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STRUCTURAL ANALYSIS COVER PAGE

Job Title: LEGEND AUTO ADDITION TO EXISTING BUILDING

Job Number: Z9-3288

Jurisdiction: CITY OF PUYALLUP

LATERAL ENGINEERING DESIGN PARAMETERS

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Wind Design Data
----------------------------------------------------------
Wind Design Speed, Vu = 110 MPH, Vasd = 85 MPH
Wind Exposure = B
Wind Importance Factor, Iw = 1.0
Internal Pressure Coefficient = +/- 0.18
Ktz = 1.38
Kd = 0.85
Seismic Design Data
__________________________________
Importance factor = 1.0
Ss = 1.26g, S1 = 0.48g
Site Class = D
SDS = 1.00g, SD1 = 0.59g
SDC = D
Seismic System = 15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance or
stee
Design Base Shear = 5.71 kips
Cs = 0.155
R = 6.5
Analysis procedure:  ASCE 11.4, 11.5 & 12.8
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GRAVITY ENGINEERING DESIGN PARAMETERS

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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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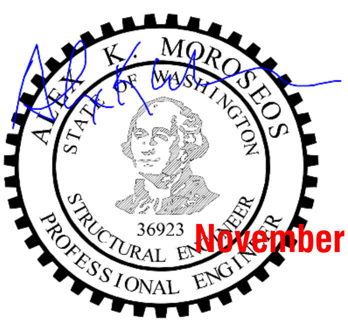


November 7, 2023

Contents

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 = 17.0 ft

Building length = 54.0 ft

Total height = 22.4 ft

Height to average roof height = (22.00 ft + [0.35 + 22.00])/2 = 22.18 ft

Bldg height = 22.00 ft

Roof height = = 0.35 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, $p = q_h\, G\, C_p - q_i\, (G\, C_{pi})$

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

		Load Case 1		Load Case 2	
Surface	Height,ft	Kz	qz (psf)	Kz	qz (psf)
	15.00	0.70	25.43	0.57	20.71
	20.00	0.70	25.43	0.62	22.53
Diaphragm	22.00	0.70	25.43	0.66	23.98
Mean Roof	22.18	0.70	25.43	0.66	23.98
Max Height	22.35	0.70	25.43	0.66	23.98

Gust effect factor G = 0.85, assume Rigid Structure (ASCE 7-10 Section 26.9.1)
Internal pressure coefficient ($G\ Cpi$) = +/- 0.18 (ASCE 7-10 Table 26.11-1)

External wall *Cp* from Figure 27.4-1

Windward wall, Cp = 0.80 for all L /B ratios
Side wall, Cp = -0.70 for all L /B ratios

Leeward wall pressure coefficient, Cp if a function of the L /B ratio
For load direction 1, B = 54.0 ft. and L = 17.0 ft.
L/B = 17.0 / 54.0 = 0.3, Cp = -0.50
For load direction 2, B = 17.0 ft. and L = 54.0 ft.
L/B = 54.0 / 17.0 = 3.2, Cp = -0.24

Surface	Wind Direction	L/B	Cp
Windward wall	All	All	0.80
Leeward wall	Direction 1	0.31	-0.50
Leeward wall	Direction 2	3.18	-0.24
Side wall	All	All	-0.70

External roof *Cp* - Load direction 1, from Figure 27.4-1

For Angle = 1.2 degrees

Windward roof : 0 to h/2, 0 to 22.2/2 = 11.1 ft, Cp = -1.30

Windward roof : h > h/2 = 11.1 ft, Cp -0.70

The above table reflects C_p values based on h/L 22.2/17.0 = 1.30

Internal pressure coefficient (G Cpi) - Load direction 1

$G\ Cpi$ = +/- 0.18 acting at 22.2 ft.
Velocity pressure at $q_i = q_h$ = 23.98 psf (Load case 2-Occurs at roof mid height)

MWFRS Net pressures - Load direction 1

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

$p = q_h\ (0.85)C_p - 23.98(+/- 0.18)$, psf

MWFRS pressures: Direction 1

Surface	z ft	q psf	G	Cp	Net pressure psf with (+Gpi)	(-Gpi)
Windward wall	15.0	20.7	0.85	0.80	9.8	18.4
	20.0	22.5	0.85	0.80	11.0	19.6
	22.0	24.0	0.85	0.80	12.0	20.6
Leeward wall	All	24.0	0.85	-0.50	-14.5	-5.9
Side wall	All	24.0	0.85	-0.70	-18.6	-10.0
Windward roof	>0-h/2	24.0	0.85	-1.30	-30.8	-22.2
Windward roof	>h/2	24.0	0.85	-0.70	-18.6	-10.0
Leeward roof	N/A					

