

FIRM LOGO

SAMPLE
FIRM LOGO

ENGINEERING FIRM

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STRUCTURAL CALCULATIONS**STRUCTURAL CALCULATIONS FOR
TWO-STORY RESIDENTIAL ADDITION****PROJECT ADDRESS / JURISDICTION**

123 Maple Street
Anytown, California 90210

OWNER / CLIENT

PREPARED FOR:
Mr. & Mrs. John Doe

GENERAL NOTES, CODE EDITION, AND SCOPE

Calculations prepared under the current California Building Code and referenced standards. This submittal is for plan check review. Field verification and contractor means and methods are not shown.

ENGINEER OF RECORD

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PROFESSIONAL STAMP (IMAGE)

Structural Calculation Report

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Dead & Live Loads — Roof, Ceiling & 2nd Floor

Dead loads

Material	Weight (psf)
Roof dead load	
Asphalt shingles	2.00
Plywood (per 1/8 in thickness) (0.5 in total)	1.60
Rigid insulation, 1/2 in	0.75
2 × 8, 16 in spacing (psf)	6.00
	10.35 psf
Ceiling dead load	
Gypsum board (per 1/8 in thickness) (0.5 in total)	2.20
Mechanical duct allowance	4.00
Suspended steel channel system	2.00
2 × 6, 16 in spacing (psf)	5.00
	13.2 psf
Floor 1	
Gypsum board (per 1/8 in thickness) (0.5 in total)	2.20
Mechanical duct allowance	4.00
Hardwood flooring, 7/8 in	4.00
Subflooring, 3/4 in	3.00
2 × 10, 16 in spacing (psf)	6.00
	19.2 psf
Interior Wall 1	
Wood or steel studs, 1/2 in gypsum board each side	8.00
	8 psf
Exterior Wall 1	
2 × 6 @ 16 in, 5/8 in gypsum, insulated, 3/8 in siding	12.00
	12 psf

Live loads

Occupancy or Use	Uniform, L_o (psf)	Live Load Reduction Permitted? (Section No.)	Multiple-Story Live Load Reduction Permitted? (Section No.)	Concentrated (lb)
Floor — living and similar areas	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)	—
Floor — sleeping areas	30 (1.44)	Yes (4.7.2)	Yes (4.7.2)	—
Roof — ordinary (L_r basis)	20 (0.96)	Yes (4.8.2)	—	—
Ceiling joist / uninhabitable attic (no storage)	10 (0.48)	Yes (4.7.2)	Yes (4.7.2)	—
Uninhabitable attic (with storage)	20 (0.96)	Yes (4.7.2)	Yes (4.7.2)	—
Stairs — one- and two-family	40 (1.92)	Yes (4.7.2)	Yes (4.7.2)	300 (1.33)
Balconies and decks	1.5 times the live load for the area served. Not required to exceed 100 psf	Yes (4.7.2)	Yes (4.7.2)	—
Handrails / guards (residential)	50 plf	—	—	200 lb concentrated
L — live-load reduction Not applied — enter reduced L if taken				

Seismic Loads — Base Shear & Story Forces

Step 1: Building geometry and weights

Level	w_x (lb) effective seismic weight	h_x (ft) height above base	h_{sx} (ft) story height
2	40,000	20	10
1	40,000	10	10
$W = \sum w_x$	80,000		

Step 2: Site Classification and Spectral Parameters

Site class	S_{DS}	S_{D1}
D	0.667 g	0.40 g

Site Class	$\bar{v}_s$ Calculated Using Measured or Estimated Shear Wave Velocity Profile (ft/s)
D. Medium dense sand or stiff clay	>700 to 1,000

S_S	1.00 g
S_{DS}	0.667 g
S_{D1}	0.40 g
S_1	0.60 g
T_L	8 s

Step 3: Risk Category, Importance Factor, and Seismic Design Category

Risk category	I_e	SDC
II	1.00	D

Risk Category	Use or Occupancy of Buildings and Structures
II	All buildings and other structures except those listed in Risk Categories I, III, and IV.

Risk Category	Seismic Importance Factor, I_e
II	1.00

Seismic Design Category	D
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Step 4: Seismic Force-Resisting System

Seismic Force-Resisting System	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n , Limits (ft) ^d				
				Seismic Design Category (SDC)				
				B	C	D ^e	E ^e	F ^f
A. BEARING WALL SYSTEMS								
16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	6.5	3	4	NL	NL	65	65	65

Step 5: Period Determination

Structure Type	C_t	x
All other structural systems	0.02 (0.0488) [*]	0.75
h_n (ft)	20.00	
Approximate period T_a	0.189 s ($T_a = C_t \cdot h_n^x$)	

Step 6: Seismic Response Coefficient C_s and Base Shear V

C_s	V, strength	0.7V, allowable stress
0.1026	8,209 lb	5,746 lb

Parameter	E, strength design	0.7E, allowable stress design
C_s	0.1026 ($T = 0.189$ s < $T_s = 0.6$ s)	
V	8,209 lb	5,746 lb

Step 7: Vertical and Horizontal Distribution

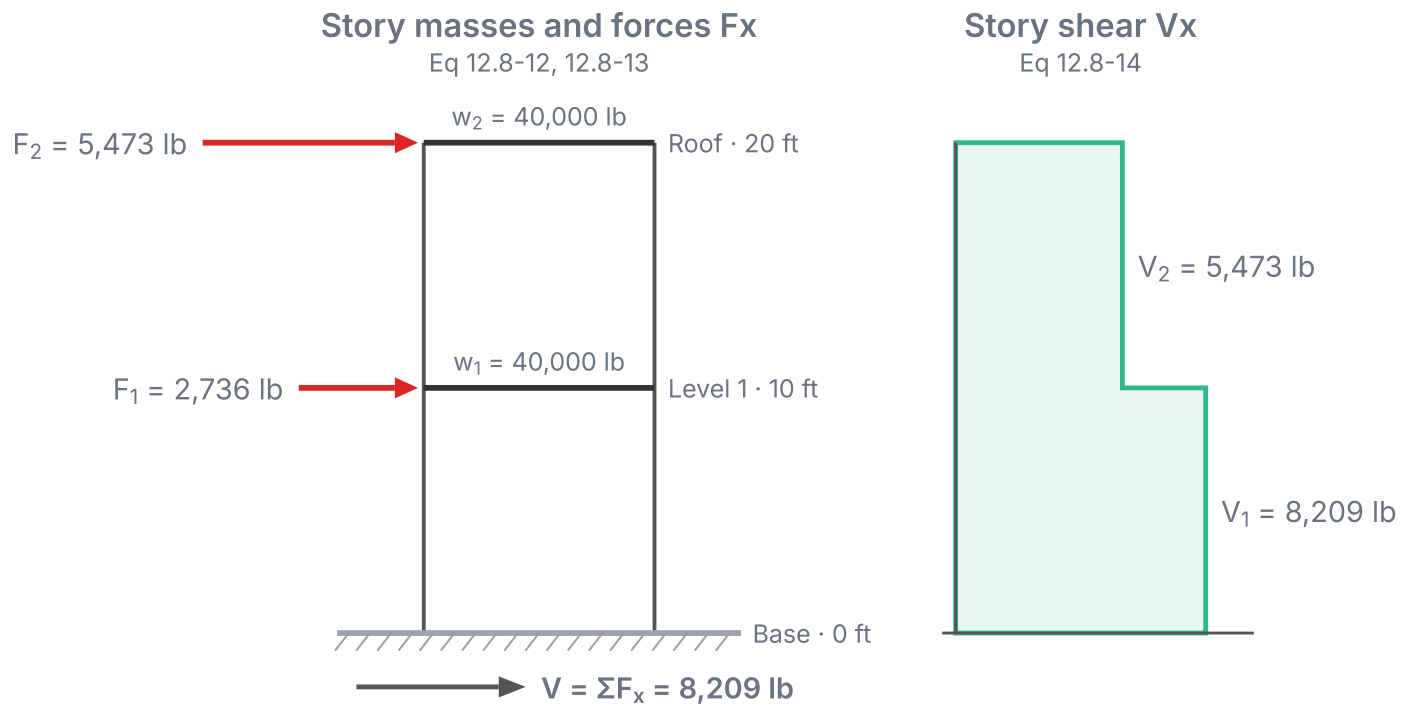
Level	E, strength design			0.7E, allowable stress design		
	F_x (lb)	V_x (lb)	M_x (lb-ft)	F_x (lb)	V_x (lb)	M_x (lb-ft)
2	5,473	5,473	0	3,831	3,831	0
1	2,736	8,209	54,728	1,915	5,746	38,310

Step 8: Diaphragm Design Forces

Level	E, strength design	0.7E, allowable stress design
	F _{px} (lb)	F _{px} (lb)
2	5,473	3,831
1	5,336	3,735

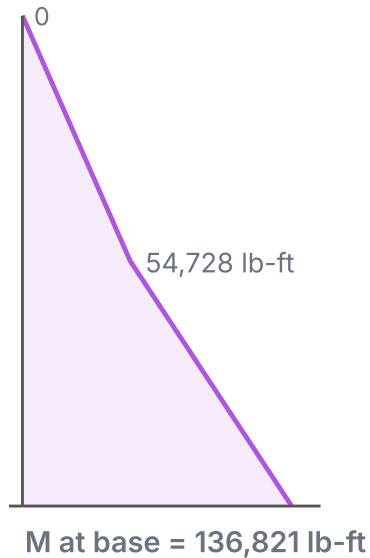
Step 9: Force Diagrams

FORCE LEVEL — E (STRENGTH DESIGN) · BASE SHEAR $V = 8,209$ LB



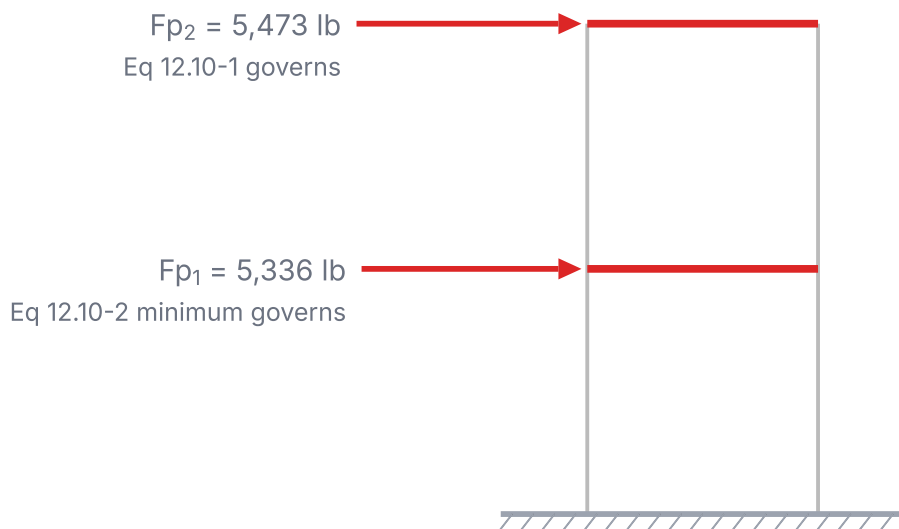
Overturning moment M_x

§12.8.5: $M_x = \sum F_i(h_i - h_x)$



Diaphragm forces F_{px}

Eq 12.10-1 to 12.10-3



Forces at the E (strength design) level. Each F_{px} is a separate design check applied to its own diaphragm — F_{px} values are not concurrent and do not accumulate into a shear or moment.

Wind Loads — MWFRS Design Pressures

Step 2: Geometry & Configuration

Length parallel to ridge	250 ft
Width normal to ridge	200 ft
Roof Angle	18.43° (4:12 rise:run)
Eave Height	20 ft
Mean Roof Height, h	36.67 ft
Ridge Height	53.33 ft

Step 3: Determine risk category of building;

Risk Category	Use or Occupancy of Buildings and Structures
II	All buildings and other structures except those listed in Risk Categories I, III, and IV.

Step 4: Determine the basic wind speed, V, for applicable risk category;

V
115 mph

Basic Wind Speed, V	115 mph
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Step 5: Determine wind load parameters:

K _d	Exposure	K _{zt}	K _e	GC _{pi}
0.85	C	1.36	0.96	+0.18, -0.18

Structure Type	Directionality Factor K _d
Main wind force resisting system	0.85

Exposure Category	Description
C	Open terrain with scattered obstructions generally < 30 ft

H — hill height	60 ft — height of the hill or escarpment
L _h — upwind distance	200 ft — from the crest to where the ground is H/2 below it
x — distance from crest	100 ft
z — height above ground	36.67 ft
Hill/escarpment shape	2D Ridge
Location	Downwind of crest

$$K_{zt} = (1 + K_1 \cdot K_2 \cdot K_3)^2 \quad 1.36$$

Ground Elevation Above Sea Level (ft)	K_e
1,000	0.96

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 ft ² (0.37 m ²), and $A_{oi} / A_{gi} \leq 0.2$	Moderate	+0.18, -0.18

Step 6: Determine velocity pressure exposure coefficient, K_z or K_h ;

$$K_h$$

1.02

z (ft)	K_z
53.33 (ridge height)	1.10
50.00	1.09
40.00	1.04
36.67 (mean roof height)	1.02
30.00	0.98
25.00	0.94
20.00 (eave height)	0.90
15.00	0.85

Step 7: Determine velocity pressure, q_z or q_h ,

z (ft)	q_z (psf)
53.33 (ridge height)	48.89
50.00	48.30
40.00	46.08
36.67 (mean roof height)	45.19
30.00	43.42
25.00	41.65
20.00 (eave height)	39.88
15.00	37.66

Step 8: Determine external pressure coefficient, (GC_{pf}),

Surface	L/B	C_p
Pressure coefficient — wind normal to ridge ($B_{\perp} = 250.0$ ft) — $\theta \geq 10^\circ$ ($\theta = 18.4^\circ$)		
Wall: Windward	all	+0.80
Wall: Leeward	0.80	-0.50
Wall: Sidewall	all	-0.70
Roof: windward		-0.36 , +0.14
Roof: leeward		-0.57
Pressure coefficient — wind parallel to ridge ($B_{\perp} = 200.0$ ft) — $\theta \geq 10^\circ$ ($\theta = 18.4^\circ$)		
Wall: Windward	all	+0.80
Wall: Leeward	1.25	-0.45
Wall: Sidewall	all	-0.70
Roof: from 0.0 to 18.3 ft		-0.90 , -0.18
Roof: from 18.3 to 36.7 ft		-0.90 , -0.18
Roof: from 36.7 to 73.3 ft		-0.50 , -0.18
Roof: from 73.3 to 250.0 ft		-0.30 , -0.18

