Structural Calculations for:

Mangini-Zaborowski Residence

8429 SE 62nd St, Mercer Island, WA 98040

Client: CTA Design Builders, Inc.

Code: 2018 International Building Code

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Scope: Structural Design of Addition to Existing Single-Family Residence

March 14, 2022

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Seattle, WA 98155
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Seismic Design Loads (ASCE 7-16)

for a Wood Framed Structure

RISK CATEGORY	II				
OCCUPANCY CAT.	II	Table 1.5-1			
IMP. FACTOR	1	Table 1.5-2			
SITE CLASS	D	Table 20.3-1	SEISMIC		
R =	6.5	Table 12.2-1	DESIGN CATEGORY	D	11.6

S_s =	1.461		T_0 =	0.12	
S_1 =	0.506		Ts =	0.62	
F_a =	1.00	Table 11.4-1	T_L =	6	Fig 22-14
F_v =	1.79	Table 11.4-2	T =	0.196	Eqn. 12.8-7
S_{DS}=	0.974		Seismic Dead Load:	15 psf	Roof
S_{D1}=	0.604			15 psf	Floor
				20 psf	Walls

Cs_{ULT}=	0.150	Eqn. 12.8-2	W_{roof}=15 + 10= 25 psf
Cs_{ASD}=	0.107		W_{floor}=15 + 10 + 10= 35 psf

Vertical Design Loads

Criteria
ASCE 7-16
IBC 2018

Dead Loads

Roof (Composit)	2.5	psf	Flooring	1	psf
1/2" Ply	1.5	psf	Sheathing	2.3	psf
Rafter/Truss	2	psf	Joist	2.6	psf
Insulation	1	psf	5/8" GWB	3.1	psf
5/8" GWB	3.1	psf	Misc. Mech	1	psf
Misc./Mech.	2	psf		10	psf
	12.1	psf			
Use	15	psf	Use	15	psf

Live Loads

Snow	25	psf
floor	40	psf

Soil Bearing

2000 psf

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Project: Mangini Zamborowski Residence
8429 SE 62nd St
Mercer Island, WA 98040

Date: 6/9/2021
Design: CEH

Wind Design Loads (ASCE 7-16)

Directional Procedure - Part 1

Exposure	B			Roof Angle =	18.43	degrees
V=	100	mph		Ground to top of roof	21	ft
K_d=	0.85		Table 26.6-1	Bottom of roof to top of roof	4.25	ft
I=	1			(mean roof height) h=	18.9	ft
G=	0.85		26.11.1			
Ke=	1.00		Table 26.9-1			

K_{zt}= 1.00

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.5
Windward Roof	0.3
Leeward Roof	-0.6

*Note= Cp values are conservative worst case values

Pressures:	Calculated using ASCE7-16 Ch. 27 (Directional Procedure)					
Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	Ultimate P_{walls} (psf)	**Allowable P_{walls} (psf)**
0-15	0.57	12.40	8.43	5.73	14.17	**8.50**
15-20	0.62	13.49	9.17	5.73	14.91	**8.94**
20-25	0.66	14.36	9.77	5.73	15.50	**9.30**
25-30	0.7	15.23	10.36	5.73	16.09	**9.65**
30-40	0.76	16.54	11.25	5.73	16.98	**10.19**

$P_{ww\ roof}$	$P_{lw\ roof}$	P_{roof} (psf)	**P_{roof} (psf)**
3.44	6.88	10.32	**6.19**

Use 10 psf on projected wind surfaces

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Project: Mangini Zamborowski Residenc
8429 SE 62nd St
Mercer Island, WA 98040

