"			11'-4"		
		Nail Qty.(1)	14	8	5	15	8	6	*	8	6	*	9	6	*	9	7	*	10	7
	9½"	Span	16'-11"			16'-6"			16'-1"			15'-9"			15'-2"			14'-8"		
		Nail Qty.(1)	*	9	7	*	10	7	*	10	7	*	11	8	*	12	8	*	13	9
	11⅞"	Span	20'-3"			19'-7"			19'-0"			18'-7"			17'-10"			17'-3"		
		Nail Qty.(1)	*	11	8	*	12	8	*	12	9	*	13	9	*	14	10	*	15	11
19.2"	5½"	Span	9'-5"			9'-1"			8'-9"			8'-6"			8'-1"			7'-9"		
		Nail Qty.(1)	12	6	4	12	6	5	13	7	5	13	7	5	14	8	5	15	8	6
	7¼"	Span	12'-11"			12'-5"			12'-1"			11'-9"			11'-2"			10'-8"		
		Nail Qty.(1)	*	9	6	*	9	6	*	9	7	*	10	7	*	10	7	*	11	8
	9½"	Span	16'-2"			15'-9"			15'-5"			15'-1"			14'-6"			13'-11"		
		Nail Qty.(1)	*	11	8	*	11	8	*	12	8	*	12	9	*	14	9	*	15	10
	11⅞"	Span	19'-1"			18'-7"			18'-2"			17'-9"			***17'-1"***			***16'-6"***		
		Nail Qty.(1)	*	13	9	*	13	9	*	14	10	*	15	10	*	*	11	*	*	12
24"	5½"	Span	8'-9"			8'-5"			8'-2"			7'-11"			7'-6"			7'-2"		
		Nail Qty.(1)	13	7	5	14	7	5	15	8	5	15	8	6	*	9	6	*	9	6
	7¼"	Span	12'-0"			11'-7"			11'-2"			10'-11"			10'-4"			9'-11"		
		Nail Qty.(1)	*	10	7	*	10	7	*	11	8	*	11	8	*	12	8	*	13	9
	9½"	Span	15'-4"			14'-11"			14'-7"			14'-2"			***13'-6"***			***12'-11"***		
		Nail Qty.(1)	*	13	9	*	13	9	*	14	10	*	15	10	*	*	11	*	*	12
	11⅞"	Span	18'-0"			17'-7"			***17'-2"***			***16'-9"***			***16'-2"***			***15'-7"***		
		Nail Qty.(1)	*	15	10	*	*	11	*	*	11	*	*	12	*	*	13	*	*	14

(1) Nail Qty. indicates required number of 16d (0.131" x 3¼") nails for heel/lap connection.
* Contact your Weyerhaeuser representative for appropriate connection information.



See details on pages 12–14 for additional connection information

See **How to Use These Tables** *and* **General Notes** *on page 4*

Directly Applied Ceiling

On-Center Spacing	Rafter Depth	Span/ Nailing	1½" TimberStrand® LSL Rafters														
			Roof Snow Load (PSF)														
			20 LL + 15 DL			25 LL + 15 DL			30 LL + 15 DL			35 LL + 15 DL			40 LL + 15 DL		
			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope		
			4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12
12"	5½"	Span	11'-3"			10'-10"			10'-6"			10'-2"			9'-11"		
		Nail Qty.(1)	6	3	3	7	4	3	7	4	3	7	4	3	8	4	3
	7¼"	Span	15'-3"			14'-9"			14'-5"			14'-0"			13'-7"		
		Nail Qty.(1)	8	4	3	9	5	4	10	5	4	10	6	4	11	6	4
	9½"	Span	18'-7"			18'-1"			17'-7"			17'-2"			16'-10"		
		Nail Qty.(1)	10	5	4	11	6	4	12	6	5	13	7	5	13	7	5
	11⅞"	Span	23'-0"			22'-1"			21'-4"			20'-9"			20'-2"		
		Nail Qty.(1)	12	7	5	13	7	5	14	8	6	15	8	6	*	9	6
16"	5½"	Span	10'-3"			9'-11"			9'-7"			9'-3"			9'-0"		
		Nail Qty.(1)	7	4	3	8	4	3	8	5	3	9	5	4	9	5	4
	7¼"	Span	14'-1"			13'-7"			13'-2"			12'-9"			12'-4"		
		Nail Qty.(1)	10	5	4	11	6	4	11	6	5	12	7	5	13	7	5
	9½"	Span	17'-4"			16'-10"			16'-5"			16'-0"			15'-8"		
		Nail Qty.(1)	12	7	5	13	7	5	14	8	6	15	8	6	*	9	6
	11⅞"	Span	20'-10"			20'-1"			19'-5"			18'-11"			18'-6"		
		Nail Qty.(1)	14	8	6	*	9	6	*	9	7	*	10	7	*	10	7
19.2"	5½"	Span	9'-8"			9'-4"			9'-0"			8'-9"			8'-6"		
		Nail Qty.(1)	8	4	3	9	5	4	9	5	4	10	5	4	11	6	4
	7¼"	Span	13'-3"			12'-9"			12'-4"			12'-0"			11'-8"		
		Nail Qty.(1)	11	6	5	12	7	5	13	7	5	14	7	5	15	8	6
	9½"	Span	16'-6"			16'-1"			15'-8"			15'-4"			15'-0"		
		Nail Qty.(1)	14	7	6	15	8	6	*	9	6	*	9	7	*	10	7
	11⅞"	Span	19'-8"			18'-11"			18'-6"			18'-0"			17'-8"		
		Nail Qty.(1)	*	9	6	*	10	7	*	10	7	*	11	8	*	12	8
24"	5½"	Span	9'-0"			8'-8"			8'-4"			8'-1"			7'-10"		
		Nail Qty.(1)	9	5	4	10	5	4	11	6	4	11	6	4	12	6	5
	7¼"	Span	12'-4"			11'-10"			11'-6"			11'-1"			10'-10"		
		Nail Qty.(1)	13	7	5	14	7	5	15	8	6	*	9	6	*	9	6
	9½"	Span	15'-8"			15'-2"			14'-10"			14'-6"			14'-1"		
		Nail Qty.(1)	*	9	6	*	10	7	*	10	7	*	11	8	*	12	8
	11⅞"	Span	18'-5"			17'-11"			17'-6"			17'-1"			16'-8"		
		Nail Qty.(1)	*	10	8	*	11	8	*	12	9	*	13	9	*	14	10

(1) Nail Qty. indicates required number of 16d (0.131" x 3¼") nails for heel/lap connection.
* Contact your Weyerhaeuser representative for appropriate connection information.

How to Use These Tables

1. Determine the roof snow load in pounds per square foot (psf).
2. Determine the rafter **On-Center Spacing**.
3. Scan down the appropriate **Roof Snow Load** column until you find a cell (within your on-center spacing) that meets or exceeds the span of your application.
4. Select the TimberStrand® LSL **Rafter Depth** and note the number of 16d (0.131" x 3¼") nails required at the heel connection for your roof slope.
5. Size ceiling joists. See page 3.

General Notes

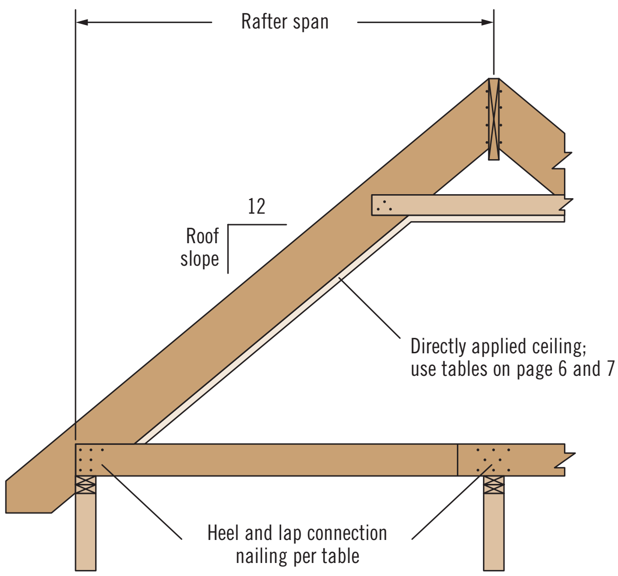
- Tables are based on:
  - Deflection criteria:
    1. For sloped rafter lengths ≤ 20': Total load is L/180
       Live load is L/240
    2. For sloped rafter lengths > 20': Total load is the greater of 1.33" or L/240
       Live load is the greater of 1" or L/360
  - Minimum rafter bearing length of 3½", assuming a top plate $F_{c\perp}$ of 425 psi.
  - Uniform loads.
  - 115% load duration.
- ***Bold italic*** values require 2x6 exterior bearing wall.
- Lateral support required at bearing and along rafter compression edge at 12" on-center maximum.
- Spans shown are the maximum horizontal distance between supports.
- Purlins may be installed (per section R802.5.1 of the 2015 IRC and section R802.4.5 of the 2018 IRC) to reduce rafter spans.
- Interpolation to determine nail quantity for other slopes is permitted.

Directly Applied Ceiling *continued*

On-Center Spacing	Rafter Depth	Span/ Nailing	1½" TimberStrand® LSL Rafters																	
			Roof Snow Load (PSF)																	
			45 LL + 15 DL			50 LL + 15 DL			55 LL + 15 DL			60 LL + 15 DL			70 LL + 15 DL			80 LL + 15 DL		
			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope			Roof Slope		
			4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12	4:12	8:12	12:12
12"	5½"	Span	9'-8"			9'-5"			9'-2"			9'-0"			8'-7"			8'-3"		
		Nail Qty.(1)	8	5	3	9	5	3	9	5	4	10	5	4	10	6	4	11	6	4
	7¼"	Span	13'-3"			12'-11"			12'-8"			12'-5"			11'-10"			11'-4"		
		Nail Qty.(1)	12	6	5	12	7	5	13	7	5	13	7	5	14	8	5	15	8	6
	9½"	Span	16'-6"			16'-3"			15'-11"			15'-8"			15'-2"			14'-8"		
		Nail Qty.(1)	14	8	6	15	8	6	*	9	6	*	9	6	*	10	7	*	10	7
	11⅞"	Span	19'-7"			19'-2"			18'-10"			18'-6"			17'-10"			17'-3"		
		Nail Qty.(1)	*	9	7	*	10	7	*	10	7	*	11	7	*	11	8	*	12	9
16"	5½"	Span	8'-9"			8'-7"			8'-4"			8'-2"			7'-10"			7'-6"		
		Nail Qty.(1)	10	5	4	10	6	4	11	6	4	11	6	4	12	7	5	13	7	5
	7¼"	Span	12'-1"			11'-9"			11'-6"			11'-3"			10'-9"			10'-3"		
		Nail Qty.(1)	14	7	5	15	8	6	15	8	6	*	8	6	*	9	6	*	10	7
	9½"	Span	15'-4"			15'-1"			14'-10"			14'-7"			14'-0"			13'-5"		
		Nail Qty.(1)	*	9	7	*	10	7	*	10	7	*	11	8	*	12	8	*	13	9
	11⅞"	Span	18'-1"			17'-10"			17'-6"			17'-3"			16'-8"			16'-1"		
		Nail Qty.(1)	*	11	8	*	12	8	*	12	9	*	13	9	*	14	10	*	15	10
19.2"	5½"	Span	8'-3"			8'-1"			7'-11"			7'-9"			7'-4"			7'-1"		
		Nail Qty.(1)	11	6	4	12	6	5	12	7	5	13	7	5	14	7	5	14	8	5
	7¼"	Span	11'-4"			11'-1"			10'-10"			10'-7"			10'-1"			9'-8"		
		Nail Qty.(1)	*	8	6	*	9	6	*	9	6	*	9	7	*	10	7	*	11	8
	9½"	Span	14'-8"			14'-5"			14'-1"			13'-10"			13'-2"			12'-7"		
		Nail Qty.(1)	*	11	8	*	11	8	*	12	8	*	12	9	*	13	9	*	14	10
	11⅞"	Span	17'-4"			17'-0"			16'-9"			16'-5"			***15'-11"***			***15'-4"***		
		Nail Qty.(1)	*	13	9	*	13	9	*	14	10	*	15	10	*	*	11	*	*	12
24"	5½"	Span	7'-8"			7'-6"			7'-4"			7'-2"			6'-10"			6'-7"		
		Nail Qty.(1)	13	7	5	13	7	5	14	7	5	15	8	6	*	8	6	*	9	6
	7¼"	Span	10'-6"			10'-3"			10'-1"			9'-10"			9'-5"			9'-0"		
		Nail Qty.(1)	*	9	7	*	10	7	*	10	7	*	11	8	*	12	8	*	12	9
	9½"	Span	13'-9"			13'-5"			13'-1"			12'-10"			12'-3"			***11'-9"***		
		Nail Qty.(1)	*	12	9	*	13	9	*	14	10	*	14	10	*	15	11	*	*	11
	11⅞"	Span	16'-4"			***16'-1"***			***15'-10"***			***15'-7"***			***15'-0"***			***14'-6"***		
		Nail Qty.(1)	*	15	10	*	*	11	*	*	12	*	*	12	*	*	13	*	*	14

(1) Nail Qty. indicates required number of 16d (0.131" x 3¼") nails for heel/lap connection.

* Contact your Weyerhaeuser representative for appropriate connection information.



See details on pages 12–14 for additional connection information

See* How to Use These Tables *and* General Notes *on page 6

Single-Span Hip Members

Member Type	No. of Plies	Member Depth	Roof Snow Load (PSF)										
			20 LL 15 DL	25 LL 15 DL	30 LL 15 DL	35 LL 15 DL	40 LL 15 DL	45 LL 15 DL	50 LL 15 DL	55 LL 15 DL	60 LL 15 DL	70 LL 15 DL	80 LL 15 DL
			Maximum Hip Spans										
1½" 1.5E TimberStrand® LSL	1	7¼"	15'-3"	14'-10"	14'-6"	14'-2"	13'-11"	13'-8"	13'-5"	13'-2"	13'-0"	12'-8"	12'-4"
		9½"	18'-7"	18'-1"	17'-8"	17'-3"	16'-11"	16'-7"	16'-4"	16'-1"	15'-10"	15'-5"	15'-0"
		11⅞"	21'-11"	21'-4"	20'-10"	20'-4"	19'-11"	19'-7"	19'-3"	18'-11"	18'-8"	18'-1"	17'-8"
	2	7¼"	17'-10"	17'-5"	17'-0"	16'-8"	16'-4"	16'-0"	15'-9"	15'-6"	15'-4"	14'-11"	14'-6"
		9½"	21'-9"	21'-3"	20'-9"	20'-3"	19'-11"	19'-7"	19'-3"	18'-11"	18'-8"	18'-2"	17'-8"
		11⅞"	25'-7"	25'-0"	24'-5"	23'-11"	23'-5"	23'-0"	22'-8"	22'-3"	21'-11"	21'-4"	20'-10"
	3	7¼"	19'-6"	19'-1"	18'-7"	18'-3"	17'-11"	17'-7"	17'-4"	17'-0"	16'-10"	16'-4"	16'-0"
		9½"	23'-9"	23'-2"	22'-8"	22'-3"	21'-10"	21'-5"	21'-1"	20'-9"	20'-6"	19'-11"	19'-6"
		11⅞"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-3"	24'-10"	24'-5"	24'-1"	23'-6"	22'-11"
1¾" 1.55E TimberStrand® LSL	1	9½"	19'-5"	18'-11"	18'-5"	18'-1"	17'-8"	17'-4"	17'-1"	16'-10"	16'-7"	16'-1"	15'-8"
		11⅞"	22'-10"	22'-3"	21'-9"	21'-3"	20'-10"	20'-6"	20'-1"	19'-9"	19'-6"	19'-0"	18'-6"
		14"	25'-8"	25'-1"	24'-6"	24'-0"	23'-6"	23'-1"	22'-8"	22'-4"	22'-0"	21'-5"	20'-10"
	2	9½"	22'-8"	22'-1"	21'-7"	21'-2"	20'-9"	20'-5"	20'-1"	19'-9"	19'-6"	18'-11"	18'-6"
		11⅞"	25'-8"	25'-8"	25'-5"	24'-11"	24'-5"	24'-0"	23'-8"	23'-3"	22'-11"	22'-4"	21'-10"
		14"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-2"	24'-7"
	3	9½"	24'-9"	24'-2"	23'-8"	23'-2"	22'-9"	22'-4"	22'-0"	21'-8"	21'-4"	20'-10"	20'-4"
		11⅞"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-6"	25'-2"	24'-6"	23'-11"
		14"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"
1¾" 2.0E Microllam® LVL	1	9¼"	20'-1"	19'-7"	19'-1"	18'-8"	18'-3"	17'-11"	17'-8"	17'-4"	17'-1"	16'-8"	16'-2"
		11¼"	23'-2"	22'-7"	22'-0"	21'-7"	21'-1"	20'-9"	20'-5"	20'-1"	19'-9"	19'-2"	18'-8"
		14"	25'-8"	25'-8"	25'-8"	25'-4"	24'-10"	24'-4"	23'-11"	23'-7"	23'-2"	22'-7"	21'-11"
		16"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-0"	23'-7"	22'-5"
		18"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	24'-11"	23'-6"	22'-5"
	2	9¼"	23'-6"	22'-11"	22'-5"	21'-11"	21'-6"	21'-1"	20'-9"	20'-5"	20'-2"	19'-7"	19'-2"
		11¼"	25'-8"	25'-8"	25'-8"	25'-4"	24'-10"	24'-5"	24'-0"	23'-7"	23'-3"	22'-8"	22'-1"
		14"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"
	3	9¼"	25'-8"	25'-1"	24'-6"	24'-0"	23'-7"	23'-2"	22'-9"	22'-5"	22'-1"	21'-6"	21'-0"
		11¼"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-6"	24'-10"	24'-3"
		14"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"

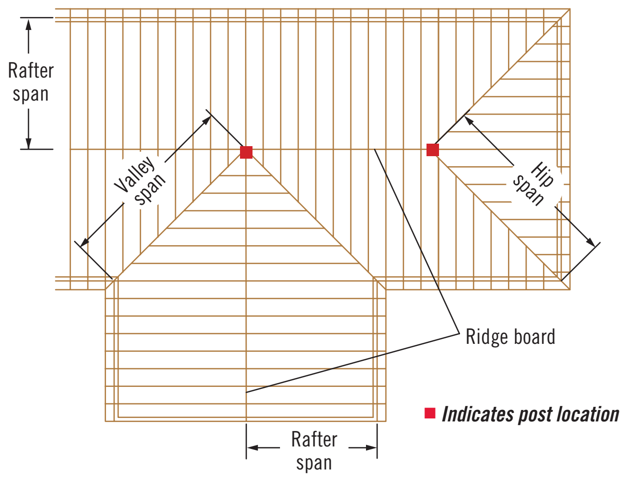
How to Use These Tables

1. Determine the roof snow load in pounds per square foot (psf).
2. Scan down the appropriate **Roof Snow Load** column until you find a cell that meets or exceeds the span of your application.
3. Select the TimberStrand® LSL or Microllam® LVL type and depth, and note the number of plies required.
4. Refer to reaction tables on page 10 to help size supporting members.

General Notes

- Tables are based on:
  - Deflection criteria of L/180 total load and L/240 live load.
  - Uniform loads.
  - 115% load duration.
  - Minimum hip/valley member bearing length of 3½", assuming a top plate $F_{c\perp}$ of 425 psi.
- Lateral support required at bearing.
- Multiple-member beam connections assume equal loads and spans from each side. For other conditions, contact your Weyerhaeuser representative.
- Spans shown are the maximum horizontal distance between supports.

Single-Span Hip/Valley



TIP: Sizing deeper beam members, rather than using multiple plies, usually results in a more economical system.

Rafter-to-Hip/Valley Span

Rafter Span	8'	9'	10'	11'	12'	13'	14'	15'	16'	17'	18'	19'	20'
Hip/Valley Span	11'-4"	12'-9"	14'-2"	15'-7"	17'-0"	18'-5"	19'-10"	21'-3"	22'-7"	24'-0"	25'-5"	26'-10"	28'-3"

- Assumes equal rafter slopes on both sides of hip/valley.
- All spans shown are horizontal spans.
- For rafter spans not shown, hip/valley span = rafter span x 1.414.

VALLEY SPAN TABLES

Single-Span Valley Beams

Member Type	No. of Plies	Member Depth	Roof Snow Load (PSF)										
			20 LL 15 DL	25 LL 15 DL	30 LL 15 DL	35 LL 15 DL	40 LL 15 DL	45 LL 15 DL	50 LL 15 DL	55 LL 15 DL	60 LL 15 DL	70 LL 15 DL	80 LL 15 DL
			Maximum Valley Spans										
1½" 1.5E TimberStrand® LSL	1	7¼"	11'-5"	11'-2"	10'-10"	10'-7"	10'-5"	10'-2"	10'-0"	9'-10"	9'-8"	9'-5"	9'-1"
		9½"	13'-11"	13'-7"	13'-3"	12'-11"	12'-8"	12'-5"	12'-2"	12'-0"	11'-10"	11'-6"	11'-1"
		11⅞"	16'-5"	16'-0"	15'-7"	15'-3"	14'-11"	14'-7"	14'-4"	14'-1"	13'-11"	13'-6"	13'-1"
	2	7¼"	13'-6"	13'-2"	12'-10"	12'-7"	12'-4"	12'-1"	11'-10"	11'-8"	11'-6"	11'-2"	10'-10"
		9½"	16'-6"	16'-0"	15'-8"	15'-4"	15'-0"	14'-8"	14'-5"	14'-3"	14'-0"	13'-7"	13'-2"
		11⅞"	19'-5"	18'-11"	18'-5"	18'-0"	17'-8"	17'-4"	17'-0"	16'-9"	16'-6"	16'-0"	15'-6"
	3	7¼"	14'-10"	14'-6"	14'-2"	13'-10"	13'-7"	13'-4"	13'-1"	12'-10"	12'-8"	12'-4"	12'-0"
		9½"	18'-2"	17'-8"	17'-3"	16'-10"	16'-6"	16'-2"	15'-11"	15'-8"	15'-5"	15'-0"	14'-7"
		11⅞"	21'-4"	20'-9"	20'-3"	19'-10"	19'-5"	19'-1"	18'-9"	18'-5"	18'-2"	17'-8"	17'-2"
1¾" 1.55E TimberStrand® LSL	1	9½"	14'-7"	14'-2"	13'-10"	13'-6"	13'-3"	13'-0"	12'-9"	12'-7"	12'-4"	12'-0"	11'-7"
		11⅞"	17'-2"	16'-8"	16'-4"	15'-11"	15'-7"	15'-4"	15'-0"	14'-9"	14'-7"	14'-2"	13'-8"
		14"	19'-5"	18'-10"	18'-5"	18'-0"	17'-7"	17'-3"	16'-11"	16'-8"	16'-5"	15'-8"	14'-11"
	2	9½"	17'-3"	16'-9"	16'-4"	16'-0"	15'-8"	15'-5"	15'-1"	14'-10"	14'-8"	14'-3"	13'-10"
		11⅞"	20'-3"	19'-9"	19'-3"	18'-10"	18'-5"	18'-1"	17'-10"	17'-6"	17'-3"	16'-9"	16'-3"
		14"	22'-11"	22'-3"	21'-9"	21'-3"	20'-10"	20'-5"	20'-1"	19'-9"	19'-5"	18'-11"	18'-4"
	3	9½"	18'-11"	18'-5"	18'-0"	17'-7"	17'-3"	16'-11"	16'-8"	16'-5"	16'-2"	15'-8"	15'-4"
		11⅞"	22'-4"	21'-9"	21'-2"	20'-9"	20'-4"	19'-11"	19'-7"	19'-4"	19'-0"	18'-6"	18'-0"
		14"	25'-2"	24'-6"	23'-11"	23'-5"	22'-11"	22'-6"	22'-2"	21'-9"	21'-5"	20'-10"	20'-4"
1¾" 2.0E Microllam® LVL	1	9¼"	15'-1"	14'-8"	14'-4"	14'-0"	13'-8"	13'-5"	13'-2"	13'-0"	12'-9"	12'-4"	11'-11"
		11¼"	17'-5"	16'-11"	16'-6"	16'-1"	15'-9"	15'-6"	15'-3"	15'-0"	14'-9"	14'-3"	13'-9"
		14"	20'-5"	19'-10"	19'-4"	18'-11"	18'-6"	18'-2"	17'-8"	17'-1"	16'-7"	15'-8"	14'-11"
		16"	22'-6"	21'-10"	20'-9"	19'-10"	19'-0"	*	*	*	*	*	*
		18"	23'-0"	*	*	*	*	*	*	*	*	*	*
	2	9¼"	17'-10"	17'-4"	16'-11"	16'-6"	16'-2"	15'-11"	15'-7"	15'-4"	15'-1"	14'-8"	14'-3"
		11¼"	20'-7"	20'-0"	19'-6"	19'-1"	18'-8"	18'-4"	18'-0"	17'-9"	17'-6"	17'-0"	16'-5"
		14"	24'-2"	23'-6"	22'-11"	22'-5"	21'-11"	21'-6"	21'-2"	20'-10"	20'-6"	19'-11"	19'-3"
		16"	25'-8"	25'-8"	25'-3"	24'-9"	24'-3"	23'-9"	23'-4"	23'-0"	22'-7"	21'-11"	20'-10"
		18"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-7"	24'-8"	23'-11"	23'-2"	*	*
	3	9¼"	19'-7"	19'-1"	18'-7"	18'-2"	17'-10"	17'-6"	17'-3"	16'-11"	16'-8"	16'-3"	15'-9"
		11¼"	22'-7"	22'-0"	21'-6"	21'-0"	20'-7"	20'-3"	19'-10"	19'-7"	19'-3"	18'-9"	18'-2"
		14"	25'-8"	25'-8"	25'-3"	24'-8"	24'-2"	23'-9"	23'-4"	22'-11"	22'-7"	22'-0"	21'-4"
		16"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-4"	24'-11"	24'-3"	23'-7"
		18"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-8"	25'-4"

* Exceeds bearing limit; contact your Weyerhaeuser representative for assistance.

See How to Use These Tables *and* General Notes *on page 8*

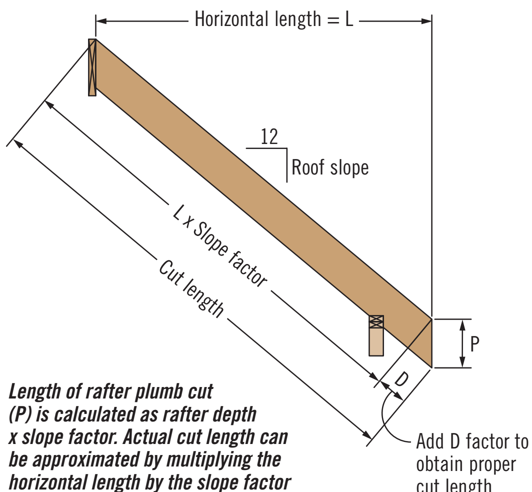
CUT LENGTH CALCULATION

Slope Factors

Type	Roof Slope								
	4:12	5:12	6:12	7:12	8:12	9:12	10:12	11:12	12:12
Rafter	1.054	1.083	1.118	1.158	1.202	1.250	1.302	1.357	1.414
Hip/Valley	1.027	1.043	1.061	1.082	1.106	1.132	1.161	1.192	1.225

D Factors

Rafter Depth	Roof Slope								
	4:12	5:12	6:12	7:12	8:12	9:12	10:12	11:12	12:12
5½"	1⅞"	2⅜"	2¾"	3¼"	3¾"	4⅛"	4⅝"	5⅛"	5½"
7¼"	2½"	3⅛"	3⅝"	4¼"	4⅞"	5½"	6⅛"	6¾"	7¼"
9¼"	3⅛"	3⅞"	4⅝"	5½"	6¼"	7"	7¾"	8½"	9¼"
9½"	3¼"	4"	4¾"	5⅝"	6⅜"	7⅛"	8"	8¾"	9½"
11¼"	3¾"	4¾"	5⅝"	6⅝"	7½"	8½"	9⅜"	10⅜"	11¼"
11⅞"	4"	5"	6"	7"	8"	9"	10"	11"	11⅞"



Length of rafter plumb cut (P) is calculated as rafter depth x slope factor. Actual cut length can be approximated by multiplying the horizontal length by the slope factor and adding the D factor.