External roof C_p-Load direction 2 (L = 54.0 ft.), from Figure 6 - 6

For Angle = 0.0 degrees

Surface : Windward roof 0 to h/2, 0 to 22.2/2 = 11.1 ft, Cp = -0.90

Surface : Windward roof h/2 to h, 11.1 to 22.2 ft, Cp = -0.90

Surface : Windward roof h to 2h, 22.2 to 2(22.2) = 44.4 ft, Cp = -0.50

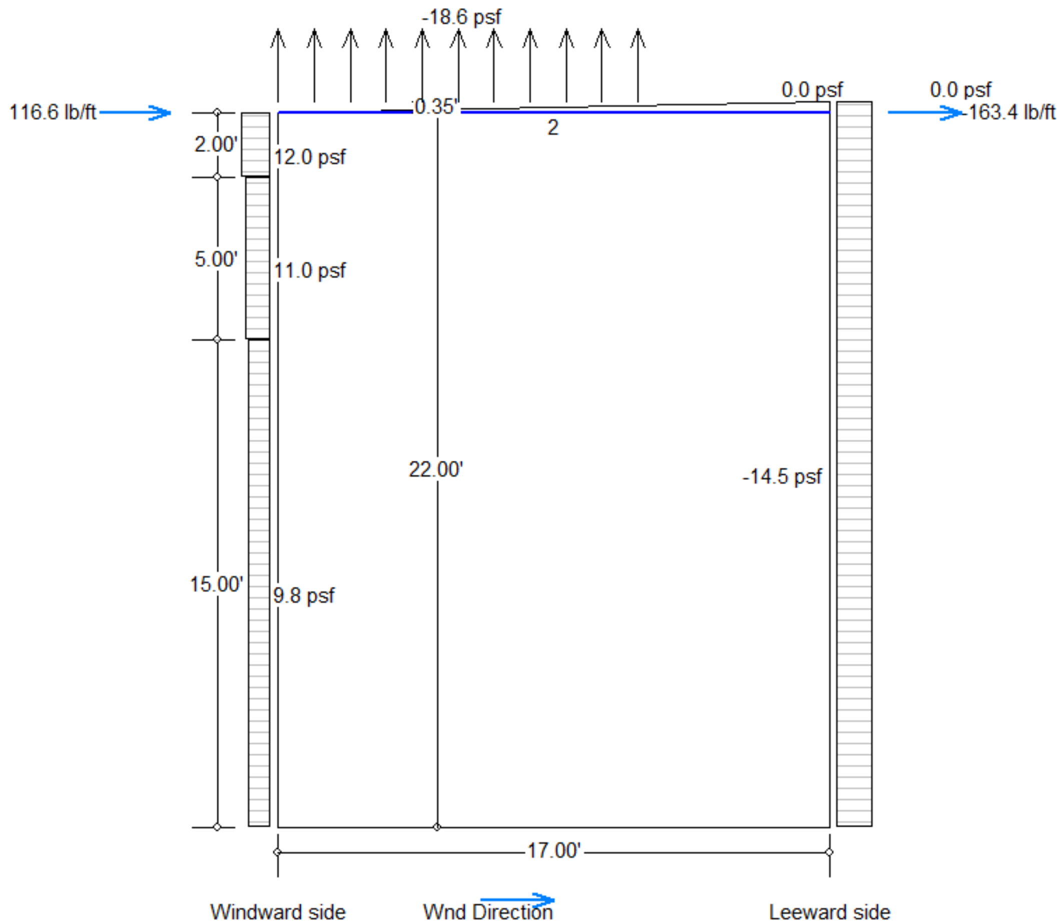
Surface : Windward roof h > 2h = 44.4 ft, Cp = -0.30

The above table reflects C_p values based on h / L of 22.2 / 54.0 = 0.4

MWFRS pressures : Direction 2

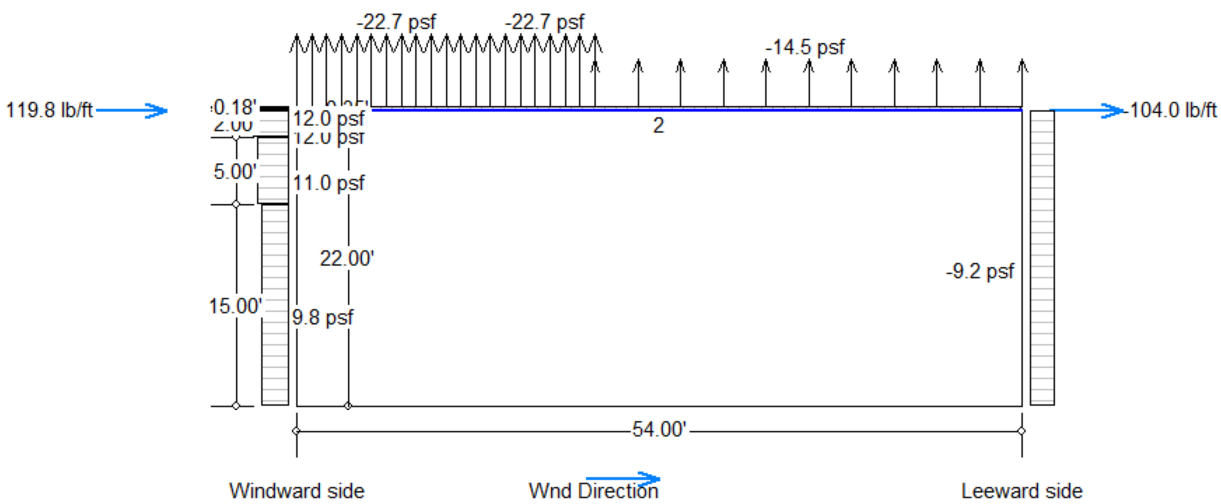
Surface	z ft	q psf	G	Cp	Net pressure psf with (+Gpi)	(-Gpi)
Windward wall	15.0	20.7	0.85	0.80	9.8	18.4
	20.0	22.5	0.85	0.80	11.0	19.6
	22.0	24.0	0.85	0.80	12.0	20.6
	22.2	24.0	0.85	0.80	12.0	20.6
	22.4	24.0	0.85	0.80	12.0	20.6
Leeward wall	All	24.0	0.85	-0.24	-9.2	-0.6
side wall	All	24.0	0.85	-0.70	-18.6	-10.0
Windward roof	0-h/2	24.0	0.85	-0.90	-22.7	-14.0

