MYERS ENGINEERING

Structural Calculations

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Mark Myers, PE
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Project: Addendum to Proposed Residence
4250 89th Avenue Southeast
Mercer Island, WA

July 9, 2021

2018 INTERNATIONAL BUILDING CODE
100 MPH WIND, EXPOSURE B, K_{zt} = 1.40
RISK CATEGORY II - SOIL SITE CLASS D
SEISMIC DESIGN CATEGORY D (IBC)

3206 50th Street Court, Suite 210-B
Gig Harbor, WA 98335
Phone: 253-858-3248
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DESIGN LOADS:

ROOF DEAD LOADS	15 PSF Total
ROOF LIVE LOADS	25 PSF (Snow)
FLOOR DEAD LOADS	15 PSF Total
FLOOR LIVE LOADS	40 PSF (Reducible)
STAIR LIVE LOADS	100 PSF

$psf := \frac{lb}{ft^2}$ $plf := \frac{lb}{ft}$

WOODS :

	WOOD TYPE:
JOISTS OR RAFTERS 2X.	HF#2
BEAMS OR HEADERS 4X - 6X OR LARGER	DF#2
LEDGERS AND TOP PLATES	HF#2
STUDS 2X4 OR 2X6	HF Stud
POSTS	
4X4	DF#2
4X6	DF#2
6X6	DF#1

GLUED-LAMINATED (GLB) BEAM & HEADER.
Fb=2,400 PSI, Fv=165 PSI, Fc (Perp) =650 PSI, E=1,800,000 PSI.

PARALLAM (PSL) 2.0E BEAM & HEADER.
Fb=2,900 PSI, Fv=290 PSI, Fc (Perp) =750 PSI, E=2,000,000 PSI.

MICROLAM (LVL) 1.9E BEAM & HEADER
Fb=2,600 PSI, Fv=285 PSI, Pc (Perp) =750 PSI, E=1,900,000 PSI.

TIMBERSTRAND (LSL) 1.3E BEAM, HEADER, & RIM BOARD
Fb=1,700 PSI, Fv=400 PSI, Pc (Perp) =680 PSI, E=1,300,000 PSI.

TRUSSES:

PREFABRICATED WOOD TRUSSES SHALL BE DESIGNED BY A REGISTERED DESIGN PROFESSIONAL REGISTERED IN THE STATE OF WASHINGTON. TRUSS DESIGNS SHALL COMPLY WITH THE REQUIREMENTS OF IBC 2303.4. SUBMITTAL PACKAGE SHALL COMPLY WITH REQUIREMENTS OF IBC 2303.4.1.4.

UNLESS OTHERWISE SPECIFIED BY LOCAL BUILDING OFFICIAL OR STATUTE, TRUSS DESIGNS BEARING THE SEAL AND SIGNATURE OF THE TRUSS DESIGNER SHALL BE AVAILABLE AT TIME OF INSPECTION.

ENGINEERED I-JOISTS

-FLOOR JOISTS & BEAMS OF EQUAL OR BETTER CAPACITY MAY BE SUBSTITUTED FOR THOSE SHOWN ON THIS PLAN, "EQUAL" IS DEFINED AS HAVING MOMENT CAPACITY, SHEAR CAPACITY, AND STIFFNESS WITHIN 3% OF THE SPECIFIED JOISTS OR BEAMS.

Myers Engineering, LLC
3206 50th Street Ct NW, Ste 210-B
Gig Harbor, WA 98335

PROJECT : **4250 89th AVE SE**

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LATERAL ANALYSIS :

BASED ON 2018 INTERNATIONAL BUILDING CODE (IBC)

Lateral Forces will be distributed along lines of Force/Resistance. Lines of Force/Resistance will be investigated for both wind and seismic lateral loads. Roof and Floor diaphragms are considered flexible.

Risk Category II per IBC 1604.5 & Soils Site Class D (Assumed)

SEISMIC DESIGN:

SEISMIC DESIGN BASED ON 2018 IBC Section 1613.1
LIGHT FRAME CONSTRUCTION LESS THAN THREE STORIES IN HEIGHT ABOVE GRADE.

Seismic Design Data:

$I_e := 1.0$ (ASCE 7-16 Table 1.5-2)

$R := 6.5$ $\Omega_0 := 3.0$ $C_d := 4$ Light-frame (wood) walls sheathed w/ wood structural panels rated for shear resistance (ASCE 7-16 Table 12.2-1)

$S_s := 1.419$ $S_1 := 0.493$ $S_{ms} := 1.702$ $S_{m1} := 0.89$

Equation 16-39 $S_{DS} := \frac{2}{3} \cdot S_{ms} = 1.13$ Equation 16-40 $S_{D1} := \frac{2}{3} \cdot S_{m1} = 0.59$

--Seismic Design Category D (S_{DS} greater than 0.50g & S_{D1} greater than 0.20g)

Roof Slope Adjustment Factor: $S_a := \dfrac{1}{\cos\left(\operatorname{atan}\left(\frac{7}{12}\right)\right)} = 1.16$ $S_b := \dfrac{1}{\cos\left(\operatorname{atan}\left(\frac{4}{12}\right)\right)} = 1.05$

Plan Area for Each Level:

$A_1 := 2245\text{ft}^2 \cdot S_a$	$A_{2a} := 1841\text{ft}^2$	$A_{2b} := 1686\text{ft}^2 \cdot S_b$
(Upper Roof)	(Upper Floor)	(Lower Roof)

Plan Perimeter for Each Level:

$P_1 := 2(55\text{ft}) + 2(55\text{ft})$	$P_2 := 2(59\text{ft}) + 2(55\text{ft})$
(Upper Floor)	(Main Floor)

W, w_x = Seismic Weight of Overall Structure, Seismic Weight of Structure above Level x (LB.)

Weight of Structure at Each Level:

Story Weight at Upper Floor:

$$w_1 := 15 \cdot \text{psf} \cdot A_1 + 12 \cdot \text{psf} \cdot 4.5 \cdot \text{ft} \cdot P_1$$

Story Weight at Main Floor:

$$w_2 := 15 \cdot \text{psf} \cdot (A_{2a} + A_{2b}) + 12 \cdot \text{psf} \cdot (4.5 \cdot \text{ft} \cdot P_1 + 5\text{ft} \cdot P_2)$$

$$W := w_1 + w_2 = 130698.67\,\text{lb}$$

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Approximate Fundamental Period, T_a:

$C_t := 0.02$ $\quad \chi := 0.75$ (per ASCE 7-16 Table 12.8-2) $\quad h_n := 24$ (Structural Height per ASCE 7-16 Sect. 11.2)

$T_a := C_t \cdot h_n^{\chi} = 0.22$ (ASCE 7-16 Eq. 12.8-7) $\quad T_L := 6$ (per ASCE 7-16 Fig. 22-12)

T_a is less than T_L, therefore Cs need not exceed: $\dfrac{S_{D1}}{\left(\dfrac{R}{I_e}\right)\cdot T_a} = 0.42$ (ASCE 7-16 Eq. 12.8-3)

C_s shall not be less than: $0.044 S_{DS} \cdot I_e = 0.05$ (ASCE 7-16 Eq. 12.8-5)

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = 0.17$$

Total Base Shear: $V_E := C_s \cdot W = 22815.3\,\text{lb}$

Vertical Shear distribution at each level:

for structures having a period of 0.5 sec or less: $k := 1$

$h_1 := 20\text{ft}$ $\quad h_2 := 10\text{ft}$ (Height from base to level x)

$$C_{v1} := \frac{(w_1 \cdot h_1)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.56$$

$F_1 := C_{v1} \cdot V_E = 12783.52\,\text{lb}$ Story Shear at Upper Floor

$$C_{v2} := \frac{(w_2 \cdot h_2)}{(w_1 \cdot h_1 + w_2 \cdot h_2)} = 0.44$$

$F_2 := C_{v2} \cdot V_E = 10031.78\,\text{lb}$ Story Shear at Main Floor

WIND DESIGN

Use analytical procedure of ASCE 7-16 Chapter 27 (Directional Procedure for buildings of all heights)

$V := 100$ 3-Sec Peak Gust (MPH) for Risk Category II (Figure 26.5-1A).

$K_d := 0.85$ Wind Directionality Factor (Table 26.6-1). $h := 24 \cdot ft$ Mean Roof Height as per Sect. 26.2

Exposure Category B (ASCE 7-16 Sect. 26.7.3)

Topographic Factor (K_{zt}) (Figure 26.8-1): 2-D Escarpment with building downwind of crest.

$x := 1177ft$ $H := 344ft$ $L_h := 890ft$ $z := h$ $\gamma := 2.5$ $\mu := 4$

$$K_1 := 0.75\left(\frac{H}{L_h}\right) = 0.29 \qquad K_2 := \left(1 - \frac{x}{\mu L_h}\right) = 0.67 \qquad K_3 := e^{\frac{(-\gamma \cdot z)}{L_h}} = 0.93 \qquad K_{zt} := \left(1 + K_1 \cdot K_2 \cdot K_3\right)^2 = 1.4$$

$G := 0.85$ Gust Effect Factor (ASCE 7-16 Sect. 26.9.1)

Building is an Enclosed Building as per ASCE 7-16 Sect. 26.10

$GC_{pi} := .18$ +/- Internal Pressure Coefficients (ASCE 7-16 Table 26.11-1)

Velocity Pressure Exposure Coefficient (Table 27.3-1):

$z_g := 1200ft$ $\alpha := 7.0$ (per ASCE 7-16 Table 26.9-1 based on Exposure Category)
z_g=1200ft, α=7.0 (Exp B), z_g=900ft, α=9.5 (Exp C), z_g=700ft, α=11.5 (Exp D)

$z_1 := 20ft$ $z_2 := 15ft$ Height from ground to level x (z_{min} = 15ft)

$$K_{z1} := 2.01\left(\frac{z_1}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.62 \qquad K_{z2} := 2.01\left(\frac{z_2}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.57 \qquad K_h := 2.01\left(\frac{h}{z_g}\right)^{\left(\frac{2}{\alpha}\right)} = 0.66$$

External Pressure Coefficients w/ Roof Pitch = 7/12 (30 degrees) Front to Back & 7/12 (30 degrees) Side to Side
Taken from Figure 27.4-1

Front to Back:

$L_{fb} := 55ft$ $B_{fb} := 55ft$ $\frac{L_{fb}}{B_{fb}} = 1$ $\frac{h}{L_{fb}} = 0.44$

$C_{pf1} := .8$ Windward Wall

$C_{pf2} := 0.23$ Windward Roof

$C_{pf3} := -.6$ Leeward Roof

$C_{pf4} := -.5$ Leeward Wall

Side to Side:

$L_{ss} := 55ft$ $B_{ss} := 55ft$ $\frac{L_{ss}}{B_{ss}} = 1$ $\frac{h}{L_{ss}} = 0.44$

$C_{ps1} := .8$ Windward Wall

$C_{ps2} := 0.23$ Windward Roof

$C_{ps3} := -.6$ Leeward Roof

$C_{ps4} := -.5$ Leeward Wall

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Velocity Pressure (q_z) Evaluated at Height (z) (Equation 23.3-1)