HIP AND VALLEY REACTION TABLES

Single-Span Hip Member (lbs)

House Width	Hip Span	Location	Roof Snow Load (PSF)										
			20 LL 15 DL	25 LL 15 DL	30 LL 15 DL	35 LL 15 DL	40 LL 15 DL	45 LL 15 DL	50 LL 15 DL	55 LL 15 DL	60 LL 15 DL	70 LL 15 DL	80 LL 15 DL
			Hip Member Reaction (lbs)										
12'	8'-6"	High	0	0	0	0	0	0	0	0	0	0	0
		Low	74	83	92	101	110	119	128	137	146	164	182
16'	11'-4"	High	0	0	0	0	0	0	0	0	20	84	148
		Low	132	148	164	180	196	212	228	244	260	292	324
20'	14'-2"	High	0	0	12	62	112	162	212	262	312	412	512
		Low	206	231	256	281	306	331	356	381	406	456	506
24'	17'-0"	High	93	165	237	309	381	453	525	597	669	813	957
		Low	297	333	369	405	441	477	513	549	585	657	729
28'	19'-10"	High	308	406	504	602	700	798	896	994	1,092	1,288	1,484
		Low	404	453	502	551	600	649	698	747	796	894	992
32'	22'-8"	High	555	683	811	939	1,067	1,195	1,323	1,451	1,579	1,835	2,091
		Low	528	592	656	720	784	848	912	976	1,040	1,168	1,296
36'	25'-5"	High	835	997	1,159	1,321	1,483	1,645	1,807	1,969	2,131	2,455	2,779
		Low	668	749	830	911	992	1,073	1,154	1,235	1,316	1,478	1,640
40'	28'-3"	High	1,149	1,349	1,549	1,749	1,949	2,149	2,349	2,549	2,749	3,149	3,549
		Low	824	924	1,024	1,124	1,224	1,324	1,424	1,524	1,624	1,824	2,024

Single-Span Valley Beam (lbs)

House Width	Valley Span	Location	Roof Snow Load (PSF)										
			20 LL 15 DL	25 LL 15 DL	30 LL 15 DL	35 LL 15 DL	40 LL 15 DL	45 LL 15 DL	50 LL 15 DL	55 LL 15 DL	60 LL 15 DL	70 LL 15 DL	80 LL 15 DL
			Valley Beam Reaction (lbs)										
12'	8'-6"	High	247	277	307	337	367	397	427	457	487	547	607
		Low	495	555	615	675	735	795	855	915	975	1,095	1,215
16'	11'-4"	High	440	493	546	600	653	706	760	813	866	973	1,080
		Low	879	986	1,093	1,199	1,306	1,413	1,519	1,626	1,733	1,946	2,159
20'	14'-2"	High	687	770	854	937	1,020	1,104	1,187	1,270	1,354	1,520	1,687
		Low	1,374	1,540	1,707	1,874	2,040	2,207	2,374	2,540	2,707	3,040	3,374
24'	17'-0"	High	989	1,109	1,229	1,349	1,469	1,589	1,709	1,829	1,949	2,189	2,429
		Low	1,978	2,218	2,458	2,698	2,938	3,178	3,418	3,658	3,898	4,378	4,858
28'	19'-10"	High	1,346	1,510	1,673	1,836	2,000	2,163	2,326	2,490	2,653	2,980	3,306
		Low	2,693	3,019	3,346	3,673	3,999	4,326	4,653	4,979	5,306	5,959	6,613
32'	22'-8"	High	1,758	1,972	2,185	2,398	2,612	2,825	3,038	3,252	3,465	3,892	4,318
		Low	3,517	3,944	4,370	4,797	5,224	5,650	6,077	6,504	6,930	7,784	8,637
36'	25'-5"	High	2,226	2,496	2,766	3,036	3,306	3,576	3,846	4,116	4,386	4,926	5,466
		Low	4,451	4,991	5,531	6,071	6,611	7,151	7,691	8,231	8,771	9,851	10,931
40'	28'-3"	High	2,748	3,081	3,414	3,748	4,081	4,414	4,748	5,081	5,414	6,081	6,748
		Low	5,495	6,162	6,828	7,495	8,162	8,828	9,495	10,162	10,828	12,162	13,495

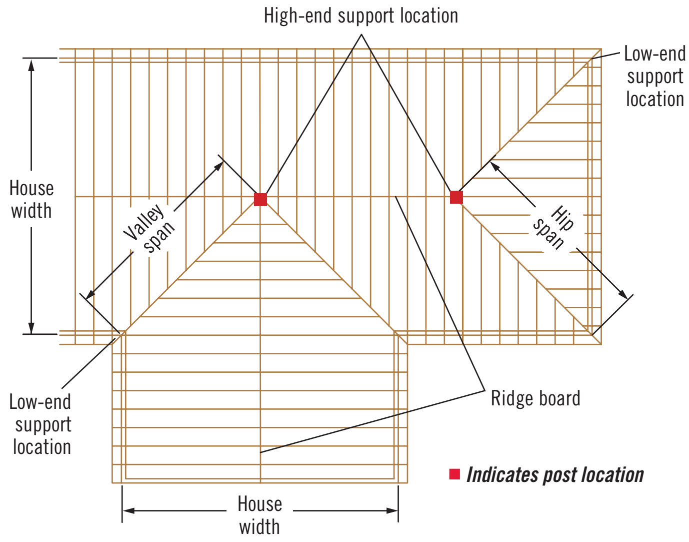
How to Use These Tables

1. Determine the roof snow load in pounds per square foot.
2. Locate the **House Width** and the corresponding **Hip** or **Valley Span** that meets or exceeds your application.
3. Identify the **High**- and **Low**-end hip member reactions (shown in pounds).
4. Where multiple hip/valley members intersect, add the corresponding reactions from each member.
5. See page 11 to size posts.

General Notes

- Tables are based on:
  - Uniform loads.
  - Equal common and jack rafter slopes.
- Interpolation between house widths to determine reactions (and spans) is permitted.
- Member weight is not included.

Single-Span Hip/Valley



POST ALLOWABLE LOADS

Allowable Axial Load (lbs)—115% Roof Snow Load

Post Length	1.3E TimberStrand® LSL Post Size				1.8E Parallam® PSL Post Size		
	2x4(1)		2x6(1)		3½" x 3½"	3½" x 5¼"	5¼" x 5¼"
	1 ply	2 ply(2)	1 ply	2 ply(2)			
6'	940	4,275	1,470	6,665	11,200	16,800	36,675
7'		3,400		5,310	9,140	13,710	32,545
8'		2,740		4,285	7,550	11,325	28,490
9'		2,250		3,520	6,320	9,480	24,835
10'		1,870		2,930	5,355	8,035	21,695
12'		1,350		2,115	3,980	5,965	16,805
14'					3,065	4,595	13,315

(1) May be angled up to 4" per foot of length.

(2) Two-ply posts to be connected with 10d (0.148" x 3") nails at 6" on-center; alternate sides and stagger.

MULTIPLE-MEMBER CONNECTIONS

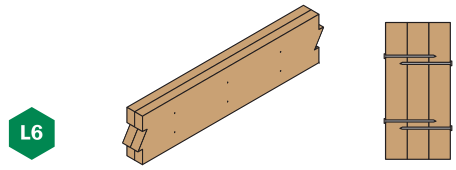
Fastener Installation Requirements

Piece Width	# of Plies	Fastener				
		Type(1)	Min. Length	Placement	# Rows	O.C. Spacing
1½" or 1¾"	2	10d nails	3"	One face	3(2)	12"
		12d–16d nails	3¼"		2(2)	
		Screws	2$\frac{15}{16}$" for 1½" members 3⅜" for 1¾" members		2	24"
	3	10d nails	3"	Both faces	3(2)	12"
		12d–16d nails	3¼"		2(2)	
		Screws	3⅜" or 3½"	Both faces	2	24"

(1) 10d nails are 0.128" diameter; 12d–16d nails are 0.148"–0.162" diameter; screws are SDS, WS, or SDW22.

(2) An additional row of nails is required with depths of 14" or greater.

- When fasteners are required on both sides, stagger fasteners on the second side so they fall halfway between fasteners on the first side.



Multiple pieces can be nailed or bolted together to form a header or beam of the required size, up to a maximum width of 5¼". Load must be applied evenly across entire beam width.

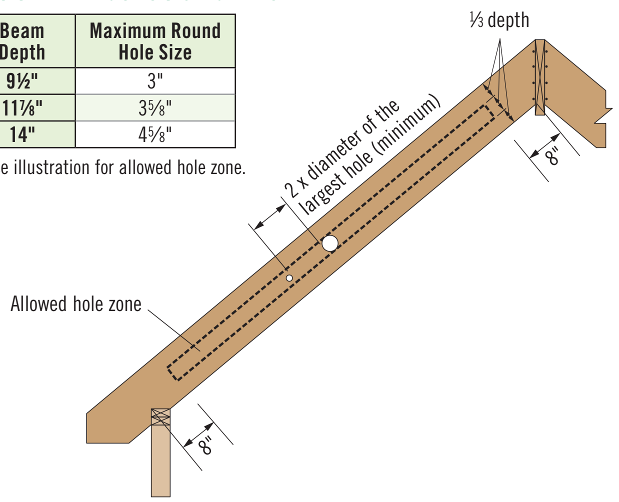
For applications that require wider members and/or uneven/side loaded beams, refer to the Trus Joist® Beam, Header and Column Specifier's Guides TJ-9000 or TJ-9020, or contact your Weyerhaeuser representative.

RAFTER, HIP AND VALLEY ALLOWABLE HOLES

1.55E TimberStrand® LSL

Beam Depth	Maximum Round Hole Size
9½"	3"
11⅞"	3⅝"
14"	4⅝"

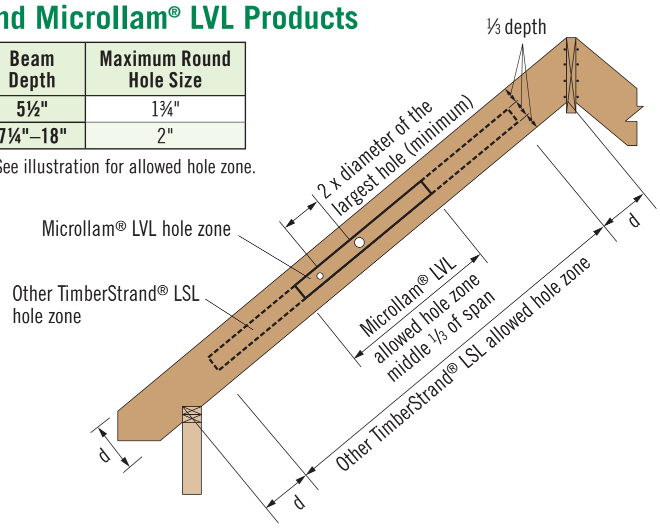
- See illustration for allowed hole zone.



All Other TimberStrand® LSL and Microllam® LVL Products

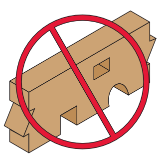
Beam Depth	Maximum Round Hole Size
5½"	1¾"
7¼"–18"	2"

- See illustration for allowed hole zone.



General Notes

- Round holes only.
- Allowed hole zone suitable for **rafters, hips, and valleys with uniform and/or concentrated loads** anywhere along the member.



DO NOT cut, notch, or drill holes in rafters, ceiling joists, hips, or valleys except at birdsmouth cut locations or as indicated in the illustrations and tables above.

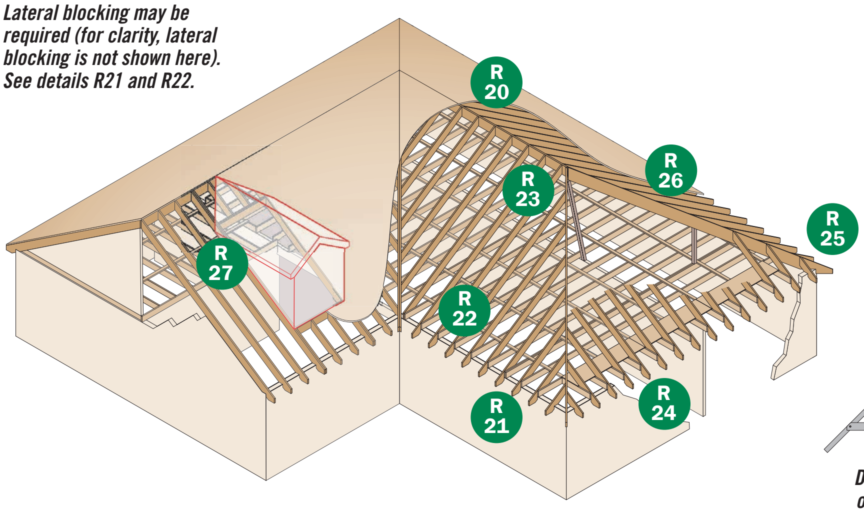
General Notes

- Allowed hole zone suitable for rafters, hips, and valleys with **uniform loads only.**
- Round holes only.
- No holes in cantilevers.

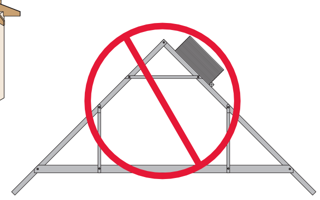
WARNING: This product can expose you to chemicals including wood dust which are known to the State of California to cause cancer, and methanol, which are known to the State of California to cause birth defects or other reproductive harm. Drilling, sawing, sanding or machining wood products can expose you to wood dust. Avoid inhaling wood dust or use a dust mask or other safeguards for personal protection. For more information go to www.P65Warnings.ca.gov and www.P65Warnings.ca.gov/wood.

TYPICAL ROOF SYSTEM

Lateral blocking may be required (for clarity, lateral blocking is not shown here). See details R21 and R22.



WARNING
Rafters and joists are unstable until laterally braced. Lack of proper bracing or insufficient roof system design can result in serious accidents.

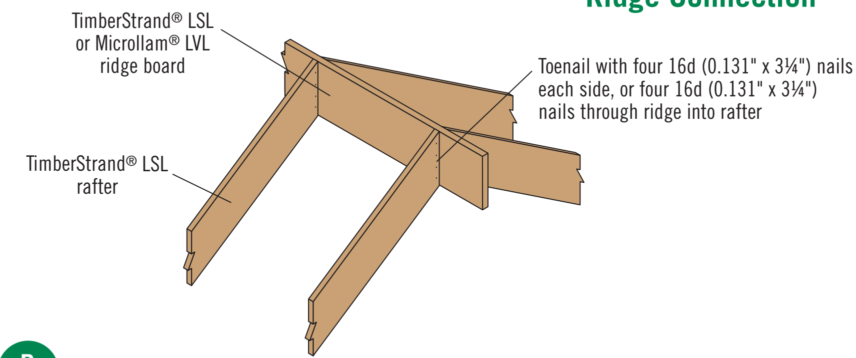


DO NOT stack building materials on unsheathed joists or rafters.

DO NOT walk on joists or rafters until braced. INJURY MAY RESULT.

ROOF FRAMING DETAILS

Ridge Connection



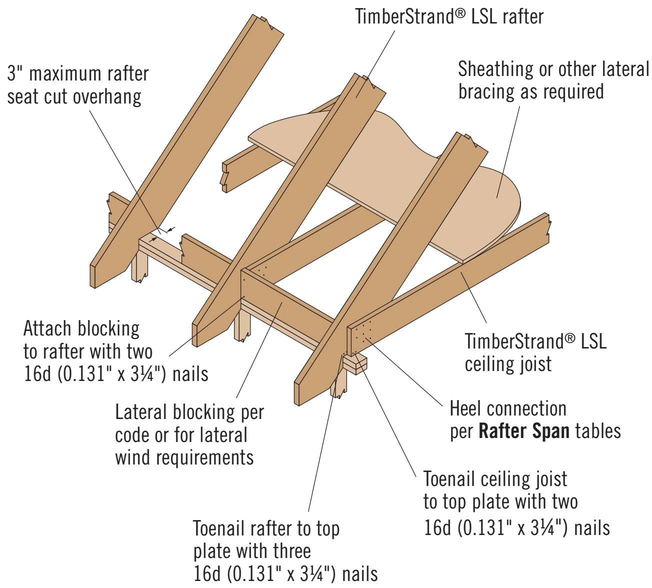
R 20

Additional connections may be required to resist wind forces in high wind zones.

Minimum Ridge Board Depth

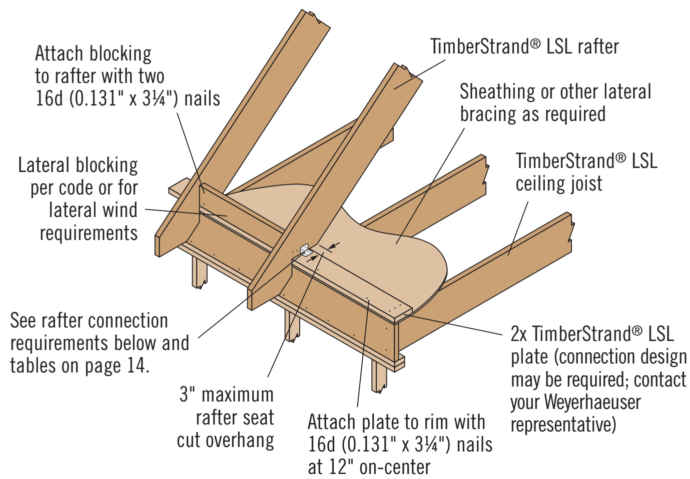
Rafter Size	Rafter Pitch		
	4:12 to 9:12	10:12 to 11:12	12:12
	Ridge Board Sizes		
2x6	2x8	2x10	2x10
2x8	2x10	2x12	2x12
2x10	14"	14"	14"
2x12	14"	16"	16"

Heel Connection (Lap)



R 21

Heel Connection (Platform)

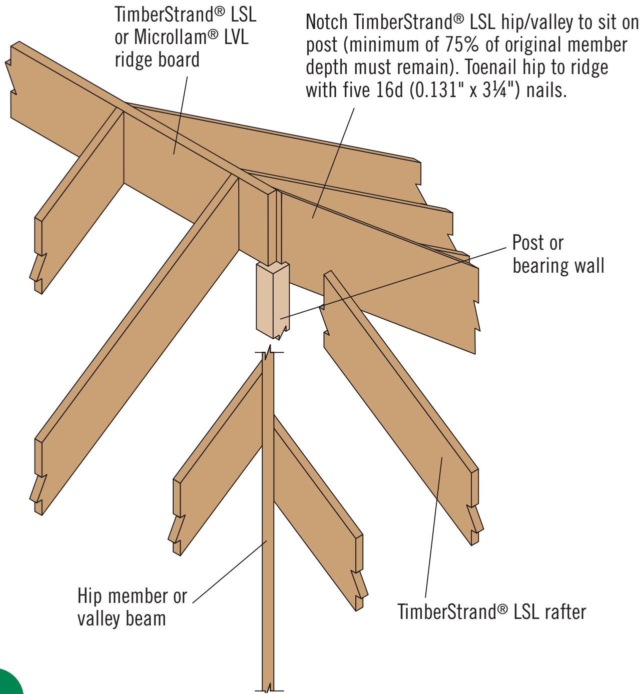


Rafter Connection for Thrust:

- Addresses lateral load only, not intended for wind uplift.
- See **Rafter Span** tables (pages 4–7) to find the nail quantity required for a lap connection. See detail R21.
  - If fewer than 8 nails are required, use one A23 or A3 clip, each side.
  - If 8–15 nails are required, use two A23 or A3 clips, each side.

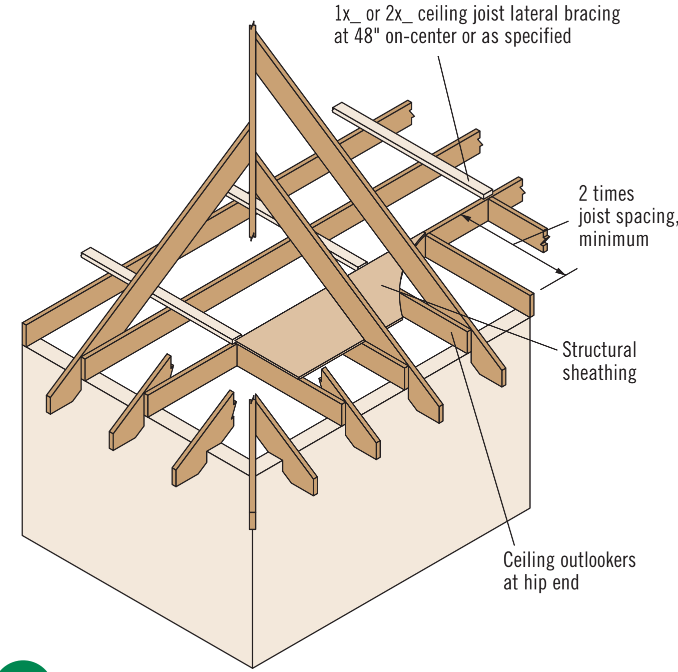
R 22

Hip/Ridge/Valley Post Detail



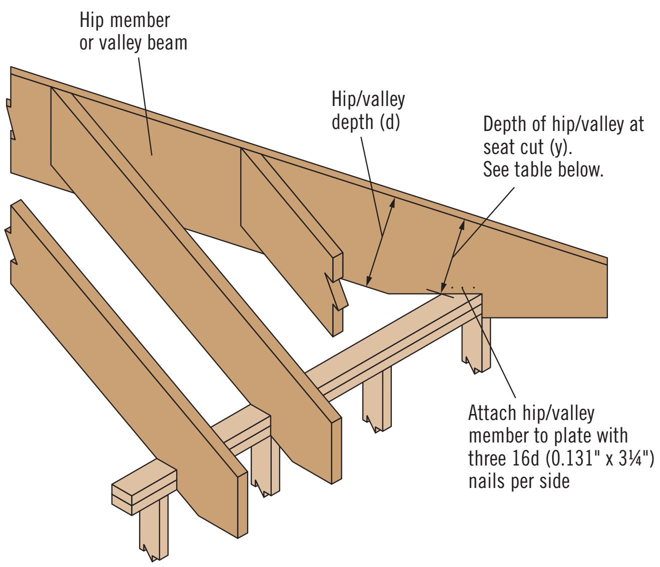
R 23

Outlooker Detail



R 24

Hip/Valley Low End



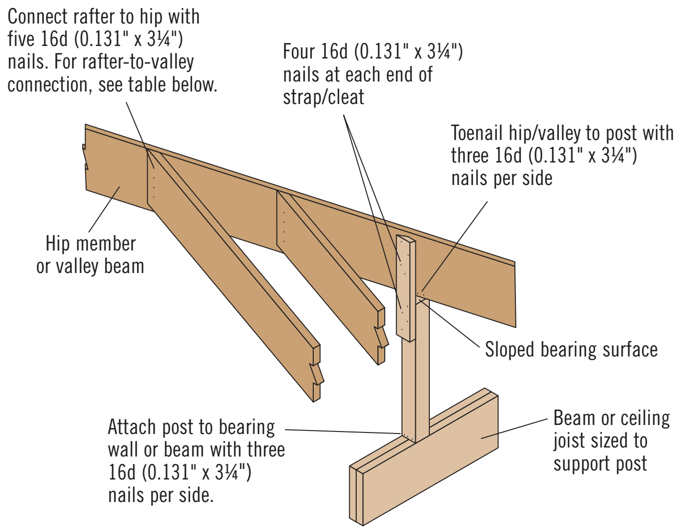
Hip/Valley Depth

Minimum Percentage of Original Hip/Valley Depth Required		
Member Type	TimberStrand® LSL	Microllam® LVL
Hip member	35%(1)	50%
Valley beam	60%	90%

(1) For net width greater than 3", minimum percentage is 30%.
- Calculate percentage as y/d x 100.

R 25

Hip/Valley Intermediate Support



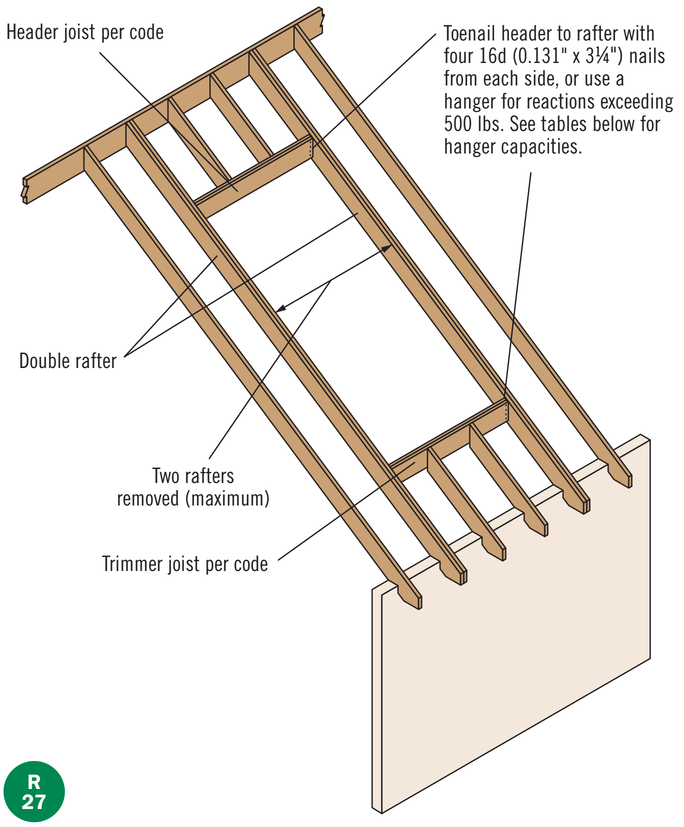
Rafter-to-Valley Connection

Rafter On-Center Spacing	Rafter Span	Roof Snow Load		
		30 LL + 15 DL	50 LL + 15 DL	80 LL + 15 DL
		Number of 16d (0.131" x 3¼") Nails Required		
16"	6'	5	5	5
	12'	5	6	8
	18'	6	9	12
	24'	8	12	*
24"	6'	5	5	7
	12'	6	8	11
	18'	9	13	*
	24'	12	*	*

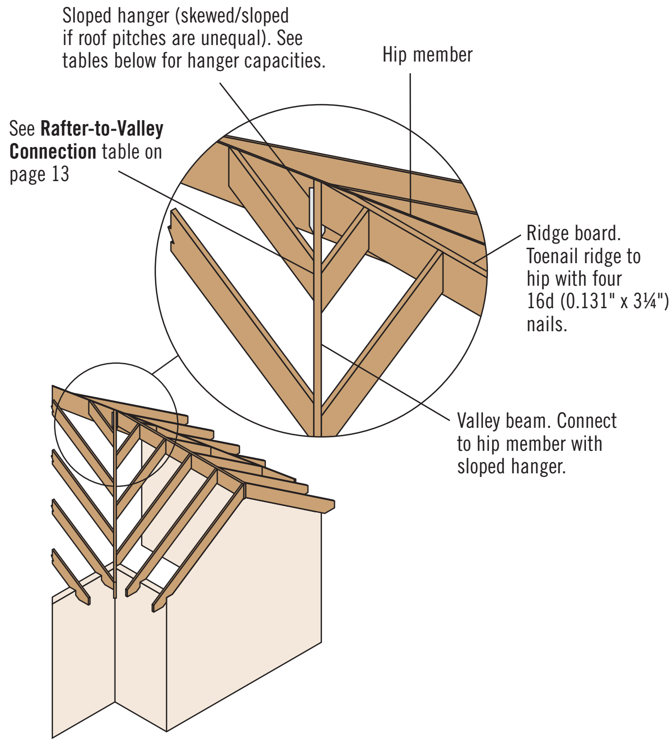
R 26

* Contact your Weyerhaeuser representative for additional connection information.

Dormer Framing



Valley-to-Hip Connection



Contact your Weyerhaeuser representative for proper sizing of a hip or valley with a point load.

FRAMING CONNECTORS

Face Mount Hangers (Detail R27)

Hanger Type	Dormer/ Trimmer Header	Depth	Hanger	Capacity (lbs)	Nailing	
					Double Rafter	Dormer Header
Simpson Strong-Tie®	1½"	5½"–7¼"	LUS26	990	10d	10d
		9¼"–11¼"	LUS210	1,530	10d	10d
	2-ply 1½"	5½"–7¼"	LUS26-2	1,170	16d	16d
		9¼"–11¼"	LUS210-2	2,075	16d	16d
USP Structural Connectors®	1½"	5½"–7¼"	JUS26	1,000	10d	10d
		9¼"–11¼"	JUS210	1,545	10d	10d
	2-ply 1½"	5½"–7¼"	JUS26-2	1,185	16d	16d
		9¼"–11¼"	JUS210-2	2,105	16d	16d

Angle Clips (Detail R22)

Hanger Type	Clip	Capacity (lbs)	Nailing	
			Top Plate	Rafter/Ceiling Joist
Simpson Strong-Tie®	A23	535	10d x 1½"	10d x 1½"
USP Structural Connectors®	A3	545	10d x 1½"	10d x 1½"

Variable Slope/Skew Hangers (Detail R28)

Hanger Type	Hip/Valley	Depth	Hanger	Capacity (lbs)		Nailing	
				Sloped	Sloped and Skewed	Hip	Valley
Simpson Strong-Tie®	1½"	All	LSSJ210LZ(1) LSSJ210RZ(1)	810	810	10d	10d x 1½"
	2-ply 1½"	All	LSSR210-2Z	2,365	1,810	16d	10d x 1½"
	1¾"	All	LSSR1.81Z	1,565	1,205	10d	10d x 1½"
	2-ply 1¾"	All	LSSR410Z	2,365	1,810	16d	10d x 1½"
USP Structural Connectors®	1½"	All	LSSH210	1,380	1,380	10d	10d x 1½"
	2-ply 1½"	All	LSSH31	3,045	1,610	16d	10d x 1½"
	1¾"	All	LSSH179	1,380	1,380	10d	10d x 1½"
	2-ply 1¾"	All	LSSH35	3,045	1,610	16d	10d x 1½"

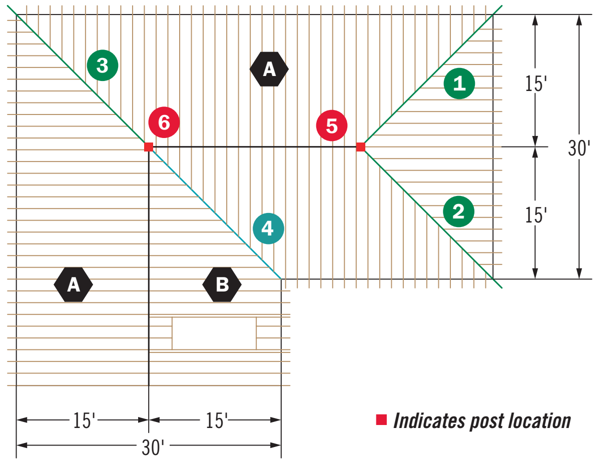
(1) LSSJ210RZ = skewed right, LSSJ210LZ = skewed left.

