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STRUCTURAL ANALYSIS COVER PAGE

Job Title: LEGEND AUTO ADDITION - SHOP

Job Number: Z9-3288

Jurisdiction: CITY OF PUYALLUP

LATERAL ENGINEERING DESIGN PARAMETERS

Wind Design Data

Wind Design Speed, $V_u = 110$ MPH, $V_{asd} = 85$ MPH

Wind Exposure = B

Wind Importance Factor, $I_w = 1.0$

Internal Pressure Coefficient = ± 0.18

$K_{tz} = 1.38$

$K_d = 0.85$

Seismic Design Data

Importance factor = 1.0

$S_s = 1.26g$, $S_1 = 0.48g$

Site Class = D

$SDS = 1.00g$, $SD_1 = 0.59g$

$SDC = D$

Seismic System = 15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel

Design Base Shear = 10.77 kips

$C_s = 0.155$

$R = 6.5$

Analysis procedure: ASCE 11.4, 11.5 & 12.8

GRAVITY ENGINEERING DESIGN PARAMETERS

Snow load = 25 psf

Roof dead load = 15 psf

Live load = 40 psf

Dead load = 12 psf

Allowable bearing pressure = 1500 psf



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November 7, 2023

Page 1 / 64

Contents

1	Lateral Analysis	3
1.1	Wind Design	3
1.2	Seismic Design.....	9
1.3	Diaphragm design shears.....	12
1.4	Compute Rho	13
2	GRAVITY DESIGN	40
2.1	Analysis of Bm 1 - (2) 2 x 6 DF #2	40
2.2	Analysis of Bm 2 - 5.125 x 12.000 GLB 24F-V4.....	42
2.3	Analysis of Bm 3 - 5.125 x 12.000 GLB 24F-V4.....	45
2.4	Analysis of Bm 4 - 3.500 x 14.000 LVL 2.0E.....	48
2.5	Analysis of Bm 5 - 3.500 x 14.000 LVL 2.0E.....	51

1 Lateral Analysis

1.1 Wind Design

ASCE 7-16 Chapter 26 & 27 (Directional Procedure)

Given data

Wind speed 110, Exposure B

Given Roof angle = 1.19 (0.2:12 Pitch)

Building width = 30.0 ft

Building length = 85.0 ft

Total height = 19.6 ft

Height to average roof height = $(19.00 \text{ ft} + [0.62 + 19.00])/2 = 19.31 \text{ ft}$

Bldg height = 19.00 ft

Roof height = 0.62 ft

Velocity pressures, $q_z = 0.00256 K_z K_{zt} K_d K_e V^2 I_w$ Eq 26.10-1

Topography factor, $K_{zt} = 1.38$

Directionality factor, $K_d = 0.85$ (Table 26.6.1)

Ground Elevation Factor, $K_e = 1.00$ (Section 26.9)

Wind pressure, $p = q_h G C_p - q_i (G C_{pi})$

$$q_z = 0.00256(1.38)(0.85)(110.00)^2 1.00 K_z = 36.33 K_z$$

Surface	Height,ft	Load Case 1		Load Case 2	
		Kz	qz (psf)	Kz	qz (psf)
	15.00	0.70	25.43	0.57	20.71
Diaphragm	19.00	0.70	25.43	0.62	22.53
Mean Roof	19.31	0.70	25.43	0.62	22.53
Max Height	19.62	0.70	25.43	0.62	22.53

Gust effect factor $G = 0.85$, assume Rigid Structure (ASCE 7-10 Section 26.9.1)

Internal pressure coefficient ($G C_{pi}$) = +/- 0.18 (ASCE 7-10 Table 26.11-1)

External wall C_p from Figure 27.4-1

Windward wall, $C_p = 0.80$ for all L/B ratios

Side wall, $C_p = -0.70$ for all L/B ratios

Leeward wall pressure coefficient, C_p is a function of the L/B ratio

For load direction 1, $B = 85.0$ ft. and $L = 30.0$ ft.

$L/B = 30.0 / 85.0 = 0.4$, $C_p = -0.50$

For load direction 2, $B = 30.0$ ft. and $L = 85.0$ ft.

$L/B = 85.0 / 30.0 = 2.8$, $C_p = -0.26$

Surface	Wind Direction	L/B	C_p
Windward wall	All	All	0.80
Leeward wall	Direction 1	0.35	-0.50
Leeward wall	Direction 2	2.83	-0.26
Side wall	All	All	-0.70

External roof C_p - Load direction 1, from Figure 27.4-1

For Angle = 1.2 degrees

Windward roof : 0 to $h/2$, 0 to $19.3/2 = 9.7$ ft, $C_p = -1.30$

Windward roof : $h > h/2 = 9.7$ ft, $C_p = -0.70$

The above table reflects C_p values based on h/L $19.3/30.0 = 0.64$

Internal pressure coefficient ($G C_{pi}$) - Load direction 1

$G C_{pi} = +/- 0.18$ acting at 19.3 ft.

Velocity pressure at $q_i = q_h = 22.53$ psf (Load case 2-Occurs at roof mid height)

MWFRS Net pressures - Load direction 1

$$p = q_h G C_p - q_i (G C_{pi})$$

$$p = q_h (0.85)C_p - 22.53(+/- 0.18), \text{ psf}$$

MWFRS pressures: Direction 1

Surface	z ft	q psf	G	C _p	Net pressure psf with (+G _{pi}) (-G _{pi})	
Windward wall	15.0	20.7	0.85	0.80	10.0	18.1
	19.0	22.5	0.85	0.80	11.3	19.4
Leeward wall	All	22.5	0.85	-0.50	-13.6	-5.5
Side wall	All	22.5	0.85	-0.70	-17.5	-9.3
Windward roof	>0-h/2	22.5	0.85	-1.30	-28.9	-20.8
Windward roof	>h/2	22.5	0.85	-0.70	-17.5	-9.3
Leeward roof	N/A					

External roof C_p-Load direction 2 (L = 85.0 ft.), from Figure 6 - 6

For Angle = 0.0 degrees

Surface : Windward roof 0 to h/2, 0 to 19.3/2 = 9.7 ft, C_p = -0.90

Surface : Windward roof h/2 to h, 9.7 to 19.3 ft, C_p = -0.90

Surface : Windward roof h to 2h, 19.3 to 2(19.3) = 38.6 ft, C_p = -0.50

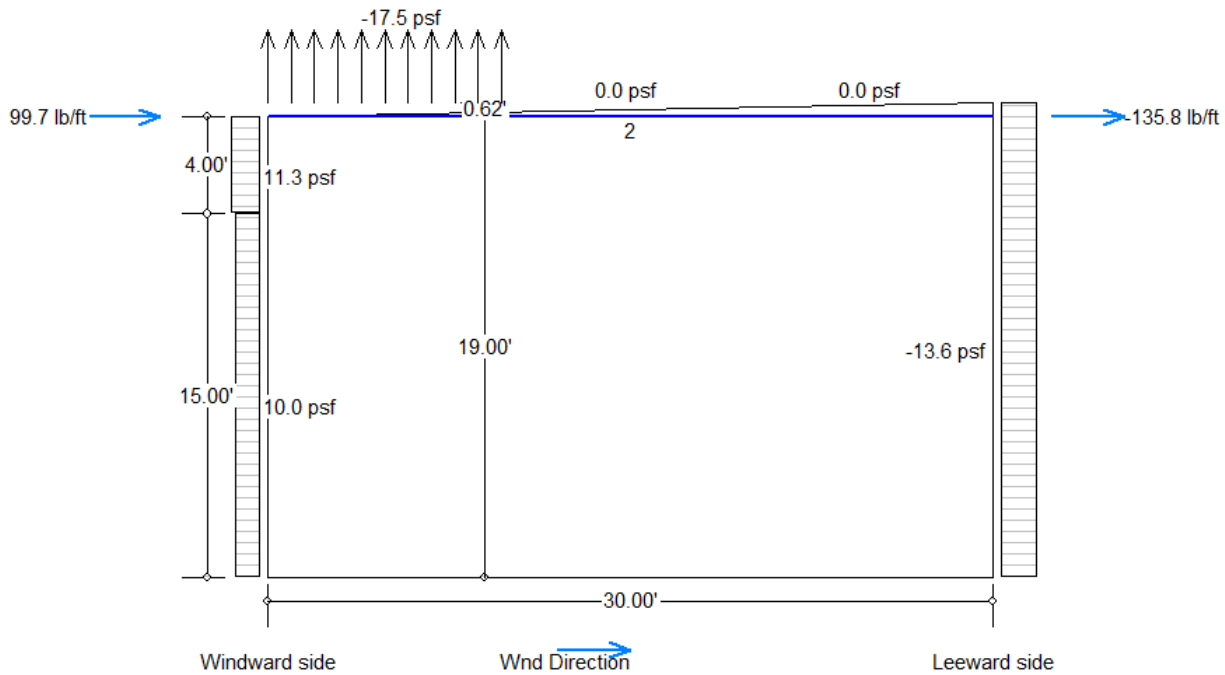
Surface : Windward roof h > 2h = 38.6 ft, C_p = -0.30

The above table reflects C_p values based on h / L of 19.3 / 85.0 = 0.2

MWFRS pressures : Direction 2

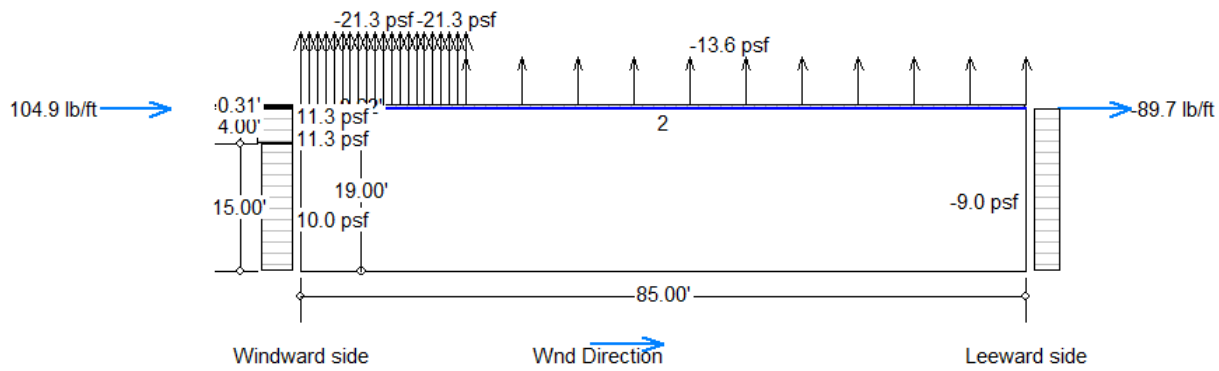
Surface	z ft	q psf	G	C _p	Net pressure psf with (+G _{pi}) (-G _{pi})	
Windward wall	15.0	20.7	0.85	0.80	10.0	18.1
	19.0	22.5	0.85	0.80	11.3	19.4
	19.3	22.5	0.85	0.80	11.3	19.4
	19.6	22.5	0.85	0.80	11.3	19.4
Leeward wall	All	22.5	0.85	-0.26	-9.0	-0.9
side wall	All	22.5	0.85	-0.70	-17.5	-9.3
Windward roof	0-h/2	22.5	0.85	-0.90	-21.3	-13.2
Windward roof	h/2-h	22.5	0.85	-0.90	-21.3	-13.2
Windward roof	h-2h	22.5	0.85	-0.50	-13.6	-5.5

Windward roof	>2h	22.5	0.85	-0.30	-9.8	-1.7	\par \ul Leeward roof N/A \ulnone
Leeward roof	>N/A						



Transverse Direction - with positive internal pressure

Diaphragm	Windward	Leeward	Total
1	99.7 lb/ft	-135.8 lb/ft	235.5 lb/ft



Longitudinal Direction - with positive internal pressure

Diaphragm	Windward	Leeward	Total
1	104.9 lb/ft	-89.7 lb/ft	194.6 lb/ft

1.2 Seismic Design

Maximum considered earthquake spectral response accelerations

Given position:

Lat = 47.205, Long = -122.320

Short period, $S_S = 125.56\%$ of g

1 second period, $S_1 = 48.31\%$ of g

Site class and adjusted maximum spectral accelerations:

Site class = D

For Site Class = D, Site coefficient, $F_a = 1.30$ Par 11.4.4

Site coefficient, $F_v = 1.82$ Table 11.4-2 - Interpolated

The adjusted maximum spectral response per §11.4.3

$$S_{MS} = F_a S_S = 1.30(1.26) = 1.51g \text{ Eq 11.4-1}$$

$$S_{M1} = F_v S_1 = 1.82(0.48) = 0.88g \text{ Eq 11.4-2}$$

Design spectral accelerations parameters:

$$S_{DS} = 2/3 S_{MS} = 2/3(1.507g) = 1.004g \text{ Eq 11.4-3}$$

$$S_{D1} = 2/3 S_{M1} = 2/3(0.878) = 0.585g \text{ Eq 11.4-4}$$

Building Risk Category and importance factors:

Category = II (per Table 1.5-1)

Category = I (as defined per Table 1.5-1)

Importance factor, $I_e = 1.00$

Seismic Design Category (SDC)

Table 11.6-1, Pg 85

For $S_{DS} = 100.45g$, SDC = D

Table 11.6-2, Pg 85

For $S_{D1} = 58.52g$, SDC = D

SDC D controls.