$q_{z1} := 0.00256 \cdot K_{z1} \cdot K_{zt} \cdot K_d \cdot V^2 = 18.95$ $\quad q_{z2} := 0.00256 \cdot K_{z2} \cdot K_{zt} \cdot K_d \cdot V^2 = 17.45$ $\quad q_h := 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 = 19.96$

Design Wind Pressures $p = qGC_p - q_i(GC_{pi})$ (Equation 27.4-1) where q_i will conservatively be taken equal to q_h

Windward Roof Front to Back $\quad p_{wr1} := q_h \cdot G \cdot C_{pf2} \cdot psf = 3.9\,ft^{-2} \cdot lb$

Leeward Roof Front to Back $\quad p_{lr1} := q_h \cdot G \cdot C_{pf3} \cdot psf = -10.18\,ft^{-2} \cdot lb$

Leeward Wall Front to Back $\quad p_{lw1} := q_h \cdot G \cdot C_{pf4} \cdot psf = -8.48\,ft^{-2} \cdot lb$

Windward Roof Side to Side $\quad p_{wr2} := q_h \cdot G \cdot C_{ps2} \cdot psf = 3.9\,ft^{-2} \cdot lb$

Leeward Roof Side to Side $\quad p_{lr2} := q_h \cdot G \cdot C_{ps3} \cdot psf = -10.18\,ft^{-2} \cdot lb$

Leeward Wall Side to Side $\quad p_{lw2} := q_h \cdot G \cdot C_{ps4} \cdot psf = -8.48\,ft^{-2} \cdot lb$

Windward Wall Both Directions

$p_{ww1} := q_{z1} \cdot G \cdot C_{pf1} \cdot psf = 12.89\,ft^{-2} \cdot lb$

$p_{ww2} := q_{z2} \cdot G \cdot C_{pf1} \cdot psf = 11.87\,ft^{-2} \cdot lb$

The Internal Pressures on Windward and Leeward Walls & Roofs will offset each other for the lateral design of the overall building and will therefore be ignored for this application.

Check net pressure not less than 16psf at walls & 8psf at roof over projected vertical plane:

$p_{wr1} - p_{lr1} = 14.08\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw1} = 21.37\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw1} = 20.35\,ft^{-2} \cdot lb$

$p_{wr2} - p_{lr2} = 14.08\,ft^{-2} \cdot lb$ $\quad p_{ww1} - p_{lw2} = 21.37\,ft^{-2} \cdot lb$ $\quad p_{ww2} - p_{lw2} = 20.35\,ft^{-2} \cdot lb$

Wind Pressure at Upper Roof (Front to Back):

$$V_{1W} := (p_{wr1} - p_{lr1})265ft^2 + (p_{ww1} - p_{lw1}) \cdot 350 \cdot ft^2 = 11211.75\,lb$$

Wind Pressure at 2nd Floor (Front to Back):

$$V_{2W} := (p_{wr1} - p_{lr1})0ft^2 + (p_{ww2} - p_{lw1}) \cdot 595 \cdot ft^2 = 12110.19\,lb$$

Wind Pressure at Upper Roof (Side to Side):

$$V_{3W} := (p_{wr2} - p_{lr2}) \cdot 300ft^2 + (p_{ww1} - p_{lw2}) \cdot 360ft^2 = 11918.38\,lb$$

Wind Pressure at 2nd Floor (Side to Side):

$$V_{4W} := (p_{wr2} - p_{lr2}) \cdot 0ft^2 + (p_{ww2} - p_{lw2}) \cdot 650ft^2 = 13229.62\,lb$$

Determine Component & Cladding loads:

Design Wind Pressures $p = q_h[(GC_p) - (GC_{pi})]$ (Equation 30.3-1)

(GC_p) is given in Figures 30.3-1 thru 30.3-7

(GC_{pi}) is given in Table 26.13-1 (See above)

$GC_{p1in} := 0.9$ $\quad GC_{p2in} := 0.9$ $\quad GC_{p3in} := 0.9$ $\quad$ Figure 30.3-2D (θ = 30 degrees)

$GC_{p1out} := -1.8$ $\quad GC_{p2out} := -2.0$ $\quad GC_{p3out} := -3.2$ $\quad GC_{p2oh} := -2.8$ $\quad GC_{p3oh} := -4.0$

$GC_{p4in} := 1.0$ $\quad GC_{p5in} := 1.0$ $\quad$ Figure 30.3-1

$GC_{p4out} := -1.1$ $\quad GC_{p5out} := -1.4$

$p_1 := q_h \cdot \left[\left(GC_{p1out}\right) - \left(GC_{pi}\right)\right] psf \qquad p_1 = -39.53\,ft^{-2} \cdot lb$ (Zone 1)

$p_2 := q_h \cdot \left[\left(GC_{p2out}\right) - \left(GC_{pi}\right)\right] psf \qquad p_2 = -43.52\,ft^{-2} \cdot lb$ (Zone 2)

$p_3 := q_h \cdot \left[\left(GC_{p3out}\right) - \left(GC_{pi}\right)\right] psf \qquad p_3 = -67.47\,ft^{-2} \cdot lb$ (Zone 3)

$p_2 := q_h \cdot \left(\left(GC_{p2oh}\right)\right) psf \qquad p_2 = -55.9\,ft^{-2} \cdot lb$ (Zone 2 Overhang)

$p_3 := q_h \cdot \left(\left(GC_{p3oh}\right)\right) psf \qquad p_3 = -79.85\,ft^{-2} \cdot lb$ (Zone 3 Overhang)

When roof pitch is less than θ=10 degrees, values of GC_p for walls may be reduced by 10%

$p_4 := q_h \cdot \left[\left(GC_{p4out}\right) - \left(GC_{pi}\right)\right] psf \qquad p_4 = -25.55\,ft^{-2} \cdot lb$ (Zone 4)

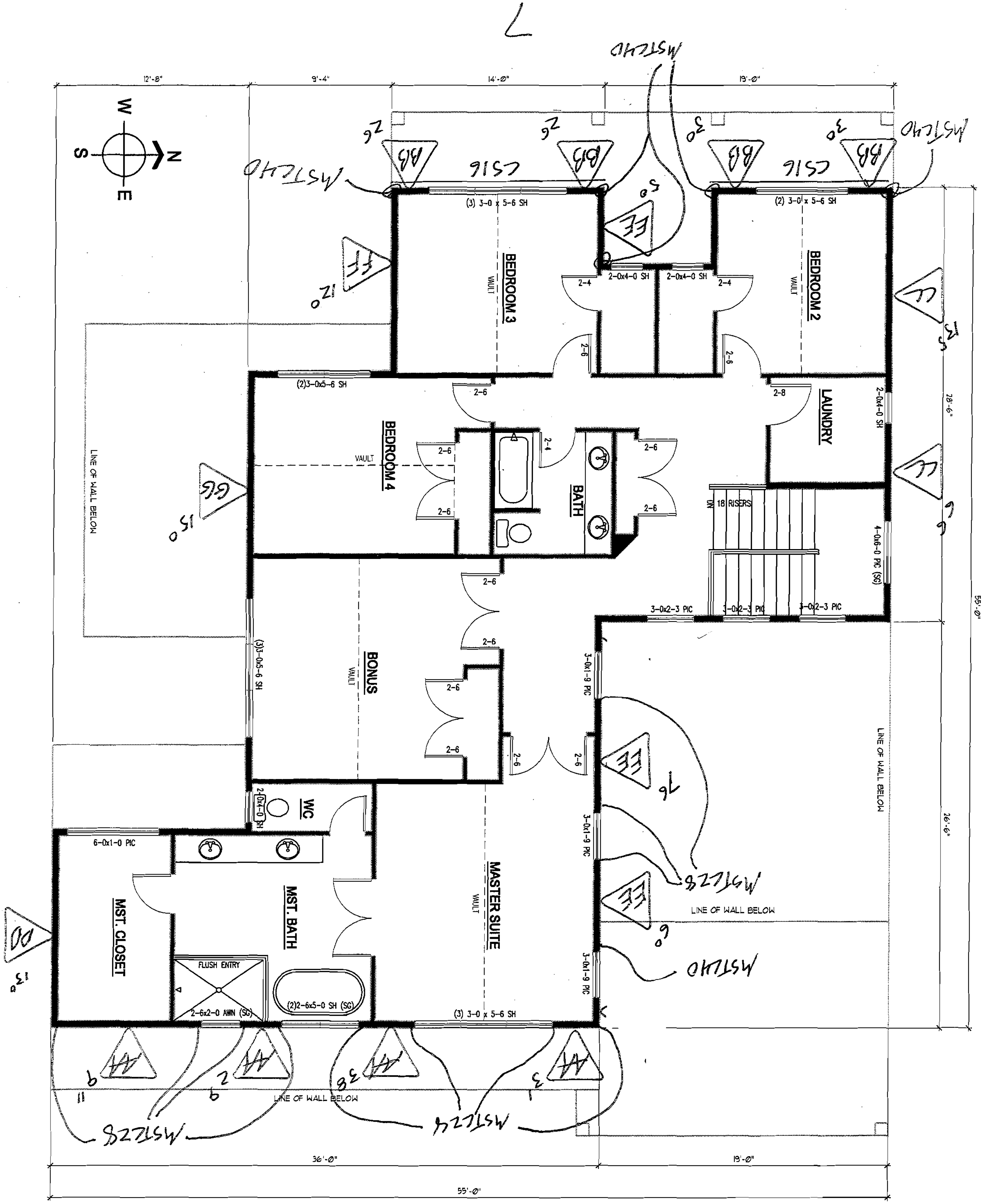
$p_5 := q_h \cdot \left[\left(GC_{p5out}\right) - \left(GC_{pi}\right)\right] psf \qquad p_5 = -31.54\,ft^{-2} \cdot lb$ (Zone 5)

Net pressure shall not be less than 16 psf for Components and Cladding (ASCE 7-16 Sec. 30.2.2)

a = 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than 4% of least horizontal dimension or 3ft

$0.1(55ft) = 5.5\,ft \qquad 0.4 \cdot h = 9.6\,ft \qquad 0.04(55ft) = 2.2\,ft$

Therefore $\quad a := 5.5ft$



7
12'-8"
9'-4"
14'-0"
19'-0"
W
S
N
E
MST240
CS16
BB
FF
EE
GG
CC
DD
(3) 3-0 x 5-6 SH
(2) 3-0 x 5-6 SH
BEDROOM 3
BEDROOM 2
VAULT
2-0x4-0 SH
2-4
2-6
2-8
LAUNDRY
2-0x4-0 SH
28'-6"
(2)3-0x5-6 SH
BEDROOM 4
BATH
18 RISERS
DN
4-0x6-0 PIC (SG)
3-0x2-3 PIC
LINE OF WALL BELOW
55'-0"
(3)3-0x5-6 SH
BONUS
3-0x1-9 PIC
26'-6"
WC
2-0x4-0 SH
6-0x1-0 PIC
MST. CLOSET
MST. BATH
MASTER SUITE
MST228
FLUSH ENTRY
2-6x2-0 AWN (SG)
(2)2-6x5-0 SH (SG)
(3) 3-0 x 5-6 SH
MST224
36'-0"
19'-0"
55'-0"