- LSSJ, LSSR and LSSH hangers can be field adjusted for slopes and skews of up to 45°. Additional lateral restraints are required for 16" and 18" members.

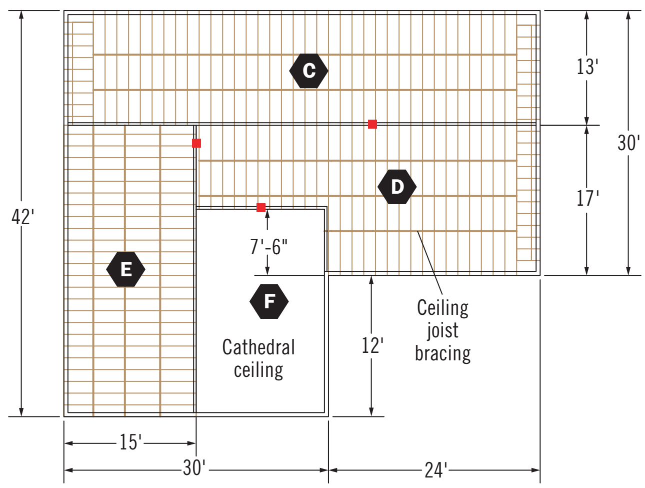
General Notes

- For additional information, please refer to Simpson Strong-Tie® and USP Structural Connectors® literature.
- Fill all round and positive-angle nail holes with the proper nails.
  - 10d x 1½" nails are 0.148" diameter by 1½" long
  - 10d nails are 0.148" diameter by 3" long
  - 16d nails are 0.162" diameter by 3½" long
- All capacities are for downward loads at 115% load duration.
- Hangers to be supported by headers of TimberStrand® LSL, Microllam® LVL, Parallam® PSL, Douglas fir, or southern pine.

Roof Plan



Ceiling Plan



Roof Plan

- One-story, single-family residence
- Roof slope = 8:12
- Roof snow load = 40 psf
- Dead load = 10 psf non-ceiling, 15 psf ceiling
- Unfinished attic area
- Architectural grade fiberglass shingles
- On-center spacing = 16" (preferred)

1 **Determine Rafter Size and Heel Connection Requirements**

Roof Plan, area A (limited access attic area):

- Find allowable member in the **Rafter Span and Heel Connection Tables, No Directly Applied Ceiling** on pages 4 and 5.
  - At 16" on-center spacing with a 40 LL + 10 DL snow load, a 1½" x 9½" 1.5E TimberStrand® LSL rafter will span 17'-5".
  - For an 8:12 roof slope, nine 16d (0.131" x 3¼") nails are required at the heel joint. See detail R21.
- See details R20 and R26 for required rafter-to-ridge and rafter-to-valley connections.

Roof Plan, area B (includes cathedral ceiling):

- Find allowable member in the **Rafter Span and Heel Connection Tables, Directly Applied Ceiling** on pages 6 and 7.
  - At 16" on-center spacing with a 40 LL + 15 DL snow load, a 1½" x 9½" 1.5E TimberStrand® LSL rafter will span 15'-8".
  - For an 8:12 roof slope, nine 16d (0.131" x 3¼") nails are required at the heel joint. See detail R21.
- See details R20 and R26 for required rafter-to-ridge and rafter-to-valley connections.

2 **Size Hip Members**

- Find hip and valley beam spans using the **Rafter-to-Hip/Valley Span** table on page 8.
  - A 15' rafter span converts to a 21'-3" hip or valley span.

Hip Members 1, 2, and 3 (identical spans and loading):

- Find allowable member in the **Single-Span Hip Members** table on page 8.
  - Both a 2-ply, 1½" x 11⅞" 1.5E TimberStrand® LSL member (23'-5") and a 1-ply, 1¾" x 14" 1.55E TimberStrand® LSL member (23'-6") will work. Use the single ply, which costs less and is easier to install.
- See details R23 and R25 for connections and seat cut.

3 **Size Valley Beam**

Valley Beam 4 (use valley beam tables only):

- Find allowable member in the **Single-Span Valley Beams** table on page 9.
  - A 3-ply, 1¾" x 14" 1.55E TimberStrand® LSL valley beam (22'-11") will work.
- See details R23 and R25 for connections and seat cut.

4 **Size Support Posts**

- Calculate post lengths for posts 5 and 6: Rafter span (15') x roof slope (8:12) = 10'.

Post 5:

- Calculate post load using the **Single-Span Hip Member** reaction table on page 10. The high-end reaction for a 32' house width (30' rounded up) is 1,067 lbs. Both hip members 1 and 2 transfer load to post 5, so the post reaction is 2 x 1,067 = 2,134 lbs.
- Find allowable member using the **Allowable Axial Load** table on page 11.
  - The allowable axial load capacity for a 10', 2-ply, 2x6 1.3E TimberStrand® LSL post (2,930 lbs) is greater than 2,134, so the post is adequate.
- Check connection requirements; see detail R26 on page 13. The post may be angled up to 4" per foot of length; therefore, install the low end of the post directly over the bearing wall, 2' from the ridge line.

Post 6:

Post member 6 receives loads from both hip member 3 (reaction = 1,067 lbs, from above) and valley beam 4.

- Find the reaction of valley beam 4.
  - In the **Single-Span Valley Beam** reaction table on page 10, the high end reaction is 2,612 lbs.
  - Total reaction on post 6 is 3,679 lbs (2,612 + 1,067).
- In the **Allowable Axial Load** table on page 11, a 10', 3½" x 3½" 1.8E Parallam® PSL post (5,355 lbs) is adequate.

5 **Size Ceiling Joists**

- Use design loads of 20 LL and 10 DL for an unfinished attic area with limited access. Specify 16" on-center joist spacing to match the roof rafter spacing, and refer to the **Ceiling Joists** table on page 3.
  - A 1½" x 7¼" 1.5E TimberStrand® LSL ceiling joist can span 15'-7", which is adequate for areas C and E. A 1½" x 9½" 1.5E TimberStrand® LSL ceiling joist can span 20'-5", which is adequate for the 17' span portion of ceiling area D. Connect to rafter per **Rafter Span and Heel Connection** tables.
- The cathedral ceiling in area F is within the limits of this guide because the room area is less than 320 ft² (15' x 19.5' = 293 ft²) and takes up less than 15% of the total ceiling area (293 / (42 x 30 + 24 x 30) = 14.8%).

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Floors and Roofs: Start with the best framing components in the industry: our Trus Joist® TJI® joists; TimberStrand® LSL rim board; and TimberStrand® LSL, Microllam® LVL, and Parallam® PSL headers and beams. Pull them all together with our self-gapping and self-draining Weyerhaeuser Edge Gold™ floor panels and durable Weyerhaeuser roof sheathing.

Walls: Get the best value out of your framing package—use TimberStrand® LSL studs for tall walls, kitchens, and bathrooms, and our traditional, solid-sawn lumber everywhere else. Cut down installation time by using TimberStrand® LSL headers for doors and windows, and Weyerhaeuser wall sheathing with its handy two-way nail lines.

Software Solutions: Whether you are a design professional or lumber dealer, Weyerhaeuser offers an array of software packages to help you specify individual framing members, create cut lists, manage inventories—even help you design a complete structural frame. Contact your Weyerhaeuser representative to find out how to get the software you need.

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LIMIT STATES DESIGN
NBCC

#TJ-9505 | SPECIFIER'S GUIDE

BEAMS, HEADERS, AND COLUMNS

Featuring Trus Joist® TimberStrand® LSL, Microllam® LVL, and Parallam® PSL

- Uniform and Predictable
- Minimal Bowing, Twisting, and Shrinking
- Strong and Straight
- Limited Product Warranty

The products in this guide are readily available through our nationwide network of distributors and dealers. For more information on other applications or other Trus Joist® products, contact your Weyerhaeuser representative.

This guide is for use with NBCC 2010, NBCC 2015, CSA 086-09 and CSA 086-14.

TABLE OF CONTENTS

SUSTAINABLE FORESTRY INITIATIVE

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Why Choose Trus Joist® Beams, Columns, and Headers?

- Reliable performance
- Consistent quality and dependable uniformity
- Flexible solutions for your beam and header needs
- Backed by a limited product warranty

Using advanced technology, Weyerhaeuser manufactures engineered lumber that is consistently straight and strong, and that resists bowing, twisting, and shrinking. That means less waste, easier installation, and higher design values for starters; plus fewer callbacks, shorter cycle times, more design flexibility, and lower overall installed cost in the end. Trus Joist® TimberStrand® LSL, Microllam® LVL, and Parallam® PSL are structural solutions you can count on—guaranteed.

This guide features Trus Joist® engineered lumber in the following widths and depths:

TimberStrand® LSL

1.55E TimberStrand® LSL header and beam sizes:
Widths: 1¾" and 3½"
Depths: 9½", 11⅞", 14", and 16"

Microllam® LVL

2.0E Microllam® LVL header and beam sizes:
Width: 1¾"
Depths: 5½", 7¼", 9¼", 9½", 11¼", 11⅞", 14", 16", 18", and 20"

Parallam® PSL

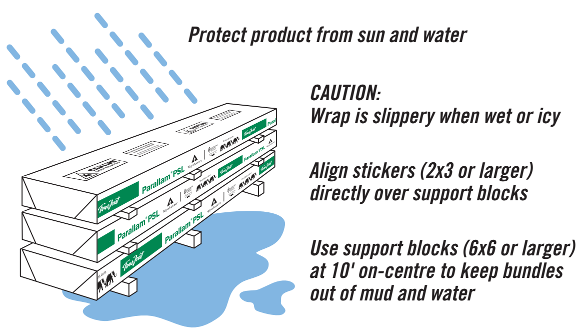
2.2E Parallam® PSL header and beam sizes:
Widths: 3½", 5¼", and 7"
Depths: 9¼", 9½", 11¼", 11⅞", 14", 16", and 19"

1.8E Parallam® PSL column and post sizes:
3½" x 3½"   3½" x 5¼"   3½" x 7"   5¼" x 5¼"   5¼" x 7"   7" x 7"

For deeper depth Parallam® PSL beams, see the *Trus Joist® 2.2E Parallam® PSL Deep Beam Technical Resource Sheet*, #TJ-7501, or contact your Weyerhaeuser representative.

Grades shown are available in Western Canada; some sizes may not be available in your region.

PRODUCT STORAGE



Trus Joist® TimberStrand® Laminated Strand Lumber (LSL)

- One-piece members reduce labor time
- Every piece is straight and strong
- Unique properties allow you to drill larger holes through 1.55E TimberStrand® LSL. See **Allowable Holes** on page 12.

Code Evaluations: See CCMC 12627-R

Trus Joist® Microllam® Laminated Veneer Lumber (LVL)

- Can easily be built up on site to reduce heavy lifting
- Offers reliable and economical solutions for beam and header applications
- Manufacturing process minimizes many of the natural inconsistencies found in wood

Code Evaluations: See CCMC 08675-R

Trus Joist® Parallam® Parallel Strand Lumber (PSL)

- Allows long spans for open floor plans without intermediate posts or columns
- Has warm, unique grain that is perfect for applications with exposed beams
- Provides ideal solutions for cantilever and multi-span applications
- Solid sections save time on site assembly

Code Evaluations: See CCMC 11161-R

General Assumptions for Products Shown in this Guide

- Specified strengths and factored resistances are based on Limit States Design per CSA O86.
- Lateral support is required at bearing and along the span at 24" on-centre, maximum.
- Bearing lengths are based on each product's bearing resistance for applicable grade and orientation.
- All members 7¼" and less in depth are restricted to a maximum deflection of 5⁄16".
- Beams that are 1¾" x 16" and deeper require multiple plies. Some exceptions allowed when using Weyerhaeuser software.
- No camber.
- Beams and columns must remain straight to within $5L^2/4608$ (in.) of true alignment. L is the unrestrained length of the member in feet.

For applications not covered in this guide, contact your Weyerhaeuser representative.

See pages 14–16 for multiple-member beam connections.

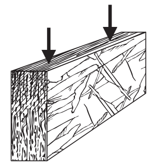
TimberStrand® LSL, Microllam® LVL, and untreated Parallam® PSL are intended for dry-use applications

Factored Resistances(1) (Standard Term)

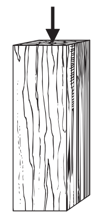
Grade	Width	Design Property	Depth 5½"	7¼"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	19"	20"
TimberStrand® LSL													
1.55E	1¾"	Factored Moment Resistance (ft-lbs)				8,665		13,260	18,155	23,425			
		Factored Shear Resistance (lbs)				5,735		7,170	8,455	9,660			
		Moment of Inertia (in.⁴)				125		244	400	597			
		Weight (plf)				5.2		6.5	7.7	8.8			
	3½"	Factored Moment Resistance (ft-lbs)				17,325		26,525	36,310	46,850			
		Factored Shear Resistance (lbs)				11,470		14,340	16,905	19,320			
		Moment of Inertia (in.⁴)				250		488	800	1,195			
		Weight (plf)				10.4		13.0	15.3	17.5			
Microllam® LVL													
2.0E	1¾"	Factored Moment Resistance (ft-lbs)	3,535	5,915	9,315	9,790	13,420	14,845	20,175	25,875	32,230		39,220
		Factored Shear Resistance (lbs)	3,060	4,035	5,150	5,285	6,260	6,610	7,790	8,905	10,015		11,130
		Moment of Inertia (in.⁴)	24	56	115	125	208	244	400	597	851		1,167
		Weight (plf)	2.8	3.7	4.7	4.8	5.7	6.1	7.1	8.2	9.2		10.2
Parallam® PSL													
2.2E	3½"	Factored Moment Resistance (ft-lbs)			20,655	21,720	29,890	33,105	45,180	58,145		80,445	
		Factored Shear Resistance (lbs)			10,490	10,775	12,760	13,465	15,875	18,145		21,545	
		Moment of Inertia (in.⁴)			231	250	415	488	800	1,195		2,001	
		Weight (plf)			10.1	10.4	12.3	13.0	15.3	17.5		20.8	
	5¼"	Factored Moment Resistance (ft-lbs)			30,980	32,580	44,840	49,660	67,775	87,220		120,665	
		Factored Shear Resistance (lbs)			15,735	16,160	19,135	20,200	23,815	27,215		32,320	
		Moment of Inertia (in.⁴)			346	375	623	733	1,201	1,792		3,001	
		Weight (plf)			15.2	15.6	18.5	19.5	23.0	26.3		31.2	
	7"	Factored Moment Resistance (ft-lbs)			41,305	43,440	59,785	66,215	90,365	116,290		160,890	
		Factored Shear Resistance (lbs)			20,980	21,545	25,515	26,935	31,750	36,290		43,090	
		Moment of Inertia (in.⁴)			462	500	831	977	1,601	2,389		4,001	
		Weight (plf)			20.2	20.8	24.6	26.0	30.6	35.0		41.6	

(1) For product in beam orientation, unless otherwise noted.

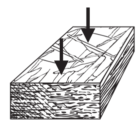
Beam Orientation



Column Orientation



Plank Orientation



Specified Strengths(1) and Moduli of Elasticity (Standard Term)

Grade	Orientation	G Shear Modulus of Elasticity (psi)	E Modulus of Elasticity (2) (psi)	f_b Flexural Stress(3) (psi)	f_t Tension Stress(4) (psi)	$f_{c\perp}$ Compression Perpendicular to Grain (psi)	$f_{c\parallel}$ Compression Parallel to Grain (psi)	f_v Horizontal Shear Parallel to Grain (psi)	SG Equivalent Specific Gravity(5)
TimberStrand® LSL									
1.55E	Beam	96,875	1.55×10^6	4,295	1,975(7)	1,635	3,465	575(7)	0.50(6)
Microllam® LVL									
2.0E	Beam	125,000	2.0×10^6	4,805	2,870	1,365	4,005	530	0.50
Parallam® PSL									
1.8E	Column	112,500	1.8×10^6	4,435(8)	3,245	990(8)	3,990	355(8)	0.50
2.2E	Beam	137,500	2.2×10^6	5,360	3,750	1,135	4,630(9)	540	0.50

(1) To obtain factored resistances, apply the appropriate formulae from CSA O86 to the specified strengths shown.

(2) To properly calculate deflections for the full range of typical SCL span and loading applications, bending and shear deflection must be considered. Use the following equation for simple span, uniformly loaded beams:

$$\Delta = \frac{270\,wL^4}{Ebd^3} + \frac{28.8\,wL^2}{Ebd}$$

Where: Δ = deflection (in.) w = uniform load (plf)
L = span (feet) b = beam thickness (in.)
d = beam depth (in.) E = modulus of elasticity (psi)

For other span and loading conditions, use engineering mechanics to account for both bending and shear deflection or use Forte®WEB software.

(3) For 12" depth. For other depths, multiply f_b by the appropriate factor as follows:

– For TimberStrand® LSL, multiply by $\left[\frac{12}{d}\right]^{0.092}$

– For Microllam® LVL, multiply by $\left[\frac{12}{d}\right]^{0.136}$

– For Parallam® PSL, multiply by $\left[\frac{12}{d}\right]^{0.111}$

(4) f_t has been adjusted to reflect the volume effects for most standard applications.

(5) For lateral connection design only.

(6) Specific gravity of 0.58 may be used for bolts installed perpendicular to face and loaded perpendicular to grain.

(7) Value accounts for large hole capabilities. See **Allowable Holes** on page 12.

(8) Value shown is for plank orientation.

(9) For column applications, use a specified strength of 800 psi. Alternatively, refer to CCMC 11161-R, Table 4.4.1, footnote 9.

TimberStrand® LSL, Microllam® LVL, and untreated Parallam® PSL are intended for dry-use applications

How to Use This Table

1. Calculate the factored and unfactored total load (TL) (neglect beam weight) and the unfactored live load (LL) on the beam or header in pounds per linear foot (plf).
2. Select appropriate **Span** (centre-to-centre of bearing).
3. Scan horizontally to find the proper width and a depth that has a capacity that meets or exceeds actual loads.
4. Review bearing length requirements to ensure adequacy.

General Notes

- Table is based on:
  - Uniform loads (beam weight considered).
  - More restrictive of simple or continuous span.
  - Deflection criteria of L/360 live load (LL) and L/240 total load (TL).
- For a live load deflection limit of L/480, multiply **Unfactored Resistance (LL)** by 0.75. For a total load limit of L/180 multiply **Unfactored Resistance (TL)** by 1.33. The resulting loads must not exceed the **Total Factored Resistance** shown.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.

*Also **General Assumptions** on page 3.*

1.55E TimberStrand® LSL: Floor and/or Snow—Standard Term (PLF)

Span	Condition	1¾" Width			3½" Width				5¼" Width (2- or 3-ply)			
		9½"	11⅞"	14"	9½"	11⅞"	14"	16"	9½"	11⅞"	14"	16"
4'	Unfactored Resistance (LL)	*	*	*	*	*	*	*	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	*	*	*	*	*	*	*	*
	Total Factored Resistance	3,350	4,738	5,140	6,701	9,477	10,278	10,278	10,052	14,215	15,417	15,417
	Min. End/Int. Bearing (in.)	2.9/7.3	4.1/10.4	4.5/11.3	2.9/7.3	4.1/10.4	4.5/11.3	4.5/11.3	2.9/7.3	4.1/10.4	4.5/11.3	4.5/11.3
5'	Unfactored Resistance (LL)	1,658	*	*	3,316	*	*	*	4,975	*	*	*
	Unfactored Resistance (TL)	*	*	*	*	*	*	*	*	*	*	*
	Total Factored Resistance	2,451	3,349	4,110	4,903	6,698	8,218	8,218	7,354	10,047	12,327	12,327
	Min. End/Int. Bearing (in.)	2.7/6.7	3.7/9.2	4.5/11.3	2.7/6.7	3.7/9.2	4.5/11.3	4.5/11.3	2.7/6.7	3.7/9.2	4.5/11.3	4.5/11.3
6'	Unfactored Resistance (LL)	1,048	*	*	2,097	*	*	*	3,146	*	*	*
	Unfactored Resistance (TL)	*	*	*	*	*	*	*	*	*	*	*
	Total Factored Resistance	1,918	2,589	3,262	3,837	5,178	6,524	6,845	5,756	7,767	9,787	10,267
	Min. End/Int. Bearing (in.)	2.5/6.3	3.4/8.5	4.3/10.7	2.5/6.3	3.4/8.5	4.3/10.7	4.5/11.3	2.5/6.3	3.4/8.5	4.3/10.7	4.5/11.3
8'	Unfactored Resistance (LL)	487	886	1,352	974	1,773	2,705	*	1,462	2,660	4,058	*
	Unfactored Resistance (TL)	725	*	*	1451	*	*	*	2,177	*	*	*
	Total Factored Resistance	1,076	1649	2,195	2,152	3,299	4,390	5,128	3,229	4,948	6,586	7,692
	Min. End/Int. Bearing (in.)	1.9/4.7	2.9/7.2	3.9/9.6	1.9/4.7	2.9/7.2	3.9/9.6	4.5/11.3	1.9/4.7	2.9/7.2	3.9/9.6	4.5/11.3
9'-6"	Unfactored Resistance (LL)	302	560	870	605	1,121	1,740	2,456	907	1,681	2,610	3,684
	Unfactored Resistance (TL)	448	*	*	897	*	*	*	1346	*	*	*
	Total Factored Resistance	761	1,167	1,599	1,522	2,334	3,199	4,130	2,284	3,502	4,799	6,196
	Min. End/Int. Bearing (in.)	1.6/4	2.4/6.1	3.3/8.3	1.6/4	2.4/6.1	3.3/8.3	4.3/10.8	1.6/4	2.4/6.1	3.3/8.3	4.3/10.8
10'	Unfactored Resistance (LL)	261	487	760	523	974	1,520	2,154	785	1,462	2,280	3,232
	Unfactored Resistance (TL)	387	724	*	775	1449	*	*	1162	2174	*	*
	Total Factored Resistance	686	1,052	1,442	1,373	2,105	2,885	3,725	2,059	3,158	4,328	5,588
	Min. End/Int. Bearing (in.)	1.5/3.8	2.3/5.8	3.2/7.9	1.5/3.8	2.3/5.8	3.2/7.9	4.1/10.2	1.5/3.8	2.3/5.8	3.2/7.9	4.1/10.2
12'	Unfactored Resistance (LL)	155	293	464	311	587	928	1,334	467	881	1393	2001
	Unfactored Resistance (TL)	228	434	688	456	868	1,,377	*	685	1,302	2,066	*
	Total Factored Resistance	474	728	999	949	1,457	1,998	2,580	1,424	2,185	2,997	3,871
	Min. End/Int. Bearing (in.)	1.5/3.5	1.9/4.8	2.6/6.6	1.5/3.5	1.9/4.8	2.6/6.6	3.4/8.5	1.5/3.5	1.9/4.8	2.6/6.6	3.4/8.5
14'	Unfactored Resistance (LL)	99	189	302	199	379	605	877	299	569	907	1316
	Unfactored Resistance (TL)	144	278	446	288	556	892	1298	433	834	1,338	1948
	Total Factored Resistance	347	533	731	694	1,066	1,462	1,890	1,041	1,599	2,194	2,835
	Min. End/Int. Bearing (in.)	1.5/3.5	1.7/4.1	2.3/5.7	1.5/3.5	1.7/4.1	2.3/5.7	2.9/7.3	1.5/3.5	1.7/4.1	2.3/5.7	2.9/7.3
16'-6"	Unfactored Resistance (LL)	61	118	189	123	236	379	555	185	354	569	832
	Unfactored Resistance (TL)	87	170	277	174	341	554	815	262	512	831	1,222
	Total Factored Resistance	248	381	523	496	763	1,047	1,354	744	1,144	1,571	2,032
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.9/4.8	1.5/3.5	1.5/3.5	1.9/4.8	2.5/6.2	1.5/3.5	1.5/3.5	1.9/4.8	2.5/6.2
18'-6"	Unfactored Resistance (LL)	44	84	136	88	169	273	401	132	254	410	601
	Unfactored Resistance (TL)	60	120	197	121	241	395	584	182	362	592	876
	Total Factored Resistance	196	301	414	392	603	829	1,073	588	905	1,244	1,609
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.7/4.3	1.5/3.5	1.5/3.5	1.7/4.3	2.2/5.5	1.5/3.5	1.5/3.5	1.7/4.3	2.2/5.5
20'	Unfactored Resistance (LL)		67	109	70	135	218	320	105	202	327	481
	Unfactored Resistance (TL)		94	156	94	189	312	463	142	284	468	695
	Total Factored Resistance		257	353	333	514	707	915	500	771	1,060	1,372
	Min. End/Int. Bearing (in.)		1.5/3.5	1.6/4	1.5/3.5	1.5/3.5	1.6/4	2/5.1	1.5/3.5	1.5/3.5	1.6/4	2/5.1

* Indicates **Total Factored Resistance** value controls.

How to Use This Table

1. Calculate the factored and unfactored total load (TL) (neglect beam weight) and the unfactored live load (LL) on the beam or header in pounds per linear foot (plf).
2. Select appropriate **Span** (centre-to-centre of bearing).
3. Scan horizontally to find the proper width and a depth that has a capacity that meets or exceeds actual loads.
4. Review bearing length requirements to ensure adequacy.