-

Building system

<15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or steel>

 $R = 6.5$ (Table 12.2-1) $\Omega_0 = 3.0$ (2.5 for flexible diaphragm - Note 9) $C_d = 4.0$ **Building element weights**

Level 1, Roof weight = 20.0 psf

Exterior wall weight = 12.0 psf

Interior partition wall weight = 10.0 psf

Building weights lumped on roof and floor diaphragms

Total levels = 1

At Roof Level

 $W_{\text{Roof}} = \text{Roof weight} \times \text{Area} + 1/2 \times \text{Partition weight} \times \text{Area} + 1/2 \times \text{Ext Wall weight} \times \text{Perim} \times \text{Height}$ $W_{\text{Roof}} = 20.0 \text{ psf} \times 2456 \text{ sq.ft.} + 1/2 \times 10.0 \text{ psf} \times 2456 \text{ sq.ft.} + 1/2 \times 12.0 \text{ psf} \times 73 \text{ ft} \times 19.0 \text{ ft} = 69722 \text{ lb}$ **Total weight = 69722 = 69722 lbs****Compute structure period**

Structure type: All other structures

 $C_T = 0.020$ (Table 12.8-2)Structure height, $h_n = 19.0 \text{ ft.}$ $T_a = C_T (h_n)^{3/4} = 0.020(19.0)^{3/4} = 0.182 \text{ sec. (Eq 12.8-7)}$ **Compute base shear**

The design value of C_s is the smaller value of

$$C_s = I_e S_{DS} / R = 1.00(1.00)/6.50 = 0.1545 \text{ EQ 12.8-2}$$

and

$$C_s = I_e S_{D1} / (R T_a) = 1.00(0.59)/[(6.50)(0.18)] = 0.4946 \text{ EQ 12.8-3}$$

but not less

$$C_s = 0.01 \text{ EQ 12.8-4}$$

Therefore $C_s = 0.1545$

Design base shear, $V = C_s W = 0.1545(69722) = 10775 \text{ lbs (10.8 kips) Eq 12.8-1}$

Vertical distribution of force

$$F_x = C_{vx} V \quad \text{Eq 12.8-11}$$

$$\text{where } C_{vx} = W_x h_x^k / (\sum W_i h_i^k) \quad \text{Eq 12.8-12}$$

Compute distribution component, k

$k = 1.0$ for $T_a \leq 0.5$ seconds, and $k = 2$ for $T_a \geq 2.5$.

$k = 1.00$ for $T_a = 0.182 \text{ sec}$

Level x	h_x	h_x^k	w_x	$w_x \times h_x^k$	C_{vx}	$F_x = C_{vx} V$	$F_x/w_x = S_a$
1	19.0	19.0	69.7	1325	1.000	10.8	0.155
SUM			69.7	1325		10.8	

Compute Diaphragm shears per ASCE 7-16 Par 12.10.1.1

$$F_p = \sum F_i / \sum w_i \times w_{px}$$

$$\text{Min } F_{px} = 0.20 S_{DS} I_e w_{px}$$

$$\text{Max } F_{px} = 0.40 S_{DS} I_e w_{px}$$

Level	w_{px}	F_i	F_{px}	Min F_{px}	Max F_{px}	Design F_{px}
1	69.7 k	10.8 k	10.8 k	14.0 k	28.0 k	14.0 k

1.3 Diaphragm design shears

The diaphragm design shears are calculated based on a unit width of diaphragm length including interior walls per the calculation:

Load between grid lines (lb/ft) = 1 ft diaphragm width x diaphragm length x (diaphragm weight + interior partition weight) + exterior wall weight x ave height above and below the diaphragm.

Analysis Direction 1

Current Level 1

Shear Forces Table

DIAPHRAGM	WIDTH	WIND LOAD	SEISMIC LOAD
SPAN	ft	lb/ft	lb/ft
A-B	44.5	235.5	149.3
B-C	40.0	235.5	149.3

Direct Shear Forces Table

DIAPHRAGM	GRID	WIND	SEISMIC	GRID	WIND	SEISMIC
SPAN	LINE	lb	lb	LINE	lb	lb
A-B	A	5240	3322	B	9946	6305
B-C	B	9946	6305	C	4706	2983

Analysis Direction 2

Current Level 1

Shear Forces Table

DIAPHRAGM	WIDTH	WIND LOAD	SEISMIC LOAD
-----------	-------	-----------	--------------

SPAN	ft	lb/ft	lb/ft
1-2	29.5	194.6	189.7

Direct Shear Forces Table

DIAPHRAGM	GRID	WIND	SEISMIC	GRID	WIND	SEISMIC
SPAN	LINE	lb	lb	LINE	lb	lb
1-2	1	2872	2800	2	2872	2800

1.4 Compute Rho

Redundancy calculation rho, per ASCE 12.3.4.2 - Summary

Level = 1

Direction	Condition		Rho
	A	B	
1	PASS	PASS	1.0
2	PASS	PASS	1.0

Design rho for Direction 1 = 1.0

Design rho for Direction 2 = 1.0

Analysis

Redundancy calculations

*** D E S I G N L E V E L = 1 ***

*** Direction 1 ***

Check condition A

Grid Line B, Height = 19.00 ft

#	Length	Height/Length
1	29.98'	0.63

Grid Line C, Height = 19.00 ft

#	Length	Height/Length
1	29.98'	0.63

Total shear wall length = 60.0 ft

Check shear wall piers that have $h/L > 1.0$. Remove that pier and check the length of removed pier ratio to total shear wall length is less than 0.33.

Removed

Grid/Pier	Length	Length/Total Length
-----------	--------	---------------------

Condition A, PASSED

Check condition B

Grid Line	Length	Height	2L/H
-----------	--------	--------	------

B	29.98'	19.00'	3.16
---	--------	--------	------

C	29.98'	19.00'	3.16
---	--------	--------	------

Sum			6.31
-----	--	--	------

There are 6.31 bays > 4 req'd, therefore OK

Condition B, PASSED

*** Direction 2 ***

Check condition A

Grid Line 1, Height = 19.00 ft

#	Length	Height/Length
---	--------	---------------

1	40.43'	0.47
---	--------	------

Grid Line 2, Height = 19.00 ft

#	Length	Height/Length
---	--------	---------------

1	6.00'	3.17
---	-------	------

Total shear wall length = 46.4 ft

Check shear wall piers that have $h/L > 1.0$. Remove that pier and check the length of removed pier ratio to total shear wall length is less than 0.33.

Removed

Grid/Pier	Length	Length/Total Length
-----------	--------	---------------------

2	6.00'	0.13 --> OK
---	-------	-------------

Condition A, PASSED

Check condition B

Grid Line	Length	Height	2L/H
-----------	--------	--------	------

1	40.43'	19.00'	4.26
---	--------	--------	------

2	6.00'	19.00'	0.63
---	-------	--------	------

Sum			4.89
-----	--	--	------

There are 4.89 bays > 4 req'd, therefore OK

Condition B, PASSED

Shear Wall at Grid 1



Design Rho = 1.0

Table 1 - Shears

Level	Sum B	H	Max Aspect	E	Ew	E+Ew	W	vE	vW	Max	MARK
	ft	ft	Ratio	lb	lb	lb	lb	plf	plf	plf	
1	40.4	19.0	0.5	2800	1425	4225	2872	73	30	73	SW-1

Notes

1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels) = rho x Qe.

4. E_w - Unfactored Wall inertia force (wall & window panels) includes ρ .
5. $E + E_w$ = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. $vE = 0.7 \times vE$ (ASD factored shear).
8. $wW = 0.6 \times vW / 1.4$.
9. * = Shear values includes effects of vertical shears due hold-down reactions
from upper levels (if applicable).

Table 2a - Vertical loads on panels

Level	Panel#/ Type	Length ft	x1 ft	x2 ft	Dead lb/ft	Snow lb/ft	Live lb/ft	Wind Uplift lb/ft
1	0/SW	40.43	0.00	40.43	228.0*	-	-	-

Notes:

1. A panel is considered an element within a braced wall line.
such as shear wall, window, filler (non-shear load), drag element.
2. length = indivisual panel length (within a braced wall line).
3. x1 = the start dimension for the distributive load - measured from LHS end of panel.
4. x2 = the end dimension for the distributive load - measured from LHS end of panel.
5. Multiple distributive loads may be supported by a panel.
6. Multiple distributive loads shown are not sorted - along the span of the panel.
7. * = Wall Dead load (wall dead load does not apply to drag elements and window panels).
Wall dead loads are summed up with framing dead loads where applicable
(which includes beam drag elements and window hdrs). See Table 2b below.
8. OPEN = Window/Door, DRAG = Drag strut, NO-SW = filler panel (no shear capacity)

SW = Shear panel.

Table 2b - Unfactored Reaction forces at panels

		DIRECTION 1				DIRECTION 2			
Reaction Location		D	S	L	W	E	W	E	W
from end					Uplift				
(ft)		lb	lb	lb	lb	lb	lb	lb	lb

1-0	0.00	4609	0	0	0	-1985	-1350	1985	1350
1-1	40.43	4609	0	0	0	1985	1350	-1985	-1350

Notes:

- Reaction X-Y, X = level, Y = panel sequence id
- D = DEAD LOAD, L = LIVE LOAD, W-UPLIFT = WIND UPLIFT LOAD
W = WIND LOAD, E = SEISMIC LOAD
- D = (Panel Height x Panel Width x Panel weight = 12.0 psf) / 2
Dead load vectors are summed at abutting panels
- DIRECTION 1 = LOAD DIRECTION LEFT TO RIGHT
- DIRECTION 2 = LOAD DIRECTION RIGHT TO LEFT
- NEGATIVE VALUES = UPLIFT OR TENSION

Table 3 - Factored Reaction forces at panels

		DIRECTION 1						DIRECTION 2						MIN
Reaction Location														
MAX														
from end		LC1	LC2	LC3	LC4	LC5	LC6	LC1	LC2	LC3	LC4	LC5	LC6	LOAD
LOAD														
(ft)		lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb
1b														

1-0	0.0		3799	3220	4002	3567	1956	750		5419	5999	5217	5651	3575	3529		750
5999																	
1-1	40.4		5419	5999	5217	5651	3575	3529		3799	3220	4002	3567	1956	750		750
5999																	

Notes

1. LC = Load combination
2. LC1 = D + 0.6W ASCE 2.4.1 - 5a
3. LC2 = D + 0.7E ASCE 2.4.1 - 5b
4. LC3 = D + 0.75L + 0.75(0.6W) + 0.75S ASCE 2.4.1 - 6a
5. LC4 = D + 0.75L + 0.75(0.7E) + 0.75S ASCE 2.4.1 - 6b
6. LC5 = 0.6D + 0.6W ASCE 2.4.1 - 7
7. LC6 = (0.6 - 0.14SDS)D + 0.7E ASCE 2.4.1 - 8, SDS = 0.970
8. MIN LOAD = Maximum negative tension force
9. MAX LOAD = Maximum positive compression force
10. W = W uplift + W shear overturning

Table 4 - Tie down schedule

Reaction Location	MIN	MAX	HOLD-DOWN	
from end	LOAD	LOAD	MARK	
(ft)	lb	lb		
1-0	0.0	750	4002	TD0
1-1	40.4	750	4002	TD0

Notes

1. N/R = Not required - compression controls.
2. NONE = Uplift exceeded specified hold-down.
3. Due to the applied dead loads, some hold-downs may differ within
a shear panel. The highest capacity hold-down will be used at
both ends.