WALL AA:

Story Shear due to Wind: $V_{3W} = 11918.38\,lb$ Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$ Distance between shear walls: $L_1 := 55\cdot ft$

Shear Wall Length: $Laa := \left[3.083\left(\frac{6.17}{9}\right) + 3.667\left(\frac{7.33}{9}\right) + 2.75\left(\frac{5.5}{9}\right) + 9.917\right]ft = 16.7\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vaa := \dfrac{\frac{0.6V_{3W}}{L_t}\cdot\frac{L_1}{2}}{Laa}$ Seismic Force: $\rho := 1.0$ $E_{aa} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Laa}$

$vaa = 214.13\,ft^{-1}\cdot lb$ $\frac{vaa}{C_o} = 214.13\,ft^{-1}\cdot lb$ $E_{aa} = 267.96\,ft^{-1}\cdot lb$ $\frac{E_{aa}}{C_o} = 267.96\,ft^{-1}\cdot lb$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 495 plf
Seismic Capacity = 353 plf

Dead Load Resisting Overturning: $L_{aa} := 2.75\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{aa} := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRaa := \frac{W_{aa}\cdot L_{aa}}{2}$ $DLRaa = 165\,lb$

Chord Force:

$CFaa_w := \frac{vaa\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_w = 1927.19\,lb$ $CFaa_s := \frac{E_{aa}\cdot L_{aa}\cdot Pt}{C_o\cdot L_{aa}}$ $CFaa_s = 2411.6\,lb$

Holdown Force:

$HDFaa_w := CFaa_w - 0.6\cdot DLRaa = 1828.19\,lb$ $HDFaa_s := CFaa_s - (0.6 - 0.14S_{DS})DLRaa = 2338.81\,lb$

Simpson MSTC28 to flush beam

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_p := \frac{(Z_N\cdot C_D\cdot C_o)}{vaa} = 0.76\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{aa}} = 0.61\,ft$

16d @ 6" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vaa} = 6.43\,ft$ $\frac{(Z_B\cdot C_o)}{E_{aa}} = 5.14\,ft$

5/8" A.B. @ 60" o.c.

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WALL BB:

Story Shear due to Wind: $V_{3W} = 11918.38\,lb$ Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$ Distance between shear walls: $L_1 := 55 \cdot ft$

Shear Wall Length: $Lbb := \left[2 \cdot 3 + 2 \cdot 2.5 \left(\frac{5}{5.5}\right)\right] ft = 10.55\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vbb := \frac{\frac{0.6 V_{3W}}{L_t} \cdot \frac{L_1}{2}}{Lbb}$ Seismic Force: $\rho := 1.0$ $E_{bb} := \frac{\rho \cdot \frac{0.7 F_1}{L_t} \cdot \frac{L_1}{2}}{Lbb}$

$vbb = 339.06\,ft^{-1} \cdot lb$ $\frac{vbb}{C_o} = 339.06\,ft^{-1} \cdot lb$ $E_{bb} = 424.28\,ft^{-1} \cdot lb$ $\frac{E_{bb}}{C_o} = 424.28\,ft^{-1} \cdot lb$

P1-3: 7/16" Sheathing w/ 8d nails @ 3" O.C.
Wind Capacity = 638 plf
Seismic Capacity = 456 plf

Dead Load Resisting Overturning: $L_{bb} := 12 \cdot ft$ Plate Height: $Pt := 9 \cdot ft$

$W_{bb} := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10 psf) \cdot 0 ft$ $DLRbb := \frac{W_{bb} \cdot L_{bb}}{2}$ $DLRbb = 720\,lb$

Chord Force:

$CFbb_w := \frac{vbb \cdot 6 ft \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_w = 1525.76\,lb$ $CFbb_s := \frac{E_{bb} \cdot 6 ft \cdot Pt}{C_o \cdot L_{bb}}$ $CFbb_s = 1909.26\,lb$

Holdown Force:

$HDFbb_w := CFbb_w - 0.6 \cdot DLRbb = 1093.76\,lb$ $HDFbb_s := CFbb_s - (0.6 - 0.14 S_{DS}) \cdot DLRbb = 1591.64\,lb$

Simpson MSTC40

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_N := \frac{(C_D \cdot Z_N \cdot C_o)}{vbb} = 0.48\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_{bb}} = 0.38\,ft$

16d @ 4" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vbb} = 4.06\,ft$ $\frac{(Z_B \cdot C_o)}{E_{bb}} = 3.24\,ft$

5/8" A.B. @ 36" o.c.

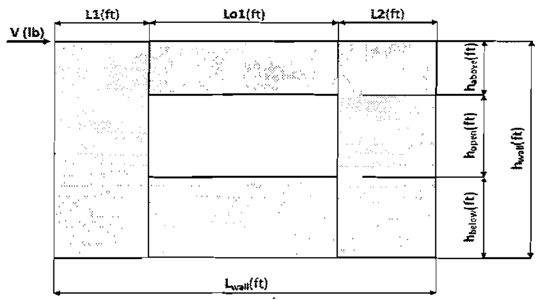
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2015 IBC/IRC	**Date:**	7/7/2021
Designer:	Mark Myers, PE		
Client:	ACH		
Project:	4250 89th AVE SE		
Wall Line:	BB at Bedroom 2		



Input Variables

				Wall Pier Aspect Ratio		Adj. Factor
V	2550 lbf	Opening 1		P1=ho1/L1=	1.83	N/A
h_{wall}	9.00 ft	ha1	1.00 ft	P2=ho1/L2=	1.83	N/A
L1	3.00 ft	ho1	5.50 ft			
L2	3.00 ft	hb1	2.50 ft			
L_{wall}	12.00 ft	Lo1	6.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ — 1913 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) =	546 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	3279 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	1639 lbf
F2 = O1(L2)/(L1+L2) =	1639 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.00 ft
T2 = (L2*Lo1)/(L1+L2) =	3.00 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 =	425 plf
V2 = (V/L)(T2+L2)/L2 =	425 plf
Check V1*L1+V2*L2=V?	2550 lbf **OK**

7. Resistance to corner forces

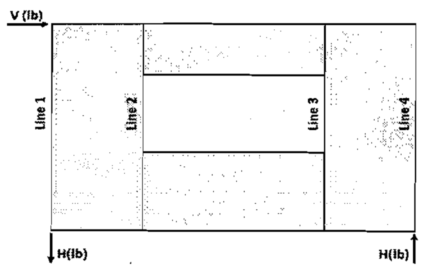
R1 = V1*L1 =	1275 lbf
R2 = V2*L2 =	1275 lbf

8. Difference corner force + resistance

R1-F1 =	-364 lbf
R2-F2 =	-364 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-121 plf
vc2 = (R2-F2)/L2 =	-121 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-425	2338	1913 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1913	-425	2338	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-425	2338	1913 lbf

Design Summary

Req. Sheathing Capacity	425 lbf	**4-Term Deflection**	0.517 in.	**3-Term Deflection**	0.544 in.
Req. Strap Force	1639 lbf	**4-Term Story Drift %**	0.019 %	**3-Term Story Drift %**	0.020 %
Req. HD Force	1913 plf		See Page 2		See Page 3

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APA Disclaimer

The information contained herein is intended for use as a resource to aid in the shear wall design based on APA – The Engineered Wood Association's testing and knowledge of wood-framed shear wall system design utilizing the force transfer around openings (FTAO) methodology. Neither APA, nor its member manufacturers, make any warranty, expressed or implied, or assume any legal liability or responsibility for the accuracy, use, application of, and/or reference to opinions, findings, conclusions, or recommendations included in this calculator. Consult your local jurisdiction or design professional to assure compliance with code, construction, and performance requirements. Because APA has no control over quality of workmanship or the conditions under which engineered wood products are used, it cannot accept responsibility of product performance or designs as actually constructed.

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	BB at Bedroom 2	

Deflection Calculation Input Variables

Sheathing:

OSB	Sheathing Material
7/16	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Hem-Fir	
E:	1.20E+06	(psi)
	Qty	Stud Size
Dimensions:	2	2x6
A:	16.5	(in.^{2})
A Override:		(in.^{2})

Nails: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2325	2325	(lbf)
HD Deflection:	0.0625	0.0625	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	425	425	425	425	(plf)
$v_{strength}$:	607	607	607	607	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	9.00	6.50	6.50	9.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
Vn:	152	152	152	152	(plf)
e:	0.0146	0.0146	0.0146	0.0146	(in.)
b:	3.00	3.00	3.00	3.00	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.060	0.065	0.098	0.441	0.022	0.047	0.071	0.230
		Sum	0.664			Sum	0.371
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.022	0.047	0.071	0.230	0.060	0.065	0.098	0.441
		Sum	0.371			Sum	0.664

Total Defl.	
0.517	(in.)
0.0192	%drift

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	BB at Bedroom 2	

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	425	425	425	425	(plf)
$v_{strength}$:	607	607	607	607	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	9.00	6.50	6.50	9.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	28.0	28.0	28.0	28.0	(kips/in.)
b:	3.00	3.00	3.00	3.00	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.060	0.195	0.441	0.022	0.141	0.230
	Sum	0.695		Sum	0.393
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.022	0.141	0.230	0.060	0.195	0.441
	Sum	0.393		Sum	0.695

Total Defl.	
0.544	(in.)
0.0202	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

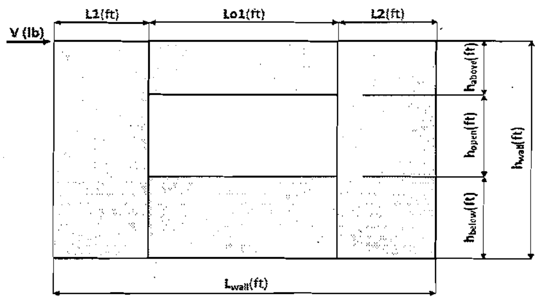
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2015 IBC/IRC	**Date:**	7/7/2021
Designer:	Mark Myers, PE		
Client:	ACH		
Project:	4250 89th AVE SE		
Wall Line:	BB at Bedroom 3		



Input Variables

					Wall Pier Aspect Ratio		Adj. Factor
V	1930 lbf		Opening 1				
h_{wall}	9.00 ft	ha1	1.00 ft		P1=ho1/L1=	2.20	0.9750
L1	2.50 ft	ho1	5.50 ft		P2=ho1/L2=	2.20	0.9750
L2	2.50 ft	hb1	2.50 ft				
L_{wall}	14.00 ft	Lo1	9.00 ft				

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1241 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 354 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3190 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1595 lbf

F2 = O1(L2)/(L1+L2) = 1595 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.50 ft

T2 = (L2*Lo1)/(L1+L2) = 4.50 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 386 plf

V2 = (V/L)(T2+L2)/L2 = 386 plf

Check V1*L1+V2*L2=V? 1930 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 965 lbf