*Also see **General Notes** on page 7.*

2.0E Microllam® LVL: Floor and/or Snow—Standard Term (PLF)

Span	Condition	1¾" Width							3½" Width (2-ply)						
		5½"	7¼"	9¼"	9½"	11¼"	11⅞"	14"	5½"	7¼"	9¼"	9½"	11¼"	11⅞"	14"
6'	Unfactored Resistance (LL)	305	660	*	*	*	*	*	611	1,319	*	*	*	*	*
	Unfactored Resistance (TL)	455	*	*	*	*	*	*	911	*	*	*	*	*	*
	Total Factored Resistance	782	1,278	1,722	1,781	2,219	2,386	2,859	1,564	2,556	3,444	3,562	4,438	4,773	5,713
	Min. End/Int. Bearing (in.)	1.5/3.5	2/5	2.7/6.8	2.8/7	3.5/8.7	3.8/9.4	4.5/11.3	1.5/3.5	2/5	2.7/6.8	2.8/7	3.5/8.7	3.8/9.4	4.5/11.3
8'	Unfactored Resistance (LL)	134	296	585	629	992	*	*	267	591	1,169	1,258	1,985	*	*
	Unfactored Resistance (TL)	154	343	*	*	*	*	*	308	686	*	*	*	*	*
	Total Factored Resistance	438	735	1,159	1,218	1,534	1,640	2,024	877	1,470	2,318	2,436	3,068	3,280	4,047
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.9	2.4/6.1	2.6/6.4	3.2/8.1	3.4/8.6	4.3/10.6	1.5/3.5	1.5/3.9	2.4/6.1	2.6/6.4	3.2/8.1	3.4/8.6	4.3/10.6
9'-6"	Unfactored Resistance (LL)	80	178	362	390	624	723	*	160	357	724	781	1248	1,447	*
	Unfactored Resistance (TL)	77	175	539	581	*	*	*	154	349	1,077	1,162	*	*	*
	Total Factored Resistance	310	520	820	862	1,182	1,308	1,624	620	1,040	1,640	1,724	2,365	2,616	3,248
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	2.1/5.1	2.2/5.4	3/7.4	3.3/8.2	4.1/10.1	1.5/3.5	1.5/3.5	2.1/5.1	2.2/5.4	3/7.4	3.3/8.2	4.1/10.1
10'	Unfactored Resistance (LL)	65	146	313	338	542	629	981	131	292	627	676	1,084	1,258	1,961
	Unfactored Resistance (TL)	62	142	465	502	*	*	*	125	285	931	1,004	*	*	*
	Total Factored Resistance	279	469	739	777	1,066	1,180	1,524	559	937	1,479	1,555	2,133	2,360	3,047
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	2/4.9	2/5.1	2.8/7	3.1/7.8	4/10	1.5/3.5	1.5/3.5	2/4.9	2/5.1	2.8/7	3.1/7.8	4/10
12'	Unfactored Resistance (LL)	32	72	186	201	326	379	599	64	143	372	402	651	758	1,198
	Unfactored Resistance (TL)	29	68	274	297	483	563	*	58	136	549	593	965	1,125	*
	Total Factored Resistance	193	324	512	538	738	817	1,112	386	648	1,023	1,076	1,477	1,634	2,224
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.6/4.1	1.7/4.3	2.3/5.9	2.6/6.5	3.5/8.8	1.5/3.5	1.5/3.5	1.6/4.1	1.7/4.3	2.3/5.9	2.6/6.5	3.5/8.8
14'	Unfactored Resistance (LL)		39	119	129	210	245	390	35	78	238	257	420	490	781
	Unfactored Resistance (TL)		35	174	188	309	361	*	29	71	348	376	618	723	*
	Total Factored Resistance		237	374	394	541	598	814	282	474	749	787	1,081	1,197	1,629
	Min. End/Int. Bearing (in.)		1.5/3.5	1.5/3.5	1.5/3.7	2/5	2.2/5.5	3/7.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2/5	2.2/5.5	3/7.5
16'-6"	Unfactored Resistance (LL)			74	80	130	153	245	18	41	147	159	261	305	490
	Unfactored Resistance (TL)			106	115	190	223	361	12	33	212	229	380	446	721
	Total Factored Resistance			268	282	387	429	584	201	338	536	563	774	857	1,168
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.7/4.3	1.9/4.7	2.6/6.4	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.7/4.3	1.9/4.7	2.6/6.4
18'-6"	Unfactored Resistance (LL)			53	57	93	109	176		26	105	114	187	219	353
	Unfactored Resistance (TL)			74	81	134	158	258		19	148	161	269	316	515
	Total Factored Resistance			212	223	307	339	463		267	424	446	613	679	925
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.5/3.8	1.7/4.2	2.3/5.7		1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.8	1.7/4.2	2.3/5.7
20'	Unfactored Resistance (LL)			42	45	74	87	141		19	84	90	149	174	282
	Unfactored Resistance (TL)			58	63	106	125	204		12	116	126	212	249	408
	Total Factored Resistance			180	190	261	289	395		227	361	380	522	579	789
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.5/3.5	1.6/3.9	2.1/5.3		1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/3.9	2.1/5.3
24'	Unfactored Resistance (LL)					43	51	83			49	53	87	102	166
	Unfactored Resistance (TL)					59	70	117			64	69	119	141	234
	Total Factored Resistance					179	199	271			247	260	358	397	543
	Min. End/Int. Bearing (in.)					1.5/3.5	1.5/3.5	1.8/4.4			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.4
28'	Unfactored Resistance (LL)							53			31	33	55	65	105
	Unfactored Resistance (TL)							72			37	40	71	85	144
	Total Factored Resistance							197			178	188	260	288	394
	Min. End/Int. Bearing (in.)							1.5/3.8			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.8

* Indicates **Total Factored Resistance** value controls.

General Notes

- Table is based on:
  - Uniform loads (beam weight considered).
  - More restrictive of simple or continuous span.
  - Deflection criteria of L/360 live load (LL) and L/240 total load (TL).
- For a live load deflection limit of L/480, multiply **Unfactored Resistance (LL)** by 0.75. For a total load limit of L r/180 multiply **Unfactored Resistance (TL)** by 1.33. The resulting loads must not exceed the **Total Factored Resistance** shown.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.

Also see ***How to Use This Table*** *on page 6 and* ***General Assumptions*** *on page 3.*

2.0E Microllam® LVL: Floor and/or Snow—Standard Term (PLF) *continued*

Span	Condition	3½" Width (2-ply)			5¼" Width (3-ply)									
		16"	18"	20"	5½"	7¼"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	20"
6'	Unfactored Resistance (LL)	*	*	*	916	1,979	*	*	*	*	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	1,366	*	*	*	*	*	*	*	*	*
	Total Factored Resistance	5,713	5,713	5,713	2,346	3,834	5,166	5,343	6,656	7,159	8,569	8,569	8,569	8,569
	Min. End/Int. Bearing (in.)	4.5/11.3	4.5/11.3	4.5/11.3	1.5/3.5	2/5	2.7/6.8	2.8/7	3.5/8.7	3.8/9.4	4.5/11.3	4.5/11.3	4.5/11.3	4.5/11.3
8'	Unfactored Resistance (LL)	*	*	*	401	887	1,754	1,887	2,977	*	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	462	1,028	*	*	*	*	*	*	*	*
	Total Factored Resistance	4,279	4,279	4,279	1,315	2,205	3,476	3,654	4,602	4,921	6,071	6,419	6,419	6,419
	Min. End/Int. Bearing (in.)	4.5/11.3	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.9	2.4/6.1	2.6/6.4	3.2/8.1	3.4/8.6	4.3/10.6	4.5/11.3	4.5/11.3	4.5/11.3
9'-6"	Unfactored Resistance (LL)	*	*	*	240	535	1,087	1,171	1,873	2,170	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	231	524	1,616	1,742	*	*	*	*	*	*
	Total Factored Resistance	3,600	3,600	3,600	930	1,560	2,460	2,586	3,547	3,924	4,872	5,401	5,401	5,401
	Min. End/Int. Bearing (in.)	4.5/11.3	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	2.1/5.1	2.2/5.4	3/7.4	3.3/8.2	4.1/10.1	4.5/11.3	4.5/11.3	4.5/11.3
10'	Unfactored Resistance (LL)	*	*	*	196	439	940	1,014	1,626	1,887	2,942	*	*	*
	Unfactored Resistance (TL)	*	*	*	187	427	1,396	1,506	*	*	*	*	*	*
	Total Factored Resistance	3,419	3,419	3,419	838	1,406	2,218	2,332	3,199	3,540	4,571	5,129	5,129	5,129
	Min. End/Int. Bearing (in.)	4.5/11.3	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	2/4.9	2/5.1	2.8/7	3.1/7.8	4/10	4.5/11.3	4.5/11.3	4.5/11.3
12'	Unfactored Resistance (LL)	*	*	*	95	215	558	603	977	1,137	1,798	2,583	*	*
	Unfactored Resistance (TL)	*	*	*	87	204	823	890	1,448	1,688	*	*	*	*
	Total Factored Resistance	2,846	2,846	2,846	579	972	1,535	1,614	2,215	2,451	3,336	4,269	4,269	4,269
	Min. End/Int. Bearing (in.)	4.5/11.3	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	1.6/4.1	1.7/4.3	2.3/5.9	2.6/6.5	3.5/8.8	4.5/11.3	4.5/11.3	4.5/11.3
14'	Unfactored Resistance (LL)	1,132	1,561	*	52	117	357	386	629	735	1,171	1,698	2,342	*
	Unfactored Resistance (TL)	*	*	*	43	106	522	565	927	1,084	*	*	*	*
	Total Factored Resistance	2,092	2,437	2,437	422	711	1,123	1,181	1,622	1,795	2,443	3,138	3,655	3,655
	Min. End/Int. Bearing (in.)	3.9/9.7	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2/5	2.2/5.5	3/7.5	3.9/9.7	4.5/11.3	4.5/11.3
16'-6"	Unfactored Resistance (LL)	716	996	1,331	27	61	221	239	391	458	735	1,074	1,493	1,996
	Unfactored Resistance (TL)	*	*	*	19	50	317	344	570	669	1,082	*	*	*
	Total Factored Resistance	1,500	1,871	2,064	301	508	804	845	1,162	1,286	1,752	2,250	2,807	3,096
	Min. End/Int. Bearing (in.)	3.3/8.2	4.1/10.2	4.5/11.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.7/4.3	1.9/4.7	2.6/6.4	3.3/8.2	4.1/10.2	4.5/11.3
18'-6"	Unfactored Resistance (LL)	518	723	971	17	39	158	171	280	328	529	777	1,084	1,456
	Unfactored Resistance (TL)	760	*	*	9	28	223	242	403	474	773	1,140	*	*
	Total Factored Resistance	1,189	1,484	1,808	237	401	636	668	920	1,018	1,388	1,784	2,226	2,712
	Min. End/Int. Bearing (in.)	2.9/7.3	3.6/9.1	4.4/11.1	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.8	1.7/4.2	2.3/5.7	2.9/7.3	3.6/9.1	4.4/11.1
20'	Unfactored Resistance (LL)	414	580	781	13	29	125	136	223	262	423	621	870	1,171
	Unfactored Resistance (TL)	605	851	*	4	17	174	189	318	374	612	907	1,277	*
	Total Factored Resistance	1,015	1,266	1,543	202	341	541	569	784	868	1,184	1,522	1,899	2,315
	Min. End/Int. Bearing (in.)	2.7/6.8	3.4/8.4	4.1/10.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/3.9	2.1/5.3	2.7/6.8	3.4/8.4	4.1/10.3
24'	Unfactored Resistance (LL)	244	344	466		14	73	79	130	153	248	367	516	698
	Unfactored Resistance (TL)	350	498	678		3	95	104	178	211	351	526	746	1,017
	Total Factored Resistance	698	872	1,064		233	371	390	538	596	814	1,048	1,308	1,596
	Min. End/Int. Bearing (in.)	2.3/5.6	2.8/7.0	3.4/8.6		1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.4	2.3/5.6	2.8/7.0	3.4/8.6
28'	Unfactored Resistance (LL)	156	220	299			46	50	83	97	158	234	330	448
	Unfactored Resistance (TL)	217	311	428			55	60	107	127	215	326	467	641
	Total Factored Resistance	508	635	775			268	282	389	432	591	761	952	1,162
	Min. End/Int. Bearing (in.)	1.9/4.8	2.4/6.0	2.9/7.3			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.8	1.9/4.8	2.4/6.0	2.9/7.3

* Indicates **Total Factored Resistance** value controls.

How to Use This Table

1. Calculate the factored and unfactored total load (TL) (neglect beam weight) and the unfactored live load (LL) on the beam or header in pounds per linear foot (plf).
2. Select appropriate **Span** (centre-to-centre of bearing).
3. Scan horizontally to find the proper width and a depth that has a capacity that meets or exceeds actual loads.
4. Review bearing length requirements to ensure adequacy.

*Also see **General Notes** on page 9.*

2.2E Parallam® PSL: Floor and/or Snow—Standard Term (PLF)

Span	Condition	3½" Width							5¼" Width						
		9¼"	9½"	11¼"	11⅞"	14"	16"	19"	9¼"	9½"	11¼"	11⅞"	14"	16"	19"
8'	Unfactored Resistance (LL)	1,286	1,383	*	*	*	*	*	1,929	2,075	*	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	*	*	*	*	*	*	*	*	*	*	*
	Total Factored Resistance	2,467	2,546	3,124	3,341	3,554	3,554	3,554	3,701	3,820	4,687	5,012	5,331	5,331	5,331
	Min. End/Int. Bearing (in.)	3.1/7.8	3.2/8.1	4/9.9	4.2/10.6	4.5/11.3	4.5/11.3	4.5/11.3	3.1/7.8	3.2/8.1	4/9.9	4.2/10.6	4.5/11.3	4.5/11.3	4.5/11.3
9'-6"	Unfactored Resistance (LL)	796	858	1,373	1,591	*	*	*	1,195	1,288	2,059	2,387	*	*	*
	Unfactored Resistance (TL)	1,185	1,278	*	*	*	*	*	1,777	1,917	*	*	*	*	*
	Total Factored Resistance	1,818	1,912	2,536	2,705	2,989	2,989	2,989	2,727	2,868	3,804	4,058	4,484	4,484	4,484
	Min. End/Int. Bearing (in.)	2.7/6.8	2.9/7.2	3.8/9.5	4.1/10.2	4.5/11.3	4.5/11.3	4.5/11.3	2.7/6.8	2.9/7.2	3.8/9.5	4.1/10.2	4.5/11.3	4.5/11.3	4.5/11.3
10'	Unfactored Resistance (LL)	689	743	1,192	1,383	*	*	*	1,034	1,115	1,788	2,075	*	*	*
	Unfactored Resistance (TL)	1,024	1,104	*	*	*	*	*	1,536	1,657	*	*	*	*	*
	Total Factored Resistance	1,639	1,724	2,376	2,543	2,839	2,839	2,839	2,459	2,586	3,564	3,815	4,259	4,259	4,259
	Min. End/Int. Bearing (in.)	2.6/6.5	2.7/6.8	3.8/9.4	4/10.1	4.5/11.3	4.5/11.3	4.5/11.3	2.6/6.5	2.7/6.8	3.8/9.4	4/10.1	4.5/11.3	4.5/11.3	4.5/11.3
12'	Unfactored Resistance (LL)	409	442	716	834	1,318	*	*	614	663	1,074	1,251	1,977	*	*
	Unfactored Resistance (TL)	604	652	1,061	1,238	*	*	*	906	979	1,592	1,857	*	*	*
	Total Factored Resistance	1,134	1,193	1,645	1,823	2,362	2,362	2,362	1,702	1,790	2,467	2,734	3,544	3,544	3,544
	Min. End/Int. Bearing (in.)	2.2/5.4	2.3/5.7	3.1/7.8	3.5/8.7	4.5/11.3	4.5/11.3	4.5/11.3	2.2/5.4	2.3/5.7	3.1/7.8	3.5/8.7	4.5/11.3	4.5/11.3	4.5/11.3
14'	Unfactored Resistance (LL)	261	283	461	538	858	1,245	*	392	424	692	808	1,288	1,868	*
	Unfactored Resistance (TL)	382	414	679	795	*	*	*	574	621	1,019	1,192	*	*	*
	Total Factored Resistance	830	873	1,204	1,335	1,825	2,022	2,022	1,245	1,310	1,807	2,002	2,737	3,033	3,033
	Min. End/Int. Bearing (in.)	1.9/4.6	2/4.9	2.7/6.7	3/7.4	4.1/10.2	4.5/11.3	4.5/11.3	1.9/4.6	2/4.9	2.7/6.7	3/7.4	4.1/10.2	4.5/11.3	4.5/11.3
16'-6"	Unfactored Resistance (LL)	162	175	287	335	539	787	*	243	262	430	503	808	1,181	*
	Unfactored Resistance (TL)	232	252	418	490	793	*	*	349	378	627	736	1,190	*	*
	Total Factored Resistance	594	625	862	956	1,308	1,686	1,712	891	937	1,294	1,434	1,962	2,530	2,569
	Min. End/Int. Bearing (in.)	1.6/3.9	1.7/4.1	2.3/5.7	2.5/6.3	3.4/8.6	4.4/11.1	4.5/11.3	1.6/3.9	1.7/4.1	2.3/5.7	2.5/6.3	3.4/8.6	4.4/11.1	4.5/11.3
18'-6"	Unfactored Resistance (LL)	115	125	205	240	388	569	925	173	187	308	361	582	854	1,388
	Unfactored Resistance (TL)	163	177	296	348	567	836	*	245	266	444	522	850	1,255	*
	Total Factored Resistance	470	494	683	757	1,036	1,337	1,525	705	742	1,025	1,136	1,555	2,005	2,287
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.7	2/5.1	2.3/5.6	3.1/7.7	4/9.9	4.5/11.3	1.5/3.5	1.5/3.7	2/5.1	2.3/5.6	3.1/7.7	4/9.9	4.5/11.3
20'	Unfactored Resistance (LL)	91	99	163	191	309	455	743	137	149	245	287	464	683	1,115
	Unfactored Resistance (TL)	127	138	233	274	449	665	*	191	208	349	412	674	998	*
	Total Factored Resistance	400	421	582	645	884	1,141	1,409	600	632	873	968	1,326	1,711	2,113
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.9/4.7	2.1/5.2	2.8/7.1	3.7/9.1	4.5/11.3	1.5/3.5	1.5/3.5	1.9/4.7	2.1/5.2	2.8/7.1	3.7/9.1	4.5/11.3
24'	Unfactored Resistance (LL)	53	57	95	112	182	268	442	80	86	143	168	273	403	663
	Unfactored Resistance (TL)	70	76	131	155	257	385	642	105	114	196	233	386	578	963
	Total Factored Resistance	274	288	399	443	608	785	1,091	411	433	599	665	912	1,178	1,636
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.6/3.9	1.7/4.3	2.4/5.9	3/7.6	4.2/10.5	1.5/3.5	1.5/3.5	1.6/3.9	1.7/4.3	2.4/5.9	3/7.6	4.2/10.5
28'	Unfactored Resistance (LL)	33	36	60	71	115	171	283	50	55	90	106	173	257	424
	Unfactored Resistance (TL)	40	44	78	93	158	239	403	61	66	117	140	237	359	605
	Total Factored Resistance	198	208	289	321	441	571	794	297	312	434	482	662	857	1,192
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2/5.1	2.6/6.5	3.6/9	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2/5.1	2.6/6.5	3.6/9
32'	Unfactored Resistance (LL)			40	47	78	115	191	34	36	61	71	117	173	287
	Unfactored Resistance (TL)			48	58	101	156	266	36	39	73	88	152	234	400
	Total Factored Resistance			218	242	333	432	602	223	235	327	363	500	648	903
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.8/4.4	2.3/5.7	3.2/7.9	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.4	2.3/5.7	3.2/7.9

* Indicates **Total Factored Resistance** value controls.

General Notes

- Table is based on:
  – Uniform loads (beam weight considered).
  – More restrictive of simple or continuous span.
  – Deflection criteria of L/360 live load (LL) and L/240 total load (TL).
- For a live load deflection limit of L/480, multiply **Unfactored Resistance (LL)** by 0.75. For a total load limit of L/180 multiply **Unfactored Resistance (TL)** by 1.33. The resulting loads must not exceed the **Total Factored Resistance** shown.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.

*Also see **How to Use This Table** on page 8 and **General Assumptions** on page 3.*

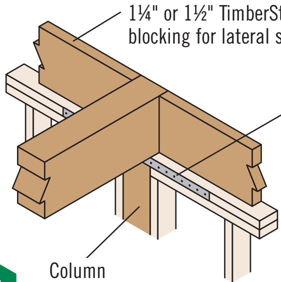
2.2E Parallam® PSL: Floor and/or Snow— Standard Term (PLF) *continued*

Span	Condition	7" Width						
		9¼"	9½"	11¼"	11⅞"	14"	16"	19"
8'	Unfactored Resistance (LL)	2,572	2,767	*	*	*	*	*
	Unfactored Resistance (TL)	*	*	*	*	*	*	*
	Total Factored Resistance	4,935	5,093	6,249	6,683	7,108	7,108	7,108
	Min. End/Int. Bearing (in.)	3.1/7.8	3.2/8.1	4/9.9	4.2/10.6	4.5/11.3	4.5/11.3	4.5/11.3
9'-6"	Unfactored Resistance (LL)	1,593	1,717	2,746	3,182	*	*	*
	Unfactored Resistance (TL)	2,370	2,556	*	*	*	*	*
	Total Factored Resistance	3,636	3,824	5,072	5,410	5,979	5,979	5,979
	Min. End/Int. Bearing (in.)	2.7/6.8	2.9/7.2	3.8/9.5	4.1/10.2	4.5/11.3	4.5/11.3	4.5/11.3
10'	Unfactored Resistance (LL)	1,378	1,486	2,385	2,767	*	*	*
	Unfactored Resistance (TL)	2,048	2,209	*	*	*	*	*
	Total Factored Resistance	3,279	3,449	4,752	5,087	5,678	5,678	5,678
	Min. End/Int. Bearing (in.)	2.6/6.5	2.7/6.8	3.8/9.4	4/10.1	4.5/11.3	4.5/11.3	4.5/11.3
12'	Unfactored Resistance (LL)	818	884	1,432	1,668	2,636	*	*
	Unfactored Resistance (TL)	1,208	1,305	2,123	2,476	*	*	*
	Total Factored Resistance	2,269	2,387	3,290	3,646	4,725	4,725	4,725
	Min. End/Int. Bearing (in.)	2.2/5.4	2.3/5.7	3.1/7.8	3.5/8.7	4.5/11.3	4.5/11.3	4.5/11.3
14'	Unfactored Resistance (LL)	523	566	922	1,077	1,717	2,490	*
	Unfactored Resistance (TL)	765	828	1,359	1,590	*	*	*
	Total Factored Resistance	1,660	1,747	2,409	2,670	3,650	4,044	4,044
	Min. End/Int. Bearing (in.)	1.9/4.6	2/4.9	2.7/6.7	3/7.4	4.1/10.2	4.5/11.3	4.5/11.3
16'-6"	Unfactored Resistance (LL)	324	350	574	671	1,078	1,575	*
	Unfactored Resistance (TL)	465	504	836	981	1,587	*	*
	Total Factored Resistance	1,188	1,250	1,725	1,913	2,617	3,373	3,425
	Min. End/Int. Bearing (in.)	1.6/3.9	1.7/4.1	2.3/5.7	2.5/6.3	3.4/8.6	4.4/11.1	4.5/11.3
18'-6"	Unfactored Resistance (LL)	231	250	411	481	776	1,139	1,851
	Unfactored Resistance (TL)	326	354	592	696	1,134	1,673	*
	Total Factored Resistance	940	989	1,366	1,515	2,073	2,674	3,050
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.7	2/5.1	2.3/5.6	3.1/7.7	4/9.9	4.5/11.3
20'	Unfactored Resistance (LL)	183	198	327	383	619	911	1,486
	Unfactored Resistance (TL)	255	277	466	549	898	1,331	*
	Total Factored Resistance	800	842	1,164	1,291	1,769	2,282	2,818
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.9/4.7	2.1/5.2	2.8/7.1	3.7/9.1	4.5/11.3
24'	Unfactored Resistance (LL)	107	115	191	224	364	537	884
	Unfactored Resistance (TL)	140	153	262	310	515	771	1,284
	Total Factored Resistance	548	577	799	887	1,216	1,571	2,182
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.6/3.9	1.7/4.3	2.4/5.9	3/7.6	4.2/10.5
28'	Unfactored Resistance (LL)	67	73	121	142	231	342	566
	Unfactored Resistance (TL)	81	89	157	187	316	479	807
	Total Factored Resistance	396	417	579	643	883	1,142	1,589
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2/5.1	2.6/6.5	3.6/9
32'	Unfactored Resistance (LL)	45	49	81	95	156	231	383
	Unfactored Resistance (TL)	48	53	97	117	203	312	533
	Total Factored Resistance	297	313	436	484	667	864	1,204
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.4	2.3/5.7	3.2/7.9

* Indicates **Total Factored Resistance** value controls.

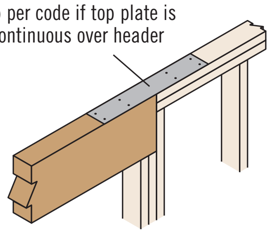
BEAM DETAILS

Bearing at Wall



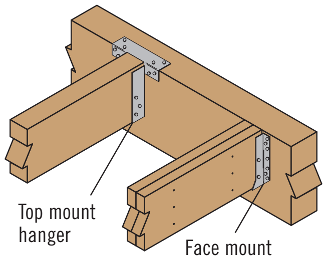
L1

Bearing for Door or Window Header



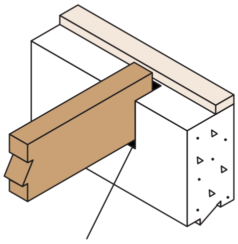
L2

Beam to Beam Connection



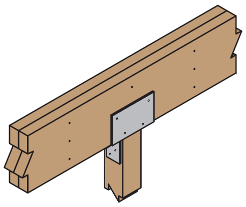
L3

Bearing at Concrete Wall



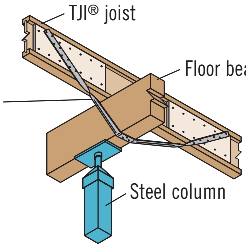
L4

Bearing at Column



L5 *Verify beam bearing length on page 12 and column factored resistance on page 17*

Beam to Column Lateral Brace

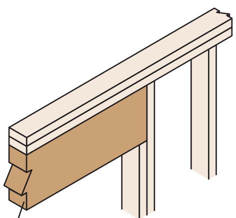


L14 *Suggested lateral bracing detail for beams when required. Verify beam bearing length on page 12.*

WINDOW AND DOOR HEADER DETAILS

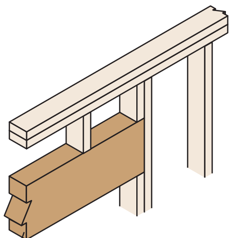
2x4 Wall Framing

Full Depth Header



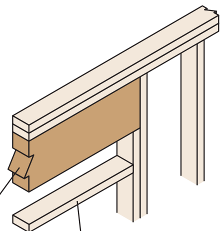
L7

Low Header



L8

High Header

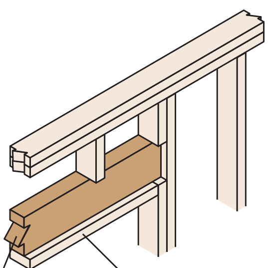


L9 **Double nailer may be required depending upon the opening size and window type*

2x6 Wall Framing

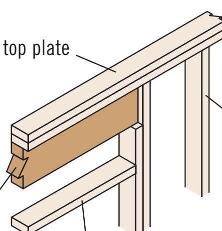
Headers not matching wall thickness may be installed flush to the inside or outside of the wall depending upon sheathing and trim attachment requirements

Low Header



L11

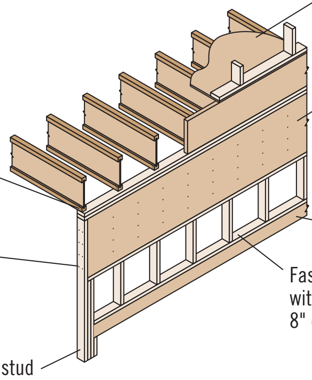
High Header



L12 **Double nailer may be required depending upon the opening size and window type*

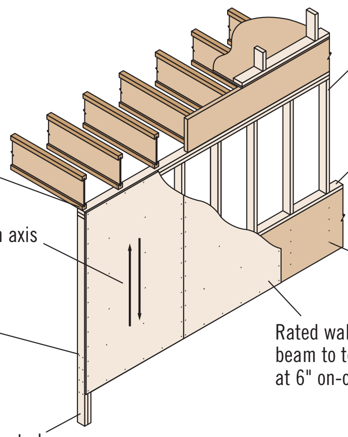
WINDOW AND DOOR HEADER DETAILS

Dropped Header with Full Lateral Bracing



L15

Dropped Header with Acceptable Lateral Bracing



L16

When framed as shown above, the following dropped headers are considered fully braced under uniform-load, simple-span conditions:

Single-ply:
– 1¾" wide headers, 11⅞" deep or less
– 3½" wide headers, 16" deep or less, with a maximum span of 18'-6"

Multiple-ply:
– Headers up to four 1¾" plies, 11⅞" deep or less
– Headers up to four 1¾" x 14" plies, with a maximum span of 8'-6"

NAILING ON NARROW FACE

Nails Installed on the Narrow Face

Nail Size	Closest On-Centre Spacing Per Row		
	TimberStrand® LSL	Microllam® LVL	Parallam® PSL
8d (0.131" x 2½") or 10d (0.128" x 3")	3"	4"	4"
10d (0.148" x 3") or 12d (0.148" x 3¼")	3"	5"	4"
16d (0.162" x 3½")	6"(1)	8"(2)	6"
(0.131" x 3"-3½")	3"	4"	4"

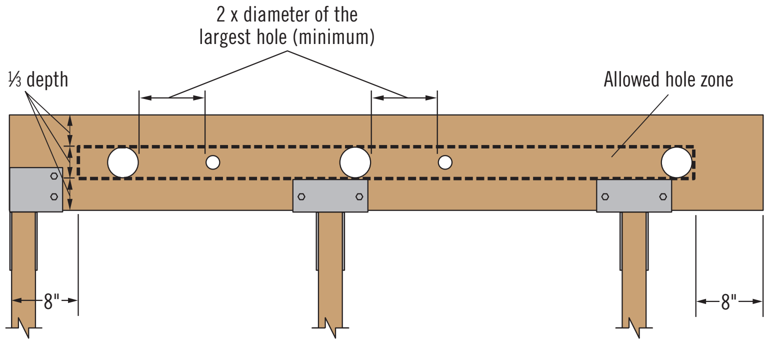
(1) Can be reduced to 3½" on-centre if nail penetration into the narrow edge is no more than 1¼" (to minimize splitting).
(2) Can be reduced to 5" on-centre if nail penetration into the narrow edge is no more than 1¼" (to minimize splitting).

- To minimize splitting, member edge distance and spacing between rows shall be 2.5 x nail diameter or ⅜", whichever is greater. Where multiple rows are used, fasteners in adjacent rows must be staggered and the rows must be equally spaced from the centreline of the narrow face axis.