Table 5 - Drag forces (Unfactored loads)

Level = 1

	q	v	dq
LOAD	lb/ft	lb/ft	lb/ft
WIND	71.84	71.04	0.80
SEISMIC	105.66	104.48	1.18

PANEL ID	TYPE	PANEL END#1		PANEL END #2	
		WIND	SEISMIC	WIND	SEISMIC
		LB	LB	LB	LB
1	SHEAR WALL	0	0	32	48

Notes:

q = Diaphragm shear.

v = Shear wall shear.

$dq = q - v$ (this level) + v (upper level)

Table 6 - Drag forces (Factored loads)

Level = 1

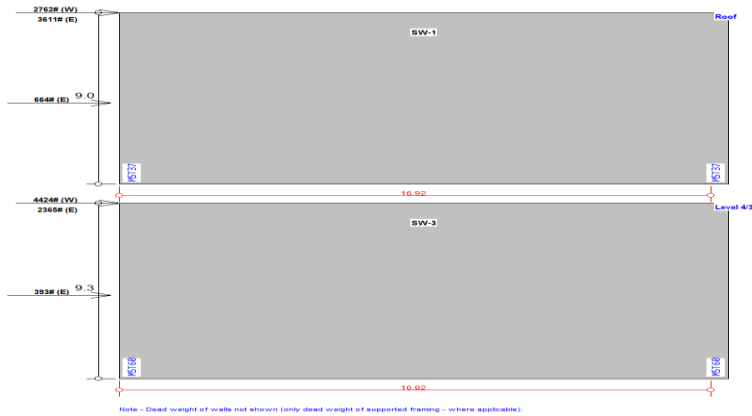
PANEL ID	TYPE	PANEL END #1		PANEL END #2	
		WIND	SEISMIC	WIND	SEISMIC
		LB	LB	LB	LB
1	SHEAR WALL	0	0	19	83

Notes

1. Wind load, $W = 0.6 \times \text{Load}$

2. Seismic load, $E = 0.7 \times 1.25 \times \text{Load}$. Apply requirements of ASCE 7-10 (SEC 12.3.3.4)

Shear Wall at Grid 2



Design Rho = 1.0

Table 1 - Shears

Level	Sum B	H	Max Aspect	E	Ew	E+Ew	W	vE	vW	Max	MARK
	ft	ft	Ratio	lb	lb	lb	lb	plf	plf	plf	
1	6.0	19.0	3.2**	2800	211	3011	2872	351	205	351	SW-3

Shear panel(s) in the braced wall line exceed aspect ratio as defined per SDPWS 4.3.4.

Reduction per SDPWS 4.3.4.2 is required. The capacity of the shear wall is reduced by

WSP = 1.25 - 0.125(h/bs) Aspect Ratio Factor. It is more convenient to increase

PROJECT NAME: LEGEND AUTO ADDITION – TO SHOP
JOB: Z9-3288

the demand load by the factor $1 / WSP$ and size the SW accordingly. Where $WSP > 1.0$.

Level	Max Aspect Ratio	WSP	1/WSP	Design Shear	Adjusted Shear	Revised SW MARK
1	3.17	0.85	1.17	351	411	SW-4

Notes

1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels) = $\rho \times Q_e$.
4. E_w - Unfactored Wall inertia force (wall & window panels) includes ρ .
5. $E + E_w$ = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. $vE = 0.7 \times vE(\text{ASD factored shear})$.
8. $wW = 0.6 \times vW / 1.4$.
9. * = Shear values includes effects of vertical shears due hold-down reactions
from upper levels (if applicable).

Table 2a - Vertical loads on panels

Level	Panel#/ Type	Length ft	x1 ft	x2 ft	Dead lb/ft	Snow lb/ft	Live lb/ft	Wind Uplift lb/ft
1	0/DRAG	34.44	0.00	34.44	0.0*	-	-	-
1	1/SW	6.00	0.00	6.00	228.0*	-	-	-

November 7, 2023

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Page 22 / 64

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Notes:

1. A panel is considered an element within a braced wall line.
such as shear wall, window, filler (non-shear load), drag element.
2. length = individual panel length (within a braced wall line).
3. x1 = the start dimension for the distributive load - measured from LHS end of panel.
4. x2 = the end dimension for the distributive load - measured from LHS end of panel.
5. Multiple distributive loads may be supported by a panel.
6. Multiple distributive loads shown are not sorted - along the span of the panel.
7. * = Wall Dead load (wall dead load does not apply to drag elements and window panels).
Wall dead loads are summed up with framing dead loads where applicable
(which includes beam drag elements and window hdrs). See Table 2b below.
8. OPEN = Window/Door, DRAG = Drag strut, NO-SW = filler panel (no shear capacity)
SW = Shear panel.

Table 2b - Unfactored Reaction forces at panels

				DIRECTION 1				DIRECTION 2			
Reaction Location	D	S	L	W	E	W		E	W		
from end				Uplift							
(ft)	lb	lb	lb	lb	lb	lb		lb	lb		

1-0 0.00		0	0	0	0	0		0	0		
1-1 34.44		684	0	0	0	-9534	-9094	9534	9094		
1-2 40.44		684	0	0	0	9534	9094	-9534	-9094		

Notes:

1. Reaction X-Y, X = level, Y = panel sequence id
2. D = DEAD LOAD, L = LIVE LOAD, W-UPLIFT = WIND UPLIFT LOAD

W = WIND LOAD, E = SEISMIC LOAD

3. $D = (\text{Panel Height} \times \text{Panel Width} \times \text{Panel weight} = 12.0 \text{ psf}) / 2$

Dead load vectors are summed at abutting panels

4. DIRECTION 1 = LOAD DIRECTION LEFT TO RIGHT

5. DIRECTION 2 = LOAD DIRECTION RIGHT TO LEFT

6. NEGATIVE VALUES = UPLIFT OR TENSION

Table 3 - Factored Reaction forces at panels

Reaction Location		DIRECTION 1						DIRECTION 2						MIN	
MAX															
	from end	LC1	LC2	LC3	LC4	LC5	LC6		LC1	LC2	LC3	LC4	LC5	LC6	LOAD
LOAD															
	(ft)	1b	1b	1b	1b	1b	1b		1b	1b	1b	1b	1b	1b	1b
1b															

1-0 0	0.0	0	0	0	0	0	0		0	0	0	0	0	0	0
1-1 7358	34.4	-4772	-5989	-3408	-4321	-5046	-6356		6140	7358	4776	5689	5867	6991	-6356
1-2 7358	40.4	6140	7358	4776	5689	5867	6991		-4772	-5989	-3408	-4321	-5046	-6356	-6356

Notes

1. LC = Load combination
2. LC1 = $D + 0.6W$ ASCE 2.4.1 - 5a
3. LC2 = $D + 0.7E$ ASCE 2.4.1 - 5b
4. LC3 = $D + 0.75L + 0.75(0.6W) + 0.75S$ ASCE 2.4.1 - 6a
5. LC4 = $D + 0.75L + 0.75(0.7E) + 0.75S$ ASCE 2.4.1 - 6b
6. LC5 = $0.6D + 0.6W$ ASCE 2.4.1 - 7

7. $LC6 = (0.6 - 0.14SDS)D + 0.7E$ ASCE 2.4.1 - 8, $SDS = 0.970$

8. MIN LOAD = Maximum negative tension force

9. MAX LOAD = Maximum positive compression force

10. $W = W$ uplift + W shear overturning

Table 4 - Tie down schedule

Reaction Location	MIN	MAX	HOLD-DOWN
from end	LOAD	LOAD	MARK
(ft)	lb	lb	

1-0	0.0	0	TD1
1-1	34.4	-6356 5689	TD4
1-2	40.4	-6356 5689	TD4

Notes

1. N/R = Not required - compression controls.
2. NONE = Uplift exceeded specified hold-down.
3. Due to the applied dead loads, some hold-downs may differ within a shear panel. The highest capacity hold-down will be used at both ends.

Table 5 - Drag forces (Unfactored loads)

Level = 1

	q	v	dq

LOAD	lb/ft	lb/ft	lb/ft

WIND	71.84	478.61	-406.77
SEISMIC	75.32	501.78	-426.46

		PANEL END#1		PANEL END #2	
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC

		LB	LB	LB	LB

1	DRAG-STRUT	0	0	2474	2594
2	SHEAR WALL	2474	2594	33	34

Notes:

q = Diaphragm shear.

v = Shear wall shear.

dq = q - v (this level) + v (upper level)

Table 6 - Drag forces (Factored loads)

Level = 1

November 7, 2023

CUSTOM DESIGN & ENGINEERING, INC.

Page 26 / 64

		PANEL END #1		PANEL END #2		
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC	

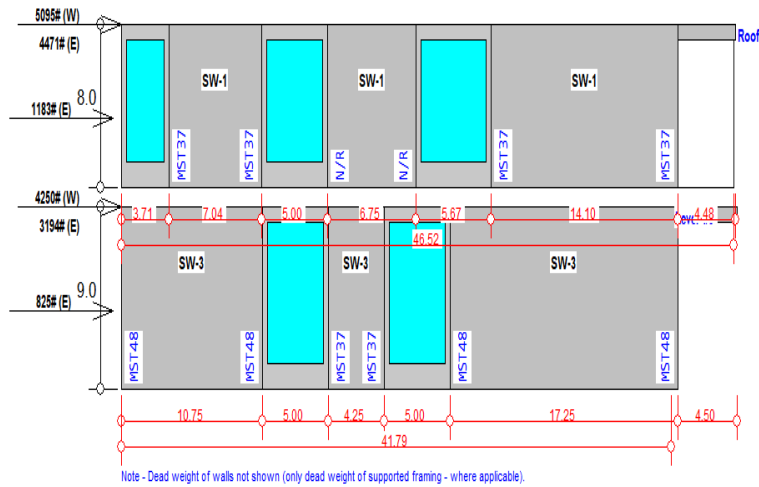
		LB	LB	LB	LB	

1	DRAG-STRUT	0	0	1484	4539	MST48
2	SHEAR WALL	1484	4539	20	60	MST48

Notes

1. Wind load, $W = 0.6 \times \text{Load}$
2. Seismic load, $E = 0.7 \times 1.25 \times \text{Load}$. Apply requirements of ASCE 7-10 (SEC 12.3.3.4)

Shear Wall at Grid B



Design Rho = 1.0

Table 1 - Shears

Level	Sum B	H	Max Aspect	E	Ew	E+Ew	W	vE	vW	Max	MARK
	ft	ft	Ratio	lb	lb	lb	lb	plf	plf	plf	
1	30.0	19.0	0.6	6305	1056	7361	9946	172	142	172	SW-1

Notes

1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels) = $\rho \times Q_e$.
4. E_w - Unfactored Wall inertia force (wall & window panels) includes ρ .
5. $E + E_w$ = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. $v_E = 0.7 \times v_E$ (ASD factored shear).
8. $w_W = 0.6 \times v_W / 1.4$.
9. * = Shear values includes effects of vertical shears due hold-down reactions
from upper levels (if applicable).

Table 2a - Vertical loads on panels

Level	Panel#/ Type	Length ft	x1 ft	x2 ft	Dead lb/ft	Snow lb/ft	Live lb/ft	Wind Uplift lb/ft
1	0/SW	29.98	0.00	29.98	228.0*	-	-	-

Notes:

1. A panel is considered an element within a braced wall line.
such as shear wall, window, filler (non-shear load), drag element.
2. length = indivisual panel length (within a braced wall line).
3. x_1 = the start dimension for the distributive load - measured from LHS end of panel.
4. x_2 = the end dimension for the distributive load - measured from LHS end of panel.
5. Multiple distributive loads may be supported by a panel.
6. Multiple distributive loads shown are not sorted - along the span of the panel.
7. * = Wall Dead load (wall dead load does not apply to drag elements and window panels).

Wall dead loads are summed up with framing dead loads where applicable

(which includes beam drag elements and window hdrs). See Table 2b below.

8. OPEN = Window/Door, DRAG = Drag strut, NO-SW = filler panel (no shear capacity)

SW = Shear panel.