Windward roof	h/2-h	24.0	0.85	-0.90	-22.7	-14.0
Windward roof	h-2h	24.0	0.85	-0.50	-14.5	-5.9
						-1.8
Windward roof	>2h	24.0	0.85	-0.30	-10.4	\par \ul Leeward roof N/A \ulnone
Leeward roof	>N/A					



Transverse Direction - with positive internal pressure

Diaphragm	Windward	Leeward	Total
1	116.6 lb/ft	-163.4 lb/ft	280.0 lb/ft



Longitudinal Direction - with positive internal pressure

Diaphragm	Windward	Leeward	Total
1	119.8 lb/ft	-104.0 lb/ft	223.8 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

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

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

Design spectral accelerations parameters:

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

$S_{D1} = 2/3S_{M1} = 2/3(0.878) = 0.585g$ 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 stee>

R = 6.5 (Table 12.2-1)

Ω_0 = 3.0 (2.5 for flexible diaphragm - Note 9)

C_d = 4.0

Building element weights

Level 1, Roof weight = 15.0 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

WRoof = Roof weight x Area + 1/2 x Partition weight x Area + 1/2 x Ext Wall weight x Perim x Height

WRoof = 15.0 psf x 916 sq.ft. + 1/2 x 10.0 psf x 916 sq.ft. + 1/2 x 12.0 psf x 141 ft x 22.0 ft = 36932 lb

Total weight = 36932 = 36932 lbs

Compute structure period

Structure type: All other structures

C_T = 0.020 (Table 12.8-2)

Structure height, h_n = 22.0 ft.

$T_a = C_T (h_n)^{3/4} = 0.020(22.0)^{3/4}$ = 0.203 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 EQ 12.8-2

and

$C_s = I_e S_{D1} / (R T_a)$ = 1.00(0.59)/[(6.50)(0.20)] = 0.4431 EQ 12.8-3

but not less

C_s = 0.01 EQ 12.8-4

Therefore C_s = 0.1545

Design base shear, $V = C_s W$ = 0.1545(36932) = 5707 lbs (5.7 kips) Eq 12.8-1

Vertical distribution of force

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

where $C_{vx} = W_x h_x^k / (\Sigma W_i h_i^k)$ 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 = 0.203 sec

Level x	h_x	h^k_x	w_x	$w_x \times h^k_x$	C_{vx}	$F_x = C_{vx} V$	$F_x/w_x = S_a$
1	22.0	22.0	36.9	813	1.000	5.7	0.155
SUM			36.9	813		5.7	

Compute Diaphragm shears per ASCE 7-16 Par 12.10.1.1

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

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

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	36.9 k	5.7 k	5.7 k	7.4 k	14.8 k	7.4 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 SPAN	WIDTH ft	WIND LOAD lb/ft	SEISMIC LOAD lb/ft
A-B	53.5	280.0	92.5

Direct Shear Forces Table

DIAPHRAGM SPAN	GRID LINE	WIND lb	SEISMIC lb	GRID LINE	WIND lb	SEISMIC lb
A-B	A	7486	2474	B	7486	2474

Analysis Direction 2

Current Level 1

Shear Forces Table

DIAPHRAGM SPAN	WIDTH ft	WIND LOAD lb/ft	SEISMIC LOAD lb/ft
1-2	16.7	223.8	206.1

Direct Shear Forces Table

DIAPHRAGM SPAN	GRID LINE	WIND lb	SEISMIC lb	GRID LINE	WIND lb	SEISMIC lb
1-2	1	1874	1725	2	1874	1725

1.4 Compute Rho

```
Redundancy calculation rho, per ASCE 12.3.4.2 - Summary
---------------------------------------------------
Level = 1
              Condition
Direction     A        B    Rho
------------------------------
   1        PASS    FAIL   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
Total shear wall length = 0.0 ft
All panels height / Length <= 1.0.
Condition A, PASSED
Check condition B
Grid Line    Length     Height    2L/H
------------------------------------------------
------------------------------------------------
Sum                                 0.00
There are 0.00 bays < 4 req'd, therefore NOT OK
Condition B, FAILED
*** Direction 2 ***
--------------------
Check condition A
Grid Line 1, Height = 22.00 ft
  #      Length    Height/Length
-------------------------------
  1      46.97'      0.47
```