R2 = V2*L2 = 965 lbf

8. Difference corner force + resistance

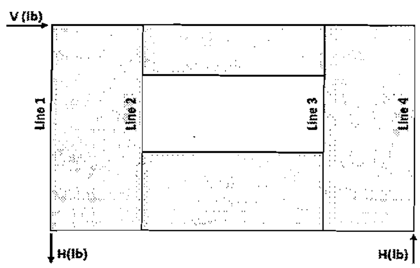
R1-F1 = -630 lbf

R2-F2 = -630 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -252 plf

vc2 = (R2-F2)/L2 = -252 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-882	2123	1241 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1241	-882	2123	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-882	2123	1241 lbf

Design Summary

Req. Sheathing Capacity	396 lbf	**4-Term Deflection**	0.525 in.	**3-Term Deflection**	0.563 in.
Req. Strap Force	1595 lbf	**4-Term Story Drift %**	0.019 %	**3-Term Story Drift %**	0.021 %
Req. HD Force	1241 plf		See Page 2		See Page 3

Req. Sheathing Capacity has been adjusted per the Aspect Ratio Factor in SDPWS 4.3.4.2

CS16

Project Information

Code:	2015 IBC/IRC	Date: 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	BB at Bedroom 3	

Deflection Calculation Input Variables

Sheathing:

OSB	Sheathing Material
7/16	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Hem-Fir	
E:	1.20E+06	(psi)
	Qty	Stud Size
Dimensions:	2	2x6
A:	16.5	(in.^{2})
A Override:		(in.^{2})

Nails: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2325	2325	(lbf)
HD Deflection:	0.0625	0.0625	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	386	386	386	386	(plf)
$v_{strength}$:	551	551	551	551	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	9.00	6.50	6.50	9.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
Vn:	138	138	138	138	(plf)
e:	0.0109	0.0109	0.0109	0.0109	(in.)
b:	2.50	2.50	2.50	2.50	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.065	0.059	0.074	0.480	0.024	0.043	0.053	0.251
		Sum	0.678			Sum	0.371
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.024	0.043	0.053	0.251	0.065	0.059	0.074	0.480
		Sum	0.371			Sum	0.678

Total Defl.	
0.525	(in.)
0.0194	%drift

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	BB at Bedroom 3	

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	386	386	386	386	(plf)
$v_{strength}$:	551	551	551	551	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	9.00	6.50	6.50	9.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	28.0	28.0	28.0	28.0	(kips/in.)
b:	2.50	2.50	2.50	2.50	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.065	0.177	0.480	0.024	0.128	0.251
	Sum	0.722		Sum	0.403
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.024	0.128	0.251	0.065	0.177	0.480
	Sum	0.403		Sum	0.722

Total Defl.	
0.563	(in.)
0.0208	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

WALL CC:

Story Shear due to Wind: $V_{1W} = 11211.75\,lb$ Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$ Distance between shear walls: $L_1 := 19\cdot ft$

Shear Wall Length: $Lcc := (13.417 + 6.5)ft = 19.92\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vcc := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1}{2}}{Lcc}$

Seismic Force: $\rho := 1.0$ $E_{cc} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Lcc}$

$vcc = 58.34\,ft^{-1}\cdot lb$ $\frac{vcc}{C_o} = 58.34\,ft^{-1}\cdot lb$

$E_{cc} = 77.6\,ft^{-1}\cdot lb$ $\frac{E_{cc}}{C_o} = 77.6\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{cc} := 6.5\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{cc} := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRcc := \frac{W_{cc}\cdot L_{cc}}{2}$ $DLRcc = 390\,lb$

Chord Force:

$CFcc_w := \frac{vcc\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_w = 525.05\,lb$

$CFcc_s := \frac{E_{cc}\cdot L_{cc}\cdot Pt}{C_o\cdot L_{cc}}$ $CFcc_s = 698.44\,lb$

Holdown Force:

$HDFcc_w := CFcc_w - 0.6DLRcc = 291.05\,lb$

$HDFcc_s := CFcc_s - (0.6 - 0.14S_{DS})\cdot DLRcc = 526.39\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vcc} = 2.8\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{cc}} = 2.1\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vcc} = 23.59\,ft$ $\frac{(Z_B\cdot C_o)}{E_{cc}} = 17.73\,ft$

5/8" A.B. @ 72" o.c.

Myers Engineering, LLC
3206 50th Street Ct NW, Ste 210-B
Gig Harbor, WA 98335

PROJECT : **4250 89th AVE SE**

Phone: 253-858-3248
Email: myengineer@centurytel.net

WALL DD:

Story Shear due to Wind: $V_{1W} = 11211.75\,\text{lb}$ Story Shear due to Seismic: $F_1 = 12783.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55\cdot ft$ Distance between shear walls: $L_1 := 12.67\cdot ft$

Shear Wall Length: $Ldd := (13)ft = 13\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vdd := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1}{2}}{Ldd}$ Seismic Force: $\rho := 1.0$ $E_{dd} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1}{2}}{Ldd}$

$vdd = 59.6\,ft^{-1}\cdot lb$ $\frac{vdd}{C_o} = 59.6\,ft^{-1}\cdot lb$ $E_{dd} = 79.28\,ft^{-1}\cdot lb$ $\frac{E_{dd}}{C_o} = 79.28\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{dd} := 13\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{dd} := (15\cdot psf)\cdot 2\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRdd := \frac{W_{dd}\cdot L_{dd}}{2}$ $DLRdd = 780\,\text{lb}$

Chord Force:

$CFdd_w := \frac{vdd\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_w = 536.42\,\text{lb}$ $CFdd_s := \frac{E_{dd}\cdot L_{dd}\cdot Pt}{C_o\cdot L_{dd}}$ $CFdd_s = 713.56\,\text{lb}$

Holdown Force:

$HDFdd_w := CFdd_w - 0.6DLRdd = 68.42\,\text{lb}$

$HDFdd_s := CFdd_s - (0.6 - 0.14S_{DS})DLRdd = 369.47\,\text{lb}$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vdd} = 2.74\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{dd}} = 2.06\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,\text{lb}$

$As := \frac{(Z_B\cdot C_o)}{vdd} = 23.09\,ft$ $\frac{(Z_B\cdot C_o)}{E_{dd}} = 17.36\,ft$

5/8" A.B. @ 72" o.c.

WALL EE:

Story Shear due to Wind: $V_{1W} = 11211.75\,\text{lb}$ Story Shear due to Seismic: $F_1 = 12783.52\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55\cdot\text{ft}$ Distance between shear walls: $L_1 := 19\cdot\text{ft}$ $L_2 := 14\text{ft}$

Shear Wall Length: $\text{Lee} := (7.5 + 6 + 5)\text{ft} = 18.5\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{10\cdot\text{ft}}{10\cdot\text{ft}}\right)\cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $\text{vee} := \dfrac{\dfrac{0.6V_{1W}}{L_t}\cdot\dfrac{L_1 + L_2}{2}}{\text{Lee}}$ Seismic Force: $\rho := 1.0$ $E_{ee} := \dfrac{\rho\cdot\dfrac{0.7F_1}{L_t}\cdot\dfrac{L_1 + L_2}{2}}{\text{Lee}}$

$\text{vee} = 109.09\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{\text{vee}}{C_o} = 109.09\,\text{ft}^{-1}\cdot\text{lb}$ $E_{ee} = 145.11\,\text{ft}^{-1}\cdot\text{lb}$ $\dfrac{E_{ee}}{C_o} = 145.11\,\text{ft}^{-1}\cdot\text{lb}$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{ee} := 5\cdot\text{ft}$ Plate Height: $\text{Pt} := 9\cdot\text{ft}$

$W_{ee} := (15\cdot\text{psf})\cdot 8\cdot\text{ft} + (10\cdot\text{psf})\cdot\text{Pt} + (10\text{psf})\cdot 0\text{ft}$ $\text{DLRee} := \dfrac{W_{ee}\cdot L_{ee}}{2}$ $\text{DLRee} = 525\,\text{lb}$

Chord Force:

$\text{CFee}_w := \dfrac{\text{vee}\cdot L_{ee}\cdot\text{Pt}}{C_o\cdot L_{ee}}$ $\text{CFee}_w = 981.79\,\text{lb}$ $\text{CFee}_s := \dfrac{E_{ee}\cdot L_{ee}\cdot\text{Pt}}{C_o\cdot L_{ee}}$ $\text{CFee}_s = 1305.99\,\text{lb}$

Holdown Force:

$\text{HDFee}_w := \text{CFee}_w - 0.6\cdot\text{DLRee} = 666.79\,\text{lb}$ $\text{HDFee}_s := \text{CFee}_s - (0.6 - 0.14S_{DS})\cdot\text{DLRee} = 1074.39\,\text{lb}$

Simpson MSTC40 to wall or MSTC28 to flush beam

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot\text{lb}$ $C_D := 1.6$

$B_R := \dfrac{(C_D\cdot Z_N\cdot C_o)}{\text{vee}} = 1.5\,\text{ft}$ $\dfrac{(C_D\cdot Z_N\cdot C_o)}{E_{ee}} = 1.12\,\text{ft}$

16d @ 12" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot\text{lb}$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,\text{lb}$

$A_s := \dfrac{(Z_B\cdot C_o)}{\text{vee}} = 12.61\,\text{ft}$ $\dfrac{(Z_B\cdot C_o)}{E_{ee}} = 9.48\,\text{ft}$

5/8" A.B. @ 72" o.c.

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PROJECT : **4250 89th AVE SE**

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WALL FF:

Story Shear due to Wind: $V_{1W} = 11211.75\,lb$

Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$

Distance between shear walls: $L_1 := 9.33\cdot ft$ $L_2 := 14ft$

Shear Wall Length: $Lff := (12)ft = 12\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vff := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lff}$

Seismic Force: $\rho := 1.0$ $E_{ff} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lff}$

$vff = 118.9\,ft^{-1}\cdot lb$ $\frac{vff}{C_o} = 118.9\,ft^{-1}\cdot lb$

$E_{ff} = 158.16\,ft^{-1}\cdot lb$ $\frac{E_{ff}}{C_o} = 158.16\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{ff} := 12\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{ff} := (15\cdot psf)\cdot 8\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$

$DLRff := \frac{W_{ff}\cdot L_{ff}}{2}$ $DLRff = 1260\,lb$

Chord Force:

$CFff_w := \frac{vff\cdot L_{ff}\cdot Pt}{C_o\cdot L_{ff}}$ $CFff_w = 1070.06\,lb$

$CFff_s := \frac{E_{ff}\cdot L_{ff}\cdot Pt}{C_o\cdot L_{ff}}$ $CFff_s = 1423.42\,lb$

Holdown Force:

$HDFff_w := CFff_w - 0.6\cdot DLRff = 314.06\,lb$

$HDFff_s := CFff_s - (0.6 - 0.14S_{DS})\cdot DLRff = 867.57\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_s := \frac{(C_D\cdot Z_N\cdot C_o)}{vff} = 1.37\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{ff}} = 1.03\,ft$

16d @ 12" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$AS := \frac{(Z_B\cdot C_o)}{vff} = 11.57\,ft$ $\frac{(Z_B\cdot C_o)}{E_{ff}} = 8.7\,ft$

5/8" A.B. @ 72" o.c.