Fastener spacing not applicable for shear wall applications. See CCMC 12627-R report for grade specific TimberStrand® LSL nailing requirements for shear walls.

ALLOWABLE HOLES

1.55E TimberStrand® LSL Headers and Beams



General Notes

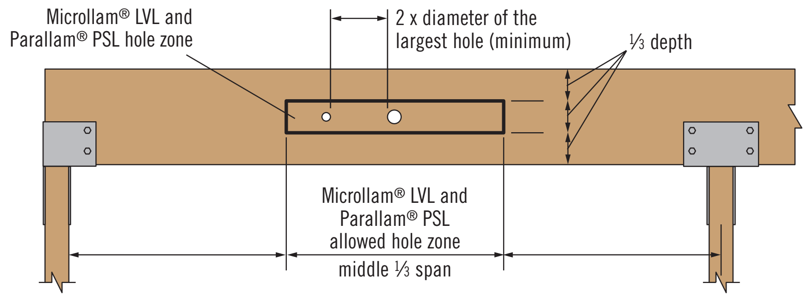
- Allowed hole zone suitable for headers and beams with **uniform and/or concentrated loads** anywhere along member.
- Round holes only.
- No holes in headers or beams in plank orientation.

1.55E TimberStrand® LSL

Header or Beam Depth	Maximum Round Hole Size
9½"	3"
11⅞"	3⅝"
14"–16"	4⅝"

- See illustration for allowed hole zone.

Microllam® LVL and Parallam® PSL Headers and Beams



General Notes

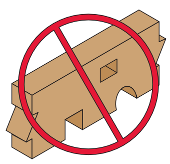
- Allowed hole zone suitable for headers and beams with **uniform loads only**.
- Round holes only.
- No holes in cantilevers.
- No holes in headers or beams in plank orientation.

Microllam® LVL and Parallam® PSL

Header or Beam Depth	Maximum Round Hole Size
5½"	1¾"
7¼"–20"	2"

- See illustration for allowed hole zone.

Larger holes in Trus Joist® structural composite lumber may be possible; refer to ForteWEB® or Javelin® software.



DO NOT cut, notch, or drill holes in headers or beams except as indicated in the illustrations and tables

Safety data sheets for all Weyerhaeuser wood products can be found on our website at:
weyerhaeuser.com/sustainability/environment/product-stewardship/safety-data-sheets.

BEARING LENGTH REQUIREMENTS

Factored Reaction (lbs)	1.55E TimberStrand® LSL			2.0E Microllam® LVL			2.2E Parallam® PSL		
	Beam Orientation			Beam Orientation			Beam Orientation		
	Width			Width			Width		
	1¾"	3½"	5¼"	1¾"	3½"	5¼"	3½"	5¼"	7"
6,000	2 ¾''	1 ½''	1 ½''	3¼"	1¾"	1½"	2"	1½"	1½"
8,000	3 ½''	1 ¾''	1 ½''	4¼"	2¼"	1½"	2¾"	1¾"	1½"
10,000	4 ½''	2 ¼''	1 ½''	5¼"	2¾"	1¾"	3¼"	2¼"	1¾"
12,000	5 ¼''	2 ¾''	1 ¾''	6½"	3¼"	2¼"	4"	2¾"	2"
14,000	6 ¼''	3 ¼''	2 ¼''	7½"	3¾"	2½"	4½"	3"	2¼"
16,000	7''	3 ½''	2 ½''		4¼"	3"	5¼"	3½"	2¾"
18,000	8''	4''	2 ¾''		4¾"	3¼"	5¾"	4"	3"
20,000		4 ½''	3''		5¼"	3½"	6½"	4¼"	3¼"
22,000		5''	3 ¼''		6"	4"	7"	4¾"	3½"
24,000		5 ¼''	3 ½''		6½"	4¼"	7¾"	5¼"	4"
26,000		5 ¾''	4''		7"	4¾"		5½"	4¼"
28,000		6 ¼''	4 ¼''		7½"	5"		6"	4½"
30,000		6 ¾''	4 ½''		8"	5¼"		6½"	4¾"
32,000		7''	4 ¾''			5¾"		6¾"	5¼"
34,000		7 ½''	5''			6"		7¼"	5½"

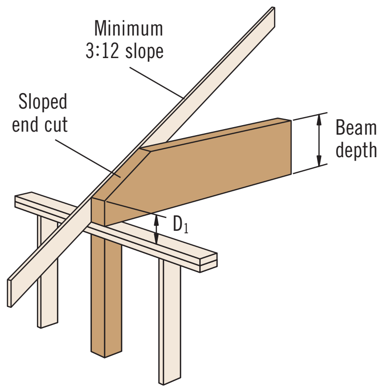
General Notes

- **Minimum bearing length:** 1½" at ends, 3½" at intermediate supports.
- Bearing across full beam width required.
- Interpolation between reaction loads is permitted for determining bearing lengths.
- Bearing lengths based on the following factored bearing resistances:
  - **1.55E TimberStrand® LSL:** 1,165 psi.
  - **2.0E Microllam® LVL:** 1,090 psi.
  - **2.2E Parallam® PSL:** 905 psi.

Factored Reactions for 3½"(1) TimberStrand® LSL Headers and Beams (lbs)

Bearing	Beam Depth	Outside Heel Height D_1							
		4½"	5"	5½"	6"	6½"	7"	7½"	8"
3½" Wood Plate(2)	9½"–11⅞"	7,535	7,535	7,535	7,535	7,535	7,535	7,535	7,535
	14"		7,535	7,535	7,535	7,535	7,535	7,535	7,535
	16"				7,535	7,535	7,535	7,535	7,535
5¼" Wood Plate(2)	9½"	8,775	9,530	10,285	11,035	11,300	11,300	11,300	11,300
	11⅞"–14"	8,775	9,530	10,285	11,035	11,300	11,300	11,300	11,300
	16"			10,285	11,035	11,300	11,300	11,300	11,300
3½" Column(3)	9½"	8,115	8,870	9,620	10,375	11,130	11,470	11,470	11,470
	11⅞"	8,115	8,870	9,620	10,375	11,130	11,885	12,640	13,395
	14"		8,870	9,620	10,375	11,130	11,885	12,640	13,395
	16"				10,375	11,130	11,885	12,640	13,395

(1) For 1¾" and 5¼" beams, multiply by 0.5 and 1.5, respectively.
(2) Bearing lengths are based on factored bearing resistance of 615 psi.
(3) Bearing lengths are based on factored bearing resistance of 1,165 psi.



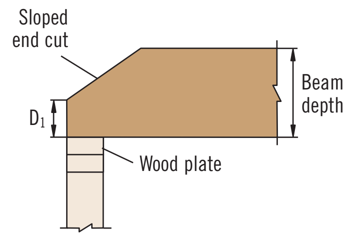
Tapered end cut detailed above is not allowed with TJI® joists

Factored Reactions for 3½"(1) Microllam® LVL and Parallam® PSL Headers and Beams (lbs)

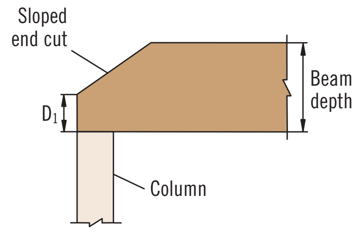
Bearing	Beam Depth	Outside Heel Height D_1								
		4½"	5"	5½"	6"	6½"	7"	7½"	8"	10"
3½" Wood Plate(2)	7¼"	7,480	7,535	7,535	7,535					
	9¼"	7,480	7,535	7,535	7,535	7,535	7,535	7,535	7,535	
	9½"	7,480	7,535	7,535	7,535	7,535	7,535	7,535	7,535	
	11¼"	7,480	7,535	7,535	7,535	7,535	7,535	7,535	7,535	7,535
	11⅞"	7,480	7,535	7,535	7,535	7,535	7,535	7,535	7,535	7,535
	14"		7,535	7,535	7,535	7,535	7,535	7,535	7,535	7,535
	16"				7,535	7,535	7,535	7,535	7,535	7,535
	18"					7,535	7,535	7,535	7,535	7,535
	19"						7,535	7,535	7,535	7,535
	20"							7,535	7,535	7,535
5¼" Wood Plate(2)	7¼"	8,070	8,070	8,070						
	9¼"	8,085	8,780	9,480	10,175	10,295	10,295	10,295		
	9½"	8,085	8,780	9,480	10,175	10,575	10,575	10,575	10,575	
	11¼"	8,085	8,780	9,480	10,175	10,870	11,300	11,300	11,300	
	11⅞"	8,085	8,780	9,480	10,175	10,870	11,300	11,300	11,300	11,300
	14"	8,085	8,780	9,480	10,175	10,870	11,300	11,300	11,300	11,300
	16"			9,480	10,175	10,870	11,300	11,300	11,300	11,300
	18"				10,175	10,870	11,300	11,300	11,300	11,300
	19"					10,870	11,300	11,300	11,300	11,300
	20"						11,300	11,300	11,300	11,300
3½" Column(3)	7¼"	7,480	8,070	8,070	8,070					
	9¼"	7,480	8,175	8,870	9,565	10,260	10,295	10,295	10,295	
	9½"	7,480	8,175	8,870	9,565	10,260	10,575	10,575	10,575	
	11¼"	7,480	8,175	8,870	9,565	10,260	10,955	11,125	11,125	11,125
	11⅞"	7,480	8,175	8,870	9,565	10,260	10,955	11,125	11,125	11,125
	14"		8,175	8,870	9,565	10,260	10,955	11,125	11,125	11,125
	16"				9,565	10,260	10,955	11,125	11,125	11,125
	18"					10,260	10,955	11,125	11,125	11,125
	19"						10,955	11,125	11,125	11,125
	20"							11,125	11,125	11,125

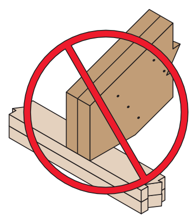
(1) For 1¾", 5¼", and 7" beams, multiply by 0.5, 1.5, and 2.0, respectively.
(2) Bearing lengths based on a factored bearing resistance of 620 psi.
(3) Bearing lengths based on factored bearing resistance of 905 psi for Microllam® LVL and Parallam® PSL.

Wood Plate Connection



Column Connection



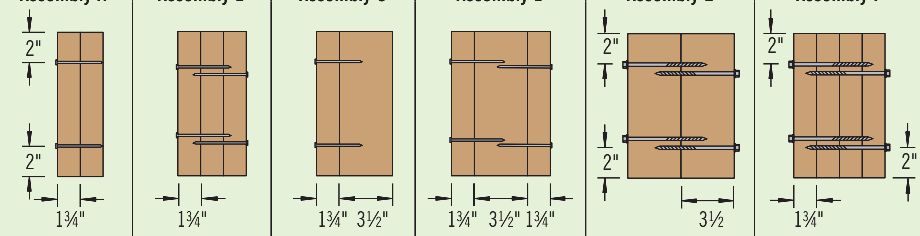


DO NOT overhang seat cuts on beams beyond inside face of support member

General Notes

- No increase for duration of load is permitted above standard term.
- No holes or concentrated load within tapered cut.
- Table considers only downward loading. Contact your Weyerhaeuser representative for assistance with uplift loading or other conditions.

L17 Factored Uniform Load—Maximum Factored Uniform Load Applied to Either Outside Member (PLF)



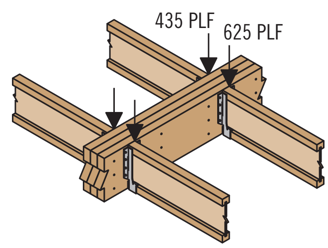
Fastener Type	Placement	Number of Rows	Fastener On-Center Spacing	Fastener Pattern					
				Assembly A	Assembly B	Assembly C	Assembly D	Assembly E	Assembly F
				3½" wide, 2-ply	5¼" wide, 3-ply	5¼" wide, 2-ply	7" wide, 3-ply	7" wide, 2-ply	7" wide, 4-ply
10d (0.128" x 3") or (0.131" x 3") Nail[1]	As shown	2[5]	12"	575	***430***	430	***385***		
		3	12"	865	***650***	650	***575***		
½" A307 Through Bolt[2][3]	–	2	24"	780	585	880	780	1,560	520
			19.2"	975	730	1,095	975	1,950	650
			16"	1,170	880	1,315	1,170	2,340	780
			Screw Length ▶	3½"	3½"	3½"	3½"	6"	6"
Simpson Strong-Tie® SDS[3]	As shown	2	24"	870	***655***	655	***580***	***2,325***	***680***
			19.2"	1,090	***815***	815	***725***	***2,905***	***850***
			16"	1,305	***980***	980	***870***	***3,485***	***1,020***
MiTek® WS[3]	As shown	2	24"	905	***680***	680	***605***		***765***[6]
			19.2"	1,130	***850***	850	***755***		***960***[6]
			16"	1,355	***1,015***	1,015	***905***		***1,150***[6]
			Screw Length ▶	3⅜"	5"	3⅜"	6¾"	6¾"	6¾"
Simpson Strong-Tie® SDW22[3][4]	One face	2	24"	680	625	585	555	1,140	555
			19.2"	850	780	730	690	1,425	690
			16"	1,020	935	880	830	1,710	830

(1) Nailed connection values may be doubled for 6" on-centre or tripled for 4" on-centre nail spacing.
(2) Washers required. Bolt holes to be 9⁄16" maximum.
(3) Factored resistance for 24" on-centre bolted or screwed connection values may be doubled for 12" on-centre spacing.
(4) When loading the head side of a SDW22 screw, assemblies A, B, D, and F can be increased by 15%.
(5) For beams up to 14" deep, maximum.
(6) Assembly F is not recommended for TimberStrand® LSL or Parallam® PSL.
• ***Bold italic*** loads indicate assemblies that require fastener placement on both faces. Stagger fasteners on the second face so they fall halfway between fasteners on the first face.

General Notes for Side-Loaded Beam Tables

- Connections are based on Limit States Design per CSA 086
- Use specific gravity of 0.5 for design of lateral connections
- Values listed are for standard term loading.
- Minimum end distance for bolts and screws is 6".
- Verify adequacy of beam in allowable load tables on pages 5–9.
- 7" wide beams should be side-loaded only when loads are applied to both sides of the members (to minimize rotation).
- Beams wider than 7" require special consideration by the design professional of record.

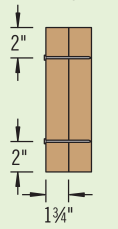
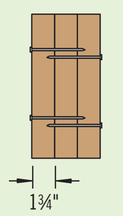
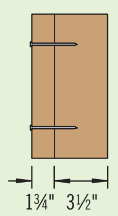
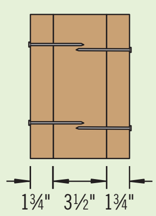
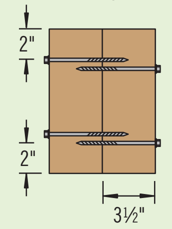
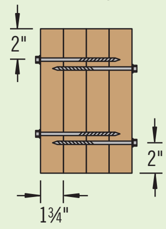
Uniform Load Design Example



First, check load tables on pages 5-9 to verify that three pieces can carry the total factored load of 1,060 plf with proper live load deflection criteria. Total factored load = (1.25 x dead load) + (1.5 x live load). Maximum factored load applied to either outside member is 625 plf. For an assembly of three 1¾" plies (Assembly B), two rows of (0.131" x 3") nails on both faces at 12" on-centre is good for only 430 plf. Therefore, use three rows of (0.131" x 3") nails on both faces at 12" on-centre (good for 650 plf).

Alternatives: Two rows of ½" A307 bolts at 19.2" on-centre or two rows of 5" SDW22 screws on one face at 24" on-centre.

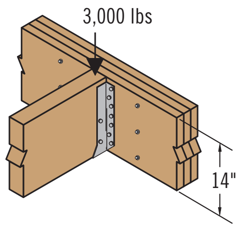
L18 Factored Point Load—Maximum Factored Point Load Applied to Either Outside Member (lbs)

Fastener Type	Placement	Number of Fasteners per Face	Fastener Pattern					
			Assembly A	Assembly B	Assembly C	Assembly D	Assembly E	Assembly F
								
			3½" wide, 2-ply	5¼" wide, 3-ply	5¼" wide, 2-ply	7" wide, 3-ply	7" wide, 2-ply	7" wide, 4-ply
10d (0.128" x 3") or (0.131" x 3") Nail	As shown	6	1,730	***1,295***	1,295	***1,150***		
		12	3,455	***2,590***	2,590	***2,305***		
		18	5,185	***3,890***	3,890	***3,455***		
		24	6,910	***5,185***	5,185	***4,610***		
		Screw Length ▶	3½"	3½"	3½"	3½"	6"	6"
Simpson Strong-Tie® SDS	As shown	4	3,480	***2,610***	2,610	***2,320***	***9,295***	***2,720***
		6	5,220	***3,915***	3,915	***3,480***	***13,945***	***4,080***
		8	6,960	***5,220***	5,220	***4,640***	***18,590***	***5,440***
MiTek® WS	As shown	4	3,615	***2,710***	2,710	***2,410***		***3,065***(2)
		6	5,425	***4,070***	4,070	***3,615***		***4,600***(2)
		8	7,230	***5,425***	5,425	***4,820***		***6,135***(2)
		Screw Length ▶	3⅜"	5"	3⅜"	6¾"	6¾"	6¾"
Simpson Strong-Tie® SDW22(1)	One face	4	2,720	2,490	2,340	2,215	4,560	2,215
		6	4,080	3,735	3,510	3,320	6,840	3,320
		8	5,440	4,980	4,680	4,425	9,120	4,425

(1) When loading the head side of a SDW22 screw, assemblies A, B, D, and F can be increased by 15%.
(2) Assembly F is not recommended for TimberStrand® LSL or Parallam® PSL.
• ***Bold italic*** loads indicate assemblies that require fastener placement on both faces. For screws required on both faces, refer to screw manufacturer's guidelines for minimum spacing requirements.

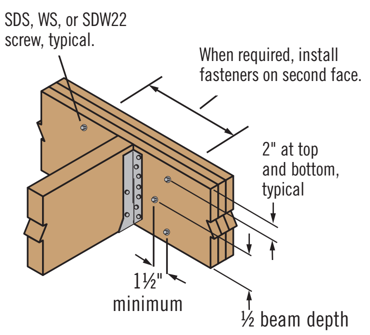
Point Load Design Example

First, verify that a 3-ply, 1¾" x 14" beam can support the factored 3,000 lb point load and all other loads applied. The factored 3,000 lb point load is being transferred to the beam with a face mount hanger. For an assembly of three 1¾" plies (Assembly B), six 5" SDW22 screws on one face are good for 3,735 lbs with a face mount hanger.



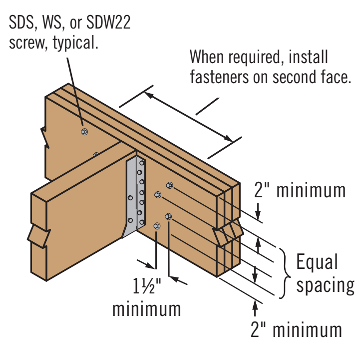
Point Load Fastener Spacing

4- or 6-Screw Connection



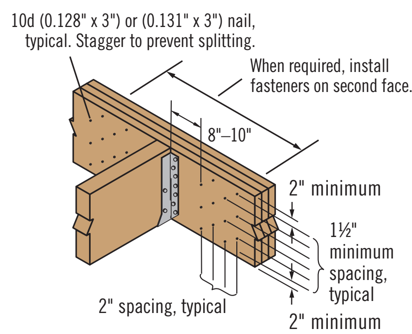
L19

8-Screw Connection



L20

Nail Connection



L21 ***There must be an equal number of nails on each side of the point load***

See table above for placement and number of fasteners per face.

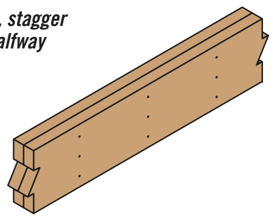
MULTIPLE-MEMBER CONNECTIONS FOR TOP-LOADED BEAMS

Fastener Installation Requirements

Piece Width	Number of Plies	Fastener				
		Type(1)	Min. Length	Placement	# Rows	O.C. Spacing
1¾"	2	10d nails	3"	One face	3(2)	12"
		12d–16d nails	3¼"		2(2)	
		Screws	3⅜" or 3½"	One face	2	24"
	3	10d nails	3"	Both faces	3(2)	12"
		12d–16d nails	3¼"		2(2)	
		Screws	3⅜" or 3½"	Both faces	2	24"
			5"	One face		
	4	10d nails(3)	3"	One face (per ply)	3(2)	12"
		12d–16d nails(3)	3¼"		2(2)	
		Screws	5" or 6"	Both faces	2	24"
			6¾"	One face		
3½"	2	Screws	5" or 6"	Both faces	2	24"
			6¾"	One face		
		½" bolts	8"	–		

(1) 10d nails are 0.128"–0.131" diameter; 12d–16d nails are 0.148"–0.162" diameter; screws are SDS, WS, or SDW22.
(2) An additional row of nails is required with depths of 14" or greater.
(3) When connecting 4-ply members, nail each ply to the other and offset nail rows by 2" from the rows in ply below.

When fasteners are required on both faces, stagger fasteners on the second face so they fall halfway between fasteners on the first face.



Load must be applied evenly across entire beam width. Otherwise, use connections for side-loaded beams

L6 *Multiple pieces can be nailed or bolted together to form a header or beam of the required size, up to a maximum width of 7"*

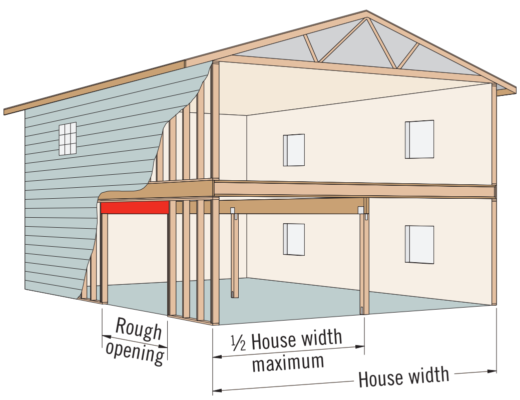
Metric to Imperial Conversions

Metric Unit	Imperial Conversion
1 kN	0.2248 kip
1 N	0.2248 lb
1 m	3.281 ft
1 mm	0.0394 in.
1 kg	2.205 lb mass
1 N • m	0.7376 lb • ft
1 N • m	8.851 lb • in.
1 mm^4	2.402 x 10^{-6} in.^{4}
1 Pa	0.0209 lb/ft^2
1 kPa	0.1450 lb/in.^{2}

Imperial to Metric Conversions

Imperial Unit	Metric Conversion
1 kip	4.448 kN
1 lb	4.448 N
1 ft	0.3048 m
1 in.	25.40 mm
1 lb mass	0.4536 kg
1 lb • ft	1.356 N • m
1 lb • in.	0.1130 N • m
1 in.^{4}	0.4162 x 10^6 mm^4
1 lb/ft^2	47.88 Pa
1 lb/in.^{2}	6.895 kPa

HEADER DESIGN EXAMPLE PROBLEM



Determine the size of 1.55E TimberStrand® LSL header required for a 10' rough opening for the given loads and assumptions:

- House width = 36'
- Trussed roof with 24" roof truss overhangs
- Roof Load = 30 psf snow + 15 psf dead
- Floor Load = 40 psf live + 12 psf dead

Calculated unfactored plf loads acting on the beam (20' roof and 9' floor tributary):

- Snow = 600 plf
- Floor = 360 plf
- Dead = 490 plf (includes wall load at 80 plf)

Next, calculate design loads per 2010 NBCC load combinations (primary load and companion load action).

1. **Unfactored live load:**
   Case 2: 1.0 x 360 + 0.5 x 600 = 660 plf
   Case 3: 1.0 x 600 + 0.5 x 360 = 780 plf
   Therefore use Case 3 at 780 plf
2. **Unfactored total load:**
   For Cases 2 and 3:
   Unfactored dead load = 1.0 x 490 = 490 plf
   Unfactored total load = 780 plf + 490 plf = 1,270 plf
3. **Factored total load:**
   Case 2: 1.5 x 360 + 0.5 x 600 = 840 plf
   Case 3: 1.5 x 600 + 0.5 x 360 = 1,080 plf
   Therefore use Case 3 at 1,080 plf
   Factored dead load = 1.25 x 490 = 613 plf
   Factored total load = 1,080 + 613 = 1,693 plf

Try using a 3½" x 11⅞" 1.55E TimberStrand® LSL header. See page 5 of this guide.

Span	Condition	1.55E Grade							
		1¾" Width			3½" Width				5½
		9½"	11⅞"	14"	9½"	11⅞"	14"	16"	9½"
10'	Unfactored Resistance (LL)	261	487	760	523	974	1,520	2,154	785
	Unfactored Resistance (TL)	387	724	*	775	1,449	*	*	1,16
	Total Factored Resistance	686	1,052	1,442	1,373	2,105	2,885	3,725	2,05
	Min. End/Int. Bearing (in.)	1.5/3.8	2.3/5.8	3.2/9.7	1.5/3.8	2.3/5.8	3.2/7.9	4.1/10.2	1.5/3.8
	Résistance non pondérée (S)	155	293	464	311	587	928	1,334	
		228			456	868			

Summary:

1. **Unfactored Resistance (LL)** = 974 > 780 OK
2. **Unfactored Resistance (TL)** = 1,449 > 1,270 OK
3. **Total Factored Resistance** = 2,106 > 1,693 OK

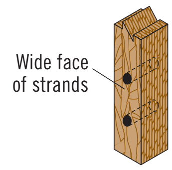
Therefore, a 3½" x 11⅞" 1.55E TimberStrand® LSL header is acceptable. The beam requires 2.6" of bearing at end supports and 6.5" of bearing at intermediate support.

Axial Factored Resistances (lbs) for 1.8E Parallam® PSL

Column Bearing Type	Effective Column Length	Column Size					
		3½" x 3½"	3½" x 5¼"	3½" x 7"	5¼" x 5¼"	5¼" x 7"	7" x 7"
On Column Base	6'	19,365	29,020	38,435	54,735	72,980	100,000
	7'	16,245	24,365	32,490	51,350	68,470	100,000
	8'	13,305	19,955	26,610	47,425	63,230	96,390
	9'	10,875	16,315	21,750	43,155	57,540	92,070
	10'	8,900	13,350	17,800	38,740	51,655	87,170
	12'	6,015	9,025	12,030	29,760	39,680	76,175
	14'	4,145	6,215	8,275	22,775	30,370	64,230
	16'	Slenderness ratio exceeds 50			17,480	23,310	52,685
	18'				13,500	17,995	43,130
	20'				10,510	14,010	35,345
	22'						29,040
	24'						23,945

The column values listed are for dry-service conditions ONLY. When wet-service conditions exist, contact your Weyerhaeuser representative for other product solutions.

DO NOT install bolts or screws into the narrow face of strands



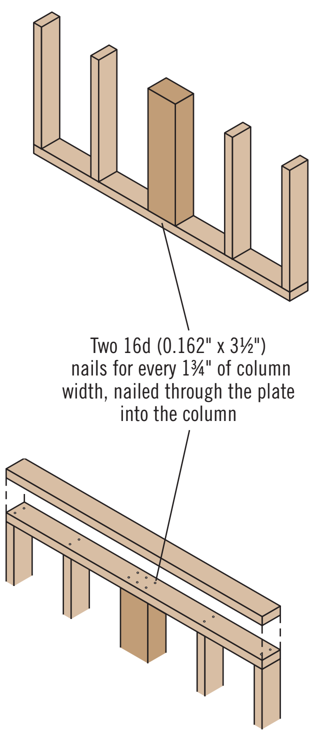
In order to use the manufacturer's published capacities when designing column caps, bases, or holdowns for uplift, the bolts or screws must be installed perpendicular to the wide face of strands, as shown above.

General Notes

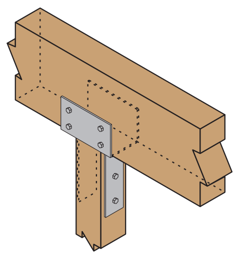
- Tables are based on:
  - Solid, one-piece column members used in dry-service conditions.
  - Bracing in both directions at column ends.
  - CSA O86.
  - Simple columns with axial loads only. For side loads or other combined bending and axial loads, see the CSA O86 provisions.
  - $K_D = 1.0$, where the specified snow or live load is greater than the specified dead load. For other load cases, use Weyerhaeuser software.
- Factored resistances have been adjusted to accommodate the worst case of the following eccentric conditions: ⅙ of column thickness (first dimension) or ⅙ of column width.
- Beams and columns must remain straight to within $5L^2/4608$ (in.) of true alignment. L is the unrestrained length of the member in feet.

For column specified strengths see page 4.