Redundancy Calculation Rho, Condition B is noted as failing [STRUCTURAL ANALYSIS - EXISTING BLDG, page 13 of 39].

```
--------------------------------
Total shear wall length = 47.0 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
-------------------------------------------------
  1           46.97'    22.00'     4.27
-------------------------------------------------
Sum                                4.27
There are 4.27 bays > 4 req'd, therefore OK
Condition B, PASSED
```


```
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	47.0	22.0	0.5	3450	2130	5580	3748	83	34	83	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. Ew - Unfactored Wall inertia force (wall & window panels) includes rho.
5. E + Ew = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. vE = 0.7 x vE(ASD factored shear).
8. wW = 0.6 x 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#/	Length	x1	x2	Dead	Snow	Live	Wind Uplift
	Type	ft	ft	ft	lb/ft	lb/ft	lb/ft	lb/ft
1	0/SW	46.97	0.00	46.97	264.0*	-	-	-
1	1/OPEN	2.99	0.00	2.99	0.0*	-	-	-
1	2/NO-SW	4.00	0.00	4.00	264.0*	-	-	-

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

Reaction	Location from end (ft)	D lb	S lb	L lb	W Uplift lb	DIRECTION 1 E lb	DIRECTION 1 W lb	DIRECTION 2 E lb	DIRECTION 2 W lb
1-0	0.00	6201	0	0	0	-2614	-1755	2614	1755
1-1	46.97	6201	0	0	0	2614	1755	-2614	-1755
1-2	49.97	528	0	0	0	0	0	0	0
1-3	53.97	528	0	0	0	0	0	0	0

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 = (Panel Height x Panel Width x Panel weight = 12.0 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)	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb	lb
1-0	0.0	5147	4371	5411	4829	2667	1049	7254	8030	6991	7573	4774	4708	1049	8030
1-1	47.0	7254	8030	6991	7573	4774	4708	5147	4371	5411	4829	2667	1049	1049	8030
1-2	50.0	528	528	528	528	317	245	528	528	528	528	317	245	245	528
1-3	54.0	528	528	528	528	317	245	528	528	528	528	317	245	245	528

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 from end (ft)	MIN LOAD lb	MAX LOAD lb	HOLD-DOWN MARK
1-0	0.0	1049	5411	TD0
1-1	47.0	1049	5411	TD0
1-2	50.0	245	528	TD1
1-3	54.0	245	528	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

LOAD	q lb/ft	v lb/ft	dq lb/ft
WIND	70.10	79.79	-9.69
SEISMIC	104.37	118.80	-14.43

PANEL ID	TYPE	PANEL END#1 WIND (LB)	PANEL END#1 SEISMIC (LB)	PANEL END #2 WIND (LB)	PANEL END #2 SEISMIC (LB)
1	SHEAR WALL	0	0	-455	-678
2	WINDOW/DOOR	-455	-678	-245	-365
3	NON-SHEAR WALL	-245	-365	35	52

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	-273	-1186
2	WINDOW/DOOR	-273	-1186	-147	-639
3	NON-SHEAR WALL	-147	-639	21	91

Notes

1. Wind load, W = 0.6 x Load
2. Seismic load, E = 0.7 x 1.25 x 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

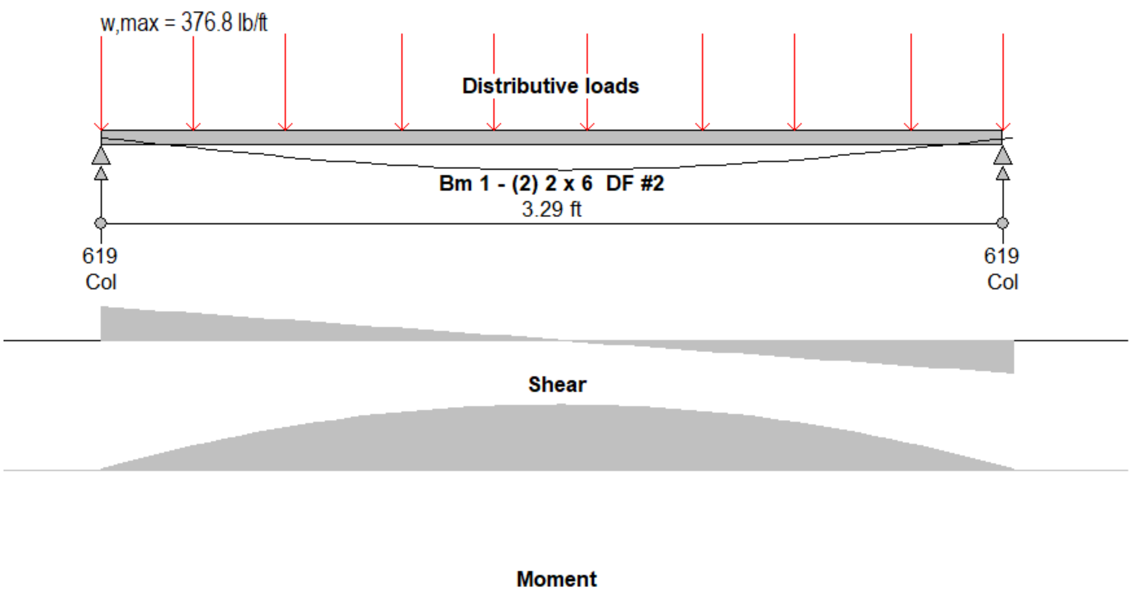


Table 1 - Point load table

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 ID	WALL HEIGHT	D	S	L
0	Floor/Roof	0	-	20.0	25.0	0.0

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB WIDTH	from loc	to loc	D	S	L
0	Floor/Roof	16.7 |	-0.0	3.3 |	167.5	209.3	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  =        619 lbs           D + S (2.4-3)
        Min shear  =       -619 lbs           D + S (2.4-3)
        Max moment =        508 ft-lbs        D + S (2.4-3)
        Min moment =          0 ft-lbs  D - (0.6)W (2.4-5b)
```

->Beam properties (2D xy axis) :

```
        Span  = 3.29 ft
        Area  = 16.50 sq.in
```

```
        Sx    = 15.12 sq.in

        Ixx   = 41.59 sq.in

->Check shear :

        fv = 1.5 x V / Area  = 619 / 16.50 = 56.27 psi

        F'v = 180.00 x 1.15 x 1.00 x 1.00 x 1.00 =  207.00 psi

          Fv = 180 psi, CD = 1.15, Cm = 1.00, Ct = 1.00, Ci = 1.00.

->Check bending :

        fb-top = M x 12 / Sx  = 6100 / 15.12 = 403.29 psi

        fb-btm = M x 12 / Sx  = 0 / 15.12 = 0.00 psi

          Fb = 900 psi, CD = 1.15, Cm = 1.00, Ct = 1.00, Cl = 1.00,

         Cf = 1.30, Cfu = 1.00, Ci = 1.00, Cr = 1.00.

          Fb'x CD x CM x CT x CL x CFx CFU x CI x CR = 1346 psi

->Check bearing :

->Check deflections :

        Number of deflection spans = 1

        Deflection span 0, Length = 3.29 ft Combined deflection = -0.015 [D + S (2.4-3)]

                Allowed = 3.29 x 12 / 360.0 = 0.110 in.

                Allowed (Seismic controled) = 3.29 x 12 / 180.0 = 0.219 in.
```