WALL GG:

Story Shear due to Wind: $V_{1W} = 11211.75\,lb$ Story Shear due to Seismic: $F_1 = 12783.52\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$ Distance between shear walls: $L_1 := 9.33\cdot ft$ $L_2 := 12.67ft$

Shear Wall Length: $Lgg := (15)ft = 15\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$ Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vgg := \dfrac{\frac{0.6V_{1W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lgg}$ Seismic Force: $\rho := 1.0$ $E_{gg} := \dfrac{\rho\cdot\frac{0.7F_1}{L_t}\cdot\frac{L_1+L_2}{2}}{Lgg}$

$vgg = 89.69\,ft^{-1}\cdot lb$ $\frac{vgg}{C_o} = 89.69\,ft^{-1}\cdot lb$ $E_{gg} = 119.31\,ft^{-1}\cdot lb$ $\frac{E_{gg}}{C_o} = 119.31\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_{gg} := 12\cdot ft$ Plate Height: $Pt := 9\cdot ft$

$W_{gg} := (15\cdot psf)\cdot 8\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 0ft$ $DLRgg := \frac{W_{gg}\cdot L_{gg}}{2}$ $DLRgg = 1260\,lb$

Chord Force:

$CFgg_w := \frac{vgg\cdot L_{gg}\cdot Pt}{C_o\cdot L_{gg}}$ $CFgg_w = 807.25\,lb$ $CFgg_s := \frac{E_{gg}\cdot L_{gg}\cdot Pt}{C_o\cdot L_{gg}}$ $CFgg_s = 1073.82\,lb$

Holdown Force:

$HDFgg_w := CFgg_w - 0.6\cdot DLRgg = 51.25\,lb$ $HDFgg_s := CFgg_s - (0.6 - 0.14S_{DS})\cdot DLRgg = 517.97\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

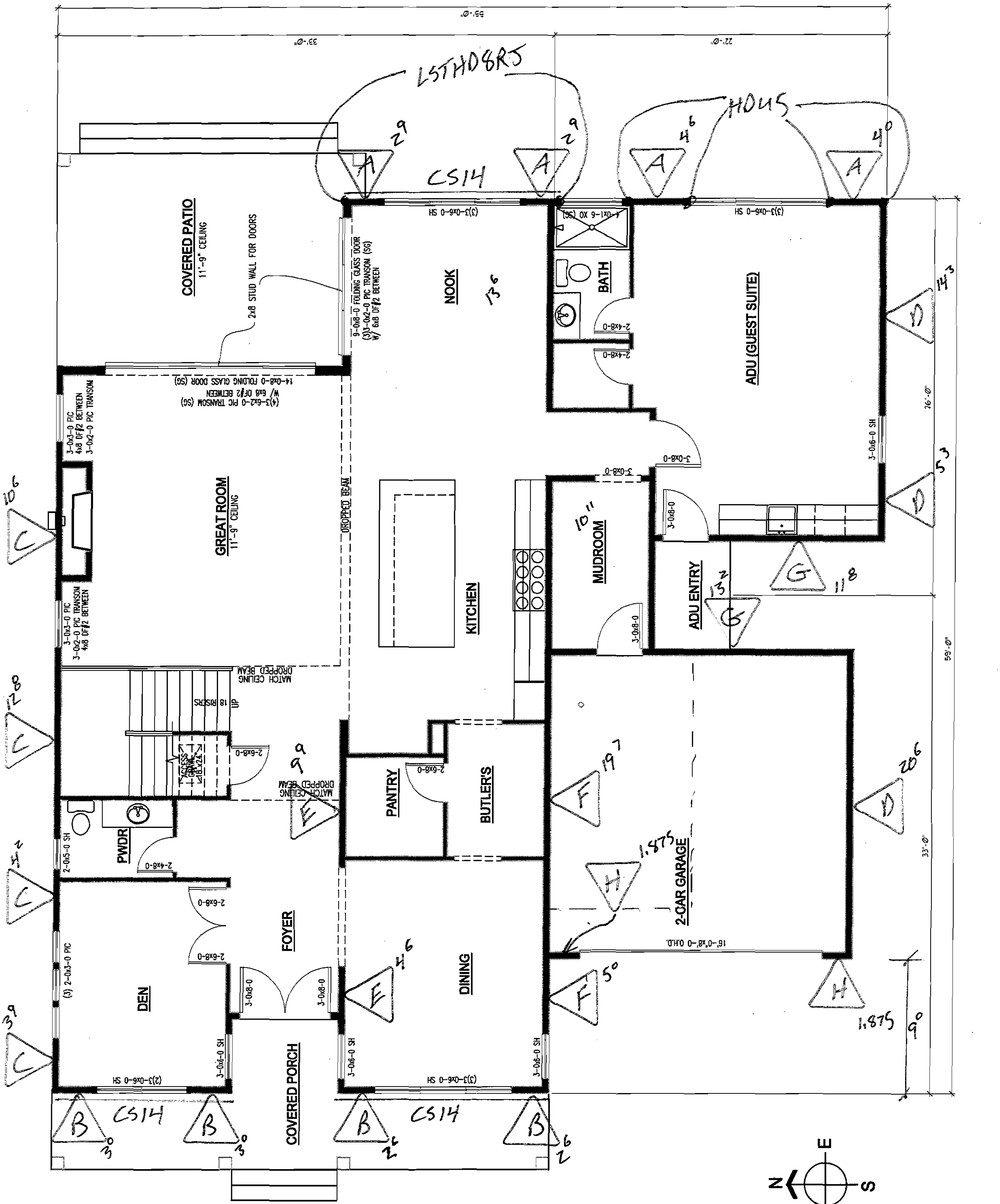
$Z_N := 102\cdot lb$ $C_D := 1.6$
$B_N := \frac{(C_D\cdot Z_N\cdot C_o)}{vgg} = 1.82\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_{gg}} = 1.37\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$
$As := \frac{(Z_B\cdot C_o)}{vgg} = 15.34\,ft$ $\frac{(Z_B\cdot C_o)}{E_{gg}} = 11.53\,ft$

5/8" A.B. @ 72" o.c.


55'-0"
33'-0"
22'-0"
LSTHD8RJ
HDU5
CS14
COVERED PATIO
11'-9" CEILING
2x8 STUD WALL FOR DOORS
9-0x8-0 FOLDING GLASS DOOR
(3)3-0x2-0 PIC TRANSOM (SG)
W/ 6x8 DF#2 BETWEEN
14-0x8-0 FOLDING GLASS DOOR (SG)
(4)3-6x2-0 PIC TRANSOM (SG)
W/ 6x8 DF#2 BETWEEN
NOOK
(3)3-0x6-0 SH
4-0x1-6 XO (SG)
BATH
2-4x8-0
ADU (GUEST SUITE)
26'-0"
3-0x6-0 SH
3-0x8-0
GREAT ROOM
11'-9" CEILING
3-0x3-0 PIC
3-0x2-0 PIC TRANSOM
4x8 DF#2 BETWEEN
DROPPED BEAM
KITCHEN
MUDROOM
ADU ENTRY
MATCH CEILING
DROPPED BEAM
18 RISERS
UP
ACCESS
2-6x8-0
PANTRY
BUTLER'S
PWDR
2-0x5-0 SH
2-4x8-0
FOYER
DEN
(3) 2-0x3-0 PIC
DINING
2-CAR GARAGE
16'-0"x8'-0" O.H.D.
59'-0"
33'-0"
9'-0"
1.875
(2)3-0x6-0 SH
COVERED PORCH
E
N
S
W

21

WALL A:

Story Shear due to Wind: $V_{4W} = 13229.62\,lb$ Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 59 \cdot ft$ Distance between shear walls: $L_1 := 26 \cdot ft$

Shear Wall Length: $La := \left[2 \cdot 2.75\left(\frac{5.5}{6}\right) + 4.5\left(\frac{9}{10}\right) + 4\left(\frac{8}{10}\right)\right]ft = 12.29\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100 \quad \% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $va := \dfrac{vaa \cdot Laa + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{La}$ Seismic Force: $\rho := 1.0 \quad E_a := \dfrac{E_{aa} \cdot Laa + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{La}$

$va = 433.18\,ft^{-1} \cdot lb \quad \frac{va}{C_o} = 433.18\,ft^{-1} \cdot lb$ $E_a = 489.89\,ft^{-1} \cdot lb \quad \frac{E_a}{C_o} = 489.89\,ft^{-1} \cdot lb$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 833 plf
Seismic Capacity = 595 plf

Dead Load Resisting Overturning: $L_a := 4 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_a := (15 \cdot psf) \cdot 2 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 0ft$ $DLRa := \frac{W_a \cdot L_a}{2} \quad DLRa = 260\,lb$

Chord Force:

$CFa_w := \frac{va \cdot L_a \cdot Pt}{C_o \cdot L_a} \quad CFa_w = 4331.81\,lb$ $CFa_s := \frac{E_a \cdot L_a \cdot Pt}{C_o \cdot L_a} \quad CFa_s = 4898.85\,lb$

Holdown Force:

$HDFa_w := CFa_w - 0.6 \cdot DLRa = 4175.81\,lb$ $HDFa_s := CFa_s - (0.6 - 0.14S_{DS}) \cdot DLRa = 4784.15\,lb$

Simpson HDU5 at DF post w/ SB5/8x24 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot lb \quad C_D := 1.6$

$B_N := \frac{(C_D \cdot Z_N \cdot C_o)}{va} = 0.38\,ft \quad \frac{(C_D \cdot Z_N \cdot C_o)}{E_a} = 0.33\,ft$

16d @ 3" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot lb \quad C_D := 1.6 \quad Z_B := A_s \cdot C_D \quad Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{va} = 3.18\,ft \quad \frac{(Z_B \cdot C_o)}{E_a} = 2.81\,ft$

5/8" A.B. @ 32" o.c.