Table 2b - Unfactored Reaction forces at panels

		DIRECTION 1				DIRECTION 2			
Reaction Location		D	S	L	W	E	W	E	W
from end					Uplift				
(ft)		lb	lb	lb	lb	lb	lb	lb	lb

1-0	0.00	3417	0	0	0	-4666	-6304	4666	6304
1-1	29.98	3417	0	0	0	4666	6304	-4666	-6304

Notes:

1. Reaction X-Y, X = level, Y = panel sequence id
2. D = DEAD LOAD, L = LIVE LOAD, W-UPLIFT = WIND UPLIFT LOAD
W = WIND LOAD, E = SEISMIC LOAD
3. $D = (\text{Panel Height} \times \text{Panel Width} \times \text{Panel weight} = 12.0 \text{ psf}) / 2$
Dead load vectors are summed at abutting panels
4. DIRECTION 1 = LOAD DIRECTION LEFT TO RIGHT
5. DIRECTION 2 = LOAD DIRECTION RIGHT TO LEFT
6. NEGATIVE VALUES = UPLIFT OR TENSION

Table 3 - Factored Reaction forces at panels

PROJECT NAME: LEGEND AUTO ADDITION – TO SHOP
JOB: Z9-3288

Reaction Location	DIRECTION 1								DIRECTION 2								MIN
MAX																	
	from end	LC1	LC2	LC3	LC4	LC5	LC6		LC1	LC2	LC3	LC4	LC5	LC6		LOAD	
LOAD																	
	(ft)	lb	lb	lb	lb	lb	lb		lb	lb	lb	lb	lb	lb		lb	
lb																	

1-0	0.0		-365	152	581	968	-1732	-1679		7200	6683	6254	5867	5833	4852		-1732
7200																	
1-1	30.0		7200	6683	6254	5867	5833	4852		-365	152	581	968	-1732	-1679		-1732
7200																	

Notes

1. LC = Load combination
2. LC1 = D + 0.6W ASCE 2.4.1 - 5a
3. LC2 = D + 0.7E ASCE 2.4.1 - 5b
4. LC3 = D + 0.75L + 0.75(0.6W) + 0.75S ASCE 2.4.1 - 6a
5. LC4 = D + 0.75L + 0.75(0.7E) + 0.75S ASCE 2.4.1 - 6b
6. LC5 = 0.6D + 0.6W ASCE 2.4.1 - 7
7. LC6 = (0.6 - 0.14SDS)D + 0.7E ASCE 2.4.1 - 8, SDS = 0.970
8. MIN LOAD = Maximum negative tension force
9. MAX LOAD = Maximum positive compression force
10. W = W uplift + W shear overturning

Table 4 - Tie down schedule

Reaction Location	MIN	MAX		HOLD-DOWN	
from end	LOAD	LOAD		MARK	
(ft)	lb	lb			

1-0	0.0		-1732	5833		TD1	
1-1	30.0		-1732	5833		TD1	

Notes

1. N/R = Not required - compression controls.
2. NONE = Uplift exceeded specified hold-down.
3. Due to the applied dead loads, some hold-downs may differ within a shear panel. The highest capacity hold-down will be used at both ends.

Table 5 - Drag forces (Unfactored loads)

Level = 1

	q	v	dq
LOAD	lb/ft	lb/ft	lb/ft
WIND	336.94	331.79	5.15
SEISMIC	249.37	245.55	3.81

		PANEL END#1		PANEL END #2	
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC

CUSTOM DESIGN & ENGINEERING, INC.

November 7, 2023

Page 32 / 64

		LB	LB	LB	LB

1	SHEAR WALL	0	0	154	114

Notes:

q = Diaphragm shear.

v = Shear wall shear.

dq = q - v (this level) + v (upper level)

Table 6 - Drag forces (Factored loads)

Level = 1					
		PANEL END #1		PANEL END #2	
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC

		LB	LB	LB	LB

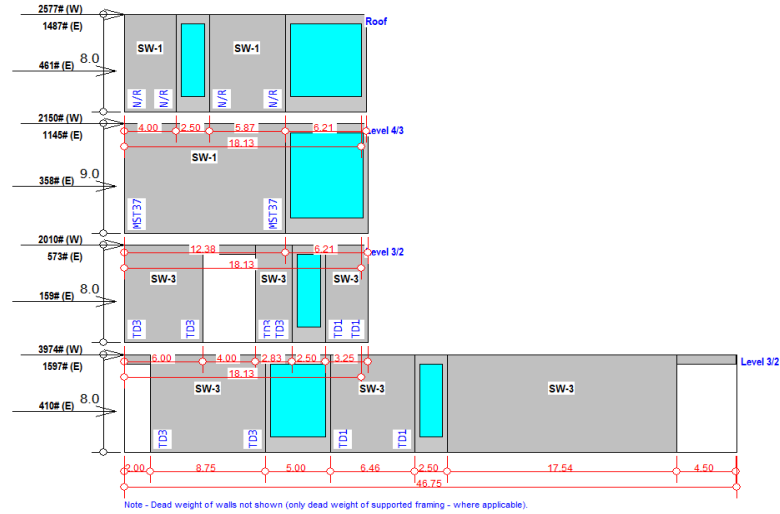
1	SHEAR WALL	0	0	93	200

Notes

1. Wind load, $W = 0.6 \times \text{Load}$

2. Seismic load, $E = 0.7 \times 1.25 \times \text{Load}$. Apply requirements of ASCE 7-10 (SEC 12.3.3.4)

Shear Wall at Grid C



Design Rho = 1.0

Table 1 - Shears

Level	Sum B	H	Max Aspect	E	Ew	E+Ew	W	vE	vW	Max	MARK
	ft	ft	Ratio	lb	lb	lb	lb	plf	plf	plf	
1	30.0	19.0	0.6	2983	1056	4040	4706	94	67	94	SW-1

Notes

1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels) = $\rho \times Q_e$.
4. E_w - Unfactored Wall inertia force (wall & window panels) includes ρ .
5. $E + E_w$ = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. $v_E = 0.7 \times v_E(\text{ASD factored shear})$.
8. $w_W = 0.6 \times v_W / 1.4$.
9. * = Shear values includes effects of vertical shears due hold-down reactions
from upper levels (if applicable).

Table 2a - Vertical loads on panels

Level	Panel#/ Type	Length ft	x1 ft	x2 ft	Dead lb/ft	Snow lb/ft	Live lb/ft	Wind Uplift lb/ft
1	0/SW	29.98	0.00	29.98	228.0*	-	-	-

Notes:

1. A panel is considered an element within a braced wall line.
such as shear wall, window, filler (non-shear load), drag element.
2. length = indivisual panel length (within a braced wall line).
3. x_1 = the start dimension for the distributive load - measured from LHS end of panel.
4. x_2 = the end dimension for the distributive load - measured from LHS end of panel.
5. Multiple distributive loads may be supported by a panel.
6. Multiple distributive loads shown are not sorted - along the span of the panel.
7. * = Wall Dead load (wall dead load does not apply to drag elements and window panels).

Wall dead loads are summed up with framing dead loads where applicable

(which includes beam drag elements and window hdrs). See Table 2b below.

8. OPEN = Window/Door, DRAG = Drag strut, NO-SW = filler panel (no shear capacity)

SW = Shear panel.

Table 2b - Unfactored Reaction forces at panels

		DIRECTION 1				DIRECTION 2			
Reaction Location		D	S	L	W	E	W	E	W
from end					Uplift				
(ft)		lb	lb	lb	lb	lb	lb	lb	lb

1-0	0.00	3417	0	0	0	-2560	-2983	2560	2983
1-1	29.98	3417	0	0	0	2560	2983	-2560	-2983

Notes:

1. Reaction X-Y, X = level, Y = panel sequence id
2. D = DEAD LOAD, L = LIVE LOAD, W-UPLIFT = WIND UPLIFT LOAD
W = WIND LOAD, E = SEISMIC LOAD
3. $D = (\text{Panel Height} \times \text{Panel Width} \times \text{Panel weight} = 12.0 \text{ psf}) / 2$
Dead load vectors are summed at abutting panels
4. DIRECTION 1 = LOAD DIRECTION LEFT TO RIGHT
5. DIRECTION 2 = LOAD DIRECTION RIGHT TO LEFT
6. NEGATIVE VALUES = UPLIFT OR TENSION

Table 3 - Factored Reaction forces at panels

PROJECT NAME: LEGEND AUTO ADDITION – TO SHOP
JOB: Z9-3288

Reaction Location		DIRECTION 1							DIRECTION 2							MIN	
MAX																	
	from end	LC1	LC2	LC3	LC4	LC5	LC6		LC1	LC2	LC3	LC4	LC5	LC6		LOAD	
LOAD																	
	(ft)	lb	lb	lb	lb	lb	lb		lb	lb	lb	lb	lb	lb		lb	
1b																	

1-0	0.0		1628	1625	2075	2073	261	-206		5207	5210	4760	4762	3840	3379		-206
5210																	
1-1	30.0		5207	5210	4760	4762	3840	3379		1628	1625	2075	2073	261	-206		-206
5210																	

Notes

1. LC = Load combination
2. LC1 = D + 0.6W ASCE 2.4.1 - 5a
3. LC2 = D + 0.7E ASCE 2.4.1 - 5b
4. LC3 = D + 0.75L + 0.75(0.6W) + 0.75S ASCE 2.4.1 - 6a
5. LC4 = D + 0.75L + 0.75(0.7E) + 0.75S ASCE 2.4.1 - 6b
6. LC5 = 0.6D + 0.6W ASCE 2.4.1 - 7
7. LC6 = (0.6 - 0.14SDS)D + 0.7E ASCE 2.4.1 - 8, SDS = 0.970
8. MIN LOAD = Maximum negative tension force
9. MAX LOAD = Maximum positive compression force
10. W = W uplift + W shear overturning

Table 4 - Tie down schedule

Reaction Location		MIN	MAX		HOLD-DOWN	
	from end	LOAD	LOAD		MARK	
	(ft)	lb	lb			

1-0	0.0		-206	3840		TD0	
1-1	30.0		-206	3840		TD0	

Notes

1. N/R = Not required - compression controls.
2. NONE = Uplift exceeded specified hold-down.
3. Due to the applied dead loads, some hold-downs may differ within a shear panel. The highest capacity hold-down will be used at both ends.

Table 5 - Drag forces (Unfactored loads)

Level = 1

	q	v	dq
LOAD	lb/ft	lb/ft	lb/ft
WIND	159.43	156.99	2.44
SEISMIC	136.85	134.75	2.09

		PANEL END#1		PANEL END #2	
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC

CUSTOM DESIGN & ENGINEERING, INC.

November 7, 2023

Page 38 / 64

		LB	LB	LB	LB
1	SHEAR WALL	0	0	73	63

Notes:

q = Diaphragm shear.

v = Shear wall shear.

dq = q - v (this level) + v (upper level)

Table 6 - Drag forces (Factored loads)

Level = 1

		PANEL END #1		PANEL END #2	
PANEL ID	TYPE	WIND	SEISMIC	WIND	SEISMIC
		LB	LB	LB	LB
1	SHEAR WALL	0	0	44	110

Notes

1. Wind load, $W = 0.6 \times \text{Load}$

2. Seismic load, $E = 0.7 \times 1.25 \times \text{Load}$. Apply requirements of ASCE 7-10 (SEC 12.3.3.4)

2 GRAVITY DESIGN

Beam Framing Analysis

2.1 Analysis of Bm 1 - (2) 2 x 6 DF #2

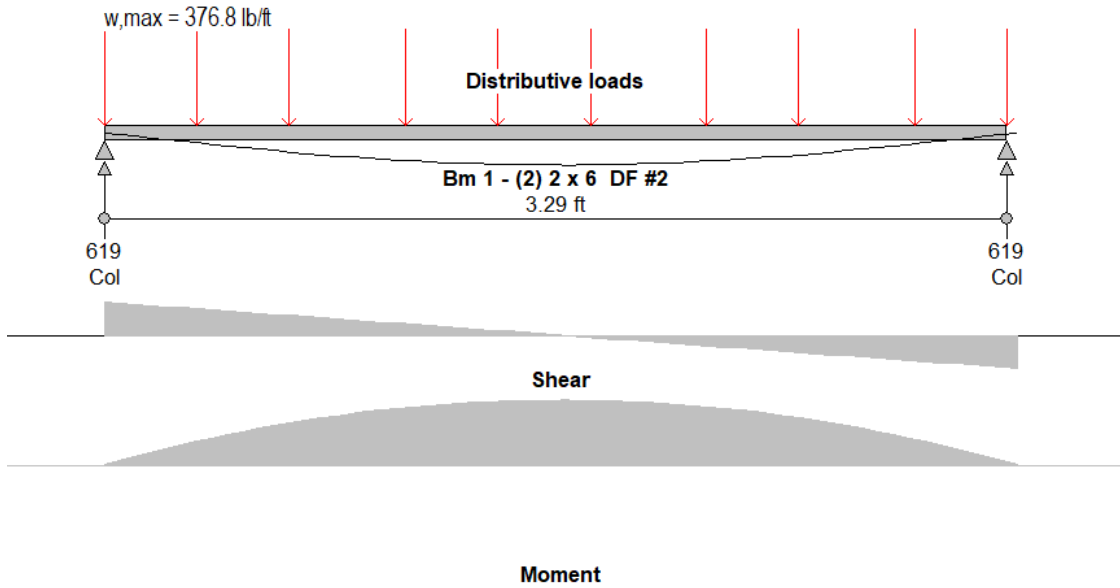


Table 1 - Point load table

LOAD	D	S	L	W+/-	E+/-	LOC	NOTES
------	---	---	---	------	------	-----	-------

No Applied point loads

(1) Un-factored loads in lbs.