2.2 Analysis of Bm 2 - 6 x 8 DF #2

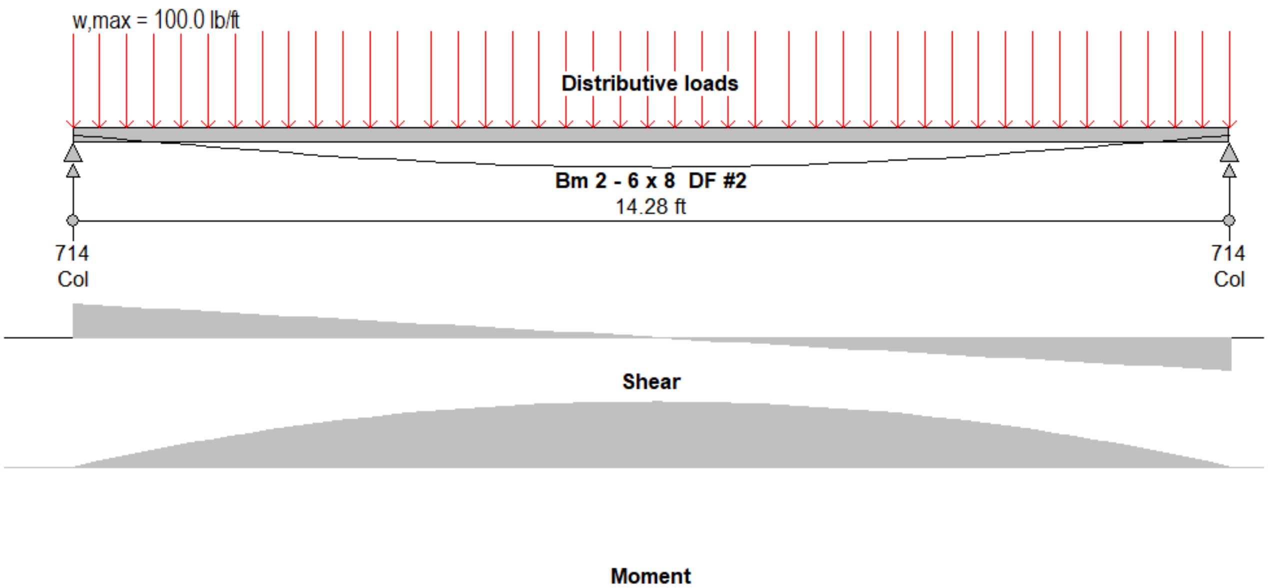


Table 1 - Point load table

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

-----------------------------------------------------------

No distributive loads

.-----------------------------------------------------------

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD  ELEMENT              TRIB     from    to       D         S      L

                           WIDTH     loc    loc

----------------------------------------------------------------------------

 0  WALL LOAD               0.0 |    0.0   14.3 |   100.0     0.0     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  =        714 lbs     D - (0.6)W (2.4-5b)

       Min shear  =       -714 lbs     D - (0.6)W (2.4-5b)

       Max moment =       2548 ft-lbs  D - (0.6)W (2.4-5b)

       Min moment =         -0 ft-lbs  D - (0.6)W (2.4-5b)

->Beam properties (2D xy axis) :