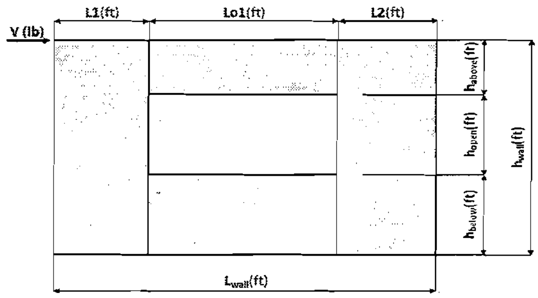
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2015 IBC/IRC	**Date:**	7/7/2021
Designer:	Mark Myers, PE		
Client:	ACH		
Project:	4250 89th AVE SE		
Wall Line:	A at Nook		



Input Variables

V	2695 lbf		Opening 1	Wall Pier Aspect Ratio		Adj. Factor
h_{wall}	10.00 ft	ha1	2.00 ft	P1=ho1/L1=	2.18	0.9773
L1	2.75 ft	ho1	6.00 ft	P2=ho1/L2=	2.18	0.9773
L2	2.75 ft	hb1	2.00 ft			
L_{wall}	14.50 ft	Lo1	9.00 ft			

1. Hold-down forces: H = Vh$_{wall}$/L$_{wall}$ 1859 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 465 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 4182 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 2091 lbf

F2 = O1(L2)/(L1+L2) = 2091 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.50 ft

T2 = (L2*Lo1)/(L1+L2) = 4.50 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 490 plf

V2 = (V/L)(T2+L2)/L2 = 490 plf

Check V1*L1+V2*L2=V? 2695 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 1348 lbf

R2 = V2*L2 = 1348 lbf

8. Difference corner force + resistance

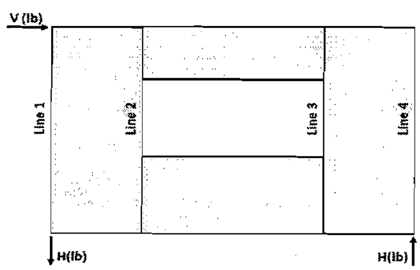
R1-F1 = -743 lbf

R2-F2 = -743 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -270 plf

vc2 = (R2-F2)/L2 = -270 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-1081	2940	1859 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1859	-1081	2940	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-1081	2940	1859 lbf

Design Summary

Req. Sheathing Capacity	501 lbf	**4-Term Deflection**	0.866 in.	**3-Term Deflection**	0.864 in.
Req. Strap Force	2091 lbf	**4-Term Story Drift %**	0.029 %	**3-Term Story Drift %**	0.029 %
Req. HD Force	1859 plf		See Page 2		See Page 3

Req. Sheathing Capacity has been adjusted per the Aspect Ratio Factor in SDPWS 4.3.4.2

LSTHD8/RJ
OR
HDU2

CS14

APA Disclaimer

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	A at Nook	

Deflection Calculation Input Variables

Sheathing:

OSB	Sheathing Material
7/16	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Hem-Fir		
E:	1.20E+06	(psi)	
	Qty	Stud Size	
Dimensions:	2	2x6	
A:	16.5	(in.^{2})	
A Override:		(in.^{2})	

Nails: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2325	2325	(lbf)
HD Deflection:	0.0625	0.0625	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	490	490	490	490	(plf)
$v_{strength}$:	700	700	700	700	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
Vn:	175	175	175	175	(plf)
e:	0.0224	0.0224	0.0224	0.0224	(in.)
b:	2.75	2.75	2.75	2.75	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.103	0.084	0.168	0.684	0.053	0.067	0.134	0.438
		Sum	1.039			Sum	0.692
Pier 2 (left)				Pier 2 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.053	0.067	0.134	0.438	0.103	0.084	0.168	0.684
		Sum	0.692			Sum	1.039

Total Defl.	
0.866	(in.)
0.0289	%drift

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	A at Nook	

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	490	490	490	490	(plf)
$v_{strength}$:	700	700	700	700	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	28.0	28.0	28.0	28.0	(kips/in.)
b:	2.75	2.75	2.75	2.75	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.103	0.250	0.684	0.053	0.200	0.438
	Sum	1.037		Sum	0.691
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.053	0.200	0.438	0.103	0.250	0.684
	Sum	0.691		Sum	1.037

Total Defl.	
0.864	(in.)
0.0288	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

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Gig Harbor, WA 98335

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Email: myengineer@centurytel.net

WALL B:

Story Shear due to Wind: $V_{4W} = 13229.62\,lb$ Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 59 \cdot ft$ Distance between shear walls: $L_1 := 9 \cdot ft$

Shear Wall Length: $Lb := \left[2 \cdot 3 + 2 \cdot 2.5\left(\frac{5}{6}\right)\right] ft = 10.17\,ft$

Percent full height sheathing: $\% := \left(\frac{10 \cdot ft}{10 \cdot ft}\right) \cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vb := \dfrac{vbb \cdot Lbb + \left(\frac{0.6V_{4W}}{L_t} \cdot \frac{L_1}{2}\right)}{Lb}$

Seismic Force: $\rho := 1.0$ $E_b := \dfrac{E_{bb} \cdot Lbb + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1}{2}\right)}{Lb}$

$vb = 411.24\,ft^{-1} \cdot lb$ $\frac{vb}{C_o} = 411.24\,ft^{-1} \cdot lb$

$E_b = 492.77\,ft^{-1} \cdot lb$ $\frac{E_b}{C_o} = 492.77\,ft^{-1} \cdot lb$

P1-2: 7/16" Sheathing w/ 8d nails @ 2" O.C.
Wind Capacity = 833 plf
Seismic Capacity = 595 plf

Dead Load Resisting Overturning: $L_b := 12 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_b := (15 \cdot psf) \cdot 2.5 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 1ft$ $DLRb := \frac{W_b \cdot L_b}{2}$ $DLRb = 885\,lb$

Chord Force:

$CFb_w := \frac{vb \cdot 6ft \cdot Pt}{C_o \cdot L_b}$ $CFb_w = 2056.2\,lb$
$CFb_w + CFbb_w = 3581.96\,lb$

$CFb_s := \frac{E_b \cdot 6ft \cdot Pt}{C_o \cdot L_b}$ $CFb_s = 2463.85\,lb$
$CFb_s + CFbb_s = 4373.11\,lb$

Holdown Force:

$HDFb_w := CFb_w - 0.6 \cdot DLRb = 1525.2\,lb$
$HDFb_w + HDFbb_w = 2618.96\,lb$

$HDFb_s := CFb_s - (0.6 - 0.14S_{DS}) \cdot DLRb = 2073.43\,lb$
$HDFb_s + HDFbb_s = 3665.07\,lb$

Simpson HDU5 w/ SB5/8x24 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vb} = 0.4\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_b} = 0.33\,ft$

16d @ 3" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vb} = 3.35\,ft$ $\frac{(Z_B \cdot C_o)}{E_b} = 2.79\,ft$

5/8" A.B. @ ~~24~~" o.c. 32"

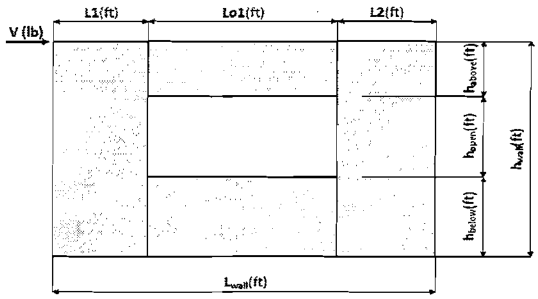
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Den	



Input Variables

				Opening 1	Wall Pier Aspect Ratio		Adj. Factor
V	2958 lbf		ha1	2.00 ft	P1=ho1/L1=	2.00	N/A
h_{wall}	10.00 ft		ho1	6.00 ft	P2=ho1/L2=	2.00	N/A
L1	3.00 ft		hb1	2.00 ft			
L2	3.00 ft		Lo1	6.00 ft			
L_{wall}	12.00 ft						

1. Hold-down forces: H = Vh_{wall}/L_{wall} 2465 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 616 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3698 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1849 lbf

F2 = O1(L2)/(L1+L2) = 1849 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.00 ft

T2 = (L2*Lo1)/(L1+L2) = 3.00 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 493 plf

V2 = (V/L)(T2+L2)/L2 = 493 plf

Check V1*L1+V2*L2=V? 2958 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 1479 lbf

R2 = V2*L2 = 1479 lbf

8. Difference corner force + resistance

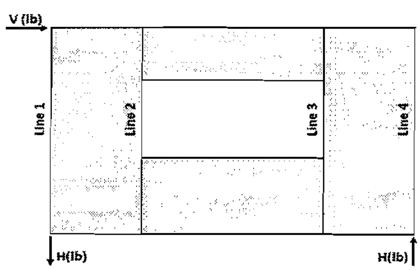
R1-F1 = -370 lbf

R2-F2 = -370 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -123 plf

vc2 = (R2-F2)/L2 = -123 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-493	2958	2465 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	2465	-493	2958	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-493	2958	2465 lbf

Design Summary

Req. Sheathing Capacity	493 lbf	**4-Term Deflection**	0.819 in.	**3-Term Deflection**	0.816 in.
Req. Strap Force	1849 lbf	**4-Term Story Drift %**	0.027 %	**3-Term Story Drift %**	0.027 %
Req. HD Force	2465 plf		See Page 2		See Page 3

CS14

APA Disclaimer

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Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Den	

Deflection Calculation Input Variables

Sheathing:

OSB	Sheathing Material
7/16	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Hem-Fir	
E:	1.20E+06	(psi)
	Qty	Stud Size
Dimensions:	2	2x6
A:	16.5	(in.^{2})
A Override:		(in.^{2})

Nails: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2325	2325	(lbf)
HD Deflection:	0.0625	0.0625	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	493	493	493	493	(plf)
$v_{strength}$:	704	704	704	704	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
Vn:	176	176	176	176	(plf)
e:	0.0228	0.0228	0.0228	0.0228	(in.)
b:	3.00	3.00	3.00	3.00	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.095	0.084	0.171	0.631	0.049	0.067	0.137	0.404
		Sum	0.982			Sum	0.657
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.049	0.067	0.137	0.404	0.095	0.084	0.171	0.631
		Sum	0.657			Sum	0.982

Total Defl.	
0.819	(in.)
0.0273	%drift

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Den	

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	493	493	493	493	(plf)
$v_{strength}$:	704	704	704	704	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	28.0	28.0	28.0	28.0	(kips/in.)
b:	3.00	3.00	3.00	3.00	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.095	0.252	0.631	0.049	0.201	0.404
	Sum	0.977		Sum	0.654
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.049	0.201	0.404	0.095	0.252	0.631
	Sum	0.654		Sum	0.977

Total Defl.	
0.816	(in.)
0.0272	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

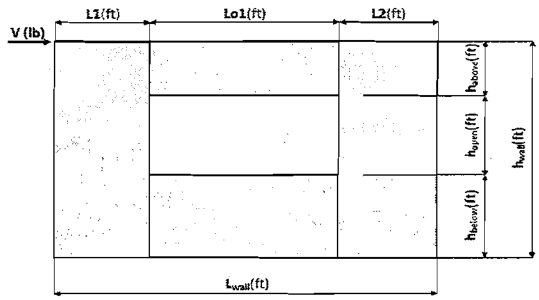
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Dining	



Input Variables

				Wall Pier Aspect Ratio		Adj. Factor
		Opening 1				
V	2465 lbf					
h_{wall}	10.00 ft	ha1	2.00 ft	P1=ho1/L1=	2.40	0.9500
L1	2.50 ft	ho1	6.00 ft	P2=ho1/L2=	2.40	0.9500
L2	2.50 ft	hb1	2.00 ft			
L_{wall}	14.00 ft	Lo1	9.00 ft			

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1761 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 440 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3962 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1981 lbf
F2 = O1(L2)/(L1+L2) = 1981 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.50 ft
T2 = (L2*Lo1)/(L1+L2) = 4.50 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 493 plf
V2 = (V/L)(T2+L2)/L2 = 493 plf
Check V1*L1+V2*L2=V? 2465 lbf **OK**

7. Resistance to corner forces

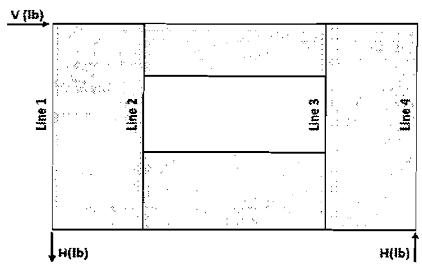
R1 = V1*L1 = 1233 lbf
R2 = V2*L2 = 1233 lbf