Top or Bottom Plate Connection

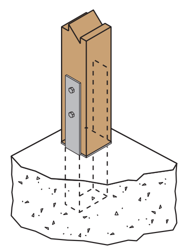


Beam on Column Cap



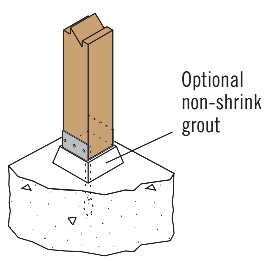
P1

Column Base



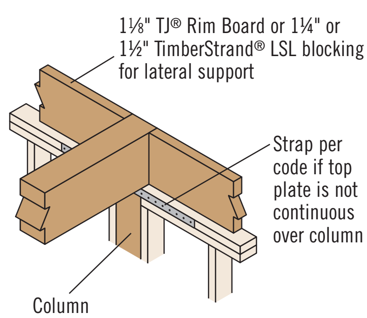
P2

Elevated Column Base



P3

Beam on Column



L1

NOTES

NOTES

WE CAN HELP YOU BUILD SMARTER

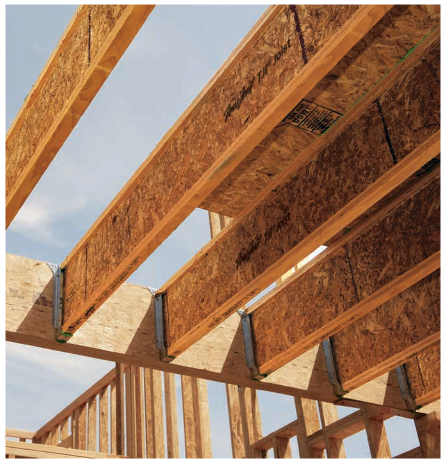
You want to build solid and durable structures—we want to help. Weyerhaeuser provides high-quality building products and unparalleled technical and field assistance to support you and your project from start to finish.

Floors and Roofs: Start with the best framing components in the industry: our Trus Joist® TJI® joists; TimberStrand® LSL rim board; and TimberStrand® LSL, Microllam® LVL, and Parallam® PSL headers and beams. Pull them all together with our self-gapping and self-draining Weyerhaeuser Edge Gold™ floor panels and durable Weyerhaeuser roof sheathing.

Walls: Get the best value out of your framing package—use TimberStrand® LSL studs for tall walls, kitchens, and bathrooms, and our traditional, solid-sawn lumber everywhere else. Cut down installation time by using TimberStrand® LSL headers for doors and windows, and Weyerhaeuser wall sheathing with its handy two-way nail lines.

Software Solutions: Whether you are a design professional or lumber dealer, Weyerhaeuser offers an array of software packages to help you specify individual framing members, create cut lists, manage inventories—even help you design a complete structural frame. Contact your Weyerhaeuser representative to find out how to get the software you need.

Technical Support: Need technical help? Weyerhaeuser has one of the largest networks of engineers and sales representatives in the business. Call us for help, and a skilled member from our team of experts will answer your questions and work with you to develop solutions that meet all your structural framing needs.



LIMITED LIFETIME
PRODUCT WARRANTY

Weyerhaeuser provides a limited warranty for the expected life of the structure for all Trus Joist® branded products. Product information, installation instructions, and the full text of each product's limited warranty (including limitations and exclusions) are available on the Weyerhaeuser website, from your Weyerhaeuser representative, or by calling toll free: 888-453-8358.

Additionally, Weyerhaeuser offers limited warranties on a broad variety of its other products. To see complete details of all Weyerhaeuser product warranties, visit weyerhaeuser.com/wood products/warranty.

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Contact your local representative or dealer at:

April 2023 • Reorder TJ-9505
This document supersedes all previous versions. If this is more than one year old, contact your dealer or Weyerhaeuser rep.

GENERAL NOTES

1. ALL CONSTRUCTION MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE REQUIREMENTS OF THE DRAWINGS, SPECIFICATIONS, AND THE CODES, RULES AND REGULATIONS OF INTERNATIONAL BUILDING CODE (IBC) 2018 EDITION.
2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS PRIOR TO CONSTRUCTION. THE ARCHITECT SHALL BE NOTIFIED OF ANY DISCREPANCIES OR INCONSISTENCIES.
3. IF ANY ERRORS OR OMISSIONS APPEAR IN THESE DRAWINGS, SPECIFICATIONS, OR OTHER DOCUMENTS; THE CONTRACTOR SHALL NOTIFY THE STRUCTURAL ENGINEER OR ARCHITECT IN WRITING OF SUCH OMISSION OR ERROR BEFORE PROCEEDING WITH THE WORK.
4. MANUFACTURED MATERIALS SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE. ALL REQUIREMENTS OF THOSE APPROVALS SHALL BE FOLLOWED.
5. ALL STRUCTURAL SYSTEMS THAT ARE TO BE COMPOSED OF MANUFACTURED COMPONENTS TO BE FIELD ERECTED SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE AND SHALL BE SUPERVISED BY THE SUPPLIER DURING MANUFACTURING, DELIVERY, HANDLING, STORAGE, AND ERECTION IN ACCORDANCE WITH INSTRUCTIONS PREPARED BY THE SUPPLIER
6. FRAMING MEMBERS THAT ARE NOT DIMENSIONED SHALL BE EQUALLY SPACED BETWEEN DIMENSIONED POINT OR MEMBERS.
7. SEE ARCHITECTURAL DRAWINGS AND PROJECT SPECIFICATIONS FOR THE FOLLOWING:
   SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS AND THRESHOLD REQUIREMENTS.
   SIZE AND LOCATION OF ALL NON-BEARING PARTITIONS.
   SIZE AND LOCATION OF ROOF, FLOOR AND WALL OPENINGS.
   SIZE AND LOCATION OF DEPRESSED AREAS, CHANGES IN ELEVATION, FLOOR AND ROOF DRAINS,
   SLOPES, CONCRETE CURBS, LEDGES, PADS AND ISLANDS, CHAMFERS, GROOVES, INSERTS, ETC.
   DIMENSIONS NOT SHOWN ON THE STRUCTURAL DRAWINGS, SIZE, WEIGHT AND LOCATION OF MACHINES AND EQUIPMENT BASES.
8. THE CONTRACT DOCUMENTS REPRESENT THE FINISHED STRUCTURE, THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT NOT BE LIMITED TO, BRACING, SHORING FOR LOADS DUE TO CONSTRUCTION EQUIPMENT, ETC. OBSERVATION VISITS TO THE SITE BY THE STRUCTURAL ENGINEER SHALL NOT INCLUDE INSPECTION OF THE ABOVE ITEMS.
9. OPENINGS, POCKETS, ETC. SHALL NOT BE PLACED IN STRUCTURAL MEMBERS UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL DRAWINGS. NOTIFY THE STRUCTURAL ENGINEER WHEN DRAWINGS BY OTHERS SHOW OPENINGS, POCKETS, ETC., LARGER THAN 6 INCHES NOT SHOWN ON THE STRUCTURAL DRAWINGS, BUT WHICH ARE LOCATED IN STRUCTURAL MEMBERS.
10. SPECIFICATIONS, CODES, AND STANDARDS NOTED IN THE CONTRACT DOCUMENTS SHALL BE OF THE LATEST APPROVED ISSUE, INCLUDING SUPPLEMENTS, UNLESS OTHERWISE NOTED. MATERIAL SPECIFICATIONS ARE ASTM LATEST EDITION.
11. CONTRACTOR SHALL PROVIDE TEMPORARY BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE PLANS.

DESIGN CRITERIA

LIVE LOADS

ROOF SNOW LOAD 25.0 PSF BASIC

DEAD LOADS

SUPERIMPOSED ROOF DEAD LOAD
FRAMING, CEILING, ETC. 15 PSF

SUPERIMPOSED WALL DEAD LOAD
EXTERIOR WALLS. 10 PSF

WIND DESIGN (PER 1615 -1622)
BASIC WIND SPEED 110 MPH
EXPOSURE B
IMPORTANCE FACTOR 1.0
TOPOGRAPHIC FACTOR 1.38

SEISMIC DESIGN (PER 1615 - 1633)
SEISMIC CATEGORY II
IMPORTANCE FACTOR= 1.0
MAPPED SPECTRAL RESPONSE ACCELERATION PARAMETERS:
S_s = 1.466
S_1 = 0.508 SITE CLASS = D
S_{ds} = 1.173 SEISMIC RISK CATEGORY = D
BASIC SEISMIC FORCE-RESISTING SYSTEMS:
LIGHT FRAMED WALLS SHEATHED WITH
WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE.
DESIGN BASE SHEAR: 47.88 KIPS
R= 6.5 - Wood Framed
R = 5.0 - Concrete
ANALYSIS METHODS USED:
WIND; METHOD 2 - ANALYTICAL PROCEDURE
SEISMIC; METHOD 2 - EQUIVALENT LATERAL FORCE

MAPPED SPECTRAL RESPONSE
ACCELERATIONS OBTAINED FROM THE USGS - SEISMIC HAZARD MAPS & DATA

FOUNDATIONS

1. ALL FOUNDATIONS SHALL BE FOUNDED A MINIMUM OF 18" BELOW LOWEST ADJACENT FINAL FINISH FLOOR OR GRADE. EXPOSED SOIL SHALL BE INSPECTED FOR COMPLIANCE BY THE ENGINEER OR HIS REPRESENTATIVE PRIOR TO CONSTRUCTING CONCRETE FORMS AND/OR PLACING REINFORCING STEEL. ANY EXCESS OR NON-COMPLYING MATERIAL AS DETERMINED BY THE ENGINEER OR HIS REPRESENTATIVE SHALL BE REMOVED AND REPLACED AS DIRECTED.
2. THE ALLOWABLE SOIL BEARING LOAD IS PER THE GEOTECHNICAL REPORT.

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE DETAILED, INCLUDING HOOKS AND BENDS, AND PLACED IN ACCORDANCE WITH ACI 315 AND ACI 318.
2. REINFORCING STEEL SHALL CONFORM TO ASTM A-615 OR A-706, GRADE 40 OR BETTER,
3. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A-185,
4. ALL REINFORCING BAR BENDS SHALL BE MADE COLD,
5. REINFORCING SPLICES SHALL BE MADE AS INDICATED ON THE DRAWINGS,
6. DOWELS BETWEEN FOOTINGS AND WALLS OR COLUMNS SHALL BE THE SAME GRADE, SIZE AND SPACING AS THE VERTICAL REINFORCING, RESPECTIVELY, UON,
7. NO BARS PARTIALLY EMBEDDED IN HARDENED CONCRETE SHALL BE FIELD BENT UNLESS SPECIFICALLY SO DETAILED AND REVIEWED BY THE STRUCTURAL ENGINEER
8. WELDING OF REINFORCEMENT SHALL BE WITH LOW HYDROGEN ELECTRODES IN CONFORMANCE WITH ACI 318-95 AND THE RECOMMENDATIONS OF THE AMERICAN WELDING SOCIETY, AWS DI,4 AND WITH THE REVIEW OF THE STRUCTURAL ENGINEER

CONCRETE

1. ALL CONCRETE CONSTRUCTION SHALL CONFORM TO THE BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE' ACI 318 AND ACI 301, WITH MODIFICATIONS AS NOTED IN THE CONTRACT DOCUMENTS,
2. PORTLAND CEMENT SHALL CONFORM TO ASTM C-150 TYPE 1 OR TYPE H,
3. COARSE AND FINE AGGREGATE FOR NORMAL WEIGHT CONCRETE SHALL CONFORM TO ASTM C-33,
4. WATER SHALL BE CLEAR AND SHALL CONFORM TO ASTM C-94,
5. CONCRETE MIXING OPERATION SHALL CONFORM TO ASTM C-94,
6. ADD TO ALL CONCRETE EXPOSED TO WEATHER MICROAIR OR MBVR AIR ENTRAINING AGENT TO ATTAIN 5 PERCENT +1- 1 PERCENT ENTRAINED AIR, BY VOLUME, CONFORMING TO ASTM C-260, ALL REFERENCE DATA USED FOR PAST PERFORMANCE DESIGN SHALL HAVE CONTAINED THE SAME ADMIXTURE BRAND AS THAT USED IN THE MIX SUBMITTED,
7. CONCRETE STRENGTHS SHALL BE VERIFIED BY 28-DAY CYLINDER TESTS, UNLESS OTHERWISE APPROVED, CONCRETE SHALL BE AS FOLLOWS:

ELEMENT TYPE	STRENGTH PSI CONCRETE
FOOTINGS, GRADE BEAMS	2,500 NORMAL WT
SLAB ON GRADE	2,500 NORMAL WT
FOUNDATION STEM WALLS	3,000 NORMAL WT
RETAINING WALLS	3,000 NORMAL WT

A MINIMUM 5 SACK MIX SHALL BE USED TO ACHIEVE THE DESIGN STRENGTHS LISTED ABOVE.

8. CONTRACTOR MAY USE AN ADMIXTURE SYSTEM TO PRODUCE FLOWABLE CONCRETE, MAXIMUM SLUMP SHALL NOT EXCEED 10 INCHES MEASURED AT THE PUMP, THE WATER/CEMENTIOUS MATERIAL RATIO OF THE APPROVED MIXES SHALL BE MAINTAINED OR LOWERED WHEN FLOWABLE CONCRETE IS USED,
9. THE FOLLOWING MINIMUM CONCRETE COVER SHALL BE PROVIDED FOR REINFORCEMENT PLACED IN CAST-IN-PLACE CONCRETE:

	CONCRETE COVER (MINIMUM)
A . CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH	3"
B. CONCRETE EXPOSED TO EARTH OR WEATHER:	
#6 THROUGH #18 BARS	2"
#5 BAR, W31 OR D31 WIRE, A1413 SMALLER	1 1/2"
C. CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH GROUND:	
SLABS, WALLS, JOISTS	
#14 AND #18 BARS	1 1/2"
#11 BARS AND SMALLER	3/4"
BEAMS, COLUMNS:	
PRIMARY REINFORCEMENT, TIES, STIRRUPS, SPIRALS	1 1/2"

10. PLACEMENT OF CONCRETE SHALL CONFORM TO ACI 304 AND THE CONTRACT DOCUMENTS. SANDBLAST ALL CONCRETE SURFACES AGAINST WHICH CONCRETE IS TO BE PLACED,
11. ALL REINFORCING BARS, ANCHOR BOLTS AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION PRIOR TO PLACING CONCRETE,
12. PROVIDE SLEEVES FOR PLUMBING AND ELECTRICAL OPENINGS IN CONCRETE BEFORE PLACING, REINFORCING SHALL NOT BE CUT, CORING OF CONCRETE IS NOT PERMITTED EXCEPT AS INDICATED,
13. CURING COMPOUNDS USED ON CONCRETE TO RECEIVE A FINISH SHALL BE APPROVED BY THE FINISH APPLICATOR BEFORE USE.

DESIGN LOADING:
REF. SOIL REPORT
EARTH SOLUTIONS NW, LLC
Dated: October 4, 2023
Pa = 42 PCF
Pp = 200 PCF
Seismic loading = 8H
Allowable Bearing Pressure = 2,500 PSF

WOOD

1. FRAMING LUMBER SHALL BE GRADED AND MARKED IN CONFORMANCE WITH WCLB STANDARD GRADING AND DRESSING RULES FOR WEST COAST LUMBER NO, 16, LATEST EDITION. UNLESS OTHERWISE NOTED ON THE DRAWINGS, LUMBER GRADES SHALL BE AS FOLLOWS:
   A. JOISTS: 2" AND 3" THICKNESS, HEM FIR NO, 1,
   B. BEAMS AND STRINGERS: DOUGLAS FIR NO, 1,
   C. POST AND TIMBERS: DOUGLAS FIR NO, 1,
   D. PLATES AND MISCELLANEOUS LIGHT FRAMING: HEM FIR STANDARD,
   E. STUDS: HEM FIR STUD.
   F. ALL BOLTED CONNECTIONS TO BE 3/4"Ø A302 BOLTS
2. MINIMUM NAILING REQUIREMENTS:

   UNLESS OTHERWISE NOTED, MINIMUM NAILING SHALL CONFORM TO THE GOVERNING CODE AND AS FOLLOWS:
   A. JOISTS OR RAFTERS TO SIDES OF STUDS 8-INCH OR LESS 3-16DB
   B. FOR EACH ADDITIONAL 4-INCH IN DEPTH OF JOISTS 1-16DC
   C. JOISTS OR RAFTERS AT ALL BEARINGS - TOENAILS EACH SIDE 2-10DD
   D. STUDS TO BEARING - TOENAILS EACH SIDE 2-10DE
   E. BLOCKING BETWEEN JOISTS OR RAFTERS
      TO JOIST OR RAFTERS - TOENAILS EACH SIDE EACH END 2-10D
      TO JOIST OR RAFTER BEARINGS - TOENAILS EACH SIDE 2-10D
   F. CROSS-BRIDGING BETWEEN JOISTS OR RAFTERS TOE NAILS EACH END 2-8D
   G. BLOCKING BETWEEN STUDS - TOENAILS EACH END 2-10D
   H. DOUBLE TOP PLATES - LOWER PLATE TO TOP OF STUD 2-16D
   J. UPPER TO LOWER PLATE - STAGGERED 16D @ 16" O,C,
   K. MULTIPLE JOISTS - STAGGERED 16D @ 12" O,C,
   L. MULTIPLE JOISTS STAGGER FOR WIDTHS MORE THAN 4 INCHES 16D @ 12" O,C.
3. INDIVIDUAL MEMBERS OF BUILT-UP POSTS AND BEAMS SHALL EACH BE ATTACHED WITH 16D SPIKES AT 12" O,C. STAGGERED, MIN,
4. ALL NAILS SHALL BE COMMON WIRE NAILS, WHENEVER POSSIBLE, NAILS DRIVEN PERPENDICULAR TO THE GRAIN SHALL BE USED, THERE SHALL BE A MINIMUM OF 2 NAILS AT ALL WOOD CONTACTS AND JOINTS USING 8D NAILS FOR 1-INCH THICK MATERIAL, 16D NAILS FOR 2-INCH THICK MATERIAL, AND 40D NAILS FOR 3-INCH THICK MATERIAL, ALL CONTINUOUS CONTACTS PROVIDE MINIMUM NAILS AT 12" O,C, WITH NAIL SIZES AS CALLED ABOVE,
5. NOTATIONS ON DRAWINGS RELATING TO FRAMING CLIPS, JOIST HANGERS, AND OTHER CONNECTING DEVICES REFER TO CATALOG NUMBERS OF STRONG-TIE CONNECTORS MANUFACTURED BY THE SIMPSON COMPANY, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER,
6. AT SAWN TIMBER JOISTS WITH THICKNESS-TO-DEPTH RATIO OF 1:6 AND GREATER, PROVIDE CROSS-BRIDGING AT 8' 0" O,C, AND SOLID BLOCKING AT BEARING POINTS,
7. ALL WOOD FRAMING DETAILS NOT SHOWN OTHERWISE SHALL BE CONSTRUCTED TO THE MINIMUM STANDARDS OF THE GOVERNING CODE,
8. ALL BEARING AND EXTERIOR STUD WALLS SHALL BE 2X6 @6"O.C. BELOW SECOND FLOOR AND 2X4 @ 16" O.C. ELSEWHERE, UNLESS OTHERWISE NOTED.
9. PROVIDE CONTINUOUS SOLID BLOCKING AT MID-HEIGHTS AND AT INTERVALS NOT TO EXCEED 8 FEET OF ALL STUD-BEARING WALLS OVER 8 FEET IN HEIGHT,
10. SEE ARCHITECTURAL DRAWINGS FOR LOCATIONS OF INTERIOR NONBEARING STUD PARTITIONS FOR LOCATION AND SIZE OF OPENINGS IN STUD WALLS, AND FOR ALL WALL FINISH DETAILS,
11. ALL CANTS AND CRICKETS SHALL BE PLACED OVER BASIC ROOF SHEATHING. SEE ARCHITECTURAL DRAWINGS FOR DETAILS AND LOCATIONS,
12. ALL WOOD STUD WALL SILL PLATES SHALL BE ATTACHED TO CONCRETE OR MASONRY WITH 1/2-INCH DIAMETER ANCHOR BOLTS AT 48" O.C,, UNLESS OTHERWISE NOTED ,
13. ALL WOOD STUD WALLS SHALL HAVE LOWER WOOD PLATE ATTACHED TO WOOD FRAMING BELOW WITH 16D NAILS AT 6" O.C, STAGGERED UNLESS SHOWN OTHERWISE.
14. FASTEN ALL POSTS TO CONCRETE WITH "CB" COLUMN BASE OR EQUAL,
15. ALL WOOD PLATES AND BLOCKING IN DIRECT CONTACT WITH CONCRETE OR MASONRY SHALL BE PRESSURE TREATED WITH AN APPROVED PRESERVATIVE IN ACCORDANCE WITH AWPS-FDN, AND BEAR THAT QUALITY MARK,
16. PROVIDE STANDARD CUT WASHERS UNDER ALL BOLTS HEADS AND NUTS IN CONTACT WITH WOOD.
17. ATTACH TIMBER JOISTS TO FLUSH HEADERS AND BEAMS WITH "U" SERIES METAL JOIST HANGERS TO SUIT THE JOIST SIZE.
18. ALL PLYWOOD SHALL BE HEM FIR, STRUCTURAL 2 OR BETTER AND SHALL CONFORM TO APA C-D INTERIOR GRADE WITH EXTERIOR GLUE, WITH UBC STANDARD 23-2 AND WITH PRODUCT STANDARD PSI, WOOD-BASED STRUCTURAL-USE PANELS SHALL CONFORM WITH UBC STANDARD 23-3 AND WITH PRODUCT STANDARD PS2, TYPE AND THICKNESS SHALL BE AS SPECIFIED ON THE PLANS.
19. PLYWOOD NAILING, USE UNLESS OTHERWISE NOTED:

A. ROOF:	8D @ 6" O.C. AT SHEET EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS
B. FLOOR:	10D @ 6" O.C. AT SHEET EDGES
	10D @ 10" O.C. AT INTERMEDIATE BEARING POINTS
C. WALLS:	8D @ 6" O.C. AT EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS

   PLYWOOD AND WOOD-BASED STRUCTURAL-USE PANELS USED FOR WALL SHEATHING SHALL HAVE SOLID BLOCKING AT ALL EDGES.
20. MACHINE APPLIED NAILING IS SUBJECT TO A SATISFACTORY DEMONSTRATION AND THE APPROVAL OF THE CHECKING AGENCY AND THE ARCHITECT, NAIL HEADS SHALL NOT PENETRATE THE OUTER PLY MORE THAN WOULD BE NORMAL FOR A HAND HAMMER. EDGE DISTANCES SHALL BE MAINTAINED, SHINERS SHALL BE REMOVED AND REPLACED, THE APPROVAL IS SUBJECT TO CONTINUED SATISFACTORY PERFORMANCE, MACHINE APPLIED NAILING ONLY ON PLYWOOD GREATER THAN 5/16".

STRUCTURAL STEEL, MISC. METAL

1. STRUCTURAL STEEL DETAILING, FABRICATION AND ERECTION SHALL BE BASED ON THE LATEST EDITION AND SUPPLEMENTS OF THE AISC "SPECIFICATION FOR STRUCTURAL STEEL FOR BUILDINGS - ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN", STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING REQUIREMENTS,

TYPE OF MEMBER	ASTM SPECIFICATION	FY
WIDE FLANGE SHAPES	A572 OR A992	50 KSI
PLATES, SHAPES, ANGLES, AND RODS	A36	36 KSI
HOLLOW STRUCTURAL SECTION (ROUND)	A53 (GRADE B)	36 KSI
HOLLOW STRUCTURAL SECTION (SQUARE OR RECTANGLE)	A500 (GRADE B)	46 KSI
ANCHOR RODS (EMBEDDED IN CONCRETE)	A307	

2. ALL WELDS SHALL BE PREQUALIFIED IN CONFORMANCE WITH AISC AND AWS STANDARDS AND SHALL BE PERFORMED BY WELDERS CERTIFIED IN THE JURISDICTION HAVING AUTHORITY OVER THIS PORTION OF THE WORK, USE E70XX ELECTRODES.3, WELD LENGTHS CALLED FOR ON THE PLANS ARE THE NET EFFECTIVE LENGTH REQUIRED, WELD SIZE SHALL BE AISC MINIMUM, UNLESS OTHERWISE NOTED.

ANCHORAGE

1. EXPANSION ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 4, CLASS 1,
2. SLEEVE ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 3, CLASS 3,
3. FLUSH SHELL ANCHORS SHALL ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP VIII, TYPE 1,
4. ADHESIVE ANCHORS SHALL CONSIST OF ALL-THREAD ANCHOR ROD, NUT, WASHER AND EPOXY INJECTION GEL OR ADHESIVE CAPSULE SYSTEM, ANCHOR RODS SHALL BE MANUFACTURED FROM A-36 MATERIAL, ZINC PLATED IN ACCORDANCE WITH ASTM B 633,
5. ALL RELATED PRODUCTS, MATERIALS AND INSTALLATION SHALL BE IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS,
6. NOTATIONS ON DRAWINGS RELATING TO EXPANSION, SLEEVE, FLUSH OR ADHESIVE ANCHORS AND OTHER CONNECTING DEVICES REFER TO CONNECTORS MANUFACTURED BY POWERS FASTENING, INC, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER

SPECIAL INSPECTION

1. SPECIAL INSPECTION BY A REGISTERED DEPUTY BUILDING INSPECTOR, APPROVED BY THE ARCHITECT AND THE CHECKING AGENCY SHALL BE REQUIRED FOR THE FOLLOWING TYPES OF WORK. SEE THE PROJECT SPECIFICATIONS FOR FURTHER REQUIREMENTS, SPECIAL INSPECTIONS SHALL NOT BE REQUIRED WHEN THE WORK IS DONE ON THE PREMISES OF A FABRICATOR REGISTERED AND APPROVED BY THE BUILDING OFFICIAL TO PERFORM SUCH WORK WITHOUT SPECIAL INSPECTION,
   SOIL
      EXCAVATION
      SOIL COMPACTION
   CONCRETE
      DESIGN STRENGTHS GREATER THAN 2,500 PSI PLACING OF REINFORCING STEEL
   WELDING
      STRUCTURAL STEEL
      REINFORCING STEEL
   FABRICATED TIMBER JOISTS
   EXPANSION TYPE ANCHOR BOLTS
   STRUCTURAL MASONRY CONSTRUCTION
   PILING, DRILLED OR DRIVEN
   STRUCTURAL STEEL FABRICATION
2. ALL PREPARED SOIL-BEARING SURFACES SHALL BE INSPECTED BY THE SOILS ENGINEER PRIOR TO PLACEMENT OF REINFORCING STEEL,
3. EXPANSION TYPE ANCHORS SHALL BE APPROVED BY THE CHECKING AGENCY FOR THEIR USE AND SHALL BE INSTALLED ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS,
4. THE OWNER, ARCHITECT, STRUCTURAL ENGINEER, AND BUILDING OFFICIAL SHALL BE FURNISHED WITH COPIES OF ALL TEST RESULTS.

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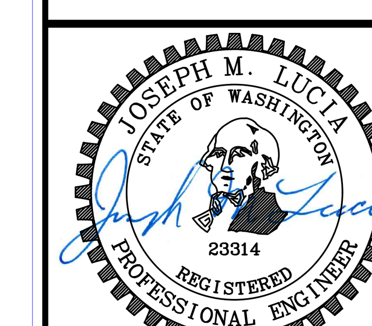

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SHEET
S-5.0

SHEAR WALL SCHEDULE

MARK	SHEATHING	NAILING (5)		LUMBER			SHEAR TRANSFER					1.4 INCREASE FOR WIND
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR	SILL PL	TOP PL'S	"A" SILL PL TO CONC.	"B" BLKG TO TOP PL.	"C" SILL PL RIM/JST/BLKG (F.N.)	"D" SHEAR WALL INTERSECTIONS	CAPACITY	CAPACITY
P1-8-6	3/8" APA RATED SHEATHING, ONE SIDE	8d@6"	8d@ 6"	2x	2x	(2)2x	5/8 @ 48"	A35@20" OR LPT4 @ 30"	16d @ 5"	16d @ 8"	270 PLF	378 PLF
P1-8-4	3/8" APA RATED SHEATHING, ONE SIDE	8d@4"	8d@ 6"	2x	2x	(2)2x	5/8 @ 40"	A35@16" OR LPT4 @ 20"	16d @ 5"	16d @ 5"	360 PLF	504 PLF
P1-8-3	3/8" APA RATED SHEATHING, ONE SIDE	8d@2-1/2"	8d@4"	2x	3x	(2)2x	5/8 @ 36"	A35@12" OR LPT4 @ 15"	20d @ 4"	16d @ 3 1/2"	530 PLF	742 PLF
P1-8-2	3/8" APA RATED SHEATHING, ONE SIDE	8d@2"	8d@ 3"	3x(9)	3x	(2)2x	5/8 @ 24"	A35@9" OR LPT4 @ 11"	20d @ 3"	1/2" x4 1/2" LAG @ 9"	610 PLF	854 PLF
P2-8-4	3/8" APA RATED SHEATHING, TWO SIDE	8d@4"	8d@ 6"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 9"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 6"	720 PLF	1008 PLF
P2-8-3	3/8" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@ 6"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 7"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 5"	980 PLF	1372 PLF
P2-8-2	3/8" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@3"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 6"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 4 1/2"	1220 PLF	1708 PLF

ROOF & FLOOR DIAPHRAGM NAILING SCHEDULE

DIA. #	DIAPHRAGM SHEATHING	NAILING (INCHES o.c.) 15/32" SHEATHING W/ 10d COMMON			
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR (KLF)	NOTES
	UNBLOCKED, OTHER		6	0.20	2x
	UNBLOCKED CASE#1		6	0.28	2x
1	BLOCKED	6	6	0.32	3x
2	BLOCKED	4	6	0.43	3x
3	BLOCKED	2.5	4	0.67	3x
4	BLOCKED	2	3	0.73	3x
5	BLOCKED	2	3	0.82	3x

DIAPHRAGM NOTES:

1. APA RATED SHEATHING, STURD-I-FLOOR EXP1/EXP2/EXT OR C-C/C-D PLYWOOD
2. STRUCTURAL 1 APA RATED SHEATHINGEXP1/EXT OR STRUCT 1 PLYWOOD
3. PROVIDE 3x'S (76mm) AT ADJOINING PANEL EDGES W/NAILS STAGGERED.
4. ALL MEMBERS TO BE 4x MINIMUM W/2 LINES OF FASTENERS (ICBO ER 1952)
5. ALL MEMBERS TO BE 4x MINIMUM W/3 LINES OF FASTENERS (ICBO ER 1952)
6. SPECIAL INSPECTION REQUIRED IN ACCORDANCE WITH ICBO ER 1952
7. PROVIDE BOUNDARY NAILING @ ALL PANEL EDGES, CASES 3,4,5 & 6.
8. ALL MEMBERS TO BE 3x (76mm) MINIMUM.