Date: 6/9/2021

Design: CEH

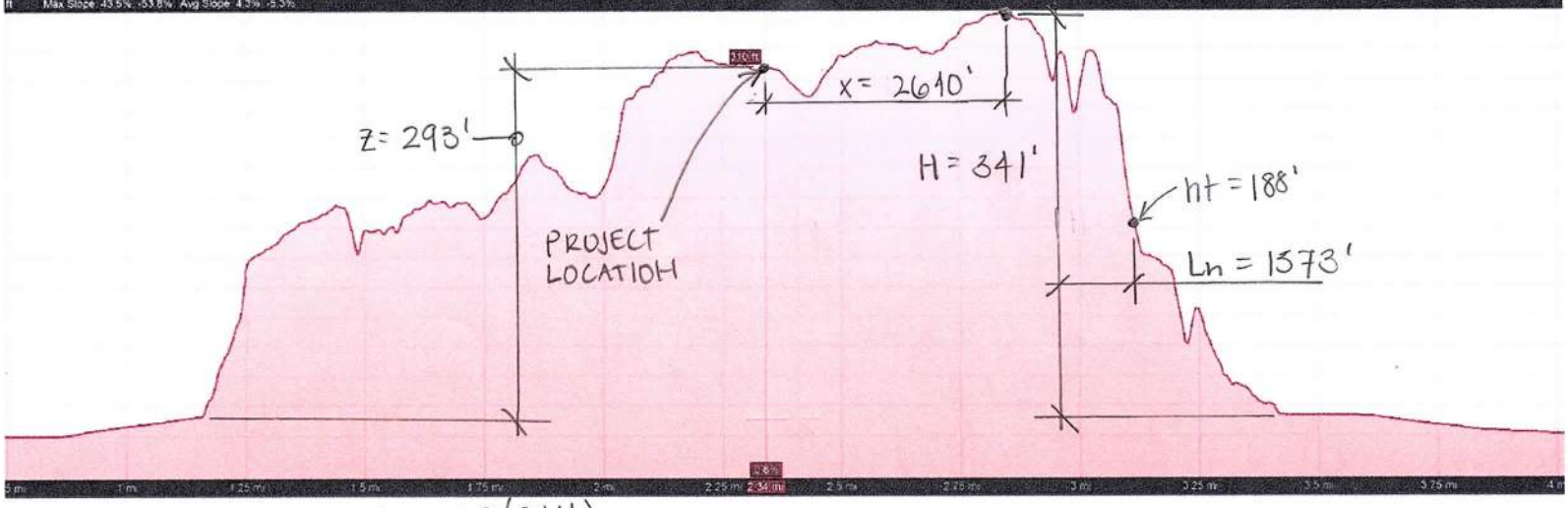
D PROPOSED WEST ELEVATION

SCALE: 1/4" = 1'-0"

0 2' 4'

$w_{ROOF} = 10\,psf\,(8.5\,ft) = 85\,lb/ft$

$w_{MAIN} = 10\,psf\,(6\,ft) = 60\,lb/ft$



$$K_1 = \frac{1.3\,H}{L_h} = \frac{1.3(341')}{1373'} = 0.32$$

$$K_2 = 1 - \frac{|x|}{\mu L_h} = 1 - \frac{2640'}{1.5(1373')} = -0.28$$

$$K_3 = e^{-az/L_h} = e^{-3(293')/1373'} = 0.53$$

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

$$= (1 + 0.32(-0.28)(0.53))^2 = 0.91$$

$\longrightarrow$ USE $K_{zt} = 1.0$

LATERAL DEMANDS - SEISMIC

ROOF

$DL = 15\ psf$

$DL_{WALL} = 20psf / 2 = 10\ psf$

$A_{ROOF} = 28\ ft(33.33\ ft) + 31.5\ ft(29.33\ ft) + 28.33\ ft(28.25\ ft) = 2657\ ft^2$

$W_{ROOF} = (15psf + 10psf)\ 2657\ ft^2 = 66.4\ k$

1ST FLR

$DL = 15\ psf$

$DL_{WALL} = 20\ psf\ (6ft / 7.67\ ft) = 15.7\ psf$

$A_{1ST} = 20\ ft(29.5\ ft) + 35.5\ ft(25.33\ ft) + 24.25\ ft(24.75\ ft) = 2089\ ft^2$

$W_{1ST} = (15\ psf + 15.7\ psf)\ 2089\ ft^2 = 64.1\ k$

BASE SHEAR

$V = 0.15\ W = 0.15(66.4\ k + 64.1\ k) = 19.6\ k$

$V_{ASD} = \frac{V}{1.4} = 14.0\ k$

VERTICAL DISTRIBUTION

LEVEL	w_x	h_x	$w_x h_x$	$C_{v,x}$	$F_{v,ASD}$
ROOF	66.4 k	11.67'	774.9	0.751	10.51 k
1ST	64.1 k	4'	256.4	0.249	3.49 k
TOTAL	130.5 k		1031.3	1.0	14.0 k