Step 9: Calculate wind pressure, p, on each building surface

Surface	$p(+GC_{pi}), \text{ psf}$	$p(-GC_{pi}), \text{ psf}$	Note
Design pressures for MWFRS — wind normal to ridge ($B_{\perp} = 250.0 \text{ ft}$) — $\theta \geq 10^{\circ}$ ($\theta = 18.4^{\circ}$)			
Wall: Windward			
$z(\text{ft})=53.33$	21.34	35.17	ridge height
$z(\text{ft})=50.00$	21.00	34.83	
$z(\text{ft})=40.00$	19.72	33.55	
$z(\text{ft})=36.67$	19.21	33.04	mean roof height
$z(\text{ft})=30.00$	18.18	32.01	
$z(\text{ft})=25.00$	17.16	30.99	
$z(\text{ft})=20.00$	16.13	29.96	eave height
$z(\text{ft})=15.00$	14.85	28.68	
Wall: Leeward	-23.24	-9.41	$q = q_h$
Wall: Sidewall	-29.77	-15.94	$q = q_h$
Roof: windward	-18.76 , -2.43	-4.93 , 11.39	
Roof: leeward	-25.48	-11.65	
Design pressures for MWFRS — wind parallel to ridge ($B_{\perp} = 200.0 \text{ ft}$) — $\theta \geq 10^{\circ}$ ($\theta = 18.4^{\circ}$)			
Wall: Windward			
$z(\text{ft})=53.33$	21.34	35.17	ridge height
$z(\text{ft})=50.00$	21.00	34.83	
$z(\text{ft})=40.00$	19.72	33.55	
$z(\text{ft})=36.67$	19.21	33.04	mean roof height
$z(\text{ft})=30.00$	18.18	32.01	
$z(\text{ft})=25.00$	17.16	30.99	
$z(\text{ft})=20.00$	16.13	29.96	eave height
$z(\text{ft})=15.00$	14.85	28.68	
Wall: Leeward	-21.61	-7.78	$q = q_h$
Wall: Sidewall	-29.77	-15.94	$q = q_h$
Roof: from 0.0 to 18.3 ft	-36.30 , -12.79	-22.47 , 1.04	

Surface	$p(+GC_{pi}), \text{ psf}$	$p(-GC_{pi}), \text{ psf}$	Note
Roof: from 18.3 to 36.7 ft	-36.30 , -12.79	-22.47 , 1.04	
Roof: from 36.7 to 73.3 ft	-23.24 , -12.79	-9.41 , 1.04	
Roof: from 73.3 to 250.0 ft	-16.71 , -12.79	-2.88 , 1.04	

Step 10: Evaluate design wind load cases

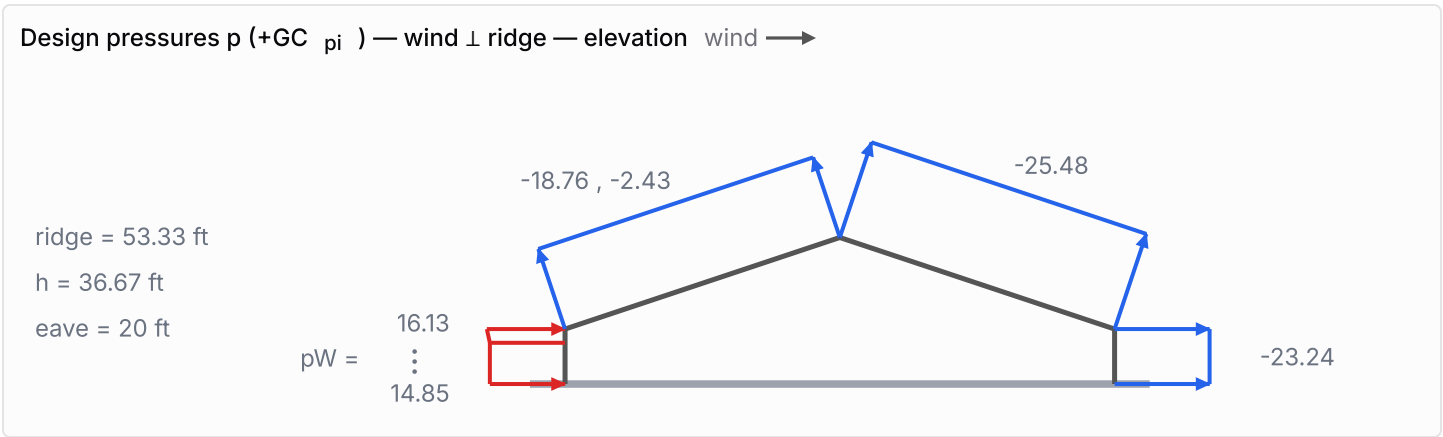
Load case	$V_x (\text{lb})$ — wind $\parallel$ ridge	$V_y (\text{lb})$ — wind $\perp$ ridge	$M_{zx} (\text{lb-ft/ft})$ — wind $\parallel$ ridge	$M_{zy} (\text{lb-ft/ft})$ — wind $\perp$ ridge	Roof uplift (lb)
Load Case 1	279,282	383,270	—	—	—
Load Case 2	209,462	287,452	$\pm 183,670$	$\pm 298,464$	—
Load Case 3	209,462	287,452	—	—	1,274,043
Load Case 4	157,236	215,781	$\pm 137,875$	$\pm 224,047$	1,274,043
Minimum Design Wind Load	117,331	146,664	—	—	0
Max (W, strength design)	279,282	383,270	—	—	1,274,043
Max (0.6W, allowable stress design)	167,569	229,962	—	—	764,426

Step 11: Pressure & Force Diagrams

Figures	Design pressures ($\pm GC_{pi}$), the four load cases, the minimum pressure
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PRESSURE & FORCE DIAGRAM — WIND $\perp$ RIDGE — $+GC_{pi}$

B = 200 ft $\times$ L = 250 ft (plan) — all pressures in psf

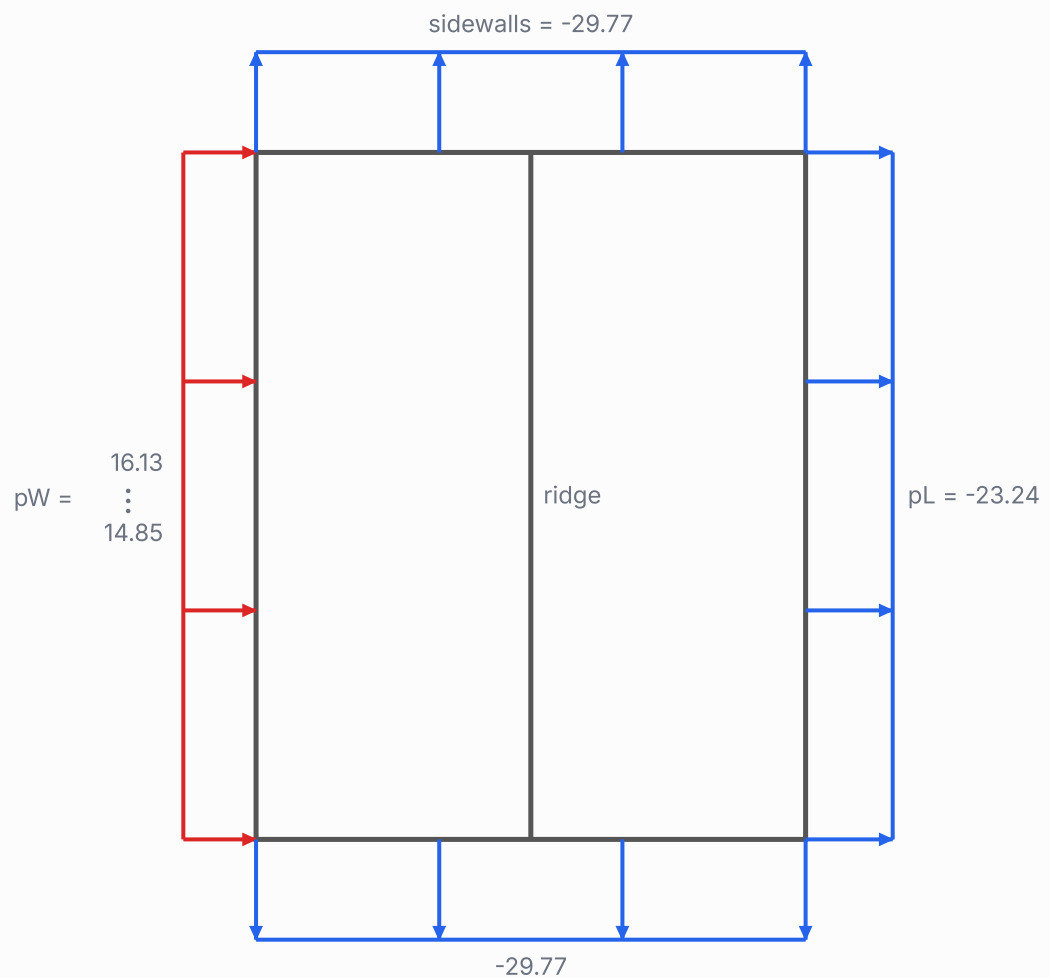


Design pressures p (+GC p_i) — wind $\perp$ ridge — sidewall elevation

Sight line along the wind (wind into the page) — both flanking sidewalls carry the same uniform suction at q_h .



Design pressures p (+GC p_i) — wind $\perp$ ridge — plan wind $\rightarrow$



Max (strength design): $V_x = 279,282 \text{ lb}$ · $V_y = 383,270 \text{ lb}$ · roof uplift = 1,274,043 lb

Max (allowable stress design): $V_x = 167,569 \text{ lb}$ · $V_y = 229,962 \text{ lb}$ · roof uplift = 764,426 lb

Minimum §27.1.5: $V = 117,331 \text{ lb}$ (wind $\parallel$ ridge) · $V = 146,664 \text{ lb}$ (wind $\perp$ ridge)

Eq. (27.3-1): $p = q K_d G C_p - q_i K_d (GC_{pi})$ · $G = 0.85$ (rigid) · $K_d = 0.85$ · $q_h = 45.19 \text{ psf}$ · $GC_{pi} = 0.18 / -0.18$ · paired roof values = both listed C_p

Beam B-1 — 2nd Floor @ Grid B/2-B/3

Step 1: Geometry & Configuration

Design material	Wood — NDS 2024 (ASD)
Configuration	Pinned support — 14 ft span — Pinned support

Step 2: Load Definition (ASCE 7-22 / NDS 2024)

D — Dead load	Self-weight — uniform 7.9 plf, full span
D — Dead load	Uniform 25 plf, full span
L — Live load	Uniform 100 plf, full span
S — Snow load	Uniform 50 plf, full span
L — live-load reduction	Not applied — enter reduced L if taken.

Step 3: Material & Section

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Specific Gravity ⁴	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity			
		F _b	F _t	F _v	F _{c⊥}	F _c	E	E _{min}	G	
DOUGLAS FIR-LARCH, DOUGLAS FIR, WESTERN LARCH										
No. 2	2" & wider	900	575	180	625	1,350	1,600,000	580,000	0.50	PLIB WWPA

Nominal Size b x d	Standard Dressed Size (S4S) b x d in x in	Area of Section A in ²	X - X AXIS		Y - Y AXIS	
			Section Modulus	Moment of Inertia	Section Modulus	Moment of Inertia
			S _x in ³	I _x in ⁴	S _y in ³	I _y in ⁴
Dimension Lumber (see NDS 4.1.3.2) and Decking (see NDS 4.1.3.5)						
4 × 10	3-1/2 × 9-1/4	32.38	49.91	230.8	18.89	33.05

Step 4: Design Parameters (NDS Adjustment Factors)

Adjustment factor	Condition	F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}
C_M (wet service)	Dry ($MC \leq 19\%$)	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_t (temperature)	$T \leq 100^\circ F$	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_i (incising)	Not incised	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_r (repetitive member)	Repetitive (3+ members $\leq 24"$ o.c.)	1.15	—	—	—	—	—	—
C_F (size)	Section 4×10	1.2	—	—	—	—	—	—
C_{fu} (flat use)	Edgewise use	1.0	—	—	—	—	—	—
C_L (beam stability)	Compression edge continuously braced	1.0	—	—	—	—	—	—
C_D (load duration)	D + L (Ten years)	1.0	—	1.0	—	—	—	—

Deflection limit	Floors — L/360 (live), L/240 (total)
Bearing length	3.5 in — $F_{c\perp}$ check enabled
Bearing $F_{c\perp}$ basis	Douglas Fir-Larch No. 2 (support material; the lower $F_{c\perp}$ governs the check)
Lateral support (C_L)	Compression edge continuously braced — $C_L = 1.0$

Step 5: Hanger Selector

Connection	Flush top · Face mount
Fastener	All
Supporting member	4 × 12 Douglas Fir-Larch
Selected hanger	HGUS410 (PASS)

Step 6: Design Verification

Controlling Load	Moment check	Shear check	Deflection check	Bearing check	Connection check
Combination	D/C ratio	D/C ratio	D/C ratio	D/C ratio	D/C ratio
D + L	0.630	0.239	0.502	0.121	0.120

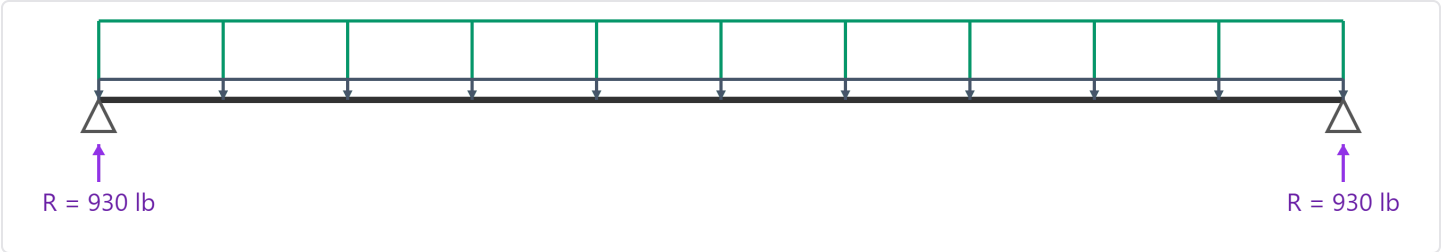
Step 7: Visualization

Check	Governing combination	Result
Flexure	D + L	$M_{\max} = 3,256 \text{ lb-ft}$, $M_{\min} = 0 \text{ lb-ft}$
Shear	D + L	$V_{\max} = 930 \text{ lb}$, $V_{\min} = -930 \text{ lb}$
Deflection	L (live load) — L/360	$\delta_{\max} = 0.234 \text{ in}$