(2) Load location measured from left end of beam.

Table 3 - Distributive load table (pressures)

LOAD	ELEMENT	AREA	WALL	D	S	L
		ID	HEIGHT			

0	Floor/Roof	0	-	20.0	25.0	0.0
---	------------	---	---	------	------	-----

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB	from	to	D	S	L
		WIDTH	loc	loc			

0	Floor/Roof	29.5	3.5	0.0	295.2	369.0	0.0
---	------------	------	-----	-----	-------	-------	-----

(1) From loc and to loc are load segments starting and ending

measured from the left of the beam

->Computed moments and shears (Factored) :

Max shear = 1149 lbs D + S (2.4-3)

Min shear = -1149 lbs D + S (2.4-3)

Max moment = 994 ft-lbs D + S (2.4-3)

Min moment = 0 ft-lbs D + S (2.4-3)

->Beam properties (2D xy axis) :

Span = 3.46 ft

Area = 16.50 sq.in

$$S_x = 15.12 \text{ sq.in}$$

$$I_{xx} = 41.59 \text{ sq.in}$$

->Check shear :

$$f_v = 1.5 \times V / \text{Area} = 1149 / 16.50 = 104.46 \text{ psi}$$

$$F'_v = 180.00 \times 1.15 \times 1.00 \times 1.00 \times 1.00 = 207.00 \text{ psi}$$

$$F_v = 180 \text{ psi}, C_D = 1.15, C_m = 1.00, C_t = 1.00, C_i = 1.00.$$

->Check bending :

$$f_{b\text{-top}} = M \times 12 / S_x = 11924 / 15.12 = 788.36 \text{ psi}$$

$$f_{b\text{-btm}} = M \times 12 / S_x = 0 / 15.12 = 0.00 \text{ psi}$$

$$F_b = 900 \text{ psi}, C_D = 1.15, C_m = 1.00, C_t = 1.00, C_l = 1.00,$$

$$C_f = 1.30, C_{fu} = 1.00, C_i = 1.00, C_r = 1.00.$$

$$F_b' \times C_D \times C_M \times C_T \times C_L \times C_F \times C_{FU} \times C_I \times C_R = 1346 \text{ psi}$$

->Check bearing :

->Check deflections :

$$\text{Number of deflection spans} = 1$$

$$\text{Deflection span 0, Length} = 3.46 \text{ ft Combined deflection} = -0.032 [D + S (2.4-3)]$$

$$\text{Allowed} = 3.46 \times 12 / 360.0 = 0.115 \text{ in.}$$

$$\text{Allowed (Seismic controled)} = 3.46 \times 12 / 180.0 = 0.231 \text{ in.}$$

2.2 Analysis of Bm 2 - 5.125 x 12.000 GLB 24F-V4

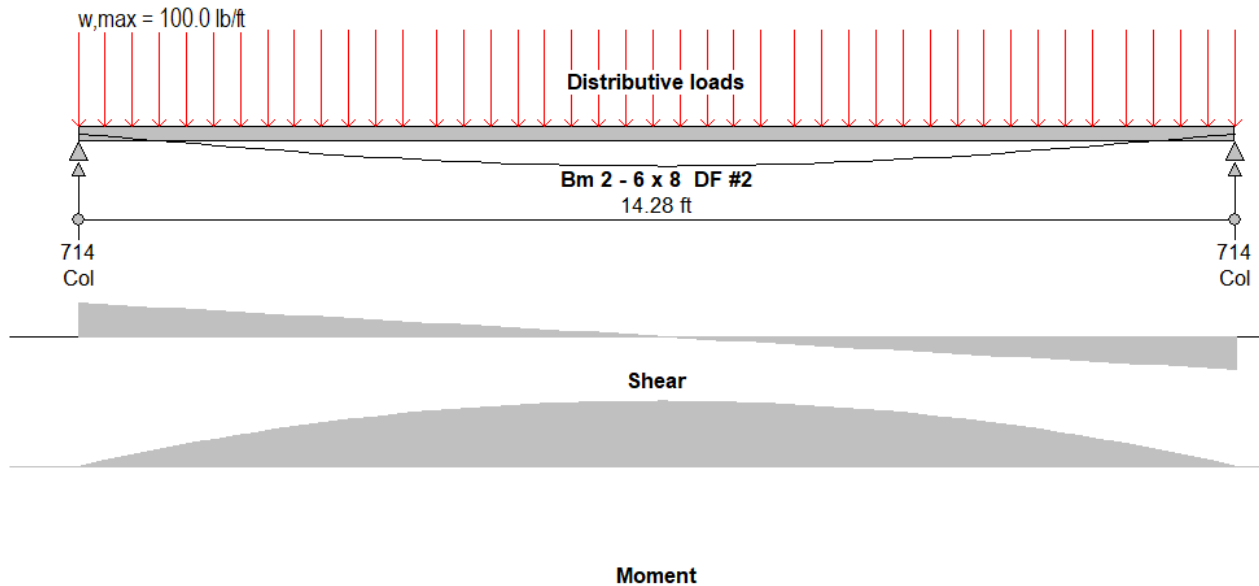


Table 1 - Point load table

LOAD	D	S	L	W+/-	E+/-	LOC	NOTES
------	---	---	---	------	------	-----	-------

No Applied point loads

(1) Un-factored loads in lbs.

(2) Load location measured from left end of beam.

Table 3 - Distributive load table (pressures)

LOAD	ELEMENT	AREA	WALL	D	S	L
------	---------	------	------	---	---	---

ID HEIGHT

0 Floor/Roof 0 - 20.0 25.0 0.0

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB	from	to	D	S	L
		WIDTH	loc	loc			

0 Floor/Roof 29.5 | 13.4 0.0 | 295.2 369.0 0.0

(1) From loc and to loc are load segments starting and ending

measured from the left of the beam

->Computed moments and shears (Factored) :

Max shear = 4465 lbs D + S (2.4-3)

Min shear = -4465 lbs D + S (2.4-3)

Max moment = 15007 ft-lbs D + S (2.4-3)

Min moment = -0 ft-lbs D + S (2.4-3)

->Beam properties (2D xy axis) :

Span = 13.45 ft

Area = 61.50 sq.in

Sx = 123.00 sq.in

Ixx = 738.00 sq.in

->Check shear :

$$f_v = 1.5 * V / \text{Area} = 4465 / 61.50 = 108.91 \text{ psi}$$

$$F'_v = 190.00 \times 1.15 \times 1.00 \times 1.00 \times 1.00 = 218.50 \text{ psi}$$

$$F_v = 190 \text{ psi}, CD = 1.15, C_m = 1.00, C_t = 1.00, C_i = 1.00.$$

->Check moment :

$$fb\text{-top} = M \times 12 / S_x = -0 / 123.00 = 0.00 \text{ psi}$$

$$fb\text{-btm} = M \times 12 / S_x = 180083 / 123.00 = 1464.09 \text{ psi}$$

$$F_b = 2400 \text{ psi}, CD = 1.15, C_m = 1.00, C_t = 1.00, C_l = 1.00,$$

$$C_v = 0.82, C_{fu} = 1.00, C_i = 1.00, C_r = 1.00.$$

Cv controls

$$F_b'_{top} \times CD \times C_M \times C_T \times C_V \times C_{FU} \times C_I \times C_R = 1125 \text{ psi}$$

$$F_b'_{btm} \times CD \times C_M \times C_T \times C_V \times C_{FU} \times C_I \times C_R = 2251 \text{ psi}$$

->Check bearing :

->Check deflections :

$$\text{Number of deflection spans} = 1$$

$$\text{Deflection span 0, Length} = 13.45 \text{ ft Combined deflection} = -0.368 [D + S (2.4-3)]$$

$$\text{Allowed} = 13.45 \times 12 / 360.0 = 0.448 \text{ in.}$$

$$\text{Allowed (Seismic controled)} = 13.45 \times 12 / 180.0 = 0.896 \text{ in.}$$

2.3 Analysis of Bm 3 - 5.125 x 12.000 GLB 24F-V4

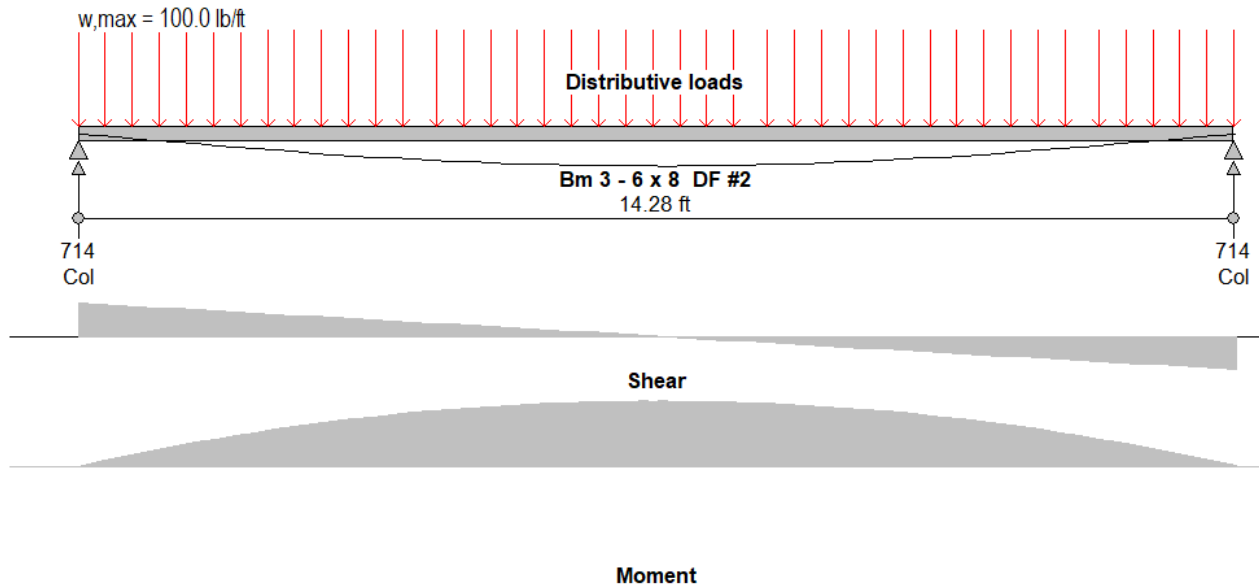


Table 1 - Point load table

LOAD	D	S	L	W+/-	E+/-	LOC	NOTES
------	---	---	---	------	------	-----	-------

No Applied point loads

(1) Un-factored loads in lbs.

(2) Load location measured from left end of beam.