       Span  = 14.28 ft

       Area  = 39.88 sq.in

       Sx    = 48.18 sq.in

       Ixx   = 174.66 sq.in

->Check shear :
```

fv = 1.5 x V / Area = 714 / 39.88 = 26.86 psi

F'v = 180.00 x 1.60 x 1.00 x 1.00 x 1.00 = 288.00 psi

Fv = 180 psi, CD = 1.60, Cm = 1.00, Ct = 1.00, Ci = 1.00.

->Check bending :

fb-top = M x 12 / Sx = 30570 / 48.18 = 634.47 psi

fb-btm = M x 12 / Sx = 0 / 48.18 = 0.00 psi

Fb = 900 psi, CD = 1.60, Cm = 1.00, Ct = 1.00, Cl = 1.00,

Cf = 1.05, Cfu = 1.00, Ci = 1.00, Cr = 1.00.

Fb'x CD x CM x CT x CL x CFx CFU x CI x CR = 1506 psi

->Check bearing :

->Check deflections :

Number of deflection spans = 1

Deflection span 0, Length = 14.28 ft Combined deflection = -0.334 [D - (0.6)W (2.4-5b)]

Allowed = 14.28 x 12 / 360.0 = 0.476 in.

Allowed (Seismic controled) = 14.28 x 12 / 180.0 = 0.952 in.

2.3 Analysis of Bm 3 - 6 x 8 DF #2

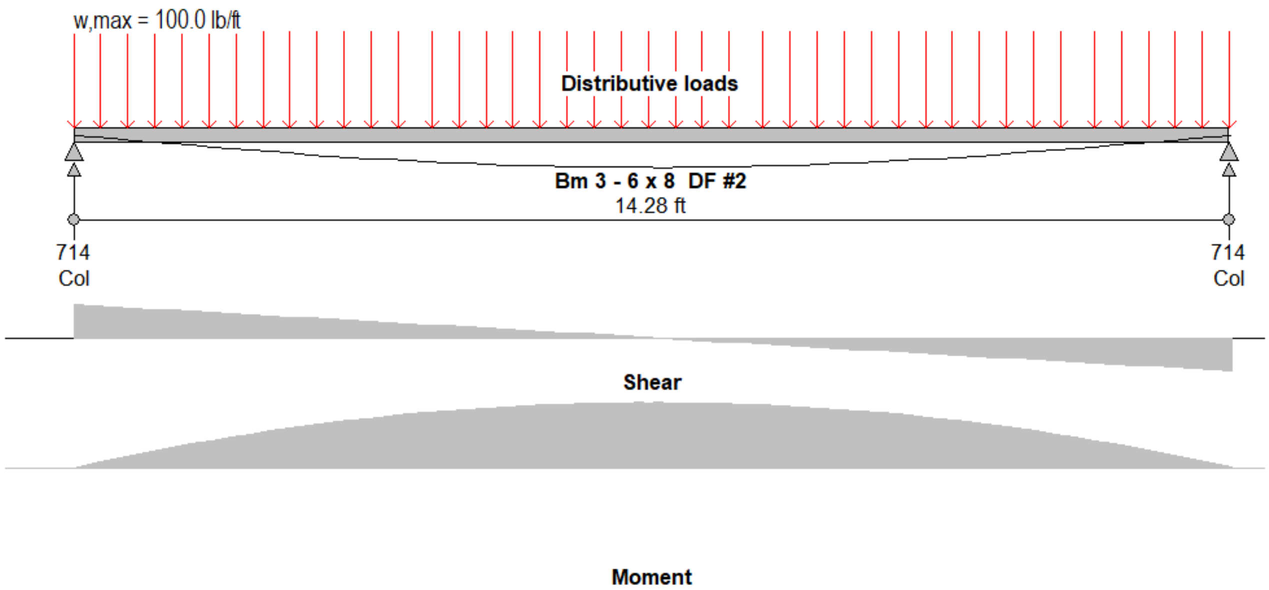


Table 1 - Point load table

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

No distributive loads

.--

(1) loads in psf.

Table 4 - Distributive load table (line loads)

LOAD	ELEMENT	TRIB WIDTH	from loc	to loc	D	S	L
0	WALL LOAD	0.0	0.0	14.3	100.0	0.0	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 = 714 lbs D - (0.6)W (2.4-5b)

Min shear = -714 lbs D - (0.6)W (2.4-5b)

Max moment = 2550 ft-lbs D - (0.6)W (2.4-5b)

Min moment = -0 ft-lbs D - (0.6)W (2.4-5b)

->Beam properties (2D xy axis) :

Span = 14.28 ft

Area = 39.88 sq.in

Sx = 48.18 sq.in

Ixx = 174.66 sq.in

->Check shear :

fv = 1.5 x V / Area = 714 / 39.88 = 26.87 psi

F'v = 180.00 x 1.60 x 1.00 x 1.00 x 1.00 = 288.00 psi

Fv = 180 psi, CD = 1.60, Cm = 1.00, Ct = 1.00, Ci = 1.00.

->Check bending :

fb-top = M x 12 / Sx = 30596 / 48.18 = 635.00 psi