Hazards by Location

Search Information

Address: 8429 SE 62nd St, Mercer Island, WA 98040, USA

Coordinates: 47.54728310000001, -122.2255649

Elevation: 304 ft

Timestamp: 2021-06-09T16:44:59.674Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.461	MCE_R ground motion (period=0.2s)
S_1	0.506	MCE_R ground motion (period=1.0s)
S_{MS}	1.461	Site-modified spectral acceleration value
S_{M1}	0.906	Site-modified spectral acceleration value
S_{DS}	0.974	Numeric seismic design value at 0.2s SA
S_{D1}	0.604	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	1.79	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.626	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.688	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)

North/South Direction - Roof

Grid	West Wall	East Wall
Vwind (kips)	1.2	2.52
Vseismic (kips)	3.02	5.26
Length of wall (ft)	19.25	14
v_wind (plf)**	62	180
v_siesmic (plf)**	157	376
h (ft)	7.67	7.67
OTF_Wind (lbs)*	478	1381
OTF_Seismic (lbs)*	1203	2882
Length of shortest wall pier (ft)	4.25	14
Apect Ratio	1.80	0.55
Aspect Ratio Penalty	1.0	1.0
Shearwall	W6	W3
Holdown	CS16	(2) CS16

***OTF does not take into account dead load and weight of the wall uno**
****v_siesmic/wind includes penalty**

North/South Direction - Main Floor

Grid	West Wall	East Wall
Vwind (kips)	1.92	4.18
Vseismic (kips)	3.52	7.01
Length of wall (ft)	19.5	23.75
v_wind (plf)**	98	176
v_siesmic (plf)**	181	295
h (ft)	6.5	3.5
OTF_Wind (lbs)*	640	616
OTF_Seismic (lbs)*	1173	1033
Length of shortest wall pier (ft)	19.5	23.75
Apect Ratio	0.33	0.15
Aspect Ratio Penalty	1.0	1.0
Shearwall	W6	W3
Holdown	HDU2	HDU4

***OTF does not take into account dead load and weight of the wall uno**
****v_siesmic/wind includes penalty**

LATERAL DESIGN

N-S DIRECTION

ROOF

DEMANDS

- ASSUME EAST WALL OF ADDITION RESISTS 50% OF LATERAL LOAD OF ENTIRE STRUCTURE.

$V_{WEST,SEIS} = 10.51\ K\ (14.13ft)(54ft)/2657ft^2 = 3.02\ K$

$V_{EAST,SEIS} = 10.51\ K/2 = 5.26\ K$

$V_{WEST,WIND} = 85\ lb/ft\ (14.13ft) = 1.20\ K$

$V_{EAST,WIND} = 85\ lb/ft\ (59.33ft)/2 = 2.52\ K$

SEISMIC LOAD GOVERNS

ASSUME HEM-FIR FRAMING

$G = 0.43$

$G_{ADJ} = 1-(0.5-G) = 1-(0.5-0.43) = 0.93$

WEST WALLS

$v_{SEIS} = 157\ lb/ft$

USE W6 SHEARWALL

$v_{all} = \frac{520\ lb/ft}{2}(0.93) = 242\ lb/ft > 157\ lb/ft \rightarrow OK$

$OTF = 1203\ lb$

USE CS16 STRAPS

$T_{all} = 1705\ lb > 1203\ lb \rightarrow OK$

EAST WALL

$v_{SEIS} = 376\ lb/ft$

USE W3 SHEARWALL

$v_{all} = \frac{980\ lb/ft}{2}(0.93) = 456\ lb/ft > 376\ lb/ft \rightarrow OK$

$OTF = 2882\ lb$

LATERAL DESIGN, CONT.

USE (2) CS16 STRAPS

T_{all} = 1705 lb (2 STRAPS) = 3410 lb > 2882 lb.