Table 3 - Distributive load table (pressures)

LOAD	ELEMENT	AREA	WALL	D	S	L
------	---------	------	------	---	---	---

ID HEIGHT

0 Floor/Roof 0 - 20.0 25.0 0.0

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB	from	to	D	S	L
		WIDTH	loc	loc			

0 Floor/Roof 29.5 | 13.4 -0.0 | 295.2 369.0 0.0

(1) From loc and to loc are load segments starting and ending

measured from the left of the beam

->Computed moments and shears (Factored) :

Max shear = 4465 lbs D + S (2.4-3)

Min shear = -4465 lbs D + S (2.4-3)

Max moment = 15004 ft-lbs D + S (2.4-3)

Min moment = -0 ft-lbs D + S (2.4-3)

->Beam properties (2D xy axis) :

Span = 13.45 ft

Area = 61.50 sq.in

Sx = 123.00 sq.in

Ixx = 738.00 sq.in

->Check shear :

$$f_v = 1.5 * V / \text{Area} = 4465 / 61.50 = 108.90 \text{ psi}$$

$$F'_v = 190.00 \times 1.15 \times 1.00 \times 1.00 \times 1.00 = 218.50 \text{ psi}$$

$$F_v = 190 \text{ psi}, CD = 1.15, C_m = 1.00, C_t = 1.00, C_i = 1.00.$$

->Check moment :

$$fb\text{-top} = M \times 12 / S_x = -0 / 123.00 = 0.00 \text{ psi}$$

$$fb\text{-btm} = M \times 12 / S_x = 180047 / 123.00 = 1463.80 \text{ psi}$$

$$F_b = 2400 \text{ psi}, CD = 1.15, C_m = 1.00, C_t = 1.00, C_l = 1.00,$$

$$C_v = 0.82, C_{fu} = 1.00, C_i = 1.00, C_r = 1.00.$$

Cv controls

$$F_b\text{'top} \times CD \times C_M \times C_T \times C_V \times C_{FU} \times C_I \times C_R = 1125 \text{ psi}$$

$$F_b\text{'btm} \times CD \times C_M \times C_T \times C_V \times C_{FU} \times C_I \times C_R = 2251 \text{ psi}$$

->Check bearing :

->Check deflections :

$$\text{Number of deflection spans} = 1$$

$$\text{Deflection span 0, Length} = 13.45 \text{ ft Combined deflection} = -0.367 [D + S (2.4-3)]$$

$$\text{Allowed} = 13.45 \times 12 / 360.0 = 0.448 \text{ in.}$$

$$\text{Allowed (Seismic controled)} = 13.45 \times 12 / 180.0 = 0.896 \text{ in.}$$

2.4 Analysis of Bm 4 - 3.500 x 14.000 LVL 2.0E

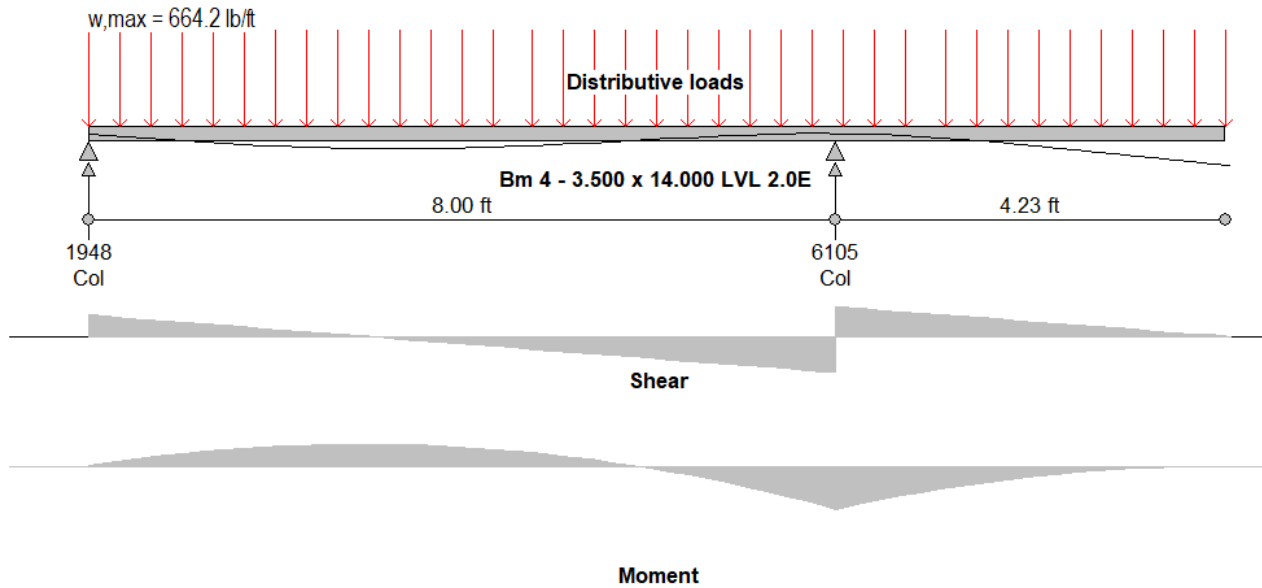


Table 1 - Point load table

LOAD	D	S	L	W+/-	E+/-	LOC	NOTES
------	---	---	---	------	------	-----	-------

No Applied point loads

(1) Un-factored loads in lbs.

(2) Load location measured from left end of beam.

Table 3 - Distributive load table (pressures)

LOAD	ELEMENT	AREA	WALL	D	S	L
------	---------	------	------	---	---	---

ID HEIGHT

```

-----
0  Floor/Roof      0      -      20.0    25.0    0.0
1  Floor/Roof      0      -      20.0    25.0    0.0
-----

```

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB	from	to	D	S	L
		WIDTH	loc	loc			
0	Floor/Roof	29.5	-0.0	12.0	295.2	369.0	0.0
1	Floor/Roof	29.5	12.0	12.2	295.2	369.0	0.0

(1) From loc and to loc are load segments starting and ending
measured from the left of the beam

->Computed moments and shears (Factored) :

```

Max shear =      2740 lbs          D + S (2.4-3)
Min shear  =     -3283 lbs          D + S (2.4-3)
Max moment =      2857 ft-lbs       D + S (2.4-3)
Min moment =     -5668 ft-lbs       D + S (2.4-3)

```

->Beam properties (2D xy axis) :

```

Span  = 12.23 ft
Area  = 49.00 sq.in
Sx    = 114.33 sq.in

```

$$I_{xx} = 800.33 \text{ sq.in}$$

->Check shear :

$$f_v = 1.5 \times V / \text{Area} = 3283 / 49.00 = 100.50 \text{ psi}$$

$$F'_v = 285 \times 1.15 = 327.75 \text{ psi}$$

$$F_v = 285 \text{ psi, CD} = 1.00$$

->Check moment :

$$f_b = M \times 12 / S_x = 68012 / 114.33 = 594.86 \text{ psi}$$

$$F_b = 2600 \text{ psi, CD} = 1.15, C_f = 0.98, C_l = 0.98.$$

$$F_b' \times CD \times C_f \times C_l = 2885 \text{ psi}$$

->Check bearing :

->Check deflections :

$$\text{Number of deflection spans} = 2$$

$$\text{Deflection span 0, Length} = 8.00 \text{ ft Combined deflection} = -0.015 [D + S (2.4-3)]$$

$$\text{Allowed} = 8.00 \times 12 / 360.0 = 0.267 \text{ in.}$$

$$\text{Allowed (Seismic controled)} = 8.00 \times 12 / 180.0 = 0.533 \text{ in.}$$

$$(2.4-3)] \text{ Cantilever Deflection span 1, Length} = 4.23 \text{ ft Combined deflection} = -0.031 [D + S$$

$$\text{Allowed} = 4.23 \times 12 / 240.0 = 0.211 \text{ in.}$$

2.5 Analysis of Bm 5 - 3.500 x 14.000 LVL 2.0E

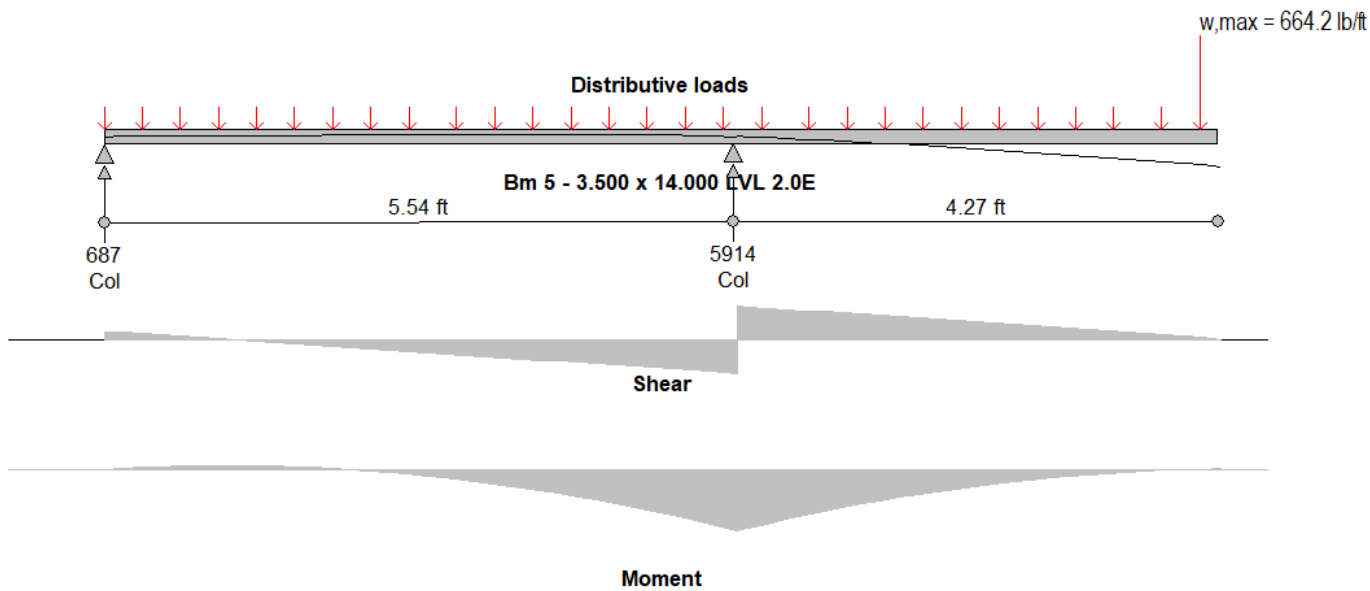


Table 1 - Point load table

LOAD	D	S	L	W+/-	E+/-	LOC	NOTES
------	---	---	---	------	------	-----	-------

No Applied point loads

(1) Un-factored loads in lbs.

(2) Load location measured from left end of beam.

Table 3 - Distributive load table (pressures)

LOAD	ELEMENT	AREA	WALL	D	S	L
------	---------	------	------	---	---	---

ID HEIGHT

0	Floor/Roof	0	-	20.0	25.0	0.0
1	Floor/Roof	0	-	20.0	25.0	0.0

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB	from	to	D	S	L
		WIDTH	loc	loc			
0	Floor/Roof	29.5	9.6	9.8	295.2	369.0	0.0
1	Floor/Roof	29.5	9.8	0.0	295.2	369.0	0.0

(1) From loc and to loc are load segments starting and ending
measured from the left of the beam

->Computed moments and shears (Factored) :

Max shear	=	2919 lbs	D + S (2.4-3)
Min shear	=	-2833 lbs	D + S (2.4-3)
Max moment	=	355 ft-lbs	D + S (2.4-3)
Min moment	=	-6396 ft-lbs	D + S (2.4-3)

->Beam properties (2D xy axis) :

Span	=	9.77 ft
Area	=	49.00 sq.in
Sx	=	114.33 sq.in

$$I_{xx} = 800.33 \text{ sq.in}$$

->Check shear :

$$f_v = 1.5 \times V / \text{Area} = 2919 / 49.00 = 89.36 \text{ psi}$$

$$F'_v = 285 \times 1.15 = 327.75 \text{ psi}$$

$$F_v = 285 \text{ psi, CD} = 1.00$$

->Check moment :

$$f_b = M \times 12 / S_x = 76751 / 114.33 = 671.29 \text{ psi}$$

$$F_b = 2600 \text{ psi, CD} = 1.15, C_f = 0.98, C_l = 0.98.$$

$$F_b' \times CD \times C_f \times C_l = 2885 \text{ psi}$$

->Check bearing :

->Check deflections :

$$\text{Number of deflection spans} = 2$$

$$\text{Deflection span 0, Length} = 5.54 \text{ ft Combined deflection} = 0.005 [D + S (2.4-3)]$$

$$\text{Allowed} = 5.54 \times 12 / 360.0 = 0.185 \text{ in.}$$

$$\text{Allowed (Seismic controled)} = 5.54 \times 12 / 180.0 = 0.370 \text{ in.}$$