fb-btm = M x 12 / Sx = 0 / 48.18 = 0.00 psi

Fb = 900 psi, CD = 1.60, Cm = 1.00, Ct = 1.00, Cl = 1.00,

Cf = 1.05, Cfu = 1.00, Ci = 1.00, Cr = 1.00.

Fb'x CD x CM x CT x CL x CFx CFU x CI x CR = 1506 psi

->Check bearing :

->Check deflections :

Number of deflection spans = 1

Deflection span 0, Length = 14.28 ft Combined deflection = -0.335 [D - (0.6)W (2.4-5b)]

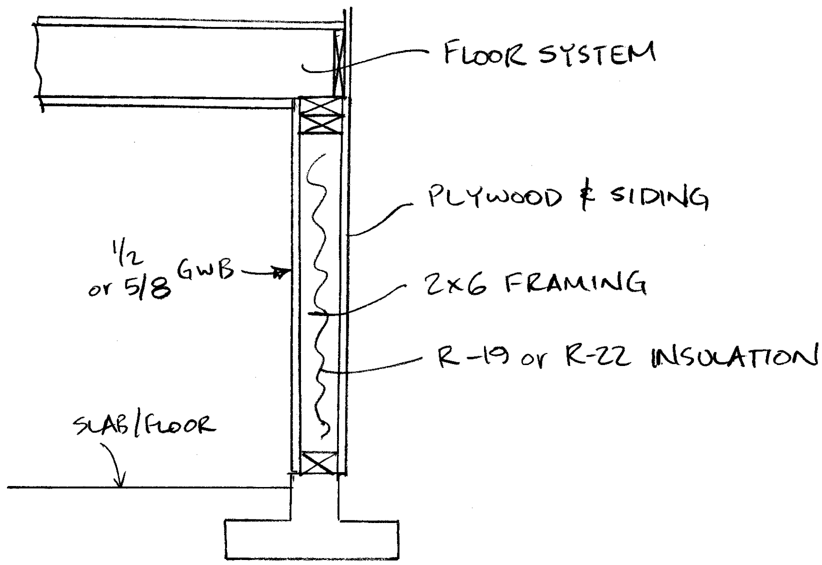
Allowed = 14.28 x 12 / 360.0 = 0.476 in.

Allowed (Seismic controled) = 14.28 x 12 / 180.0 = 0.952 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 × 6 Cedar siding1.5 psf × .75 thick	= 1.13 psf
15# paper	= 1.00 psf
½ CDX plywood 3.0 psf /inch	= 1.5 psf
R-22 Insulation	= 1.0 psf
2 × 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 ½″ GWB	= 2.5 psf
Total with ½ GWB	**= 8.63 psf (Use 10 psf)**

With 2-1/12 Face brick (25 psf) & ½ GWB = **32.5 psf**

Typical light floor system with 2 × 10 floor joists 16″ O.C.

¾ T & G Sub floor (3.0 psf/inch)	= 2.25 psf
2 × 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)

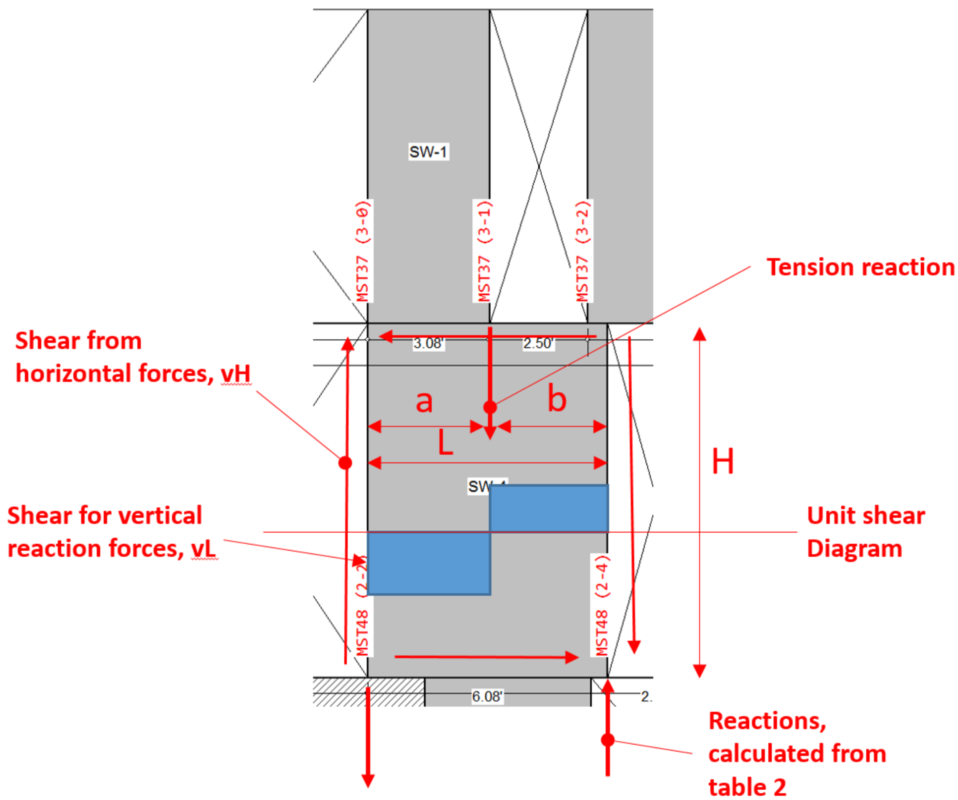
Notes
1. b = sum of all solid panels.
2. H / W = Maximum aspect ratio of all panels within a SW.
3. E - Unfactored seismic forces(Summed between levels).
4. Ew - Unfactored Wall inertia force (wall & window panels).
5. E + Ew = Total unfactored seismic load.
6. W - Unfactored wind forces(Summed between levels).
7. vE = 0.7 x vE(ASD factored shear).
8. wW = 0.6 x 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



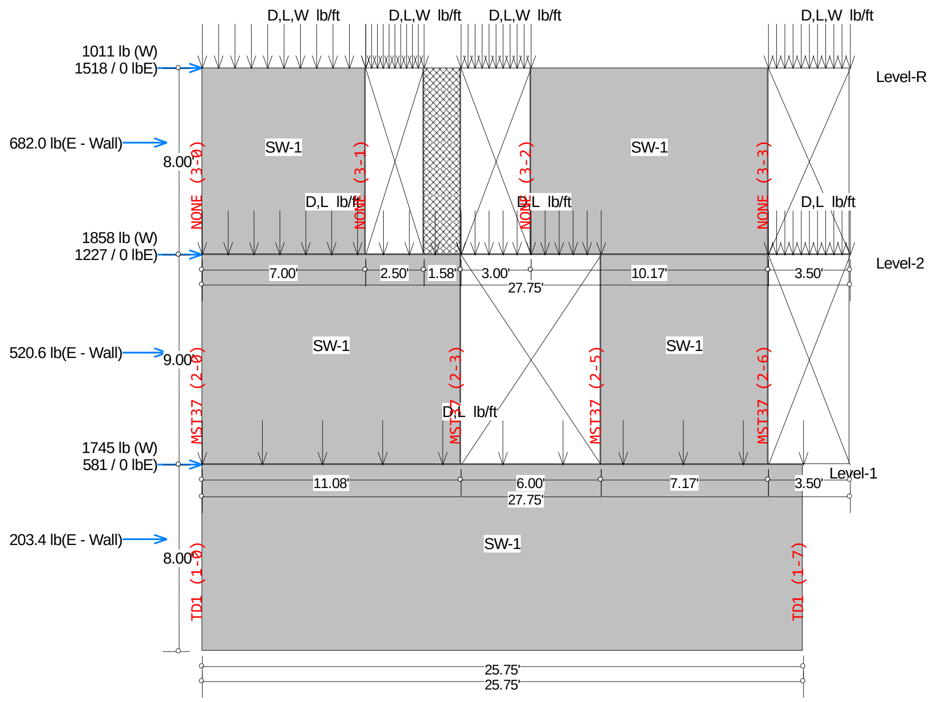
Free body diagram of a typical shear panel

To compute vL,

vL(LHS) = (Tension reaction x b / L) / H

Total shear = Max LHS or RHS reactions + vH.