8. Difference corner force + resistance

R1-F1 = -748 lbf
R2-F2 = -748 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -299 plf
vc2 = (R2-F2)/L2 = -299 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-1197	2958	1761 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1761	-1197	2958	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-1197	2958	1761 lbf

Design Summary

Req. Sheathing Capacity	519 lbf	**4-Term Deflection**	0.937 in.	**3-Term Deflection**	0.933 in.
Req. Strap Force	1981 lbf	**4-Term Story Drift %**	0.031 %	**3-Term Story Drift %**	0.031 %
Req. HD Force	1761 plf		See Page 2		See Page 3

Req. Sheathing Capacity has been adjusted per the Aspect Ratio Factor in SDPWS 4.3.4.2

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Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Dining	

Deflection Calculation Input Variables

Sheathing:

OSB	Sheathing Material
7/16	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Hem-Fir	
E:	1.20E+06	(psi)
	Qty	Stud Size
Dimensions:	2	2x6
A:	16.5	(in.^{2})
A Override:		(in.^{2})

Nails: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2325	2325	(lbf)
HD Deflection:	0.0625	0.0625	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_a + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	493	493	493	493	(plf)
$v_{strength}$:	704	704	704	704	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
Vn:	176	176	176	176	(plf)
e:	0.0228	0.0228	0.0228	0.0228	(in.)
b:	2.50	2.50	2.50	2.50	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.114	0.084	0.171	0.757	0.058	0.067	0.137	0.485
		Sum	1.127			Sum	0.747
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.058	0.067	0.137	0.485	0.114	0.084	0.171	0.757
		Sum	0.747			Sum	1.127

Total Defl.	
0.937	(in.)
0.0312	%drift

Project Information

Code:	2015 IBC/IRC	**Date:** 7/7/2021
Designer:	Mark Myers, PE	
Client:	ACH	
Project:	4250 89th AVE SE	
Wall Line:	B at Dining	

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	7/16	7/16	.7/16	7/16	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	493	493	493	493	(plf)
$v_{strength}$:	704	704	704	704	(plf)
E:	1.20E+06	1.20E+06	1.20E+06	1.20E+06	(psi)
h:	10.00	8.00	8.00	10.00	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	28.0	28.0	28.0	28.0	(kips/in.)
b:	2.50	2.50	2.50	2.50	(ft)
HD Capacity:	2325	2325	2325	2325	(lbf)
HD Defl:	0.0625	0.0625	0.0625	0.0625	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.114	0.252	0.757	0.058	0.201	0.485
	Sum	1.123		Sum	0.744
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.058	0.201	0.485	0.114	0.252	0.757
	Sum	0.744		Sum	1.123

Total Defl.	
0.933	(in.)
0.0311	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

WALL C:

Story Shear due to Wind: $V_{2W} = 12110.19\,lb$

Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$

Distance between shear walls: $L_1 := 19\cdot ft$

Shear Wall Length: $Lc := \left[3.75\left(\frac{7.5}{10}\right) + 4.17\left(\frac{8.33}{10}\right) + 12.67 + 10.5\right]ft = 29.46\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100 \qquad \% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vc := \dfrac{vcc\cdot Lcc + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Lc}$

Seismic Force: $\rho := 1.0 \qquad E_c := \dfrac{E_{cc}\cdot Lcc + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Lc}$

$vc = 82.05\,ft^{-1}\cdot lb \qquad \frac{vc}{C_o} = 82.05\,ft^{-1}\cdot lb$

$E_c = 93.65\,ft^{-1}\cdot lb \qquad \frac{E_c}{C_o} = 93.65\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_c := 3.75\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_c := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 6ft$

$DLRc := \frac{W_c\cdot L_c}{2} \qquad DLRc = 300\,lb$

Chord Force:

$CFc_w := \frac{vc\cdot L_c\cdot Pt}{C_o\cdot L_c} \qquad CFc_w = 820.54\,lb \qquad CFc_w + CFcc_w = 1345.6\,lb$

$CFc_s := \frac{E_c\cdot L_c\cdot Pt}{C_o\cdot L_c} \qquad CFc_s = 936.5\,lb \qquad CFc_s + CFcc_s = 1634.94\,lb$

Holdown Force:

$HDFc_w := CFc_w - 0.6\cdot DLRc = 640.54\,lb$

$HDFc_s := CFc_s - (0.6 - 0.14S_{DS})\cdot DLRc = 804.16\,lb$

No Holdown Required

$HDFc_w + HDFcc_w = 931.6\,lb$

$HDFc_s + HDFcc_s = 1330.55\,lb$

Simpson LSTHD8RJ

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb \quad C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vc} = 1.99\,ft \qquad \frac{(C_D\cdot Z_N\cdot C_o)}{E_c} = 1.74\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb \quad C_D := 1.6 \quad Z_B := A_s\cdot C_D \quad Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vc} = 16.77\,ft \qquad \frac{(Z_B\cdot C_o)}{E_c} = 14.69\,ft$

5/8" A.B. @ 72" o.c.

WALL D:

Story Shear due to Wind: $V_{2W} = 12110.19\,lb$ Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 55\cdot ft$ Distance between shear walls: $L_1 := 22\cdot ft$

Shear Wall Length: $Ld := (5.25 + 14.25 + 20.5)ft = 40\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force:

$$vd := \frac{\frac{10}{22}(vgg\cdot Lgg) + vdd\cdot Ldd + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1}{2}\right)}{Ld}$$

$vd = 70.99\,ft^{-1}\cdot lb$ $\frac{vd}{C_o} = 70.99\,ft^{-1}\cdot lb$

Seismic Force: $\rho := 1.0$

$$E_d := \frac{\frac{10}{22}\left(E_{gg}\cdot Lgg\right) + E_{dd}\cdot Ldd + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1}{2}\right)}{Ld}$$

$E_d = 81.22\,ft^{-1}\cdot lb$ $\frac{E_d}{C_o} = 81.22\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_d := 5.25\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_d := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 8ft$

$DLRd := \frac{W_d\cdot L_d}{2}$ $DLRd = 472.5\,lb$

Chord Force:

$CFd_w := \frac{vd\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_w = 709.9\,lb$

$CFd_s := \frac{E_d\cdot L_d\cdot Pt}{C_o\cdot L_d}$ $CFd_s = 812.16\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = 426.4\,lb$

$HDFd_s := CFd_s - \left(0.6 - 0.14S_{DS}\right)\cdot DLRd = 603.72\,lb$

No Holdown Required

Dead Load Resisting Overturning: $L_d := 14.25 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_d := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 8ft$

$DLRd := \frac{W_d \cdot L_d}{2}$ $DLRd = 1282.5\,lb$

Chord Force:

$CFd_w := \frac{vd \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_w = 709.9\,lb$

$CFd_w + CFdd_w = 1246.32\,lb$

$CFd_s := \frac{E_d \cdot L_d \cdot Pt}{C_o \cdot L_d}$ $CFd_s = 812.16\,lb$

$CFd_s + CFdd_s = 1525.72\,lb$

Holdown Force:

$HDFd_w := CFd_w - 0.6DLRd = -59.6\,lb$

$HDFd_w + HDFdd_w = 8.82\,lb$

$HDFd_s := CFd_s - (0.6 - 0.14S_{DS}) \cdot DLRd = 246.39\,lb$

$HDFd_s + HDFdd_s = 615.86\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D \cdot Z_N \cdot C_o)}{vd} = 2.3\,ft$ $\frac{(C_D \cdot Z_N \cdot C_o)}{E_d} = 2.01\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B \cdot C_o)}{vd} = 19.38\,ft$ $\frac{(Z_B \cdot C_o)}{E_d} = 16.94\,ft$

5/8" A.B. @ 72" o.c.

WALL E:

Story Shear due to Wind: $V_{2W} = 12110.19\,\text{lb}$ Story Shear due to Seismic: $F_2 = 10031.78\,\text{lb}$

Bldg Width in direction of Load: $L_t := 55\cdot\text{ft}$ Distance between shear walls: $L_1 := 19\cdot\text{ft}$ $L_2 := 14\text{ft}$

Shear Wall Length: $Le := \left[9.75 + 4.5\left(\frac{9}{10}\right)\right]\text{ft} = 13.8\,\text{ft}$

Percent full height sheathing: $\% := \left(\frac{10\cdot\text{ft}}{10\cdot\text{ft}}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$
per AF&PA SDPWS Table 4.3.3.5

Wind Force: $ve := \dfrac{vee\cdot Lee + \left(\frac{0.6V_{2W}}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Le}$

Seismic Force: $\rho := 1.0$ $E_e := \dfrac{E_{ee}\cdot Lee + \left(\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}\right)}{Le}$

$ve = 304.2\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{ve}{C_o} = 304.2\,\text{ft}^{-1}\cdot\text{lb}$ $E_e = 347.19\,\text{ft}^{-1}\cdot\text{lb}$ $\frac{E_e}{C_o} = 347.19\,\text{ft}^{-1}\cdot\text{lb}$

P1-4: 7/16" Sheathing w/ 8d nails @ 4" O.C.
Wind Capacity = 495 plf
Seismic Capacity = 353 plf

Dead Load Resisting Overturning: $L_e := 4.5\cdot\text{ft}$ Plate Height: $Pt := 10\cdot\text{ft}$

$W_e := (15\cdot\text{psf})\cdot 0\cdot\text{ft} + (10\cdot\text{psf})\cdot Pt + (10\text{psf})\cdot 10\text{ft}$

$DLRe := \frac{W_e\cdot L_e}{2}$ $DLRe = 450\,\text{lb}$

Chord Force:

$CFe_w := \frac{ve\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_w = 3041.99\,\text{lb}$

$CFe_s := \frac{E_e\cdot L_e\cdot Pt}{C_o\cdot L_e}$ $CFe_s = 3471.89\,\text{lb}$

Holdown Force:

$HDFe_w := CFe_w - 0.6\cdot DLRe = 2771.99\,\text{lb}$

$HDFe_s := CFe_s - (0.6 - 0.14S_{DS})\cdot DLRe = 3273.38\,\text{lb}$

Simpson HDU4 w/ SB5/8x24 or PAB5 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot\text{lb}$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{ve} = 0.54\,\text{ft}$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_e} = 0.47\,\text{ft}$

16d @ 4" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot\text{lb}$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,\text{lb}$

$A_s := \frac{(Z_B\cdot C_o)}{ve} = 4.52\,\text{ft}$ $\frac{(Z_B\cdot C_o)}{E_e} = 3.96\,\text{ft}$

5/8" A.B. @ 48" o.c.