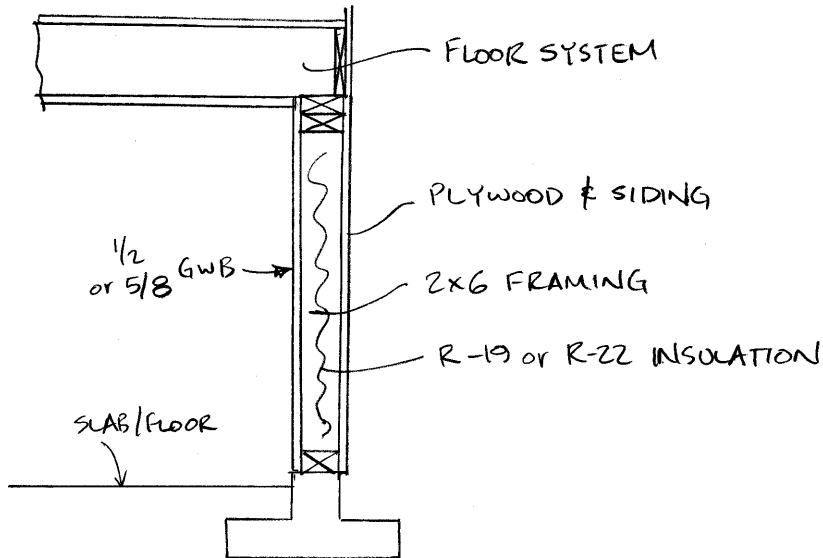
$$(2.4-3)] \text{ Cantilever Deflection span 1, Length} = 4.23 \text{ ft Combined deflection} = -0.064 [D + S$$

$$\text{Allowed} = 4.23 \times 12 / 240.0 = 0.211 \text{ in.}$$

APPENDIX A – TYPICAL DEAD WEIGHTS OF LIGHT WEIGHT STRUCTURES

Typical dead weights of light wood framed structures

Typical exterior wall



Weight of wall:

1 x 6 Cedar siding	1.5 psf x .75 thick	= 1.13 psf
15# paper		= 1.00 psf
1/2 CDX plywood	3.0 psf /inch	= 1.5 psf
R-22 Insulation		= 1.0 psf
2 x 6 Studs (16" o.c. - 2.0 lb/ft)		= 1.50 psf
Sub Total		= 6.13 psf
With 5/8" GWB (5 psf/inch)		= 3.10 psf
Total with 5/8 GWB		= 9.23 psf (Use 10 psf)
With 1/2" GWB		= 2.5 psf
Total with 1/2 GWB		= 8.63 psf (Use 10 psf)

With 2-1/12 Face brick (25 psf) & 1/2 GWB = **32.5 psf**

Typical light floor system with 2 x 10 floor joists 16" O.C.

3/4 T & G Sub floor (3.0 psf/inch)	= 2.25 psf
2 x 10 Joists (16" o.c - 3.37 lb/ft)	= 2.53 psf

Carpeting	= 2.0 psf
5/8 GWB	= 3.1 psf
or ½ GWB	= 2.0 psf

Total with 5/8 GWB = 10 psf (Use 12 psf min)

Total with ½ GWB = 9.23psf (Use 10 psf min)

Typical heavy floor system with TJI floor joists 16" O.C. & 1-1/2" of Gyp-crete 2000.

¾ T & G Sub floor (3.0	
psf/inch)	2.3
TJI 150 PRO 2.3 lb/ft @ 16 o.c.	1.7
Carpeting	2.0
5/8 GWB	3.1
1.5" Gyp-crete @ 115 pcf	14.4
Mech ceiling load	3.0
Total	26.5 psf

Typical interior 2 × 4 wall with 16" o.c. studs

½ GWB (both sides)	= 5.0 psf
2 × 4 studs (16" o.c. - 1.28 lb/ft)	= 0.96 psf
Insulation (optional)	= 1.0 psf
Total	= 7.00 psf

Truss roof system - with light roofing ≤ 6 psf

Max truss weight	= 3.5 psf
½ GWB	= 2.5 psf
R-38 insulation	= 1.5 psf
Roof covering (includes paper)	= 6.0 psf
½ CDX Sheathing	= 1.50 psf
Total	= 15 psf

Stick Frame roof system - with light roofing ≤ 6 psf

½ GWB	= 2.5 psf
R-38 Insulation	= 1.5 psf
Roof covering (includes paper)	= 6.0 psf
½ CDX Sheathing	= 1.5 psf

Subtotal = 11.5 psf

with

2 × 8 - 2' o.c. (1.32 psf) = 12.82 psf

2 × 8 - 16" o.c.	(1.92 psf)	= 13.42 psf
2 × 10 - 2' o.c.	(1.69 psf)	= 13.19 psf
2 × 10 - 16" o.c.	(2.53 psf)	= 14.03 psf

APPENDIX B – WOOD SHEAR WALL DESIGN

The shear wall design consists of a static analysis by applying lateral and gravity loads. The following describes the tables per a typical analysis.

SHEAR PANEL ASPECT RATIO CHECK**SHEAR PANEL VERTICAL SHEAR
COMBINED WITH THE HORIZONTAL
SHEAR****Analysis of Grid Line A**

Table 1 - Shears

Level	Sum B ft	H ft	Max Aspect Ratio	E lb	Ew lb	E+Ew lb	W lb	vE plf	vW plf	Max plf	MARK
3	28.8	8.0	2.6**	4348	1475	5823	1607	141	24	141	SW-1
2	28.8	9.0	3.0**	7724	2600	10324	4697	415*	98*	415	SW-4
1	12.8	8.0	1.9	9323	2885	12207	7633	681	257	681	SW-6 (DBL)

Notes

1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels).
4. Ew - Unfactored Wall inertia force (wall & window panels).
5. E + Ew = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. vE = $0.7 \times vE(\text{ASD factored shear})$.
8. vW = $0.6 \times vW / 1.4$.
9. * = Shear values includes effects of vertical shears due hold-down reactions from upper levels (if applicable).
10. ** = Design shear has been factored up by the maximum $h/2w$ (Section 2305.3.4) for SW with $h / w > 2.0$. H = to the story height, however some shear panels may have reduced heights by utilizing a continuous header beam per plan.

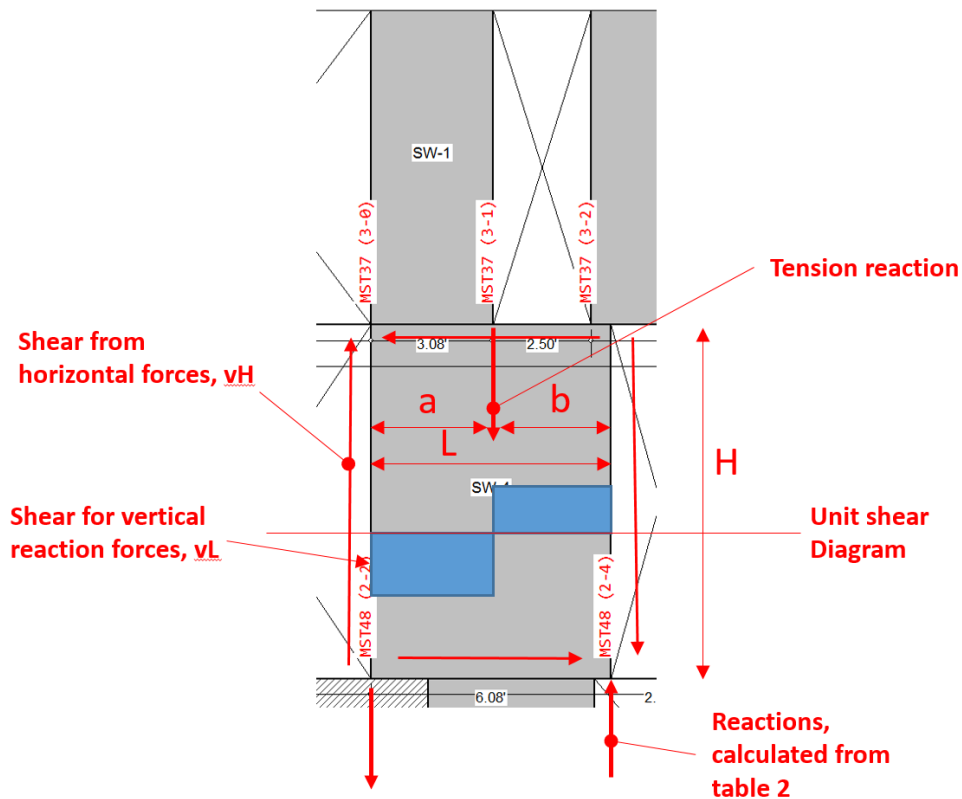
Table 1

This table shows the total shear at a shear wall. The lateral forces are summed from top to bottom for the seismic and wind lateral forces. The lengths of the shear panel per level is the "Sum B".

Note 10 - Aspect ratio check – Per NDS Table 4.3.4, the shear panel unit shears must be reduced by the $2b_s / h$ if the h/b_s ratio exceeds 2:1. The Max aspect ratio check on Table 1, increases the applied load by a factor

of $h/2w$, $w = b_s$. The table above denotes when the requirement is exceeded, by the ** footnote and note 10. If all the panels meet the aspect ratio, this note will be shown on the table.

Note 9 – Combined horizontal and vertical unit shears – In order to reduce the number of hold-downs and the uplift can be resisted by the shear wall panel, then the vertical shear from the tension force(s) acting in beam action will be added to the horizontal shear. The applied forces are shown below



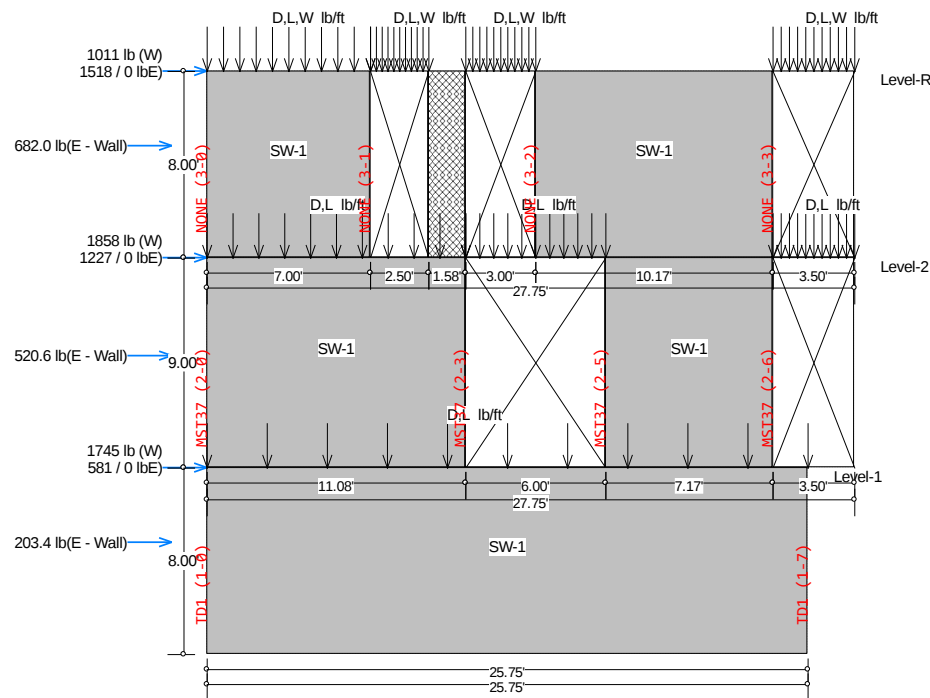
Free body diagram of a typical shear panel

To compute v_L ,

$$v_L(\text{LHS}) = (\text{Tension reaction} \times b / L) / H$$

$$\text{Total shear} = \text{Max LHS or RHS reactions} + v_H.$$

Table 2a & 2b



Typical shear panel with all the forces acting

Table 2a - Vertical loads on panels

Level	Panel	Length ft	Dead lb/ft	Snow lb/ft	Live lb/ft	Wind Uplift lb/ft
3	1	7.00	147	0	392	-245
3	2	2.50	147	0	392	-245
3	3	1.58	0	0	0	0
3	4	3.00	121	0	323	-202
3	5	10.17	0	0	0	0
3	6	3.50	147	0	392	-245
2	1	11.08	147	0	392	0
2	2	6.00	121	0	323	0
2	3	7.17	0	0	0	0
2	4	3.50	147	0	392	0
1	1	25.75	78	0	209	0

The dead load, D is calculated from weight of the shear panel, wall with windows and walls not considered shear panels, and the weight on the framing (if any) that is supported by the three elements noted above. Snow and live loads are shown, but they do not control to the hold-down design (uplift). The snow load and the live load play a significant role when designing support beams for discontinuous elements. This analysis is performed in the beam design stage, where the shear wall overturning forces are combined with the vertical forces on the beam. This will be shown later. The wind uplift is additive to the hold-down reaction forces.

Typical dead weight component acting where the hold-down occurs is:

$D = \text{weight of wall} \times \text{width} \times \text{height} / 2$, where the weight of the wall = 12 psf for example. The dead weights are cumulative from top to bottom.