BEAM MODEL & DIAGRAMS — 2A. D + L (GOVERNING)

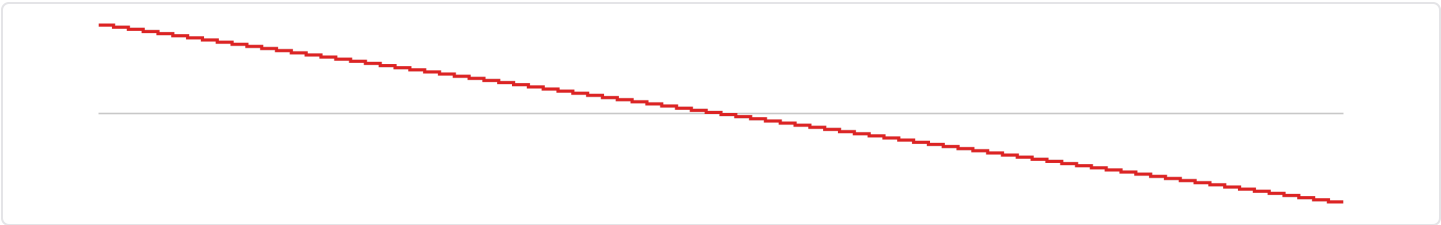
Beam Model

D L

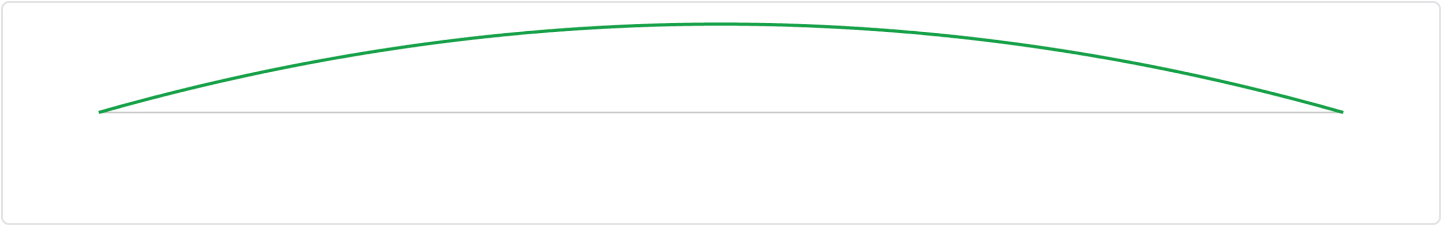


A uniform dead load of 7.9 plf (member self-weight for section 4×10) is applied to the beam; it is not shown in the load display.

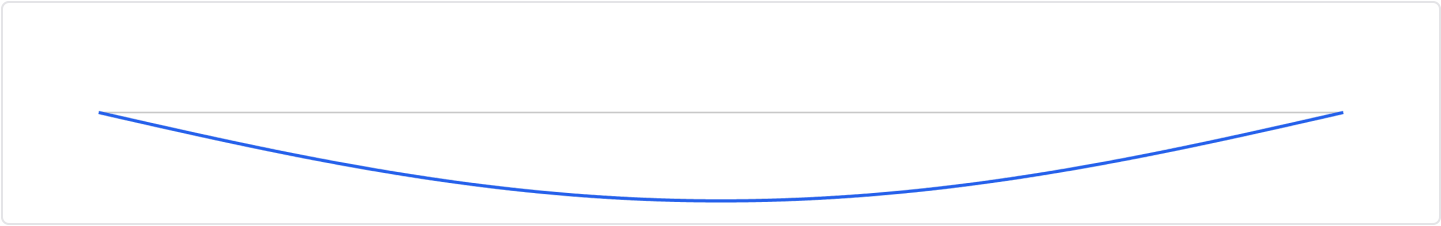
Shear Force — Min: -930 lb, Max: 930 lb



Bending Moment — Min: 0.31 lb-ft, Max: 3,256 lb-ft



Deflection — $\delta_{\max} = 0.311 \text{ in}$ (peak magnitude)



Floor Joists FJ-1 — 2nd Floor, Grid 1-3

JOIST & RAFTER SPAN LOOKUP

Step 1: Geometry & Configuration

Member type	Floor Joist
Design basis	Common lumber species
Span	14 ft 0 in

Step 2: Loads & Section

Live load	30 psf, L/Δ = 360
Dead load	10 psf
Joist spacing	16 in o.c.
Joist size	2 × 8
Species / Grade	Douglas Fir - Larch #2

Step 3: Hanger Selector

Model	Width (in)	Height (in)	Bearing (in)	Fasteners (qty)	Allowable download (lb)	Allowable uplift (lb)	Result
APLH26	1.563	5.125	1.875	Face 6 · Joist 4	1040	690	✓ PASS

Step 4: Design Verification

Result	✓ PASS
Maximum span (table)	14 ft 1 in
Entered span	14 ft 0 in

Post P-2 — 1st Floor @ Grid C/2

Step 1: Geometry & End Conditions

Design material	Wood — NDS 2024 (ASD)
Height	10 ft
Unbraced length per axis	$\ell_1 = 10\text{ ft}; \ell_2 = 5\text{ ft}$
K_e (strong, x-x)	1.00 (Pinned-Pinned, no sway)
K_e (weak, y-y)	1.00 (Pinned-Pinned, no sway)
$\ell_{e1}; \ell_{e2}$	10 ft; 5 ft

Step 2: Load Definition (ASCE 7-22 / NDS 2024)

D — Dead load	Self-weight — 74 lb (7.4 plf × 10 ft)
D — Dead load	6,000 lb
L — Live load	3,500 lb
E — Earthquake load	3,500 lb
W — Wind load — lateral	150 plf uniform (strong-axis bending — load on the narrow face)
Discontinuous wall/frame	Supports a discontinuous wall or frame — $\Omega_0 = 3, \rho = 1.0$ ($E_{mh} = \Omega_0 \cdot Q_E$)
L — live-load reduction	Not applied — enter reduced L if taken.

Step 3: Material & Section

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Specific Gravity ⁴	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity			
		F _b	F _t	F _v	F _{c⊥}	F _c	E	E _{min}	G	
		DOUGLAS FIR - LARCH, DOUGLAS FIR, WESTERN LARCH								
No. 2 (Posts & Timbers)	Posts and Timbers	750	475	170	625	700	1,300,000	470,000	0.50	PLIB, WWPA

Nominal Size b x d	Standard Dressed Size (S4S) b x d in x in	Area of Section A in ²	X - X AXIS		Y - Y AXIS	
			Section Modulus	Moment of Inertia	Section Modulus	Moment of Inertia
			S _x	I _x	S _y	I _y
			in ³	in ⁴	in ³	in ⁴
Timbers (5" x 5" and larger) ² Post and Timber (see NDS 4.1.3.4 and NDS 4.1.5.3)						

Nominal Size b x d	Standard Dressed Size (S4S) b x d in x in	Area of Section A in ²	X - X AXIS		Y - Y AXIS	
			Section Modulus S_x in ³	Moment of Inertia I_x in ⁴	Section Modulus S_y in ³	Moment of Inertia I_y in ⁴
6 × 6	5 - 1/2 × 5 - 1/2	30.25	27.73	76.26	27.73	76.26

Step 4: Design Parameters (NDS Adjustment Factors)

Adjustment factor	Condition	F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}
C_M (wet service)	Dry ($MC \leq 19\%$)	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_t (temperature)	$T \leq 100^\circ F$	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_i (incising)	Not incised	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C_F (size)	Section 6×6	—	—	—	—	1.0	—	—
C_F (size)	Section 6×6 — combined M+P	1.0	—	—	—	—	—	—
C_{fu} (flat use)	Edgewise use — combined M+P bends about the strong axis	1.0	—	—	—	—	—	—
C_L (beam stability)	$d \leq b$ — no lateral support required	1.0	—	—	—	—	—	—
C_D (load duration)	Axial compression: 9m. D + $0.525E_{mh} + 0.75L$ (Ten minutes); Combined M+P: D + $0.75L$ + $0.75 \times 0.6W$ (Ten minutes); Shear: D + $0.6W$ (Ten minutes)	1.6	—	1.6	—	1.6	—	—
ℓ_e/d	21.8 (strong) / 10.9 (weak) — governs on strong axis. $\ell_e/d = 21.8 \leq 50 \checkmark$							
C_P	0.572 (governing combination 9m. D + $0.525E_{mh} + 0.75L$, $C_D = 1.6$)							

Step 5: Design Verification

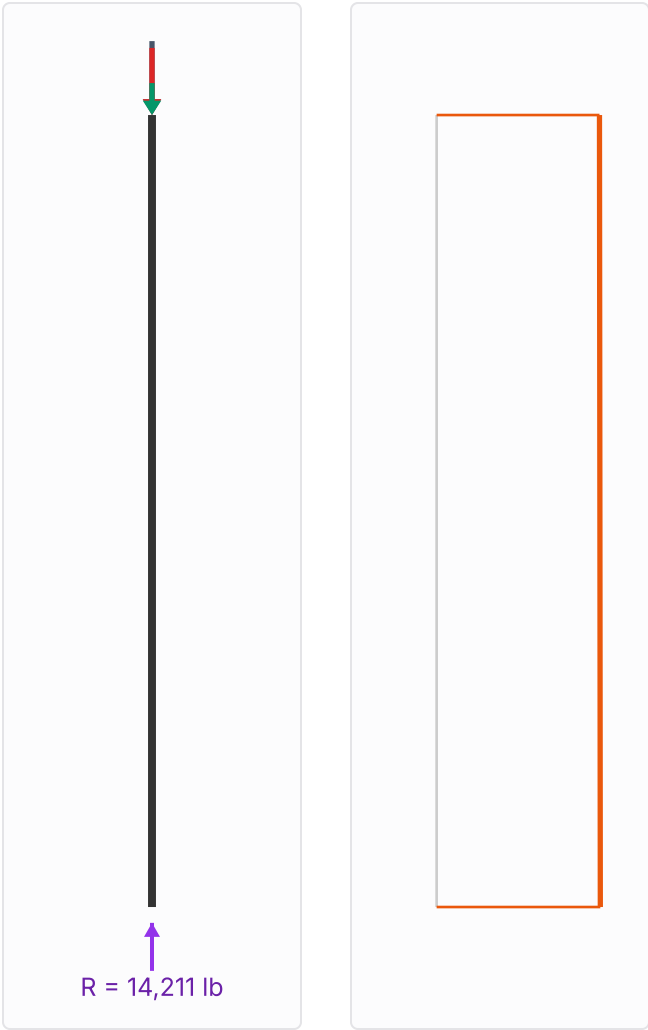
Controlling Load	P (lb)	f (psi)	F' (psi)	Axial check	Combined M+P	Shear	Result
Combination				D/C ratio			
9m. D + 0.525E _{mh} + 0.75L	14,211	470	640	0.733	no lateral in this combination	no lateral in this combination	PASS
Bearing — F _{c⊥} of Douglas Fir-Larch	—	—	—	0.704	—	—	PASS

Step 6: Visualization

Shown case	9m. D + 0.525E _{mh} + 0.75L (governing)
Axial load	14,211 lb
Section	6×6

COLUMN MODEL — 9M. D + 0.525E_{MH} + 0.75L (GOVERNING)

Column Model Axial — max = 14,211 lb



- D = 6,074 lb
 - 0.525×E_{mh} = 5,513 lb
 - 0.75×L = 2,625 lb
- 9m. D + 0.525E_{mh} + 0.75L (governing)

Shear Wall SW-1 — Line A, 1st Floor

Step 1: Geometry & Configuration

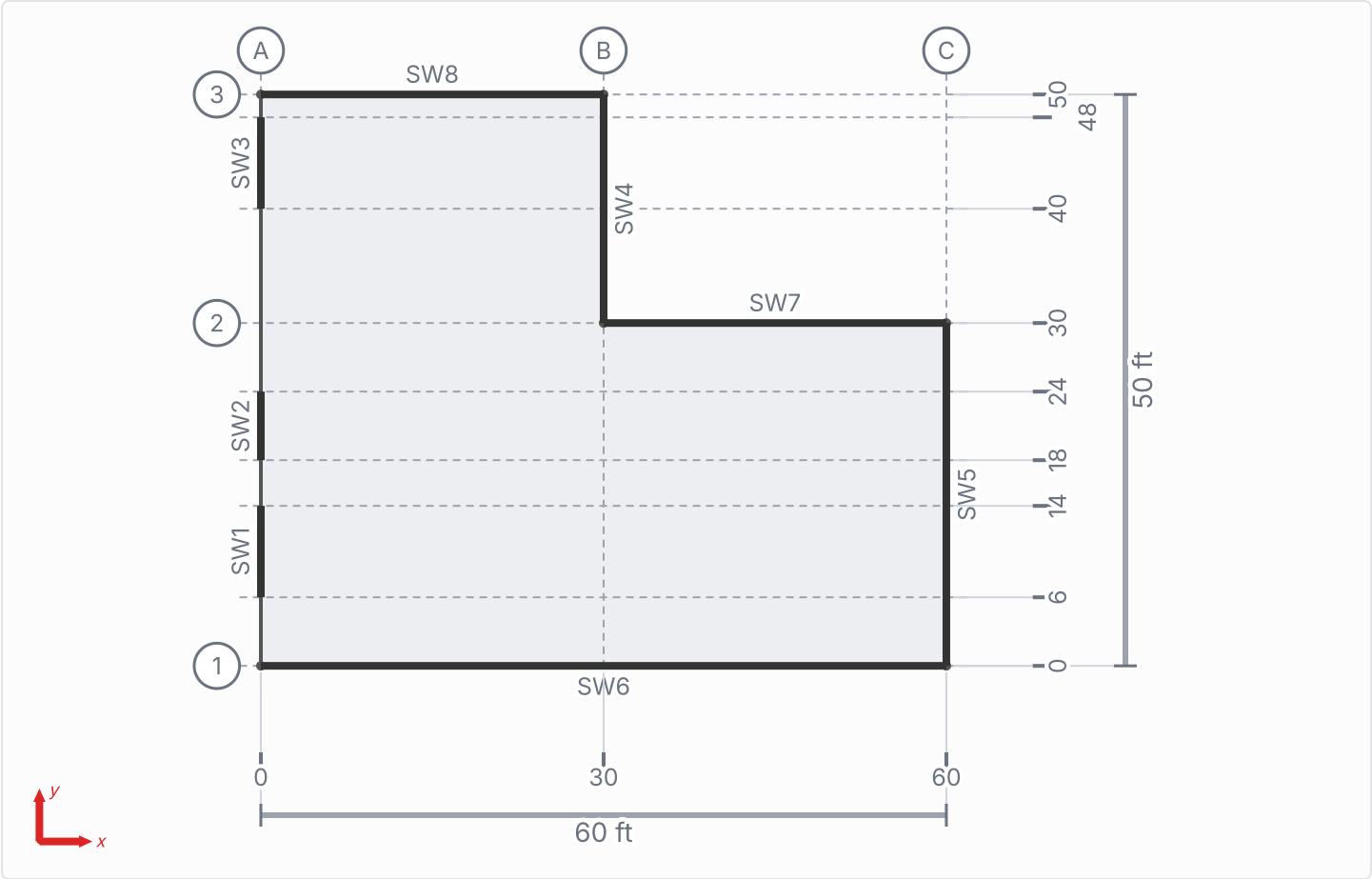
Wall height h	9 ft			
Lines of resistance	6 on the plan (A, B, C, 1, 2, 3)			
Wall	Grids	Gap before (ft)	b (ft)	h/b
DIRECTION X · LINE 1				
SW6	1/A-C	0	60	0.15
Line length		60 ft (footprint) · Σb = 60 ft · 0 / 0 ft to the edges		
Share of the story force		E 37.5% · W 30%		
DIRECTION X · LINE 2				
SW7	2/B-C	30	30	0.30
Line length		60 ft (footprint) · Σb = 30 ft · 30 / 0 ft to the edges		
Share of the story force		E 50% · W 50%		
DIRECTION X · LINE 3				
SW8	3/A-B	0	30	0.30
Line length		30 ft (footprint) · Σb = 30 ft · 0 / 0 ft to the edges		
Share of the story force		E 12.5% · W 20%		
DIRECTION Y · LINE A				
SW1	A/6 ft-14 ft	6	8	1.13
SW2	A/18 ft-24 ft	4	6	1.50
SW3	A/40 ft-48 ft	16	8	1.13
Line length		50 ft (footprint) · Σb = 22 ft · 6 / 2 ft to the edges		
Share of the story force		E 31.25% · W 25%		
DIRECTION Y · LINE B				
SW4	B/2-3	30	20	0.45
Line length		50 ft (footprint) · Σb = 20 ft · 30 / 0 ft to the edges		
Share of the story force		E 50% · W 50%		
DIRECTION Y · LINE C				
SW5	C/1-2	0	30	0.30