SHEAR WALL FRAMING NOTES:

1. IN ADDITION TO THE TYPICAL WALL FRAMING REQUIREMENTS PROVIDE FRAMING AT SHEAR WALLS AS INDICATED.
2. SEE SCHEDULE FOR SHEATHING AND NAILING REQUIRED. SCHEDULE ASSUMES HEM-FIR OR BETTER LUMBER. STAGGER PANEL JOINTS EACH SIDE OF WALL WHERE SHEATHING IS REQUIRED BOTH SIDE OF WALL.
3. STUD BLOCKING THICKNESS SHOWN ARE MINIMUM SIZES BASED ON SHEAR WALL NAILING REQUIREMENT. PROVIDE LARGER STUD WHERE REQUIRED OTHERWISE.
4. BLOCK ALL PANEL EDGES.
5. 10d SHALL BE 0.148x3". 8d SHALL BE 0.131X2 ½". DRIVE ALL NAILS FLUSH WITH THE FACE OF . TOLERANCE IS +1/16 to -0
6. PLATES ON CONCRETE SHALL BE TREATED. SEE GENERAL STRUCTURAL NOTES.
7. NAIL OR LAG SHEATHING & STUD AT SHEAR WALL INTERSECTION AS INDICATED.
8. WHERE ONLY ONE HOLDOWN IS SPECIFIED LOCATE ON OPENING SIDE OF HOLDOWN STUDS. SEE WALL ELEVATION AT RIGHT.
9. (2)2x MAY BE USED IN LIEU OF 3x AT PANEL JOINTS. STITCH NAIL THE STUDS TOGETHER PER SHEAR TRANSFER 'C'. SEE 'PLAN VIEW 1'. REFER TO APA TECHNICAL PUBLICATION TT-076.

TYPICAL WALL FRAMING NOTES:

1. PROVIDE TYPICAL WALL FRAMING INDICATED, EXCEPT WHERE NOTED OTHERWISE.
2. SEE ARCHITECTURAL DRAWINGS FOR FIRE BLOCKING AND BACKING FOR FINISHES AND FURNISHINGS.

TYPICAL ROOF & FLOOR DIAPHRAGM FRAMING NOTES:

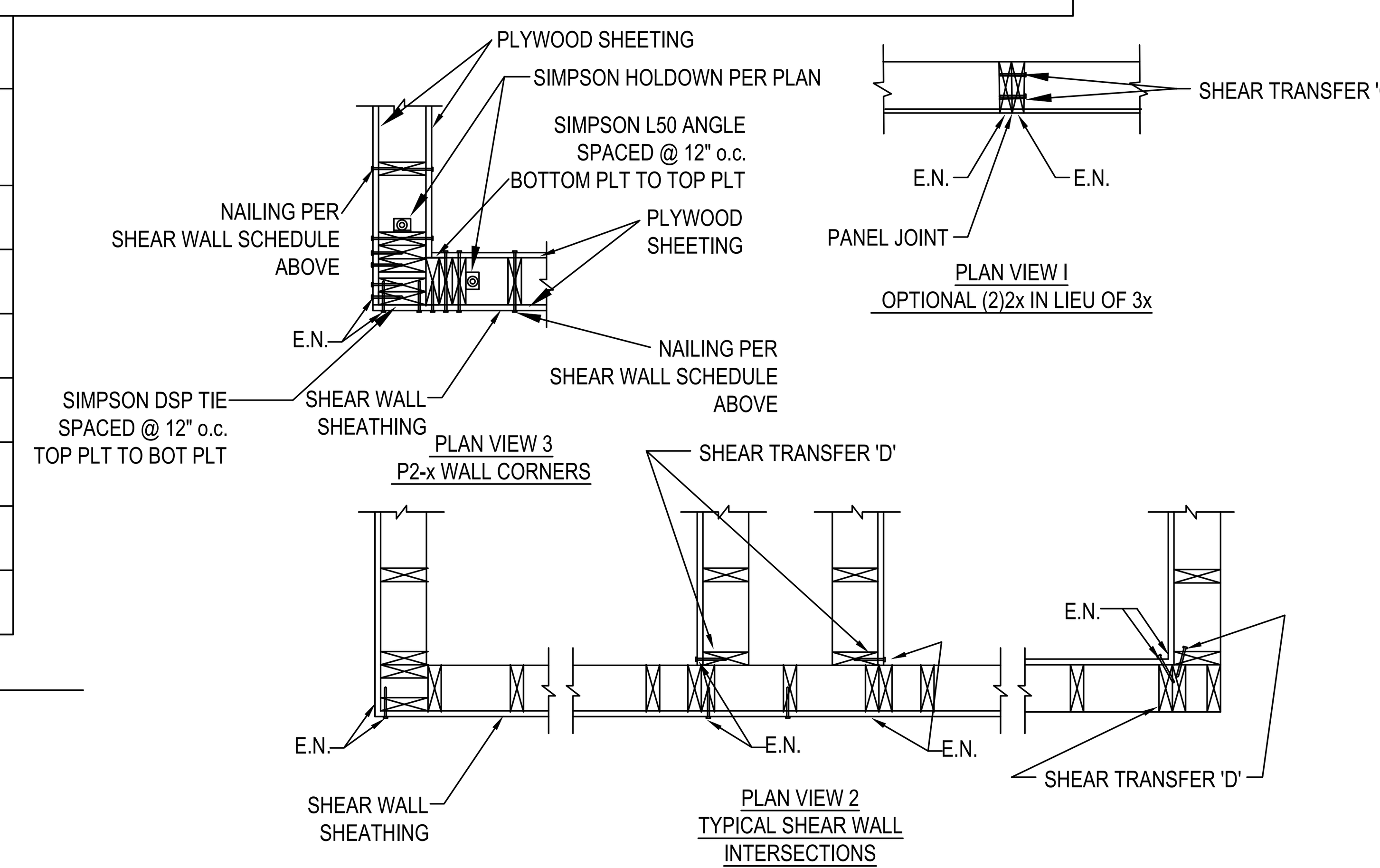
1. ROOF AND FLOOR DIAPHRAGMS ARE UNBLOCKED, U.L.N. AND NAILED ACCORDING TO THE FASTENING SCHEDULE OF IBC TABLE 2304.9.1.

HEADER END NAILING

NOMINAL DEPTH	END ATTACHMENT
4	(4)16d
6	(6)16d
8	(8)16d
10	(10)16d
12	(12)16d
14	(14)16d
16	(16)16d
18	(18)16d

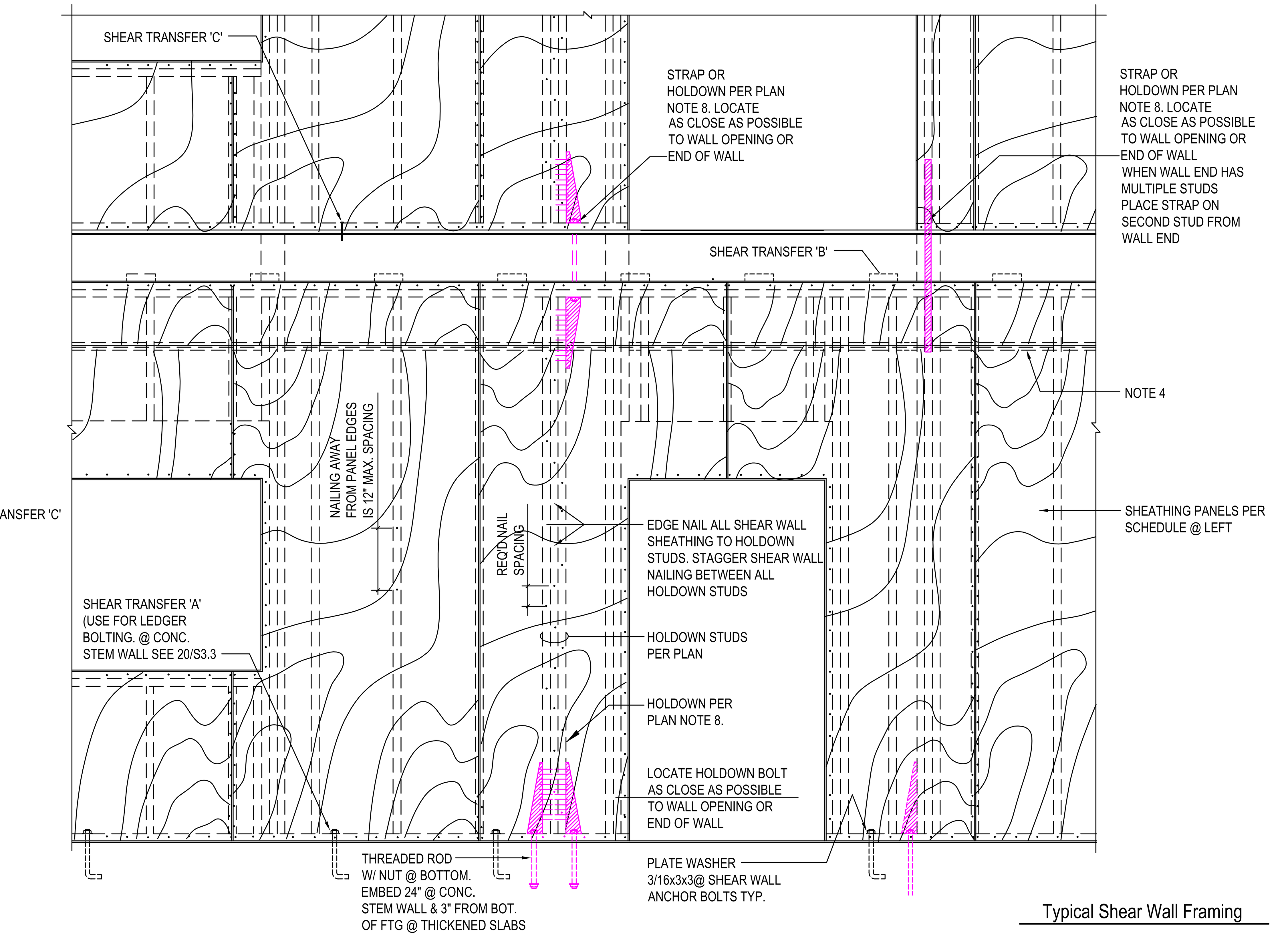
ROUGH WINDOW SILL

HORIZ ROUGH OPENING	NUMBER OF SILLS REQUIRED	END ATTACHMENT	REF.
0 TO 6'	1	(2)16d END NAIL	20/S6.1
> 6'	2	(2)16d END NAIL, +A35 EA END @ EA SILL	20/S6.1

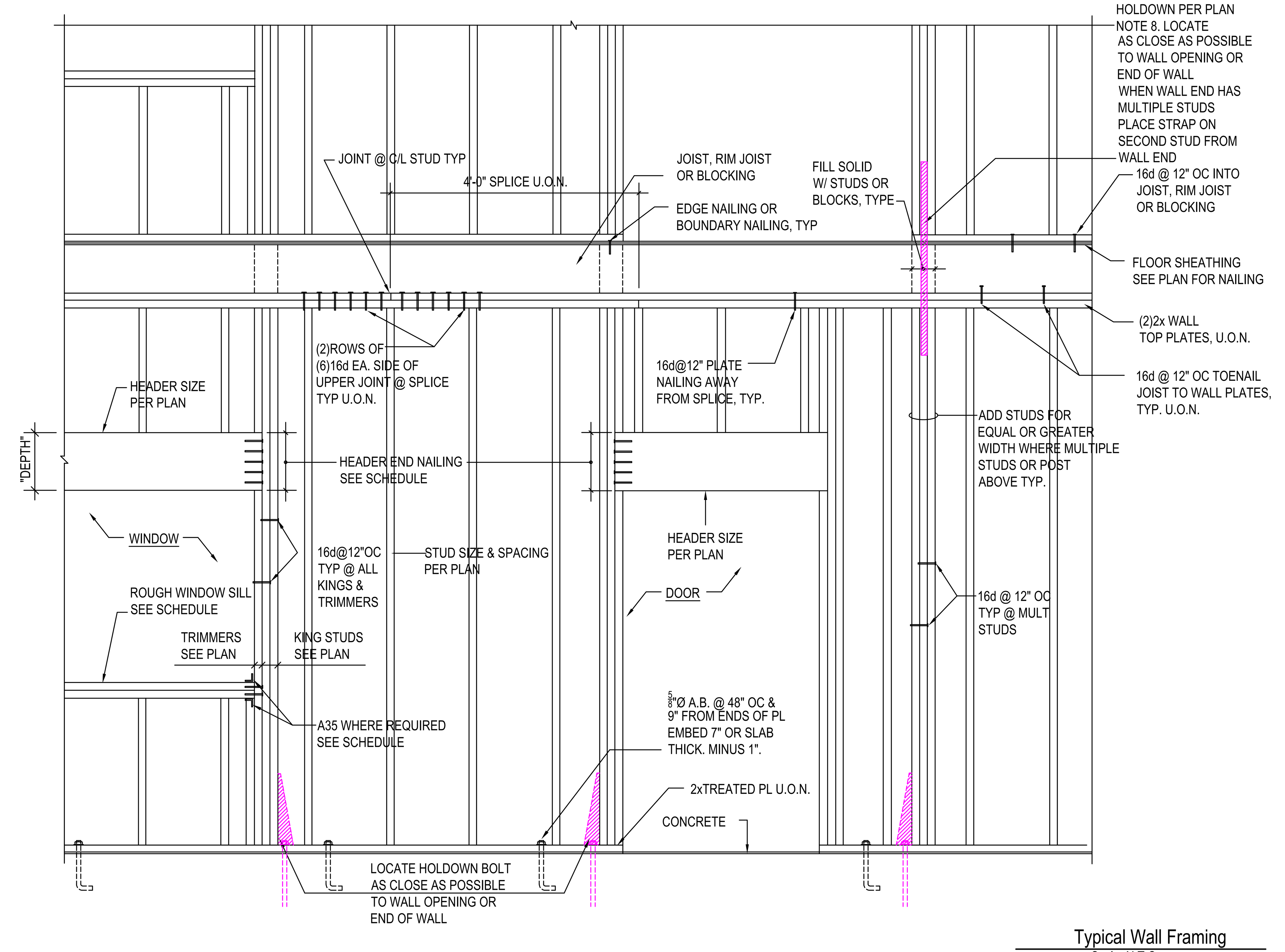


MINIMUM NAILING SCHEDULE

CONNECTION	NAILS
1. Joist to sill or girder, toenail	(3) 8d
2. Bridging to joist, toenail each end	(2) 8d
3. 1" x 6" sub floor or less to each joist, face nail	(2) 8d
4. Wider than 1"x6" sub floor to each joist, face nail	(3)8d
5. 2" subfloor to joist or girder, blind and face nail	(2)16d
6. Sole plate to joist or blocking, typical face nail	16d at 16" o.c.
Sole plate to joist or blocking, at braced wall panels	(3)16d per 16"
7. Top plates to stud, end nail	(4)16d
8. Stud to sole plate	(4)8d, toenail or (2) 16d, end nail
9. Double stud, face nail	16d at 24" o.c.
10. Double top plates, typical face nail	16d at 16" o.c.
Double top plates, lap splice	(8)16d
11. Blocking between joist or rafters to top plate, toenail	(3)8d
12. Rim joist to top plate, toenail	8d at 6" o.c.
13. Top plates, laps and intersections, face nail	(2)16d
14. Continuous header, two pieces	16d at 16" o.c. along each edge
15. Ceiling joist to plate, toenail	(3)8d
16. Continuous header to studs, toenail	(4)8d
17. Ceiling joist, lap over partitions face nail	(3)16d
18. Ceiling joist to parallel rafters, face nail	(3)16d
19. Rafter to plate, toenail	(3)8d
20. 1" brace to each stud and plate, face nail	(2)8d
21. 1"X8" sheathing or less to each bearing , face nail	(2)8d
22. Wider than 1"x8" sheathing to each bearing face nail	(5)8d
23. Built up corner studs	16d at 24" o.c.
24. Built up girder and beams	



Typical Shear Wall Framing



Typical Wall Framing
Scale: N.T.C.

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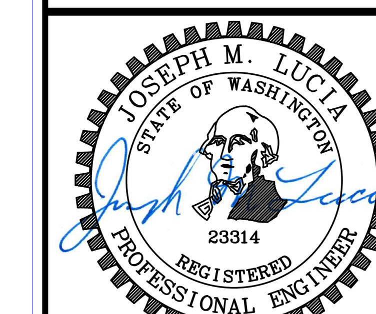

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SHEET
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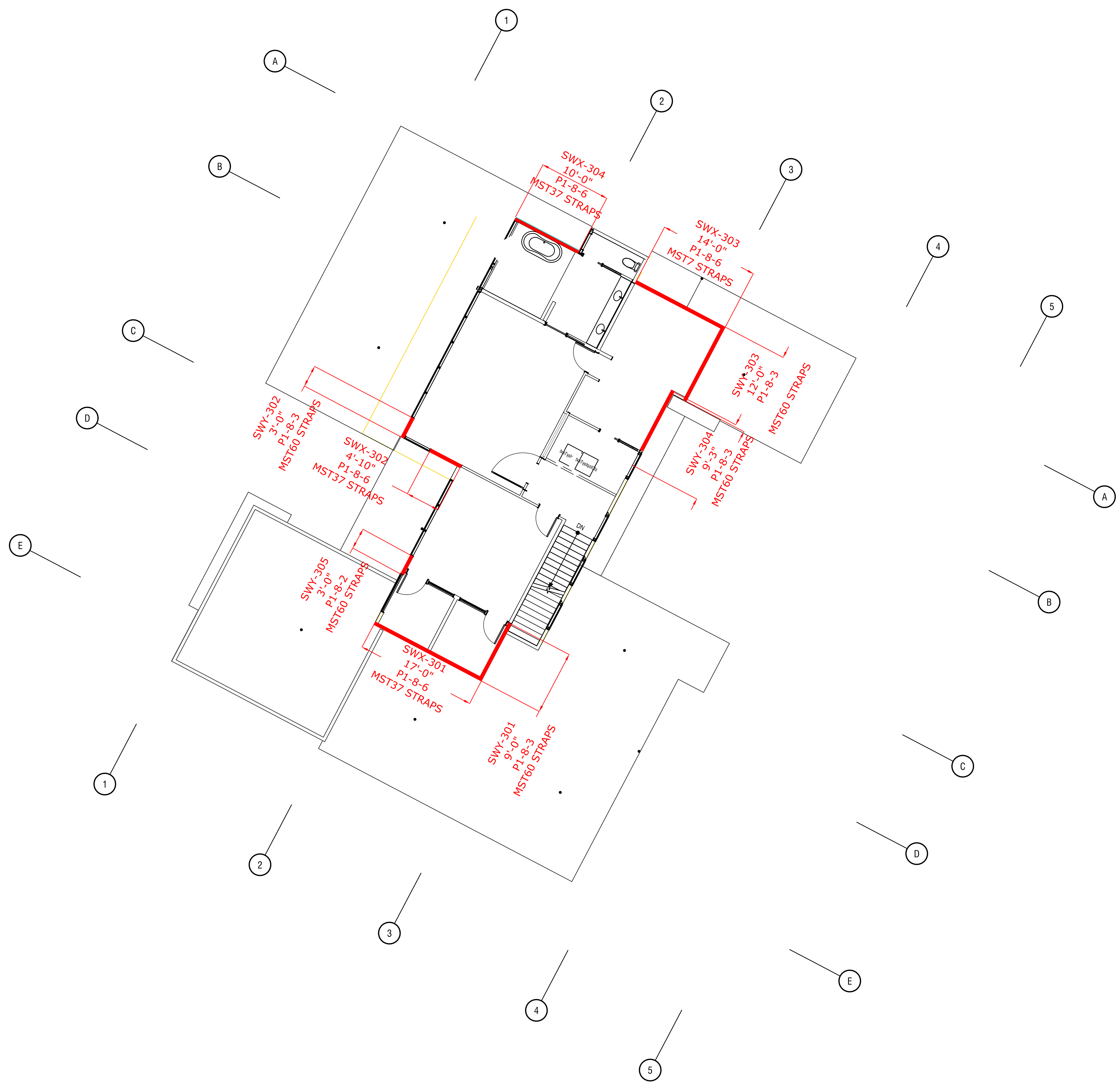