Snow, Live and Wind uplift load = load, lb/ft x L / 2

Table 2b - Unfactored Reaction forces at panels

Reaction	Location from end (ft)					DIRECTION 1		DIRECTION 2	
		D	S	L	W	E	W	E	W
		lb	lb	lb	Uplift lb	lb	lb	lb	lb
3-0	0.0	850	0	1371	-857	-1025	-471	1025	471
3-1	7.0	1154	0	1860	-1163	1025	471	-1025	-471
3-2	14.1	813	0	484	-302	-1025	-471	1025	471
3-3	24.3	913	0	685	-428	1025	471	-1025	-471
3-4	9.5	380	0	490	-306	0	0	0	0
3-5	11.1	401	0	484	-302	0	0	0	0
3-6	27.8	425	0	685	-428	0	0	0	0
2-0	0.0	2262	0	3541	-857	-2594	-1712	2594	1712
2-1	7.0	1154	0	1860	-1163	0	0	0	0
2-2	9.5	380	0	979	-612	0	0	0	0
2-3	11.1	3309	0	4105	-605	2081	1476	-2081	-1476
2-4	14.1	0	0	484	-302	0	0	0	0
2-5	17.1	1481	0	968	0	-2459	-1650	2459	1650
2-6	24.3	2659	0	2056	-857	2972	1885	-2972	-1885
2-7	27.8	871	0	1371	-428	0	0	0	0
1-0	0.0	4508	0	6234	-857	-3533	-2750	3533	2750
1-1	7.0	1154	0	1860	-1163	0	0	0	0
1-2	9.5	380	0	1469	-918	0	0	0	0
1-3	11.1	3309	0	4105	-605	0	0	0	0
1-4	14.1	0	0	484	-302	0	0	0	0
1-5	17.1	1481	0	968	0	0	0	0	0
1-6	24.3	2659	0	2056	-857	0	0	0	0
1-7	25.8	2246	0	2693	0	3533	2750	-3533	-2750

Notes

1. Reaction X-Y, X = level, Y = panel sequence id
2. D = DEAD LOAD, L = LIVE LOAD, W-UPLIFT = WIND UPLIFT LOAD
W = WIND LOAD, E = SEISMIC LOAD
3. $D = (\text{Panel Height} \times \text{Panel Width} \times \text{Panel weight} = 12.0 \text{ psf} + \text{Roof or Dead load}) / 2$
Dead load vectors are summed at abutting panels
4. DIRECTION 1 = LOAD DIRECTION LEFT TO RIGHT
5. DIRECTION 2 = LOAD DIRECTION RIGHT TO LEFT
6. NEGATIVE VALUES = UPLIFT OR TENSION

The table above shows the point loads applied at each panel (solid, open, etc).

The Reaction location 3-0, denotes 3rd level, 1st panel located 0 ft from the left,

The Reaction location 2-4, denotes 2nd level, 5th panel located 14.1 ft from the left.

Overturning forces for wind and seismic are calculated for both directions (alternating directions). All these forces must add to zero, since they are pure rotation reactions. The wind uplift is combined with the wind overturning as per Table 3 below.

Table 3 combination of loads

Table 3 - Factored Reaction forces at panels

Reaction Location	from end (ft)	DIRECTION 1						DIRECTION 2						MIN LOAD	MAX LOAD
		LC1	LC2	LC3	LC4	LC5	LC6	LC1	LC2	LC3	LC4	LC5	LC6		
		lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb
3-0	0.0	567	132	1281	1340	227	-323	1133	1568	1705	2416	793	1112	-323	1281
3-1	7.0	1436	1871	2238	3087	975	1253	871	436	1814	2011	410	-182	-182	1814
3-2	14.1	531	96	828	638	205	-340	1096	1531	1252	1714	771	1095	-340	1095
3-3	24.3	1196	1631	1446	1965	830	1141	630	195	1022	889	265	-294	-294	1141
3-4	9.5	380	380	609	747	228	176	380	380	609	747	228	176	176	380
3-5	11.1	401	401	628	764	241	186	401	401	628	764	241	186	186	401
3-6	27.8	425	425	746	939	255	197	425	425	746	939	255	197	197	425
2-0	0.0	1235	447	3763	3557	330	-766	3290	4078	5303	6280	2385	2866	-766	3557
2-1	7.0	1154	1154	2026	2549	692	536	1154	1154	2026	2549	692	536	536	1154
2-2	9.5	380	380	839	1114	228	176	380	380	976	1114	228	176	176	380
2-3	11.1	4195	4766	6780	7481	2871	2993	2423	1852	5451	5295	1099	79	79	4766
2-4	14.1	0	0	227	363	0	0	0	0	227	363	0	0	0	0
2-5	17.1	491	-241	1464	915	-102	-1034	2471	3202	2949	3497	1878	2409	-1034	2409
2-6	24.3	3790	4739	4664	5761	2727	3315	1528	579	2967	2641	464	-846	-846	3315
2-7	27.8	871	871	1706	1899	523	404	871	871	1706	1899	523	404	404	871
1-0	0.0	2858	2035	7561	7329	1055	-380	6158	6982	10036	11039	4355	4566	-380	6982
1-1	7.0	1154	1154	2026	2549	692	536	1154	1154	2026	2549	692	536	536	1154
1-2	9.5	380	380	1068	1481	228	176	380	380	1343	1481	228	176	176	380
1-3	11.1	3309	3309	6116	6388	1985	1536	3309	3309	6116	6388	1985	1536	1536	3309
1-4	14.1	0	0	227	363	0	0	0	0	227	363	0	0	0	0
1-5	17.1	1481	1481	2206	2206	888	687	1481	1481	2206	2206	888	687	687	1481
1-6	24.3	2659	2659	3816	4201	1595	1234	2659	2659	3816	4201	1595	1234	1234	2659
1-7	25.8	3896	4719	5503	6121	2997	3516	596	-227	3028	2411	-302	-1431	-1431	3516

Notes

1. LC = Load combination
2. LC1 = D + 0.6W ASCE 2.4.1 - 5a
3. LC2 = D + 0.7E ASCE 2.4.1 - 5b
4. LC3 = D + 0.75L + 0.75(0.6W) + 0.75S ASCE 2.4.1 - 6a
5. LC4 = D + 0.75L + 0.75(0.7E) + 0.75S ASCE 2.4.1 - 6b
6. LC5 = 0.6D + 0.6W ASCE 2.4.1 - 7
7. LC6 = (0.6 - 0.14SDS)D + 0.7E ASCE 2.4.1 - 8, SDS = 0.970
8. MIN LOAD = Maximum negative tension force
9. MAX LOAD = Maximum positive compression force
10. W = W uplift + W shear overturning

All the forces shown on Table 2b are combined here. Load cases 6 & 7 size the hold-downs, since they produce the highest negative numbers. Any uplift force < 500 lb is considered small and therefore no hold-down is assigned to it.

The next page shows the application of the overturning forces on beams supporting discontinuous shear walls

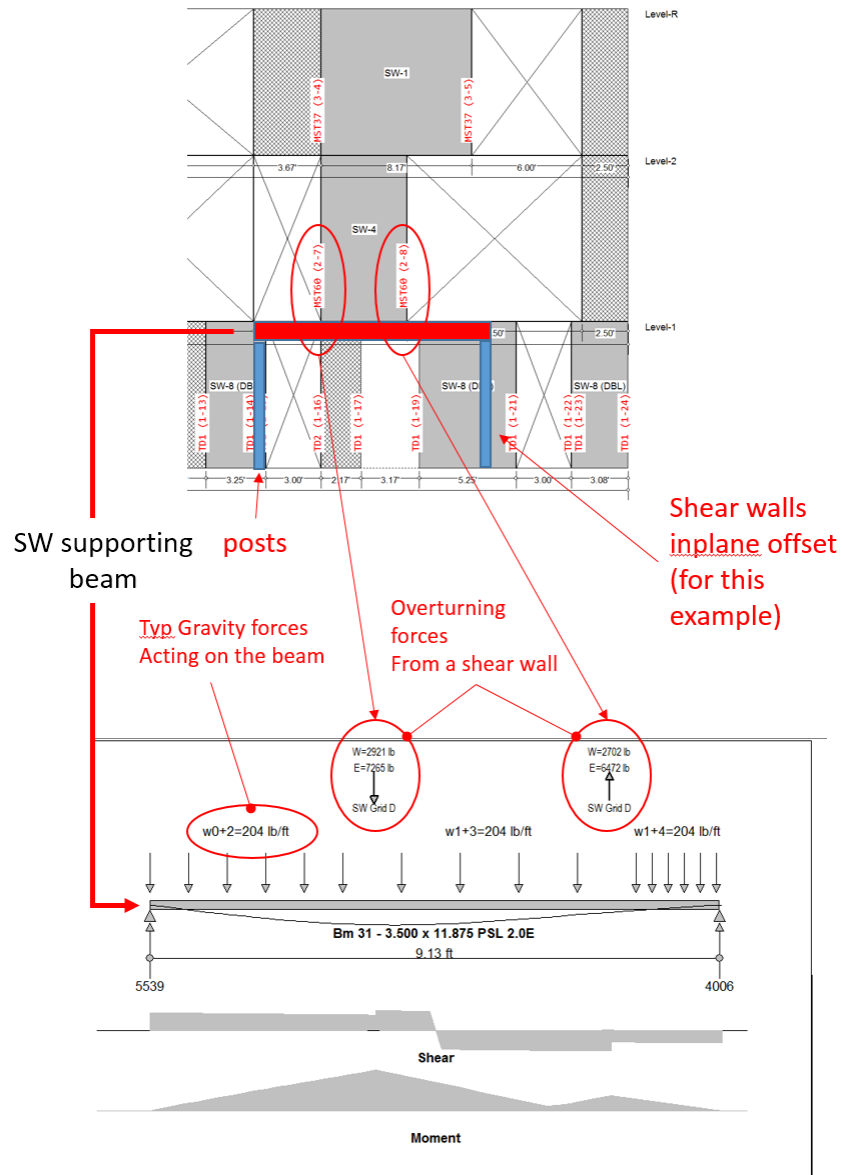


Illustration how shear wall forces are applied to the supporting beam
(See next page for typical calcs)

Excerpt from the beam analysis where
the overturning forces are applied w/
the omega-not factored

->From location 7.81 ft to 9.13 ft

->Distributed load from wall cladding loads, $w = 108.0 \text{ lb/ft}$

-> Weight of wall = $12.00 \text{ psf} \times \text{Height} = 9.00 \text{ ft} = 108.00 \text{ lb/ft}$

Point load 5 from shear wall overturning

P-WIND_POS = 2920.5 lb @ loc = 3.60 ft

P-SEISMIC_POS = 7264.6 lb @ loc = 3.60 ft

E (SEISMIC) = $\text{OMEGA} \times \text{QE} \text{ ***} = 0.7 \times 3.00 \times 7265 \text{ lb} = 15256 \text{ lb}$

P-WIND_NEG = -2920.5 lb @ loc = 3.60 ft

P-SEISMIC_NEG = -7264.6 lb @ loc = 3.60 ft

E (SEISMIC) = $\text{OMEGA} \times \text{QE} \text{ ***} = 0.7 \times 3.00 \times -7265 \text{ lb} = -15256 \text{ lb}$

Point load 6 from shear wall overturning

P-WIND_POS = -2701.9 lb @ loc = 7.35 ft

P-SEISMIC_POS = -6472.5 lb @ loc = 7.35 ft

E (SEISMIC) = $\text{OMEGA} \times \text{QE} \text{ ***} = 0.7 \times 3.00 \times -6472 \text{ lb} = -13592 \text{ lb}$

P-WIND_NEG = 2701.9 lb @ loc = 7.35 ft

P-SEISMIC_NEG = 6472.5 lb @ loc = 7.35 ft

E (SEISMIC) = $\text{OMEGA} \times \text{QE} \text{ ***} = 0.7 \times 3.00 \times 6472 \text{ lb} = 13592 \text{ lb}$

->Computed moments and shears (Factored) :

Max shear = 6619 lbs $0.6D - 0.7E \text{ (2.4-8b)}$

Min shear = -6991 lbs $D + 0.7E \text{ (2.4-5c)}$

Max moment = 18738 ft-lbs $D + 0.7E \text{ (2.4-5c)}$

Min moment = -16632 ft-lbs $0.6D - 0.7E \text{ (2.4-8b)}$

Controlling factored loads. In this
example only cladding and wall dead
loads are acting on this beam.