Wall	Grids	Gap before (ft)	b (ft)	h/b
Line length	30 ft (footprint) · $\Sigma b = 30 \text{ ft} \cdot 0 / 0 \text{ ft to the edges}$			
Share of the story force	E 18.75% · W 25%			

Basis & assumptions

Diaphragm

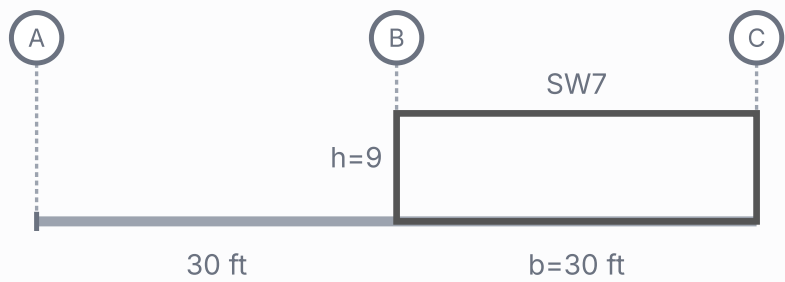
Idealized as flexible: the story force reaches each line by tributary area (seismic) and by tributary length along the span (wind). The engineer confirms the flexible-diaphragm condition applies — light-frame construction with no topping of concrete or similar materials on the panels except nonstructural topping of 1.5 in or less, each line within the allowable story drift.



Line 1 — direction X



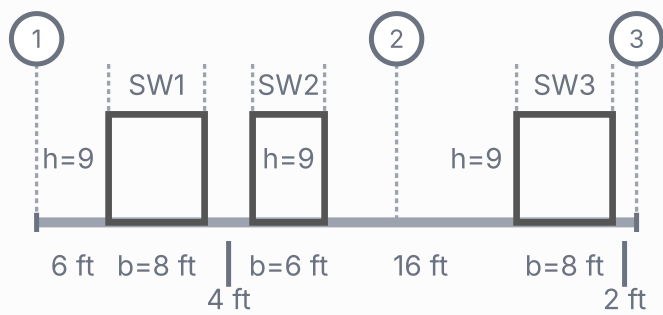
Line 2 — direction X



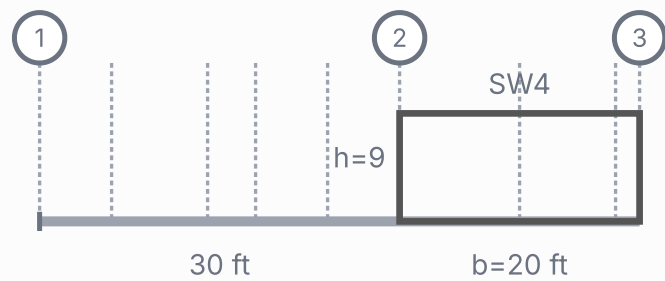
Line 3 — direction X



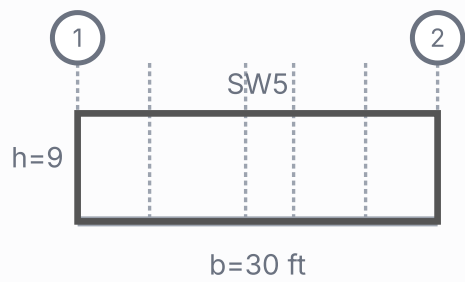
Line A — direction Y



Line B — direction Y



Line C — direction Y



Step 2: Load Definition (ASCE 7-22 / SDPWS 2021)

SEISMIC PARAMETERS — WHOLE BUILDING					
Seismic Design Category		D			
S _{DS}		0.667 g			
STORY FORCES					
Direction	W (lb)	E (lb)	ρ	Distribution	
X	24,000 lb	24,000 lb	1.0	Tributary area (E), length (W)	
Y	24,000 lb	24,000 lb	1.3	Tributary area (E), length (W)	
LINE FORCES					
Line	W (lb)	E (lb)	E _h = ρE (lb)	D on the line (lb)	E _v (lb)
DIRECTION X					
Line 1	7,200 lb	9,000 lb	9,000 lb	18,000 lb	2,401 lb
Line 2	12,000 lb	12,000 lb	12,000 lb	9,000 lb	1,201 lb
Line 3	4,800 lb	3,000 lb	3,000 lb	9,000 lb	1,201 lb

LINE FORCES					
Line	W (lb)	E (lb)	$E_h = \rho E$ (lb)	D on the line (lb)	E_v (lb)
DIRECTION Y					
Line A	6,000 lb	7,500 lb	9,750 lb	17,600 lb	2,348 lb
Line B	12,000 lb	12,000 lb	15,600 lb	16,000 lb	2,134 lb
Line C	6,000 lb	4,500 lb	5,850 lb	24,000 lb	3,202 lb

GRAVITY LOADS ON THE WALLS			
Lines	Load	Applied	Value
Lines A, B, C	D — Dead load	Uniform on all walls	800 plf
Lines A, B, C	L — Live load	Uniform on all walls	400 plf
Lines 1, 2, 3	D — Dead load	Uniform on all walls	300 plf
Lines 1, 2, 3	L — Live load	Uniform on all walls	150 plf

L — live-load reduction	Not applied — enter reduced L if taken
W — roof wind uplift	Not included — W is the horizontal story force only

Basis & assumptions	
Story forces	W and E act horizontally at the top of the wall line; E is the strength-level story force before ρ . Each line takes its share of the direction's force by tributary area (E) and tributary length (W) — flexible diaphragm.
E_h and E_v	$E_h = \rho \times E$. $E_v = 0.2 \times S_{DS} \times D = 0.2 \times 0.667 \times D$, with D the dead load on the line's walls (the sum of their dead-load reactions).
ρ	1.0 in direction X (SDC D) — a redundancy condition of ASCE 7-22 §12.3.4.2 the engineer has verified.
Roof wind uplift	Not included — W is the horizontal story force only; MWFRS roof uplift acting with the same wind is not deducted from the resisting dead load. Add it to the wind hold-down demand by hand, or provide a separate uplift load path.

Step 3: Sheathing & Fasteners

Sheathing Material	Minimum Nominal Panel Thickness (in)	Minimum Nail Bearing Length in Framing Member or Blocking, l_m (in)	Nail Type & Size ⁹ Length (in) x Shank diameter (in) x Head diameter (in)	Panel Edge Nail Spacing (in)		
				6		
				V_n (plf)	G_a (kips/in.)	
					OSB	PLY
DIRECTION X · LINES 1, 2, 3						
Wood Structural Panels - Structural I ^{4,5}	15/32	1- 3/8	8d common nail (2-1/2 × 0.131 x 0.281) ⁸	785	14	11
DIRECTION Y · LINES A, B, C						
Wood Structural Panels - Structural I ^{4,5}	15/32	1- 3/8	8d common nail (2-1/2 × 0.131 x 0.281) ⁸	785	14	11

DIRECTION X · LINES 1, 2, 3	
V_n used in design	1,570 plf (2 × 785, both sides)
ASD allowable unit shear	785 plf wind · 560.7 plf seismic
DIRECTION Y · LINES A, B, C	
V_n used in design	1,570 plf (2 × 785, both sides)
ASD allowable unit shear	785 plf wind · 560.7 plf seismic

Basis & assumptions	
V_n used in design	785 plf tabulated per side; both sides: 2 × 785 = 1,570 plf.
ASD allowable unit shear	$V_n / 2.0$ for wind and $V_n / 2.8$ for seismic (SDPWS 2021 §4.3.3), before each wall's aspect-ratio factor.
Sheathing sides	Both sides of common framing, same row and same nail spacing — the footnote adjustments apply to each side before the two sides combine (SDPWS 2021 §4.3.5.4.1).
Construction basis	All panel edges supported by and fastened to framing members or blocking; panels at least 4 ft × 8 ft except at boundaries and changes in framing; nails at least 3/8 in from panel edges; field nails the edge-nail size at 6 in o.c. maximum (12 in where panels are thicker than 7/16 in or studs are spaced less than 24 in o.c.); nailed face of framing and blocking 2 in nominal or greater; studs 24 in o.c. maximum; panels to DOC PS 1 or PS 2.

Step 4: Hold-downs, anchors, end posts

SUBSTEP 4A: HOLD-DOWNS

Walls	Model No.	Anchor Bolt Dia.	CL (in)	Wood Fasteners	Min. Wood Member (in)	Species	Allowable Tension (lb)	Deflection (in)
SW1-SW8	DTT2Z®	1/2	13/16	(8) 1/4 x 1 1/2 SDS	3 x 3 1/2	DF/SP	2145	0.128

SUBSTEP 4B: SABR ANCHOR BOLT AT STEM WALL

Walls	Model No.	Dimensions (in)				Allowable Tension Loads (lb.)					
		Stem Wall Width	Diameter	Length	Min. Embed. (l _e)	Wind and SDC A&B			SDC C-F		
						Middle	Corner	End	Middle	Corner	End
SW1-SW8	SABR5/8X24	6	5/8	24	18	8,630	7,560	6,360	8,630	6,300	5,575

SUBSTEP 4B: HOLD-DOWN ANCHOR ROD TO SABR

Walls	Hold-down Anchor Dia. (in)	SABR Dia. (in)	Transition Coupler	Allowable Tension (lb)
SW1-SW8	1/2	5/8	CNW5/8-1/2 (5/8 to 1/2)	4,265

SUBSTEP 4C: END POSTS (CHORDS)

Walls	Post	Bottom Plate / Sill	Bolt Holes	C _b
SW1-SW8	6x6 · Douglas Fir-Larch No. 1	Douglas Fir-Larch	None — follows 4A: DTT2Z® (screw-fastened)	Not applied

Step 5: Collector (Drag Strut)

COLLECTOR INPUTS — WHOLE BUILDING

Collector amplification	Light-frame exception — F _{px} with ρ = 1.0, no Ω ₀
Collector increase	× 1.25 — Type 2 reentrant corner, SDC D
w _{px} — diaphragm weight	233,900 lb
Importance factor I _e	1.00

DIAPHRAGM FORCE

DIRECTION X

F _{px} — diaphragm force	31,202 lb — the minimum governs
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DIAPHRAGM FORCE

DIRECTION Y

F_{px} — diaphragm force **31,202 lb** — the minimum governs

COLLECTOR PER LINE

Line	Delivery	Seismic share	Collector seismic force
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DIRECTION X

Line 1	Full line	37.5%	14,626 lb
Line 2	Full line	50%	19,501 lb
Line 3	Full line	12.5%	4,875 lb

DIRECTION Y

Line A	Full line	31.25%	12,188 lb
Line B	Full line	50%	19,501 lb
Line C	Full line	18.75%	7,313 lb

Basis & assumptions

Collector amplification	Exception — a structure braced entirely by wood light-frame shear walls: the collectors are designed for the diaphragm force F_{px} with $\rho = 1.0$; Ω_0 is not applied (ASCE 7-22 §12.10.2.1 Exception).
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Collector increase	$\times 1.25$ on the collector's F_{px} force — Type 2 reentrant corner at (30, 30) ft: 30 ft of 60 ft (50%) along x and 20 ft of 50 ft (40%) along y, both over 20%, SDC D (ASCE 7-22 §12.3.3.5).
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F_{px} — direction X	$(\sum F_i / \sum w_i, \text{levels x to roof}) \times w_{px} = 24,000 \text{ lb}$; not less than $0.2 S_{DS} I_e w_{px} = 31,202 \text{ lb}$; need not exceed $0.4 S_{DS} I_e w_{px} = 62,405 \text{ lb} \rightarrow F_{px} = 31,202 \text{ lb}$, the minimum governs (ASCE 7-22 §12.10.1.1). Each line takes its seismic share of it.
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F_{px} — direction Y	$(\sum F_i / \sum w_i, \text{levels x to roof}) \times w_{px} = 24,000 \text{ lb}$; not less than $0.2 S_{DS} I_e w_{px} = 31,202 \text{ lb}$; need not exceed $0.4 S_{DS} I_e w_{px} = 62,405 \text{ lb} \rightarrow F_{px} = 31,202 \text{ lb}$, the minimum governs (ASCE 7-22 §12.10.1.1). Each line takes its seismic share of it.
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Collector delivery — lines A, C, 1, 3	Along the full diaphragm line — seismic by the line's tributary depth in the footprint (constant along this line); wind uniform.
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Collector delivery — lines B, 2	Along the full diaphragm line — seismic by the line's tributary depth in the footprint; wind uniform.
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Basis & assumptions	
Collector seismic force — line A	Light-frame exception — $\text{share} \times F_{px} = 31.25\% \times 31,202 \times 1.25$ (irregularity increase) = 12,188 lb ($\rho = 1.0$; strength level — each combination applies its own factor, 0.7 or 0.525).
Collector seismic force — lines B, 2	Light-frame exception — $\text{share} \times F_{px} = 50\% \times 31,202 \times 1.25$ (irregularity increase) = 19,501 lb ($\rho = 1.0$; strength level — each combination applies its own factor, 0.7 or 0.525).
Collector seismic force — line C	Light-frame exception — $\text{share} \times F_{px} = 18.75\% \times 31,202 \times 1.25$ (irregularity increase) = 7,313 lb ($\rho = 1.0$; strength level — each combination applies its own factor, 0.7 or 0.525).
Collector seismic force — line 1	Light-frame exception — $\text{share} \times F_{px} = 37.5\% \times 31,202 \times 1.25$ (irregularity increase) = 14,626 lb ($\rho = 1.0$; strength level — each combination applies its own factor, 0.7 or 0.525).
Collector seismic force — line 3	Light-frame exception — $\text{share} \times F_{px} = 12.5\% \times 31,202 \times 1.25$ (irregularity increase) = 4,875 lb ($\rho = 1.0$; strength level — each combination applies its own factor, 0.7 or 0.525).