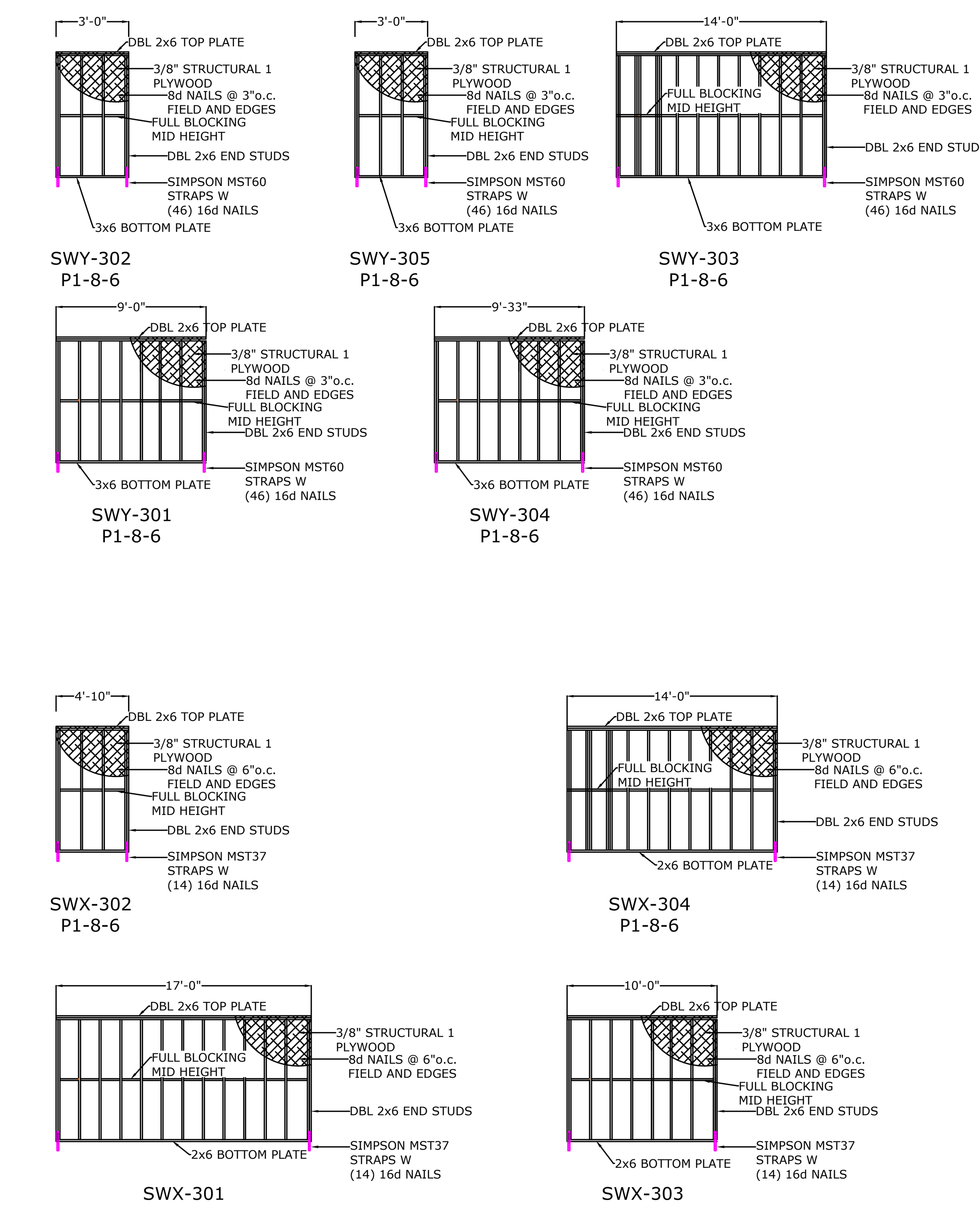

SECOND FLOOR LEVEL - SHEAR WALLS

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JOSEPH M. LUCIA
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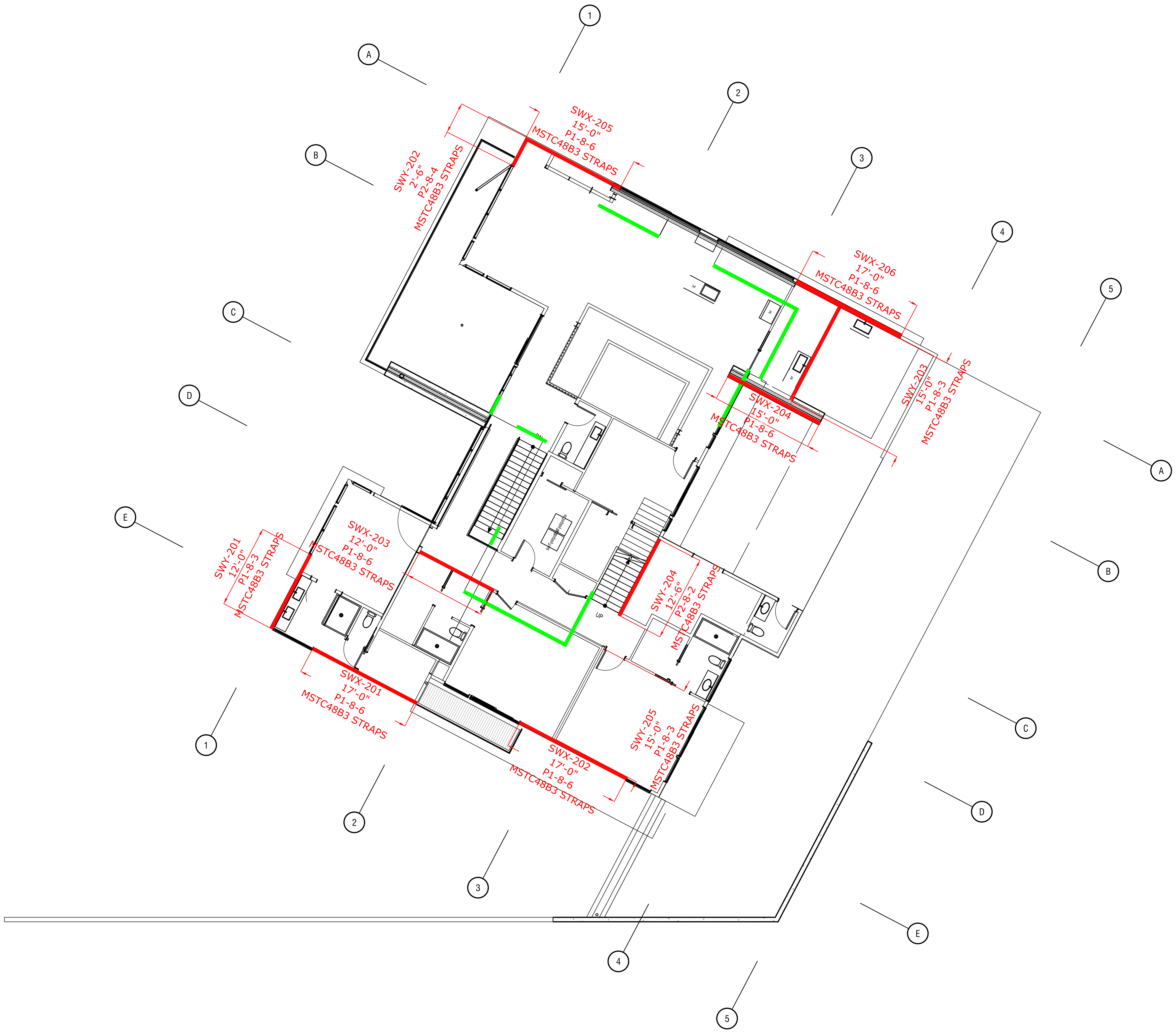
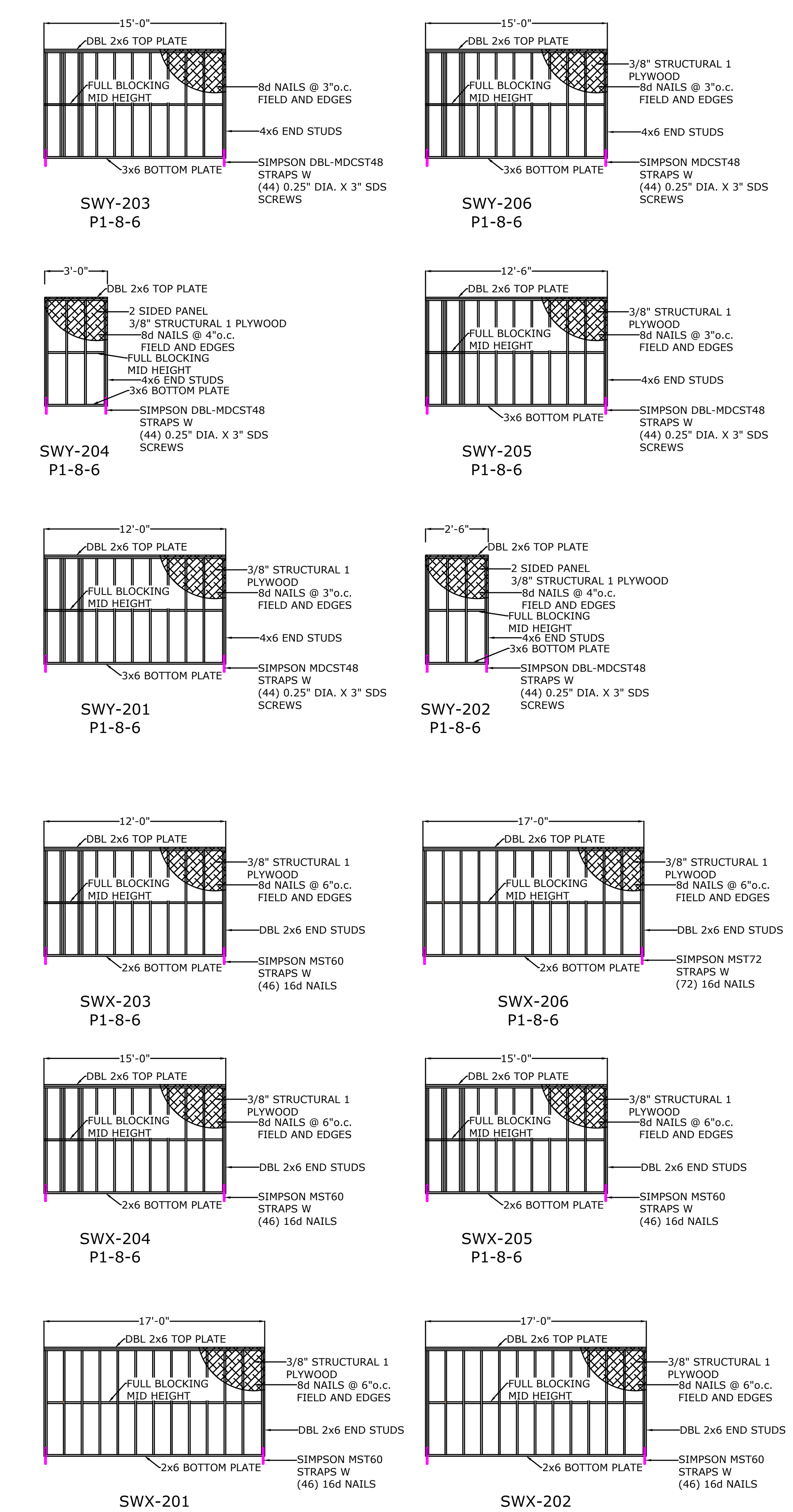
SHEET
S-7.0





FIRST FLOOR LEVEL - SHEAR WALLS

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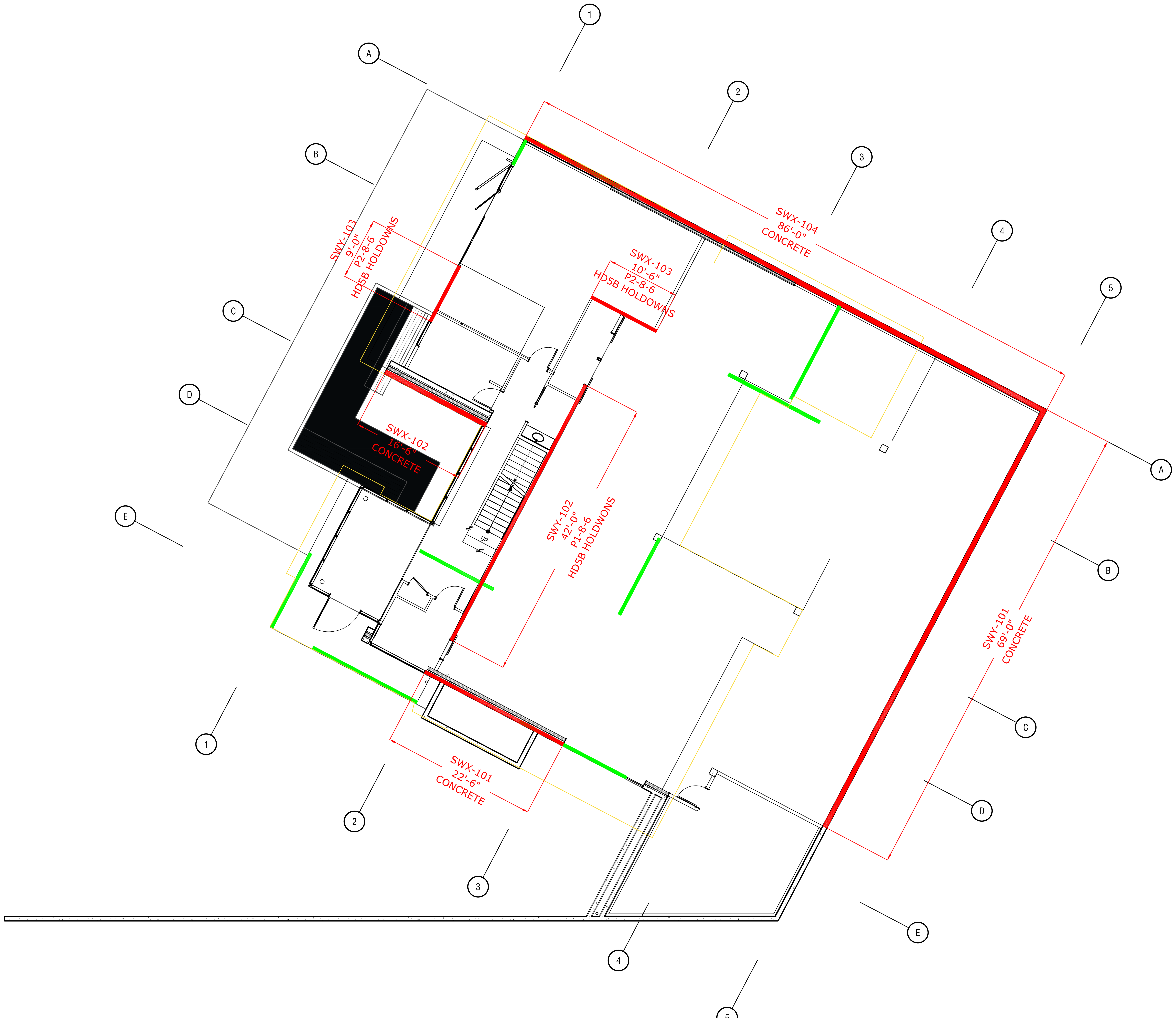
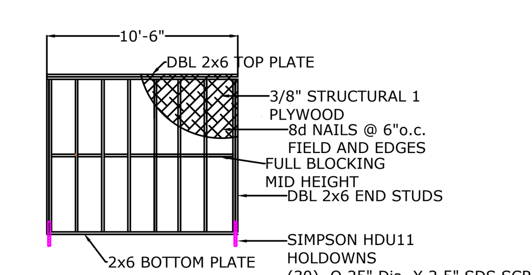
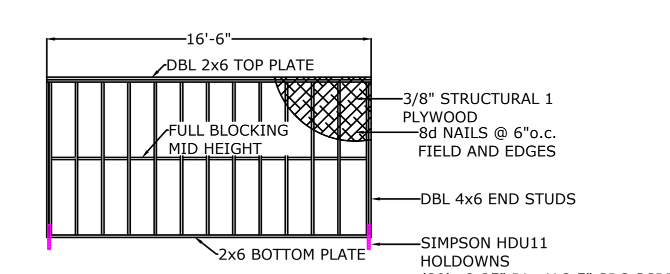
SHEET
S-8.0





SWX-03
P1-8-6



SWX-102
P1-8-6

GARAGE-BASEMENT LEVEL - SHEAR WALLS

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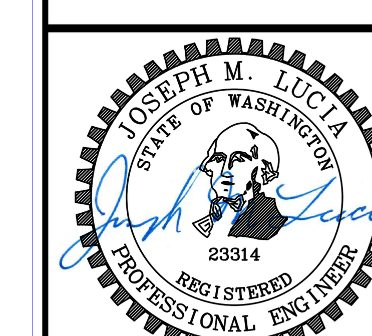

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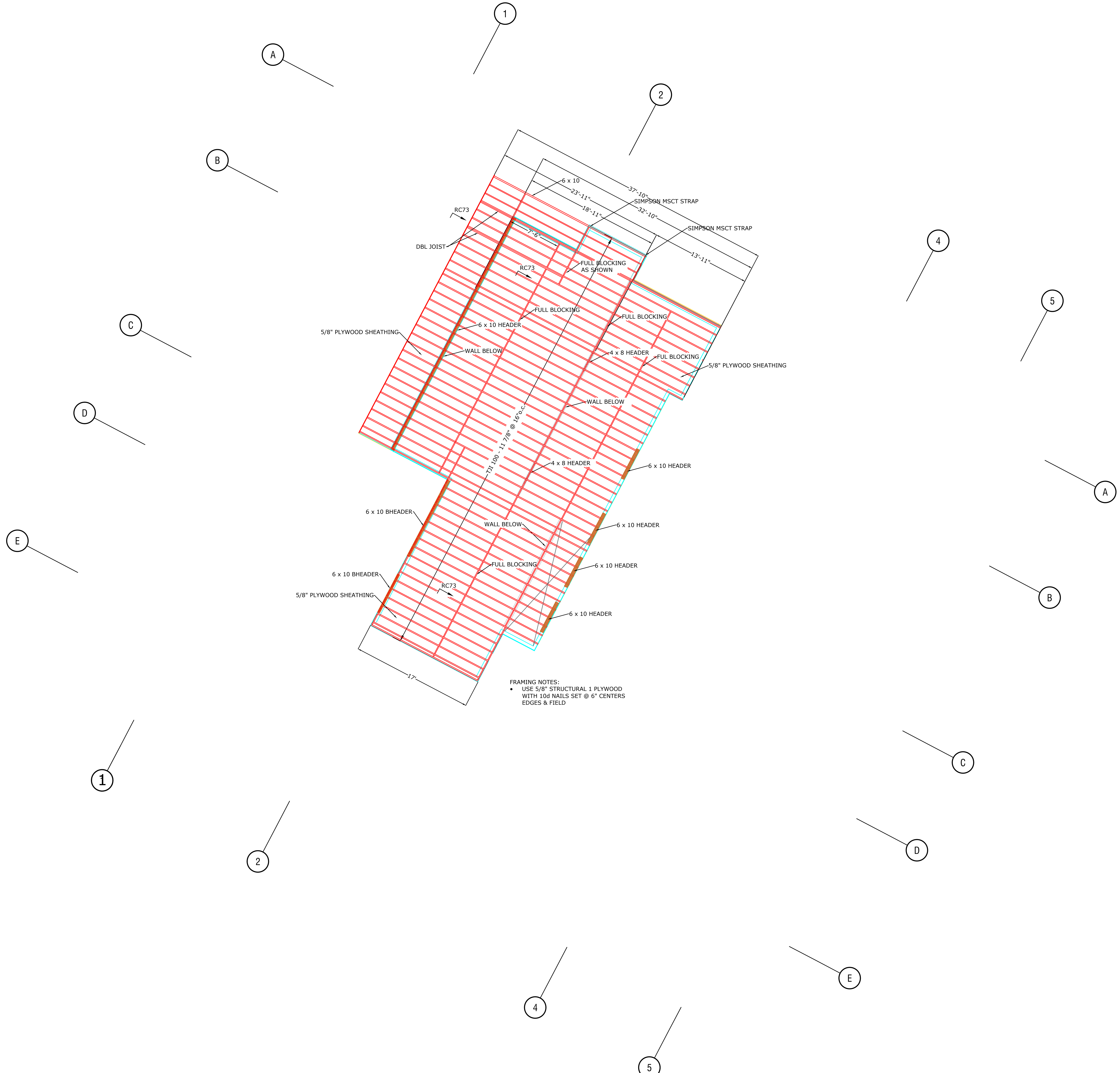

FRAMING NOTES:

- USE 5/8" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

ROOF FRAMING

LANZ RESIDENCE
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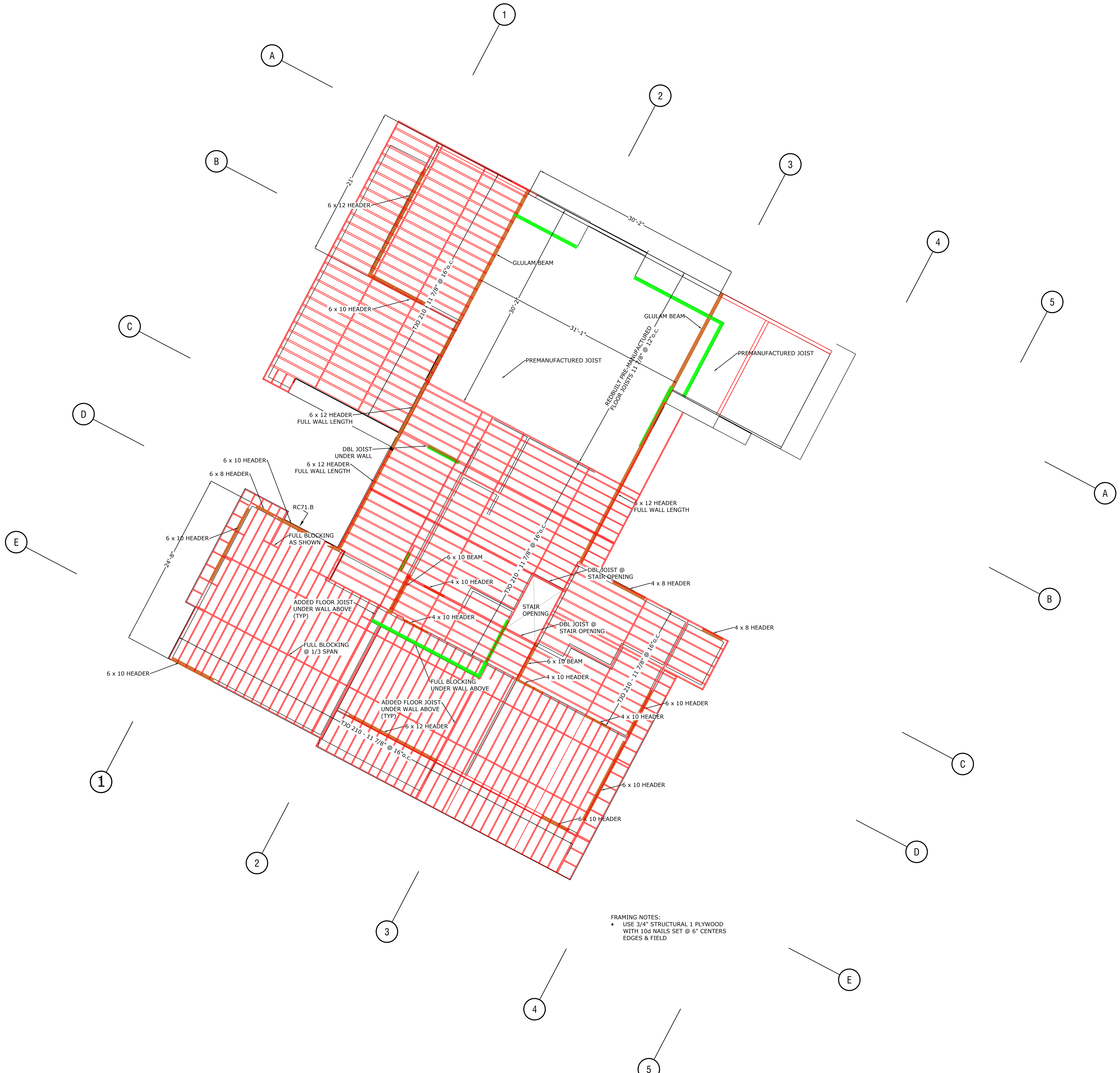

SECOND FLOOR - FLOOR FRAMING

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FRAMING NOTES:
- USE 3/4" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

FIRST FLOOR - FLOOR FRAMING

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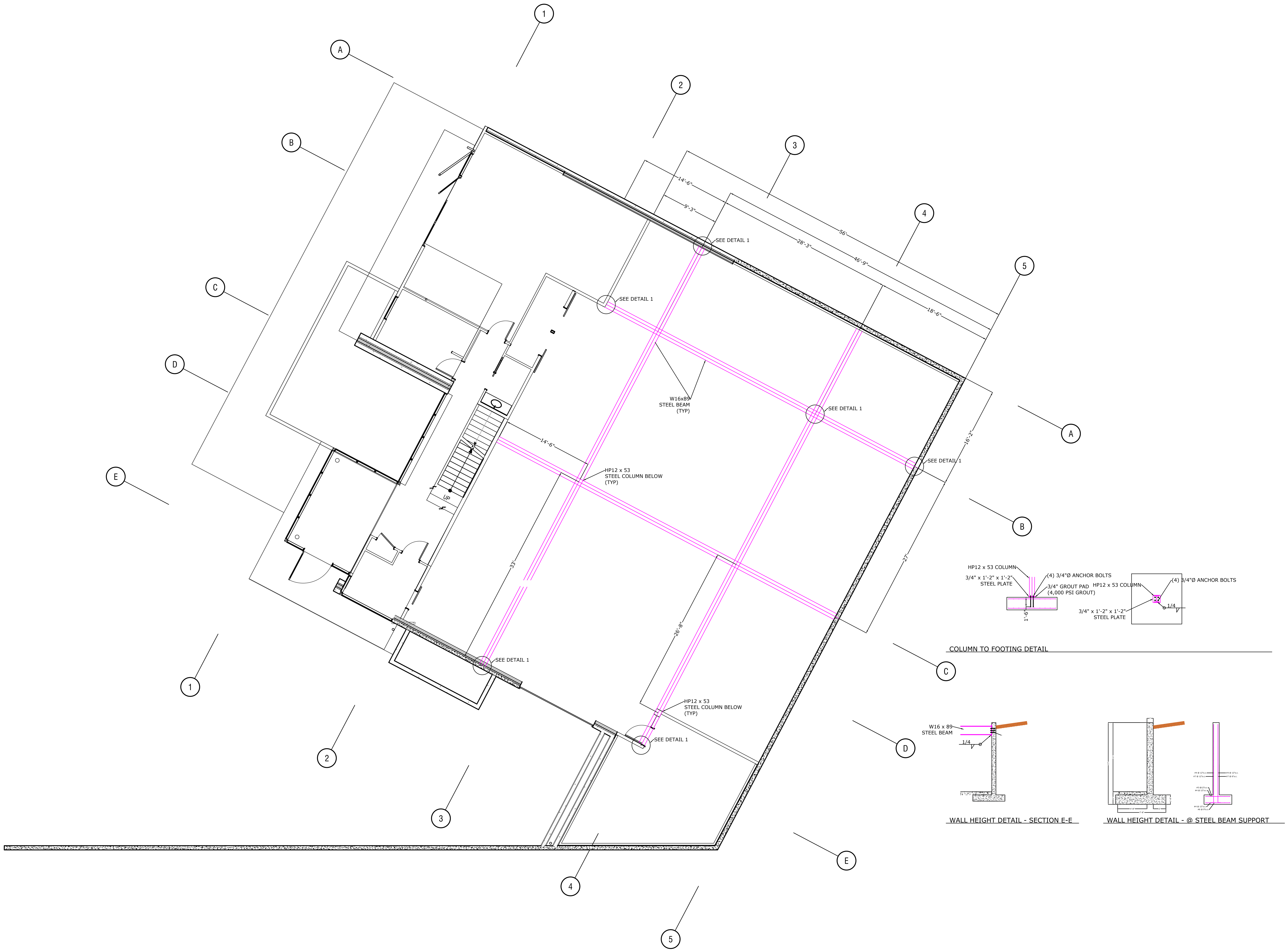

FIRST FLOOR - FLOOR FRAMING

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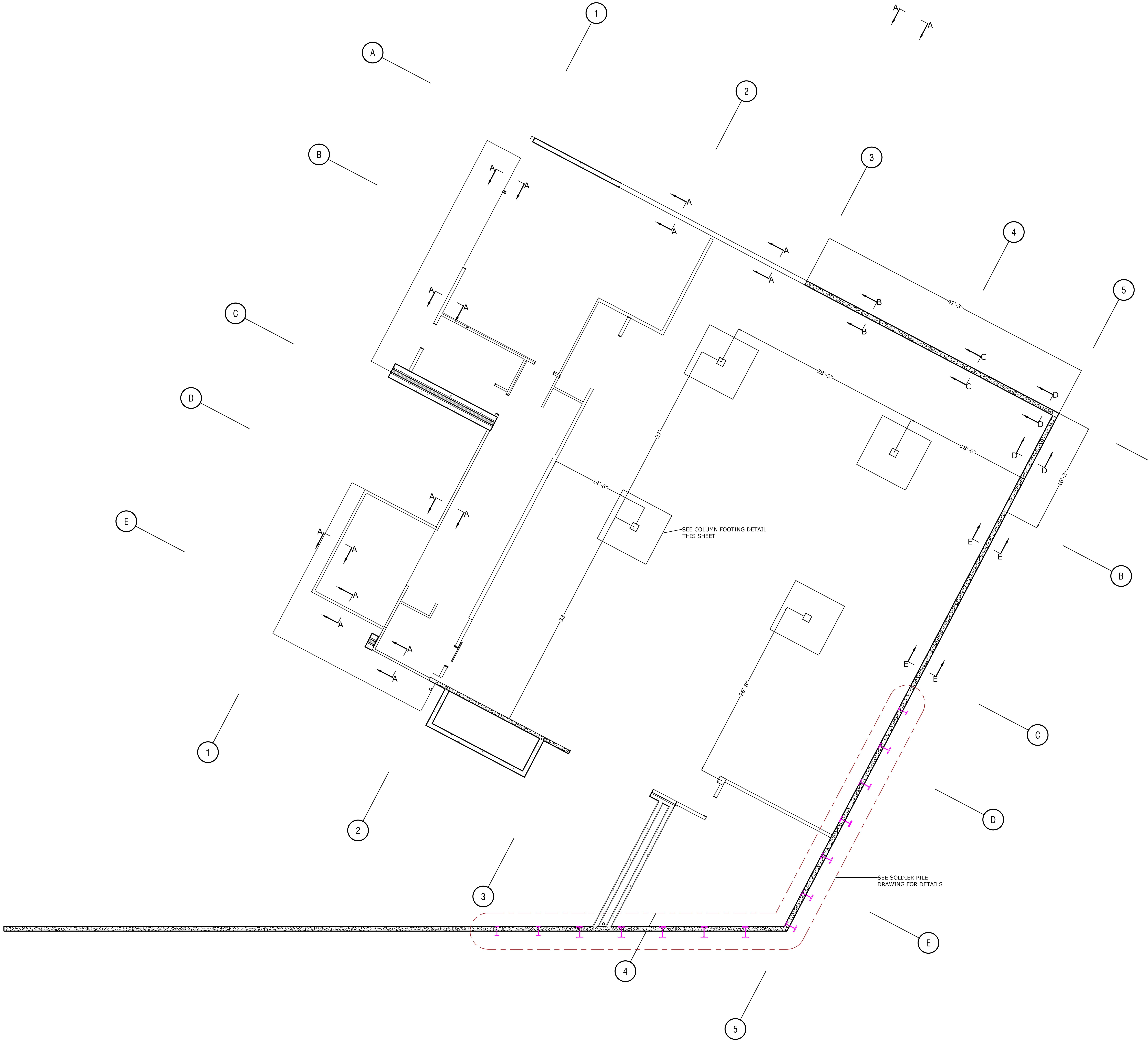
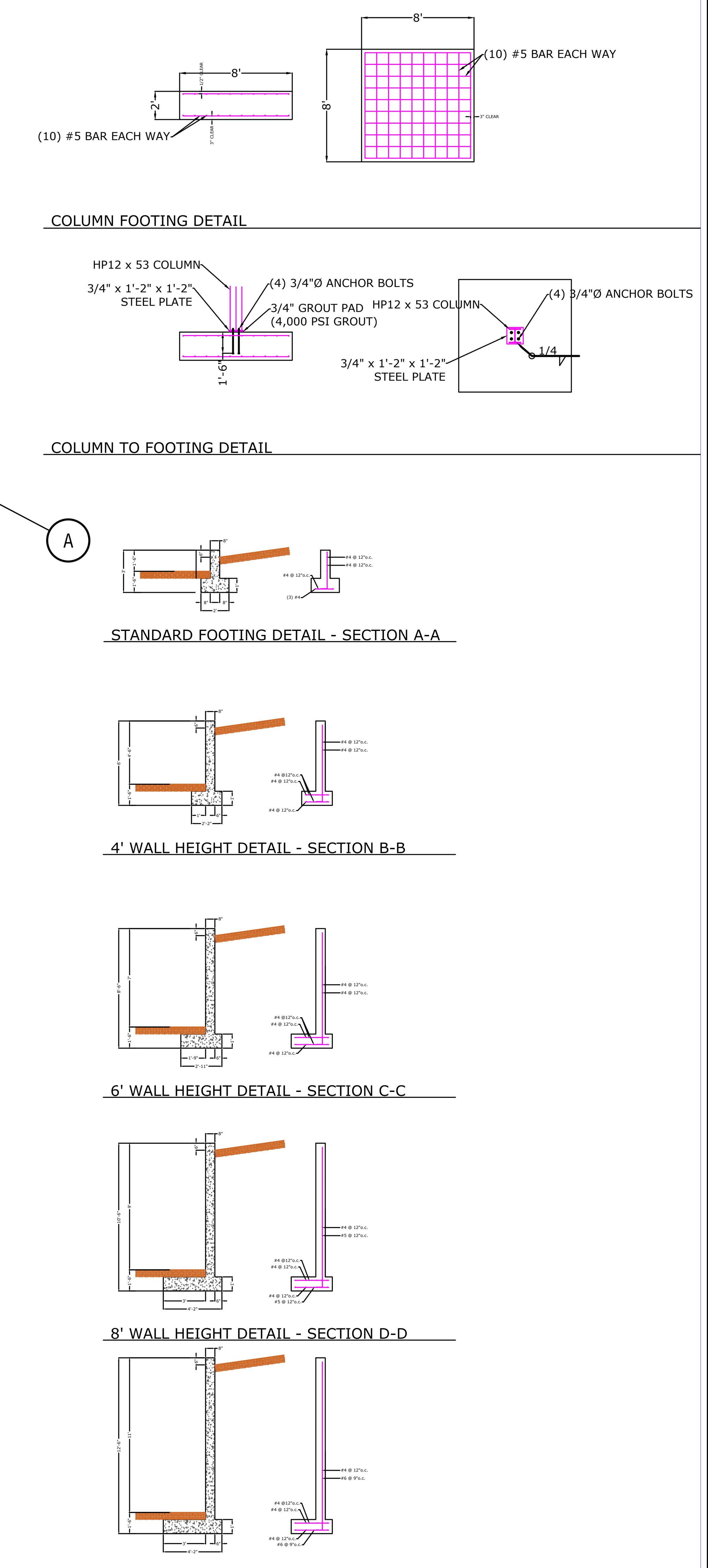


FIRST FLOOR - FLOOR FRAMING



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SHEET
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