WALL F:

Story Shear due to Wind: $V_{2W} = 12110.19\,lb$

Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 55 \cdot ft$

Distance between shear walls: $L_1 := 14 \cdot ft$ $L_2 := 22ft$

Shear Wall Length: $Lf := (5 + 19.58)ft = 24.58\,ft$

Percent full height sheathing: $\% := \left(\frac{24.58 \cdot ft}{25.08 \cdot ft}\right) \cdot 100$ $\% = 98.01$

Max Opening Height = 10ft-0in, Therefore $C_o := 0.95$
per AF&PA SDPWS Table 4.3.3.5

Wind Force:

$$vf := \frac{vff \cdot Lff + \frac{13}{22}(vgg \cdot Lgg) + \left(\frac{0.6V_{2W}}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lf}$$

$vf = 187.13\,ft^{-1} \cdot lb$ $\frac{vf}{C_o} = 196.98\,ft^{-1} \cdot lb$

Seismic Force: $\rho := 1.0$

$$E_f := \frac{E_{ff} \cdot Lff + \frac{13}{22}\left(E_{gg} \cdot Lgg\right) + \left(\rho \cdot \frac{0.7F_2}{L_t} \cdot \frac{L_1 + L_2}{2}\right)}{Lf}$$

$E_f = 213.74\,ft^{-1} \cdot lb$ $\frac{E_f}{C_o} = 224.98\,ft^{-1} \cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_f := 24.58 \cdot ft$ Plate Height: $Pt := 10 \cdot ft$

$W_f := (15 \cdot psf) \cdot 0 \cdot ft + (10 \cdot psf) \cdot Pt + (10psf) \cdot 7ft$

$DLRf := \frac{W_f \cdot L_f}{2}$ $DLRf = 2089.3\,lb$

Chord Force:

$CFf_w := \frac{vf \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_w = 1969.84\,lb$

$CFf_s := \frac{E_f \cdot L_f \cdot Pt}{C_o \cdot L_f}$ $CFf_s = 2249.85\,lb$

Holdown Force:

$HDFf_w := CFf_w - 0.6 \cdot DLRf = 716.26\,lb$

$HDFf_s := CFf_s - \left(0.6 - 0.14S_{DS}\right) \cdot DLRf = 1328.16\,lb$

Simpson LSTHD8/RJ or HDU2 w/ SSTB16 anchor

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102 \cdot lb$ $C_D := 1.6$

$B_R := \frac{\left(C_D \cdot Z_N \cdot C_o\right)}{vf} = 0.83\,ft$ $\frac{\left(C_D \cdot Z_N \cdot C_o\right)}{E_f} = 0.73\,ft$

16d @ 16" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860 \cdot lb$ $C_D := 1.6$ $Z_B := A_s \cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{\left(Z_B \cdot C_o\right)}{vf} = 6.99\,ft$ $\frac{\left(Z_B \cdot C_o\right)}{E_f} = 6.12\,ft$

5/8" A.B. @ 72" o.c.

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WALL G:

Story Shear due to Wind: $V_{4W} = 13229.62\,lb$ Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 59\cdot ft$ Distance between shear walls: $L_1 := 26\cdot ft$ $L_2 := 24ft$

Shear Wall Length: $Lg := (13.17 + 11.67)ft = 24.84\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vg := \dfrac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lg}$

Seismic Force: $\rho := 1.0$ $E_g := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lg}$

$vg = 135.41\,ft^{-1}\cdot lb$ $\frac{vg}{C_o} = 135.41\,ft^{-1}\cdot lb$ $E_g = 119.79\,ft^{-1}\cdot lb$ $\frac{E_g}{C_o} = 119.79\,ft^{-1}\cdot lb$

P1-6: 7/16" Sheathing w/ 8d nails @ 6" O.C.
Wind Capacity = 339 plf
Seismic Capacity = 242 plf

Dead Load Resisting Overturning: $L_g := 11.67\cdot ft$ Plate Height: $Pt := 10\cdot ft$

$W_g := (15\cdot psf)\cdot 0\cdot ft + (10\cdot psf)\cdot Pt + (10psf)\cdot 2ft$

$DLRg := \frac{W_g\cdot L_g}{2}$ $DLRg = 700.2\,lb$

Chord Force:

$CFg_w := \frac{vg\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_w = 1354.05\,lb$ $CFg_s := \frac{E_g\cdot L_g\cdot Pt}{C_o\cdot L_g}$ $CFg_s = 1197.88\,lb$

Holdown Force:

$HDFg_w := CFg_w - 0.6\cdot DLRg = 933.93\,lb$ $HDFg_s := CFg_s - (0.6 - 0.14S_{DS})\cdot DLRg = 888.99\,lb$

No Holdown Required

Base Plate Nail Spacing (2015 NDS Table 12N)
16d Sinker (0.148"x3.25") Nails & 1-1/2" Plate Hem-Fir

$Z_N := 102\cdot lb$ $C_D := 1.6$

$B_n := \frac{(C_D\cdot Z_N\cdot C_o)}{vg} = 1.21\,ft$ $\frac{(C_D\cdot Z_N\cdot C_o)}{E_g} = 1.36\,ft$

16d @ 12" o.c.

Anchor Bolt Spacing (2015 NDS Table 12E)
5/8" Dia. Bolt (6" Embed) & 1-1/2" Plate Hem-Fir

$A_s := 860\cdot lb$ $C_D := 1.6$ $Z_B := A_s\cdot C_D$ $Z_B = 1376\,lb$

$As := \frac{(Z_B\cdot C_o)}{vg} = 10.16\,ft$ $\frac{(Z_B\cdot C_o)}{E_g} = 11.49\,ft$

5/8" A.B. @ 72" o.c.

WALL H:

Story Shear due to Wind: $V_{4W} = 13229.62\,lb$

Story Shear due to Seismic: $F_2 = 10031.78\,lb$

Bldg Width in direction of Load: $L_t := 59\cdot ft$

Distance between shear walls: $L_1 := 9\cdot ft$ $L_2 := 24ft$

Shear Wall Length: $Lh := (2\cdot 1.875)ft = 3.75\,ft$

Percent full height sheathing: $\% := \left(\frac{10\cdot ft}{10\cdot ft}\right)\cdot 100$ $\% = 100$

Max Opening Height = 0ft-0in, Therefore $C_o := 1.00$ per AF&PA SDPWS Table 4.3.3.5

Wind Force: $vh := \dfrac{\frac{0.6V_{4W}}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lh}$

Seismic Force: $\rho := 1.0$ $E_h := \dfrac{\rho\cdot\frac{0.7F_2}{L_t}\cdot\frac{L_1 + L_2}{2}}{Lh}$

$vh = 591.97\,ft^{-1}\cdot lb$ $\frac{vh}{C_o} = 591.97\,ft^{-1}\cdot lb$

$E_h = 523.69\,ft^{-1}\cdot lb$ $\frac{E_h}{C_o} = 523.69\,ft^{-1}\cdot lb$

See APA Technical Topic TT-100
"A Portal Frame with Hold Downs for
Engineered Applications" (Emphasis Added)

Restraint Panel Height = 10ft Maximum

Restraint Panel Width = 1ft-10-1/2 in Minimum

Allowable Shear per Panel = 1046 lbs Seismic & 1465 lbs Wind

Shear per Panel: $V_s := (1.875ft\cdot E_h) = 981.92\,lb$ O.K.

$V_w := (1.875ft\cdot vh) = 1109.94\,lb$ O.K.

Diapragm Shear Check:

Assume 2x HF Roof Framing, 7/16" Sheathing w/ 8d (0.131" x 2.5") nails, 6" o.c Edge nailing

Unblocked Diapraghm Case 1 Wind Capacity = 300 plf & Seismic Capacity = 214 plf

Unblocked Diapraghm Case 2-6 Wind Capacity = 221 plf & Seismic Capacity = 158 plf

Wall Lines AA:

$$vaa\cdot\frac{Laa}{36ft} = 99.32\,ft^{-1}\cdot lb \qquad E_{aa}\cdot\frac{Laa}{36ft} = 124.28\,ft^{-1}\cdot lb$$

Wall Lines BB:

$$vbb\cdot\frac{Lbb}{33ft} = 108.35\,ft^{-1}\cdot lb \qquad E_{bb}\cdot\frac{Lbb}{33ft} = 135.58\,ft^{-1}\cdot lb$$

Wall Lines CC:

$$vcc\cdot\frac{Lcc}{28ft} = 41.5\,ft^{-1}\cdot lb \qquad E_{cc}\cdot\frac{Lcc}{28ft} = 55.2\,ft^{-1}\cdot lb$$

Wall Lines DD:

$$vdd\cdot\frac{Ldd}{13ft} = 59.6\,ft^{-1}\cdot lb \qquad E_{dd}\cdot\frac{Ldd}{13ft} = 79.28\,ft^{-1}\cdot lb$$

Wall Lines EE:

$$vee\cdot\frac{Lee}{55ft} = 36.69\,ft^{-1}\cdot lb \qquad E_{ee}\cdot\frac{Lee}{55ft} = 48.81\,ft^{-1}\cdot lb$$

Wall Lines FF:

$$vff\cdot\frac{Lff}{24ft} = 59.45\,ft^{-1}\cdot lb \qquad E_{ff}\cdot\frac{Lff}{24ft} = 79.08\,ft^{-1}\cdot lb$$

Wall Lines GG:

$$vgg\cdot\frac{Lgg}{43ft} = 31.29\,ft^{-1}\cdot lb \qquad E_{ee}\cdot\frac{Lee}{43ft} = 62.43\,ft^{-1}\cdot lb$$

Wall Lines A:

$$\frac{va\cdot La - vaa\cdot Laa}{55ft} = 31.8\,ft^{-1}\cdot lb \qquad \frac{E_a\cdot La - E_{aa}\cdot Laa}{55ft} = 28.13\,ft^{-1}\cdot lb$$

Wall Lines B:

$$\frac{vb\cdot Lb - vbb\cdot Lbb}{33ft} = 18.35\,ft^{-1}\cdot lb \qquad \frac{E_b\cdot Lb - E_{bb}\cdot Lbb}{33ft} = 16.23\,ft^{-1}\cdot lb$$

Wall Lines C:

$$\frac{vc\cdot Lc - vcc\cdot Lcc}{59ft} = 21.27\,ft^{-1}\cdot lb \qquad \frac{E_c\cdot Lc - E_{cc}\cdot Lcc}{59ft} = 20.56\,ft^{-1}\cdot lb$$

Wall Lines D:

$$\frac{vd\cdot Ld - vdd\cdot Ldd}{50ft} = 41.3\,ft^{-1}\cdot lb \qquad \frac{E_d\cdot Ld - E_{dd}\cdot Ldd}{50ft} = 44.36\,ft^{-1}\cdot lb$$

Wall Line E:

$$\frac{ve\cdot Le - vee\cdot Ldd}{55ft} = 50.54\,ft^{-1}\cdot lb \qquad \frac{E_e\cdot Le - E_{ee}\cdot Ldd}{55ft} = 52.81\,ft^{-1}\cdot lb$$

Wall Line F:

$$\frac{vf\cdot Lf}{59ft} = 77.96\,ft^{-1}\cdot lb \qquad \frac{E_f\cdot Lf}{59ft} = 89.04\,ft^{-1}\cdot lb$$

Wall Line G:

$$\frac{vg\cdot Lg}{34ft} = 98.93\,ft^{-1}\cdot lb \qquad \frac{E_g\cdot Lg}{34ft} = 87.52\,ft^{-1}\cdot lb$$

Wall Line H:

$$\frac{vh\cdot Lh}{34ft} = 65.29\,ft^{-1}\cdot lb \qquad \frac{E_h\cdot Lh}{34ft} = 57.76\,ft^{-1}\cdot lb$$