Step 6: Design Verification

DEMAND / CAPACITY AS D/C — EACH CHECK UNDER ITS COLUMN'S COMBINATION UNLESS THE CELL NAMES ANOTHER

Wall	Sheathing (line shear D/C)	Hold-down (overturning T D/C)	End post (compression C D/C)	End post (net- section T D/C)	Anchor bolts — stem wall W: Wind and SDC A&B · E: SDC C- F (SDC D)		
					Middle	Corner	End
Combination	8. $D + 0.7E_v \pm 0.7E_h$	10. $0.6D - 0.7E_v \pm 0.7E_h$	9. $D + 0.525E_v \pm 0.525E_h + 0.75L$	10. $0.6D - 0.7E_v \pm 0.7E_h$			
DIRECTION X · LINE 1							
SW6	19% V = 6,300 lb	Not required T = 0	72% C = 13,658 lb	0%	0%	0%	0%
DIRECTION X · LINE 2							
SW7	50% V = 8,400 lb	13% T = 282 lb	44% C = 8,375 lb	1%	3%	4%	5%
DIRECTION X · LINE 3							
SW8	12% V = 2,100 lb	Not required T = 0	37% C = 6,921 lb	0%	0%	0%	0%
DIRECTION Y · LINE A							
SW1	55% V = 2,482 lb	65% T = 1,395 lb	36% C = 6,747 lb	3%	16%	22%	25%
SW2	55% V = 1,861 lb	89% T = 1,909 lb	30% 8. $D + 0.7E_v \pm 0.7E_h$ C = 5,670 lb	5%	22%	30%	34%
SW3	55% V = 2,482 lb	65% T = 1,395 lb	36% C = 6,747 lb	3%	16%	22%	25%
DIRECTION Y · LINE B							
SW4	97% V = 10,920 lb	46% T = 991 lb	80% C = 15,212 lb	2%	11%	16%	18%
DIRECTION Y · LINE C							
SW5	24% V = 4,095 lb	Not required T = 0	96% C = 18,110 lb	0%	0%	0%	0%

DIRECTION X · LINE 1

Collector (drag strut) **None** — the walls run the full line, so no drag force develops

DIRECTION X · LINE 2

Anchor rod SW7 **CNW5/8-1/2 coupler** — D/C 7% ✓ PASS

Collector (drag strut) **max F = 8,532 lb at SW7 left end (8. $D + 0.7E_v + 0.7E_h$)**

DIRECTION X · LINE 3	
Collector (drag strut)	None — the walls run the full line, so no drag force develops
DIRECTION Y · LINE A	
Anchor rod SW1	CNW5/8 - 1/2 coupler — D/C 33% ✓ PASS
Anchor rod SW2	CNW5/8 - 1/2 coupler — D/C 45% ✓ PASS
Anchor rod SW3	CNW5/8 - 1/2 coupler — D/C 33% ✓ PASS
Collector (drag strut)	max F = 1,396 lb at SW3 left end (8. D + 0.7Ev + 0.7Eh)
DIRECTION Y · LINE B	
Anchor rod SW4	CNW5/8 - 1/2 coupler — D/C 23% ✓ PASS
Collector (drag strut)	max F = 10,238 lb at SW4 left end (8. D + 0.7Ev + 0.7Eh)
DIRECTION Y · LINE C	
Collector (drag strut)	None — the walls run the full line, so no drag force develops

Basis & assumptions	
Shear wall model	Individual full-height (segmented) shear walls; each line force is shared in proportion to the walls' design capacities. Perforated and force-transfer-around-opening shear walls are not modelled.
Stories	One story — the story force of Step 2 acts at the top of the wall; the overturning of stories above is not carried down a stacked wall.
Diaphragm	Flexible, as idealized in Step 1; the diaphragm itself is not designed here.
Out-of-plane loads	Not checked — in-plane shear and overturning only; wind and seismic on the face of the wall are a separate design.
Deflection and drift	Not computed — the shear wall deflection and the story drift are not checked in this package.
Orthogonal directions	Not combined — each direction is designed for its own story force; an element shared by a wall of each direction (a corner post, its hold-down or footing) is not checked for the two acting together.
Base shear transfer	Not designed in this package — the connection carrying the wall shear into the foundation or the framing below is a separate design.

Basis & assumptions

D/C	Demand over capacity under the ASD combination that governs each check: the line shear against the line capacity (the same D/C on every wall of the line; V is the wall's share of the line force, in proportion to its capacity), a hold-down against its tabulated allowable tension, an anchor bolt location against its SABR table allowable in the column that combination reads (named over the anchor columns), a designed end post in compression against the lesser of its column capacity at the C_D of that combination (NDS 2.3.2.2) and the bearing of the bottom plate, and in tension against its net section (the gross area less any bolt hole through the post).
Combinations and forces	The Combination row under the column headings names the combination most walls are checked under in that column; a cell names its own only where it differs. Each wall's force is stated once — V under the sheathing, T under the hold-down, C under the end post. The net-section and anchor cells give the D/C alone. The net-section check takes the hold-down's T . An anchor location takes the wall's uplift under its own combination — the hold-down's T wherever the two share a combination. An anchor rod coupler carries the hold-down's T against the allowable of the Substep 4B rod table.
End posts — demand	Every ASD combination at the C_D of its shortest-duration load: the lateral combinations ($C_D = 1.6$) with the overturning chord forces, and the gravity-only combinations with the wall's gravity reaction at each end — the wall taken as a simple span between its ends ($w \cdot b/2$ for a uniform load), conservative for a long bearing wall whose studs bear on a continuous plate. Bearing: the bottom plate's $F_{c\perp}$, no C_D .
Moment arm b_{eff} — line A	SW1 7.245 ft · SW2 5.245 ft · SW3 7.245 ft — $b_{eff} = b - x_T - x_C$; x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3-7).
Moment arm b_{eff} — line B	SW4 19.245 ft — $b_{eff} = b - x_T - x_C$; x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3-7).
Moment arm b_{eff} — line C	SW5 29.245 ft — $b_{eff} = b - x_T - x_C$; x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3-7).
Moment arm b_{eff} — line 1	SW6 59.245 ft — $b_{eff} = b - x_T - x_C$; x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3-7).
Moment arm b_{eff} — line 2	SW7 29.245 ft — $b_{eff} = b - x_T - x_C$; x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3-7).

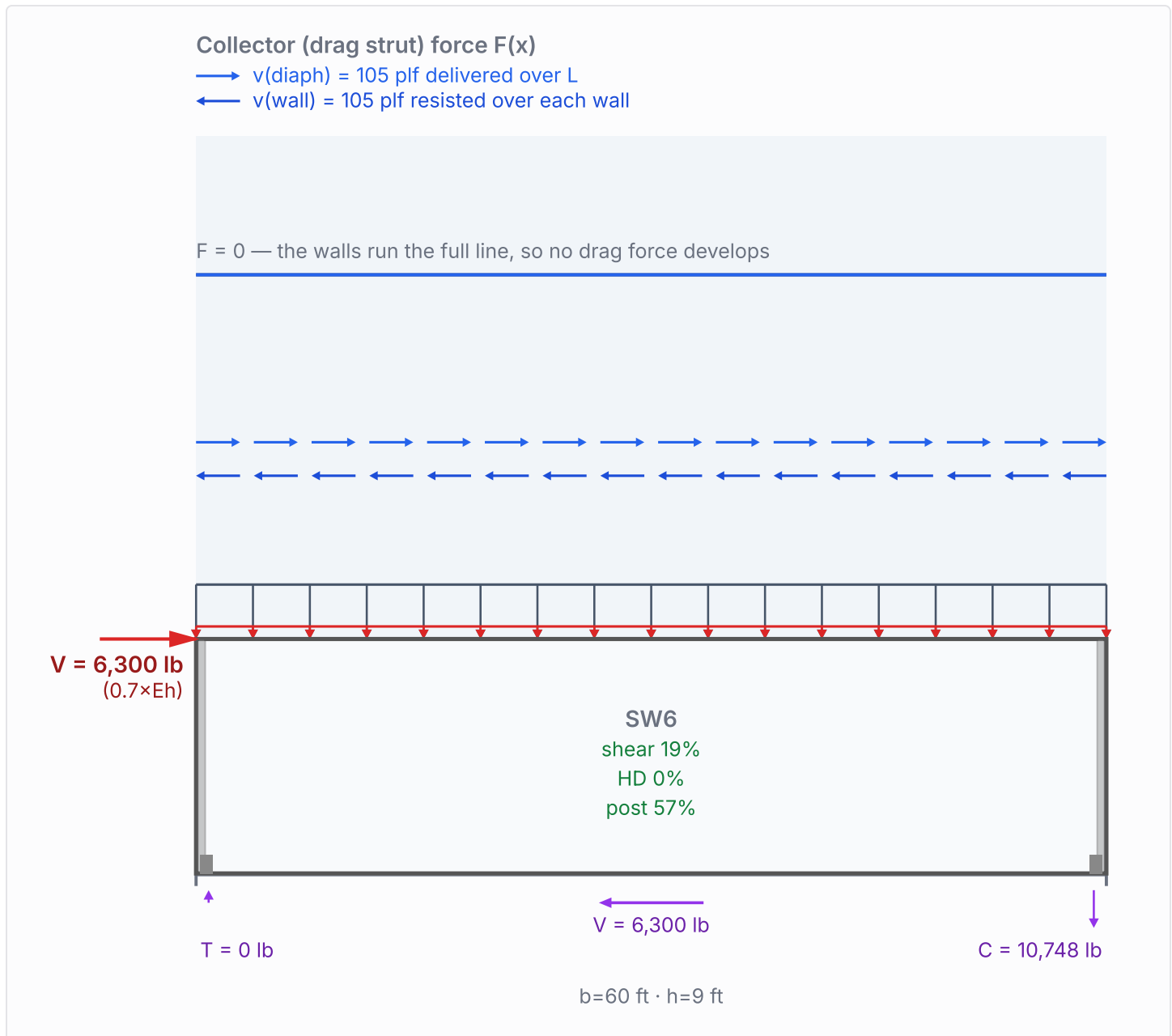
Basis & assumptions	
Moment arm b_{eff} — line 3	SW8 29.245 ft — $b_{eff} = b - x_T - x_C$: x_T = chord width + hold-down CL, x_C = chord width / 2; the chord is the Substep 4C post, else the hold-down row's minimum wood member (SDPWS 2021 Eq. 4.3 - 7).
End posts SW1–SW3 — line A	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.
Collector drag — lines A, B, 2	F_{px} basis, $\rho = 1.0$, $\times 1.25$ irregularity increase; the strap / top-plate splice capacity is by the engineer.
End posts SW4 — line B	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.
End posts SW5 — line C	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.
End posts SW6 — line 1	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.
End posts SW7 — line 2	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.
End posts SW8 — line 3	6×6 · Douglas Fir-Larch No. 1; plate Douglas Fir-Larch ($F_{c\perp} = 625$ psi). NDS 2024 allowables — C_D 1.6 (W/E): I_e/d 18.8, C_P 0.629, F_c' 1,006 psi, column P 30,426 lb, plate bearing 18,906 lb, net-section T 39,930 lb; C_D 1.0 (gravity): I_e/d 18.8, C_P 0.783, F_c' 783 psi, column P 23,675 lb, plate bearing 18,906 lb; C_D 0.9 (gravity): I_e/d 18.8, C_P 0.809, F_c' 728 psi, column P 22,032 lb, plate bearing 18,906 lb.