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 = weight of wall x width x 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)	D lb	S lb	L lb	W Uplift lb	DIRECTION 1 E lb	DIRECTION 1 W lb	DIRECTION 2 E lb	DIRECTION 2 W 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 = (Panel Height x Panel Width x Panel weight = 12.0 psf + 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 LC1 lb	LC2 lb	LC3 lb	LC4 lb	LC5 lb	LC6 lb	DIRECTION 2 LC1 lb	LC2 lb	LC3 lb	LC4 lb	LC5 lb	LC6 lb	MIN LOAD lb	MAX LOAD 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

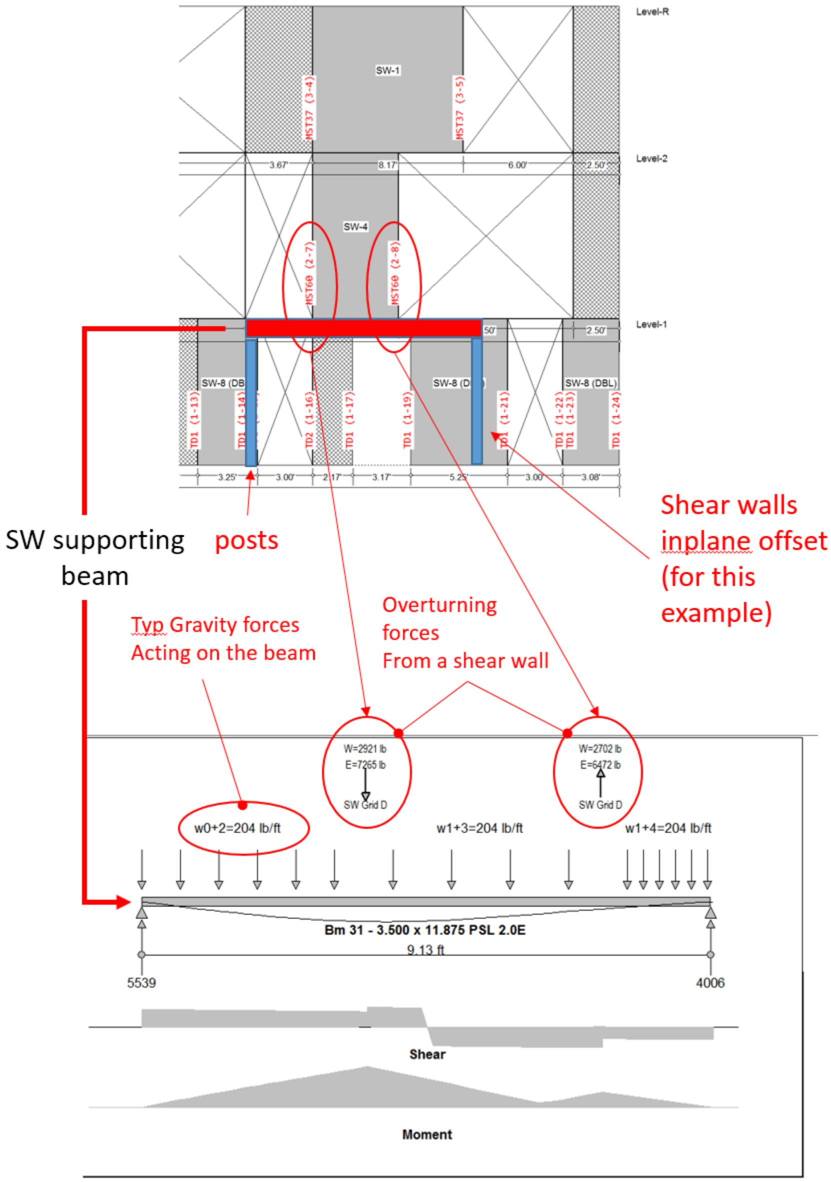


Illustration how shear wall forces are applied to the supporting beam

(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 lb/ft

->      Weight of wall = 12.00 psf x Height = 9.00 ft = 108.00 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) = OMEGA X QE ***  = 0.7 x 3.00 X 7265 lb = 15256 lb

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

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

                      E (SEISMIC) = OMEGA X QE ***  = 0.7 x 3.00 X -7265 lb = -15256 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) = OMEGA X QE ***  = 0.7 x 3.00 X -6472 lb = -13592 lb

              P-WIND_NEG = 2701.9 lb @ loc = 7.35 ft

              P-SEISMIC_NEG = 6472.5 lb @ loc = 7.35 ft

                      E (SEISMIC) = OMEGA X QE ***  = 0.7 x 3.00 X 6472 lb = 13592 lb

->Computed moments and shears (Factored) :

       Max shear  =       6619 lbs     0.6D - 0.7E (2.4-8b)

       Min shear  =      -6991 lbs        D + 0.7E (2.4-5c)

       Max moment =      18738 ft-lbs     D + 0.7E (2.4-5c)

       Min moment =     -16632 ft-lbs 0.6D - 0.7E (2.4-8b)
```

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