Step 7: Visualization

Figures

Wall-line elevation, unit shear, and collector force

Line 1 (direction X) — free-body diagram



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 6,300 \text{ lb}$ · Diaphragm line $L = 60 \text{ ft}$

Applied loads:

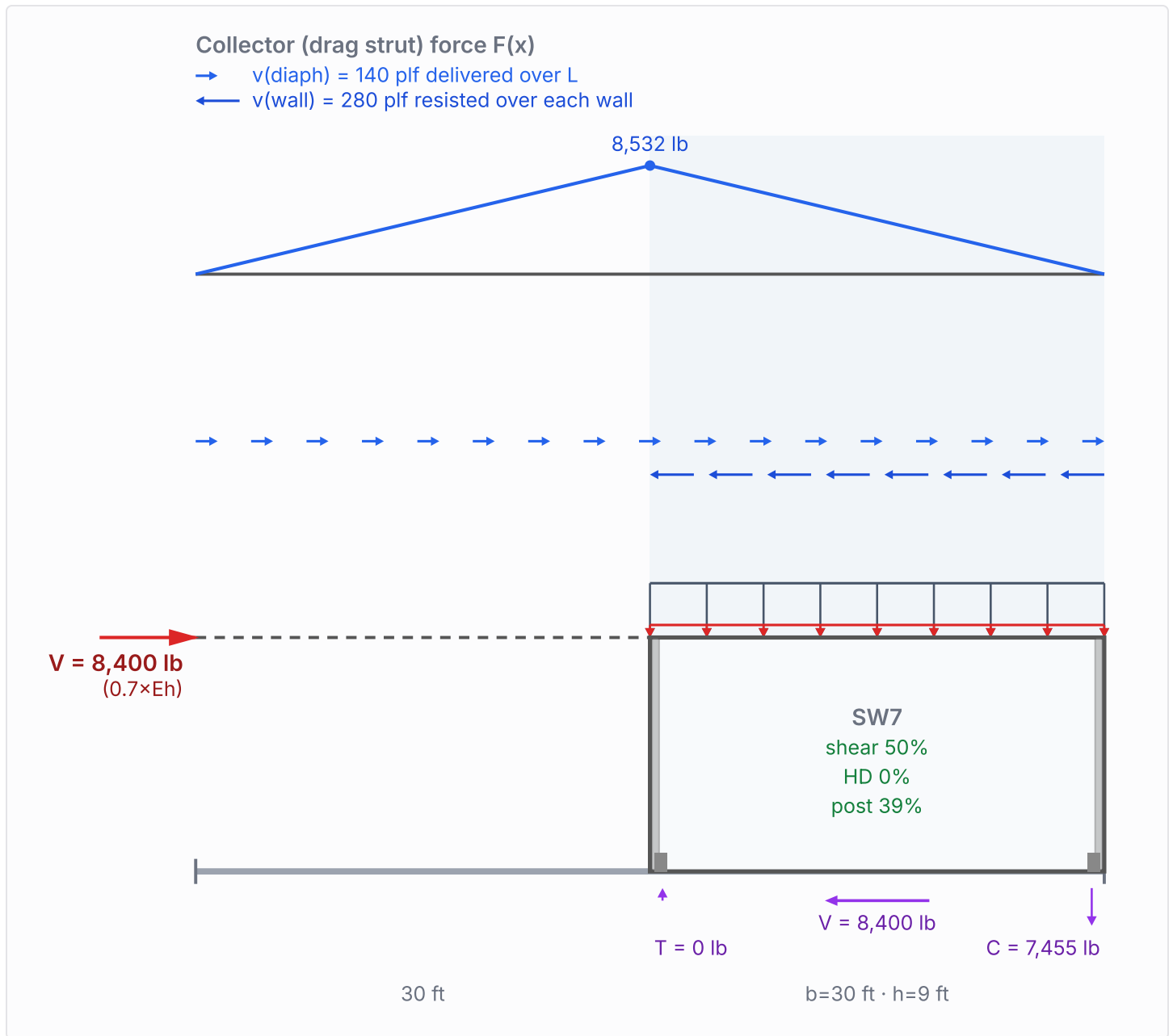
- $0.7 \times Eh = 0.7 \times 9000 = 6,300 \text{ lb}$
- $D = 300 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 2401 = 1,681 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW6 28 plf $\downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$.

Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): 6×6 · Douglas Fir-Larch No. 1.

Line 2 (direction X) — free-body diagram



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 8,400 \text{ lb} \cdot \text{Diaphragm line } L = 60 \text{ ft}$

Applied loads:

- $0.7 \times Eh = 0.7 \times 12000 = 8,400 \text{ lb}$
- $D = 300 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 1201 = 840 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW7 28 plf $\downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; hold-down tension: 10. $0.6D - 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$; collector drag: 8. $D + 0.7Ev + 0.7Eh$.

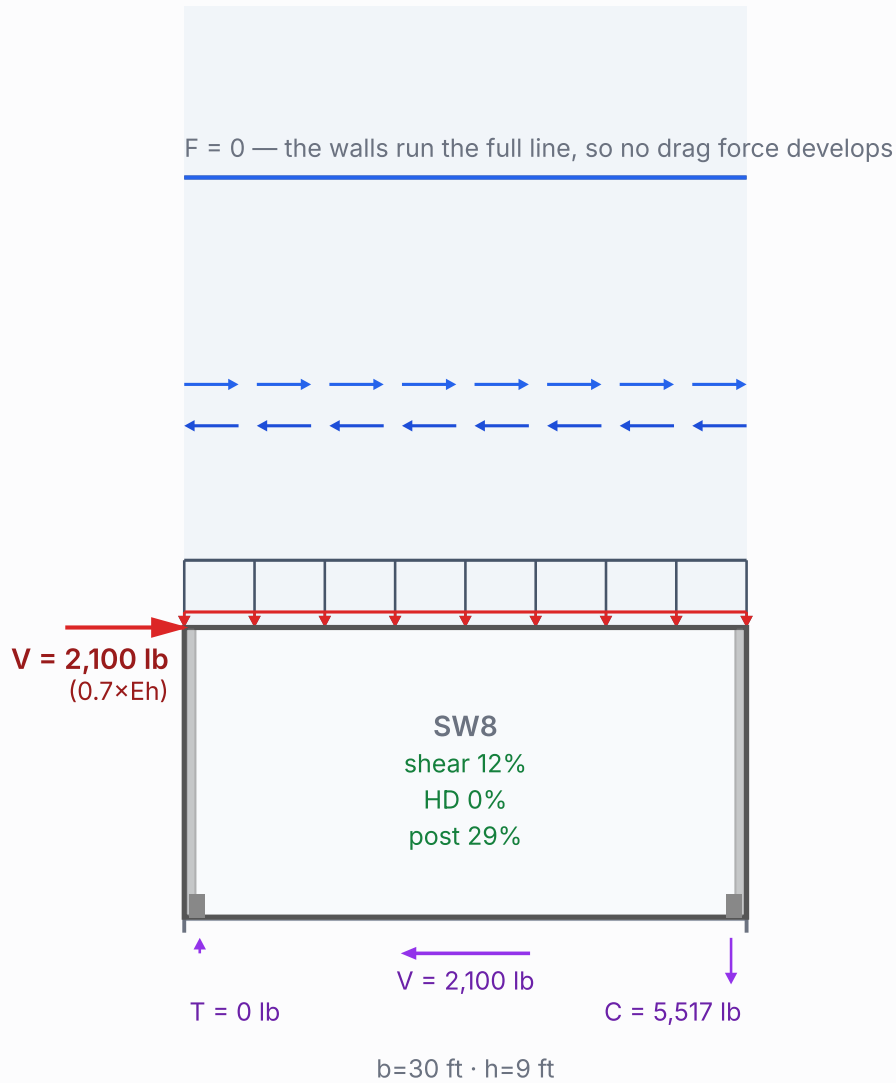
Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): $6 \times 6 \cdot \text{Douglas Fir-Larch No. 1}$.

Line 3 (direction X) — free-body diagram

Collector (drag strut) force $F(x)$

- $v(\text{diaph}) = 70 \text{ plf}$ delivered over L
- ← $v(\text{wall}) = 70 \text{ plf}$ resisted over each wall



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 2,100 \text{ lb} \cdot \text{Diaphragm line } L = 30 \text{ ft}$

Applied loads:

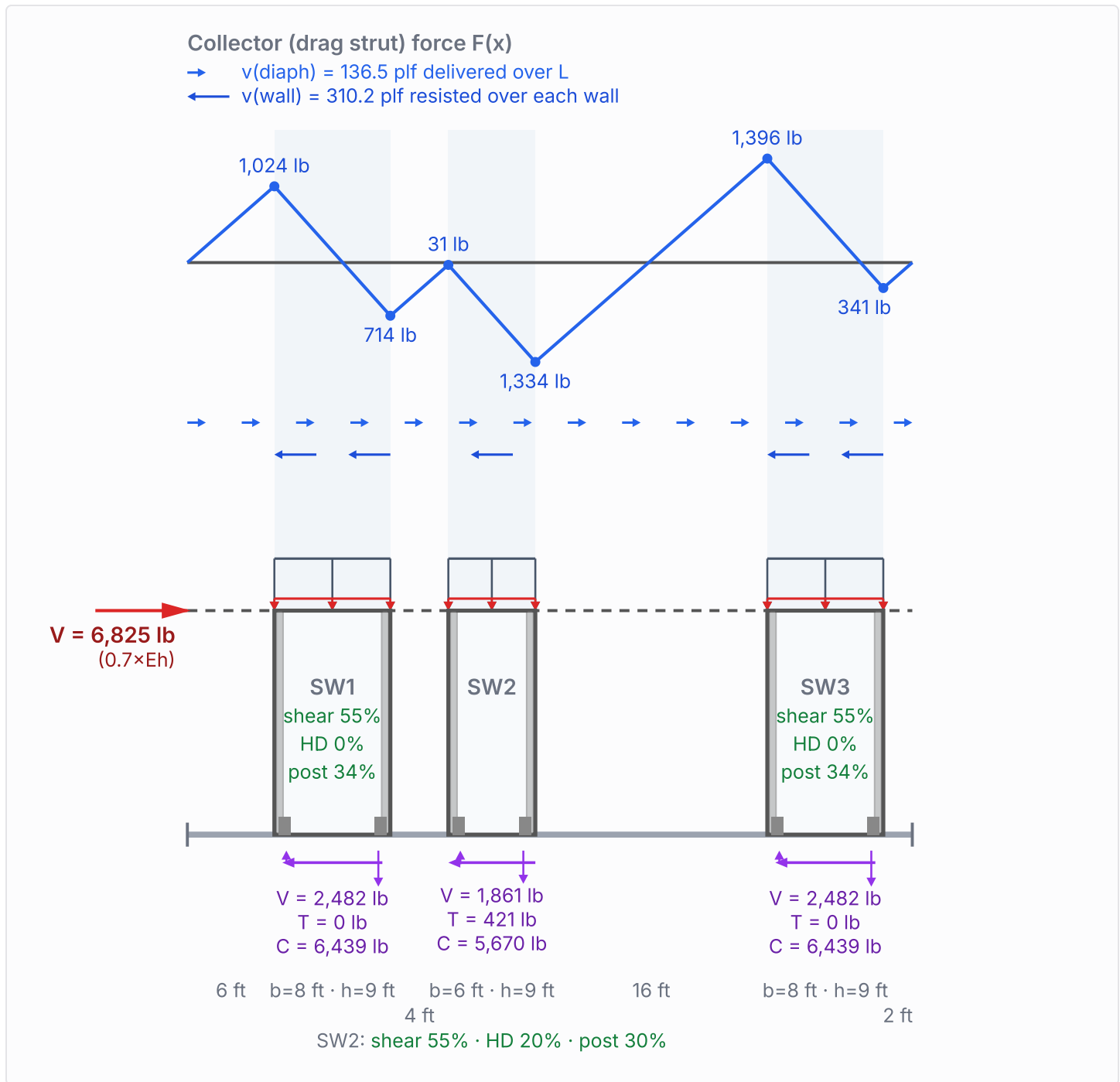
- $0.7 \times Eh = 0.7 \times 3000 = 2,100 \text{ lb}$
- $D = 300 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 1201 = 840 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW8 28 plf $\downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$.

Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): $6 \times 6 \cdot \text{Douglas Fir-Larch No. 1}$.

Line A (direction Y) — free-body diagram



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 6,825 \text{ lb}$ · Diaphragm line $L = 50 \text{ ft}$

Applied loads:

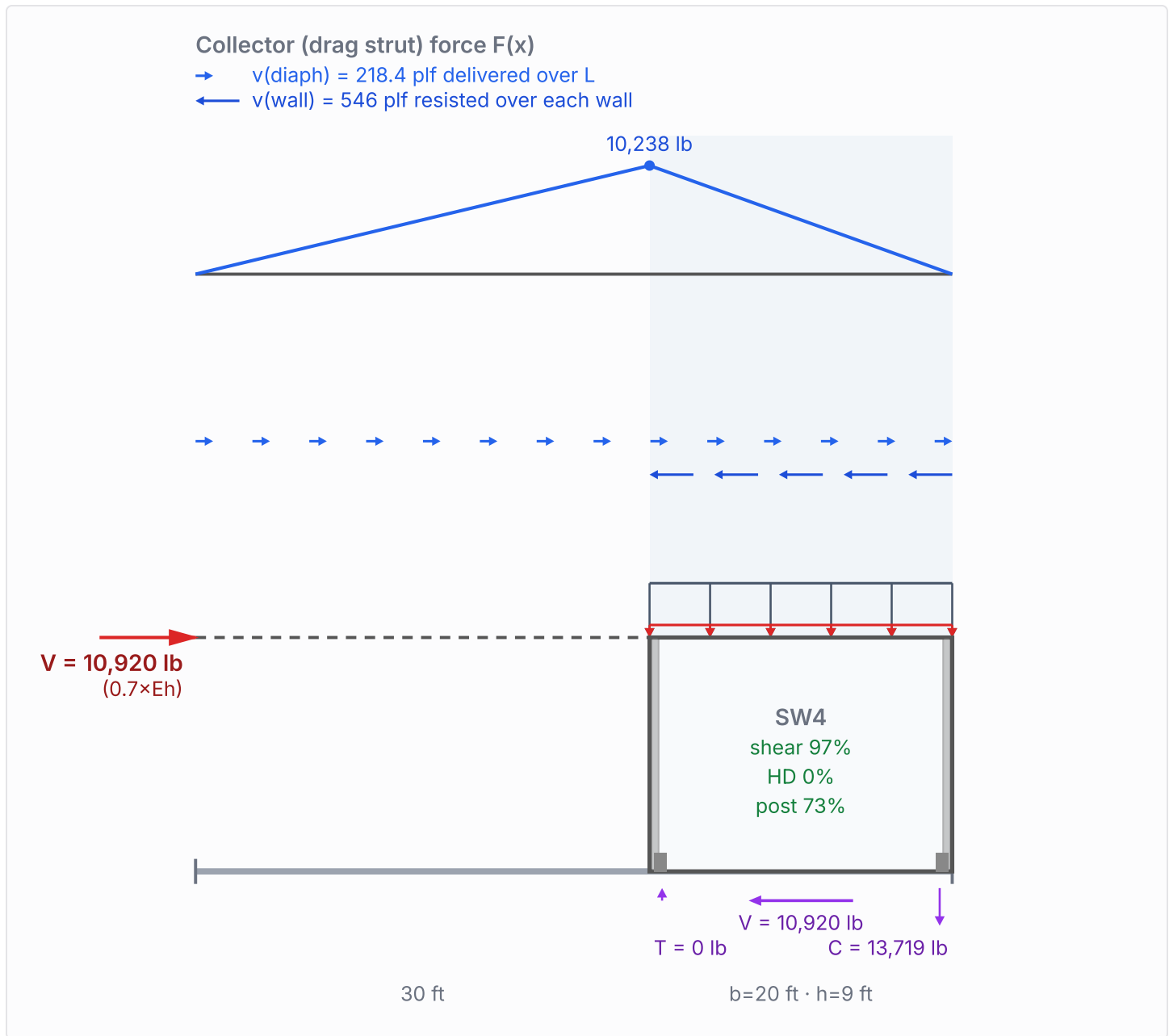
- $0.7 \times Eh = 0.7 \times 9750 = 6,825 \text{ lb}$
- $D = 800 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 2348 = 1,643 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW1 $74.7 \text{ plf} \downarrow$; SW2 $74.7 \text{ plf} \downarrow$; SW3 $74.7 \text{ plf} \downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; hold-down tension: 10. $0.6D - 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$; collector drag: 8. $D + 0.7Ev + 0.7Eh$.

Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): 6×6 · Douglas Fir-Larch No. 1.

Line B (direction Y) — free-body diagram



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 10,920 \text{ lb} \cdot \text{Diaphragm line } L = 50 \text{ ft}$

Applied loads:

- $0.7 \times Eh = 0.7 \times 15600 = 10,920 \text{ lb}$
- $D = 800 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 2134 = 1,494 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW4 74.7 plf $\downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; hold-down tension: 10. $0.6D - 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$; collector drag: 8. $D + 0.7Ev + 0.7Eh$.

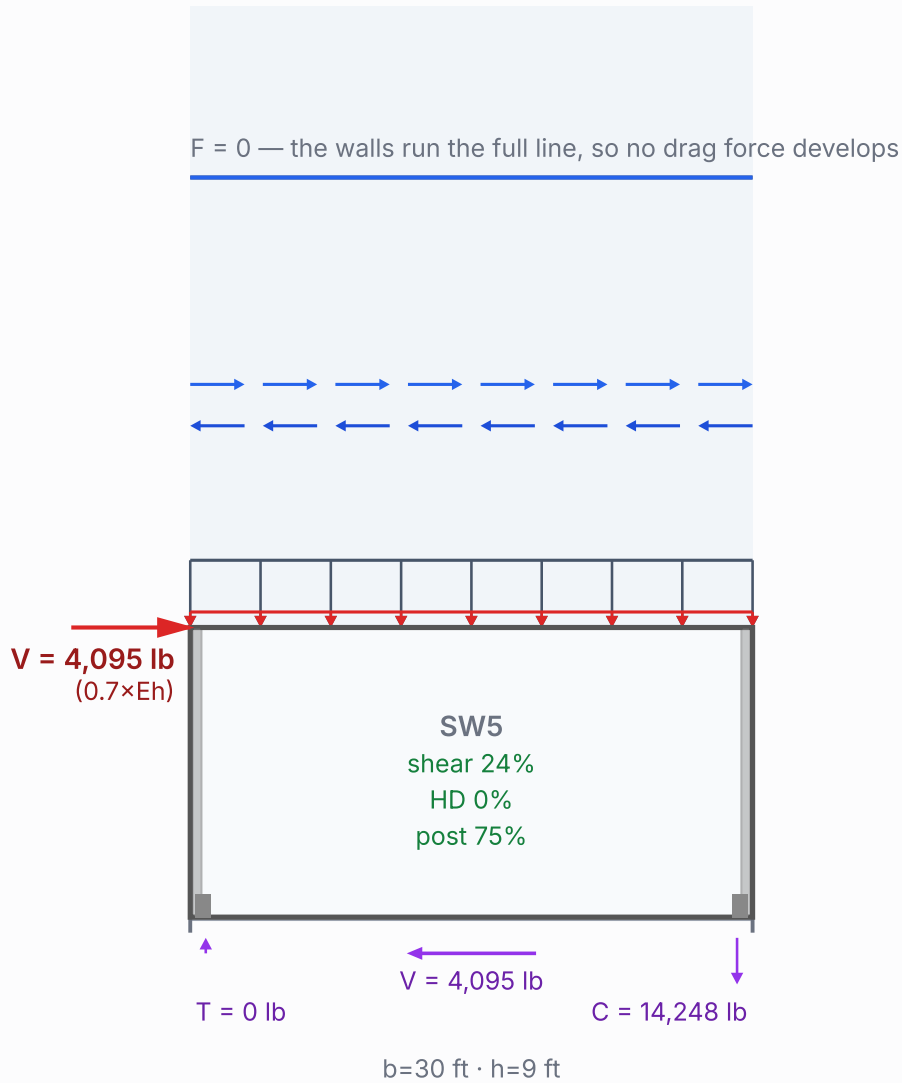
Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): $6 \times 6 \cdot \text{Douglas Fir-Larch No. 1}$.

Line C (direction Y) — free-body diagram

Collector (drag strut) force $F(x)$

- $v(\text{diaph}) = 136.5 \text{ plf}$ delivered over L
- ← $v(\text{wall}) = 136.5 \text{ plf}$ resisted over each wall



8. $D + 0.7Ev + 0.7Eh$ (governing line shear) — $V = 4,095 \text{ lb} \cdot \text{Diaphragm line } L = 30 \text{ ft}$

Applied loads:

- $0.7 \times Eh = 0.7 \times 5850 = 4,095 \text{ lb}$
- $D = 800 \text{ plf}$ (all walls)
- $0.7 \times Ev = 0.7 \times 3202 = 2,241 \text{ lb} \downarrow$ (line)
- Ev bands per wall: SW5 74.7 plf $\downarrow$

Governing combinations — sheathing (line shear): 8. $D + 0.7Ev + 0.7Eh$; end-post compression: 9. $D + 0.525Ev + 0.525Eh + 0.75L$.

Applied loads and reactions are factored per the shown combination; reactions (purple) reverse with load direction.

End posts (shaded bars at wall ends): $6 \times 6 \cdot \text{Douglas Fir-Larch No. 1}$.

Footing F-1 — Steel Column @ Grid C/3

Step 1: Geometry & Configuration

Footing type	Isolated footing (column)
Footing size	12×12 ft (plan)
Thickness	t = 36 in
Overlying fill	2 ft
Slab over footing	6 in
Pedestal	2×2×2 ft (Lx×Ly×H)
Eccentricity	$e_x = 3$ in, $e_y = 3$ in

Step 2: Materials & Reinforcement

f'_c	4,000 psi
f_y	60 ksi
q_a	6,000 psf
Density class (λ)	Normalweight — $\lambda = 1.0$
Sliding μ	$\mu = 0.35$; passive = 8 kips
Concrete unit wt	150 pcf (footing and pedestal)
Soil unit wt	120 pcf
Concrete (slab) unit wt	150 pcf
Reinf. (X)	12-#9 @ 12.4 in o.c.
Reinf. (Y)	12-#9 @ 12.4 in o.c.
Mat layering	Bottom = X-direction bars — d, X mat = 32.436 in; d, Y mat = 31.308 in
Cover	3 in

Step 3: Load Definition (ASCE 7-22)

D — Dead load	P = 340 kips
L — Live load	P = 200 kips
W — Wind load	P = 0 kips; $V_x = 20$ kips, $M_y = 60$ kip-ft; $V_y = 20$ kips, $M_x = 60$ kip-ft
E — Earthquake load	P = 30 kips; $V_x = 12$ kips, $M_y = 40$ kip-ft; $V_y = 12$ kips, $M_x = 40$ kip-ft
Self-weight	footing + pedestal + soil + slab included in D
SDC	D
Overstrength	$\Omega_0 = 3$, $\rho = 1.0$ — the entered E is Q_E ; $E_{mh} = \Omega_0 \cdot Q_E$ in the seismic combinations
L — live-load reduction	Not applied — enter reduced L if taken

4. Design Verification (ACI 318-25)

Bearing (sizing) D/C 0.94	One-way shear D/C 0.44	Two-way shear D/C 0.52	Flexure D/C 0.59	Bearing (loaded area) D/C 0.27
Development D/C 0.94				

Bearing (sizing)	OK (D/C 0.94)
One-way shear	OK (D/C 0.44)
Two-way shear	OK (D/C 0.52)
Flexure	OK (D/C 0.59)
Bearing (loaded area)	OK (D/C 0.27)
Dowels	OK
Development	OK (D/C 0.94)
Overturing	SR $9.11 \geq 1.5$ (8m. D + $0.7E_{mh}$) — OK
Sliding	SR $3.20 \geq 1.0$ (10m. $0.6D - 0.7E_{mh}$) — OK

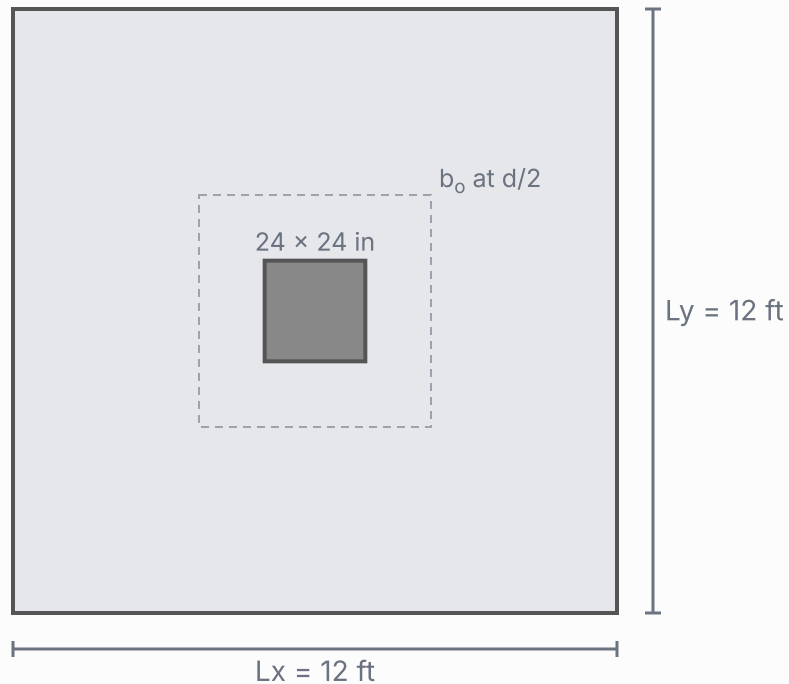
Step 5: Visualization

Figures	Plan view, side views, pressure, DCR
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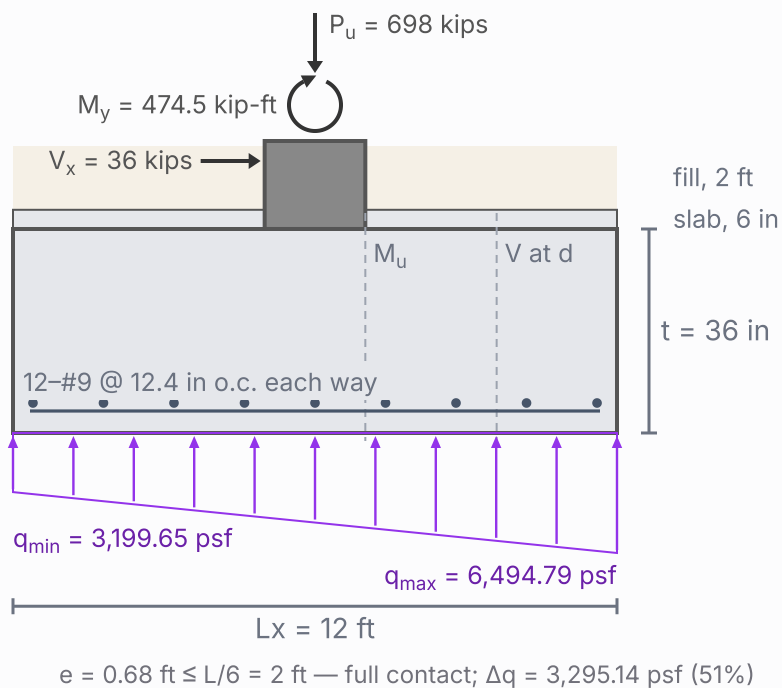
FOOTING VIEWS & SOIL PRESSURE — 6M. 1.2D + E_{MH} + L

Factored (LRFD) — 6m. 1.2D + E_{mh} + L (governing bearing combination): $P = 698$ kips, $q_{conc} = 4,847.22$ psf. The diagram shows THIS combination — pick a wind/seismic combination in the list to see its eccentric (trapezoidal) pressure.

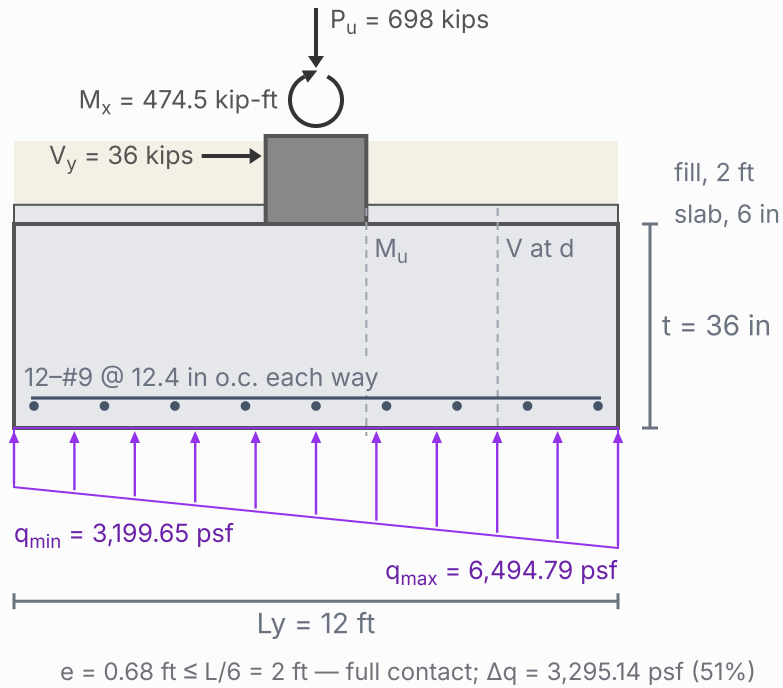
Plan



Section X-Z



Section Y-Z



Dashed guides: flexural critical section per ACI 318-25 Table 13.2.7.1 and one-way shear at d beyond it (§13.2.7.2); plan shows the punching perimeter at $d/2$ (§22.6.4.1). Soil pressure shown for the selected combination; moments are non-concurrent (each section shows its own axis). Both mats are at the bottom: the X mat sits on the 3 in cover and the Y mat rests on it one bar diameter higher, so each direction has its own d . Section depth not to scale.

Retaining Wall RW-1 — East Property Line

Step 1: Geometry & Configuration

Stem height	10 ft
Retained height	10 ft
Stem material	Reinforced concrete — ACI 318-25
Stem thickness	12 in
Toe length	3 ft
Heel length	6 ft
Footing thickness	18 in
Footing width B	10 ft
Soil cover over toe	1 ft
Backfill slope β	18.43° — rises 2 ft over the heel
Shear key	12 × 42 in · toe face 3 ft from the toe edge (at the stem face)

Step 2: Soil & Lateral Earth Pressure

Design lateral soil load	30 psf/ft — Geotechnical report
Pressure condition	Active
Soil unit weight	120 pcf
Groundwater	None above the footing base — drained backfill
Surcharge	100 psf
Surcharge nature	Variable — weight is L (excluded from resisting)
Sliding resistance	$\mu = 0.35$
Passive resistance	200 psf/ft
Allowable bearing	3,000 psf
Seismic earth pressure	Included — ΔP_{AE} as earthquake load E
Seismic method	Mononobe-Okabe (full K_{AE})
S_{DS}	0.667 g
k_h basis	Yielding wall — $k_h = S_{DS}/2.5 = 0.267$ g
Backfill angles	$\phi = 40^\circ$, $\delta = 18.43^\circ$, $\beta = 18.43^\circ$ (Step 1)
Wall self-weight inertia	Included — $F_{wi} = k_h \cdot (W_{stem} + W_{footing} + W_{key})$
Seismic increment ΔP_{AE}	3,404 plf at $0.60 \cdot H$

Step 3: Concrete & Reinforcement

Concrete f'_c	3,000 psi
Steel f_y	60,000 psi
Concrete unit weight	150 pcf
Density class	normalweight
Covers	stem 2.0 / bottom 3.0 / top 2 in
Stem vertical bars	#6 @ 8 in
Stem horizontal bars	#4 @ 9 in
Heel top bars	#6 @ 8 in
Toe bottom bars	#6 @ 12 in
Footing longitudinal bars	#5 @ 9 in
Straight development ℓ_d	General equation — actual cover and spacing, $K_{tr} = 0$

Step 4: Design Verification (ACI 318-25 & IBC 1807.2)

Overturning

FS 5.15 $\geq$ 1.5 ✓

Sliding

FS 2.60 $\geq$ 1.5 ✓

Bearing

0.46 ✓

Overturning, seismic

FS 2.08 $\geq$ 1.1 ✓

Sliding, seismic

FS 1.28 $\geq$ 1.1 ✓

Bearing, seismic

0.91 ✓

Check	Governing combination	Demand	Capacity	D/C or FS
Overall	✓ PASS — all checks satisfied			
STABILITY				
Overturning	D + H + surcharge (lateral only)	14.58 kip-ft/ft	75.04 kip-ft/ft	FS 5.15 ≥ 1.5 ✓
Sliding	D + H + surcharge (lateral only)	3.07 klf	7.99 klf	FS 2.60 ≥ 1.5 ✓
Bearing	D + H + surcharge	1,382.46 psf	3,000 psf	0.46 ✓
Overturning, seismic	D + H + 0.7E	36.01 kip-ft/ft	75.04 kip-ft/ft	FS 2.08 ≥ 1.1 ✓
Sliding, seismic	D + H + 0.7E	6.25 klf	7.99 klf	FS 1.28 ≥ 1.1 ✓
Bearing, seismic	1.0D + 0.7E + H (ASD)	2,729.11 psf	3,000 psf	0.91 ✓
MEMBER STRENGTH				
Stem flexure	1.2D + 1.6H + 1.6L	10 kip-ft/ft	26.66 kip-ft/ft	0.38 ✓
Stem shear	1.2D + 1.6H + 1.6L	2.8 klf	9.49 klf	0.30 ✓
Heel flexure	1.4D	39.94 kip-ft/ft	44.48 kip-ft/ft	0.90 ✓
Heel shear	1.4D	12.98 klf	15.4 klf	0.84 ✓
Toe flexure	1.4D + 1.6H	7.07 kip-ft/ft	28.1 kip-ft/ft	0.25 ✓
Toe shear	1.4D + 1.6H	2.81 klf	14.42 klf	0.19 ✓
Key flexure	1.6H	9.47 kip-ft/ft	23.69 kip-ft/ft	0.40 ✓
Key shear	1.6H	4.76 klf	8.5 klf	0.56 ✓
Stem flexure, seismic	1.2D + Ev + Eh + L + 1.6H	23.21 kip-ft/ft	26.66 kip-ft/ft	0.87 ✓
Stem shear, seismic	1.2D + Ev + Eh + L + 1.6H	5.07 klf	9.49 klf	0.53 ✓
Heel flexure, seismic	1.2D + Ev + Eh + L + 1.6H	39.84 kip-ft/ft	44.48 kip-ft/ft	0.90 ✓

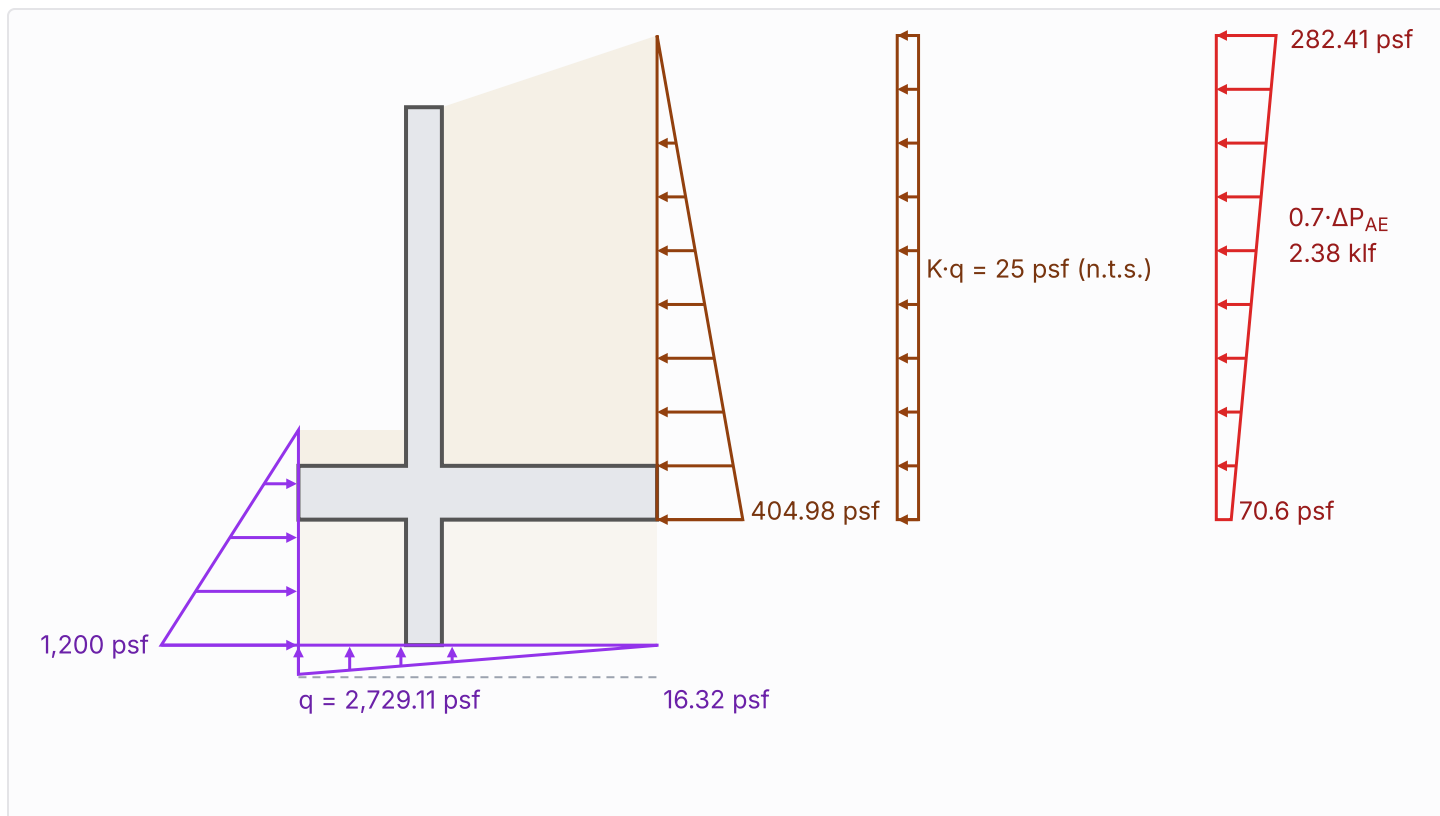
Check	Governing combination	Demand	Capacity	D/C or FS
Heel shear, seismic	$1.2D + E_v + E_h + L + 1.6H$	12.96 klf	15.4 klf	0.84 ✓
Toe flexure, seismic	$0.9D - E_v + E_h + 1.6H$	22.87 kip-ft/ft	28.1 kip-ft/ft	0.81 ✓
Toe shear, seismic	$0.9D - E_v + E_h + 1.6H$	9.15 klf	14.42 klf	0.63 ✓
DETAILING & MINIMUMS				
Stem dowel hook	—	11.502 in	14.25 in	0.81 ✓
Heel bar development	—	25.633 in	69 in	0.37 ✓
Toe bar development	—	19.718 in	33 in	0.60 ✓
Interface dowels	—	0.216 in ² /ft	0.660 in ² /ft	0.33 ✓
Stem horizontal steel	—	0.259 in ² /ft	0.267 in ² /ft	0.97 ✓
Footing S+T steel	—	0.389 in ² /ft	0.827 in ² /ft	0.47 ✓

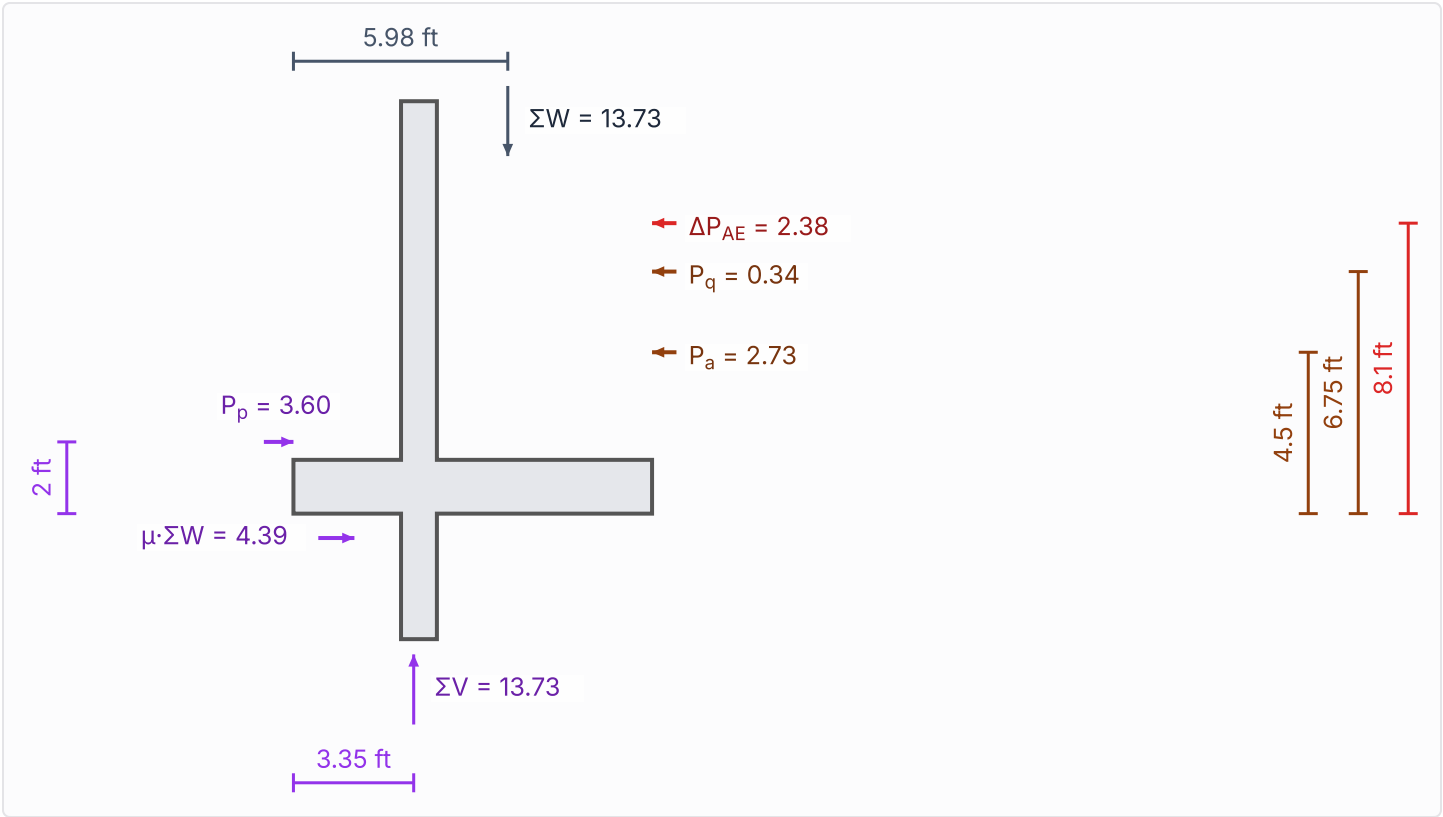
Step 5: Visualization

Figures

Cross-section with lateral earth pressure, passive resistance and bearing distribution at the governing case, the same case as resultants and lever arms, and the reinforcement detail of the wall section

WALL CROSS-SECTION & PRESSURES — $1.0D + 0.7E + H$ (ASD)





15'-0"

10'-0"

1'-6"

3'-6"

#4 @ 9" O.C. HORIZ.
S+T, SINGLE CURTAIN

#6 @ 8" O.C. DOWELS
STD. HOOK, $\ell_{dh} = 1'-0"$
 $12d_b = 9"$ EXT., FREE END TO TOE

#6 @ 12" O.C. BOTT.
TOE MAT
3" CLR. TO EARTH

#6 @ 8" O.C. KEY DOWELS
STD. HOOK, $\ell_{dh} = 1'-0"$

3'-0"

1'-0"

6'-0"

#6 @ 8" O.C. VERT.
STEM BARS, EARTH FACE
2" CLR. TO EARTH FACE

LAP 2'-2"

#6 @ 8" O.C. TOP
HEEL MAT
2" CLR. TO TOP

#5 @ 9" O.C. LONGIT.
FOOTING S+T, E.F.

(8) #5 LONGIT. S+T IN KEY (4x2)
 $A_s = 2.48 \geq 0.91 \text{ IN}^2$

10'-0"

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