MAIN FLR

DEMANDS

$V_{WEST,SEIS}$ = 3.02 k + 3.49 k (12 ft)(25 ft) / 2089 ft² = 3.52 k

$V_{EAST,SEIS}$ = 5.26 k + 3.49 k/2 = 7.01 k

$V_{WEST,WIND}$ = 1.20 k + 60 lb/ft (12 ft)(1k/1000 ft) = 1.92 k

$V_{EAST,WIND}$ = 2.52 k + 60 lb/ft (55.33 ft)/2 (1k/1000 ft) = 4.18 k

CHECK DIAPHRAGM SHEAR STRESS

$v = \frac{1745\ k}{24.33\ ft} = 72\ lb/ft$

$v_{all} = \frac{360\ lb/ft}{2}(0.93) = 167\ lb/ft > 72\ lb/ft \rightarrow OK$

WEST WALLS

v_{SEIS} = 181 lb/ft.

USE W6 SHEARWALL

v_{all} = 242 lb/ft > 80 lb/ft → OK

OTF = 1173 lb + 1203 lb = 2376 lb.

USE HDU2

T_{all} = 3075 lb > 1376 lb → OK

EAST WALL

v_{SEIS} = 295 lb/ft

USE W3 SHEARWALL

$v_{all} = \frac{980\ lb/ft}{2}(0.93) = 456\ lb/ft > 295\ lb/ft \rightarrow OK$

OTF = 1033 lb + 2882 lb = 3915 lb.

USE HDU4

T_{all} = 4565 lb > 3915 lb → OK

East/West Direction - Roof

Grid	North Wall	Middle Wall	South Wall
Vwind (kips)	1.15	2.3	1.15
Vseismic (kips)	0.64	3.02	1.51
Length of wall (ft)	see FTAO sheet	8	14
v_wind (plf)**	see FTAO sheet	288	82
v_siesmic (plf)**	see FTAO sheet	378	108
h (ft)	7.67	7.67	7.67
OTF_Wind (lbs)*	see FTAO sheet	2205	630
OTF_Seismic (lbs)*	see FTAO sheet	2895	827
Length of shortest wall pier (ft)	4.5	8	7
Apect Ratio	1.70	0.96	1.10
Aspect Ratio Penalty	1.0	1.0	1.0
Shearwall	W6	W3	W6
Holdown	CS16	(2) CS16	CS16

***OTF does not take into account dead load and weight of the wall uno**
****v_siesmic/wind includes penalty**

East/West Direction - Main Floor

Grid	North Wall	Middle Wall	South Wall
Vwind (kips)	1.96	3.92	1.96
Vseismic (kips)	0.85	4.02	2.01
Length of wall (ft)	3.5	8	18.25
v_wind (plf)**	653	490	107
v_siesmic (plf)**	283	503	110
h (ft)	3.5	3.5	3.5
OTF_Wind (lbs)*	1960	1715	376
OTF_Seismic (lbs)*	850	1759	385
Length of shortest wall pier (ft)	1.5	8	9
Apect Ratio	2.33	0.44	0.39
Aspect Ratio Penalty	0.86	1.0	1.0
Shearwall	W2	W2	W6
Holdown	HDU2	HDU5	HDU2

***OTF does not take into account dead load and weight of the wall uno**
****v_siesmic/wind includes penalty**

LATERAL DESIGN, CONT.

E-W DIRECTION

ROOF

DEMANDS

- ASSUME EXISTING WALLS RESIST SEISMIC DEMANDS OF EXISTING STRUCTURE.

$V_{NORTH,\,WIND} = 85\ lb/ft\ (13.5\ k) = 1.15\ k$

$V_{MID,\,WIND} = 85\ lb/ft\ (27\ ft) = 2.30\ k$

$V_{SOUTH,\,WIND} = 1.15\ k$

$V_{NORTH,\,SEIS} = 10.5\ k\ (12ft)(13.5ft)/2657\ ft^2 = 0.64\ k$

$V_{MID,\,SEIS} = 10.51\ k\ (54')(28.25')/2657\ ft^2/2 = 3.02\ k$

$V_{SOUTH,\,SEIS} = 10.51\ k\ (27\ ft)(28.25ft)/2657\ ft^2/2 = 1.51\ k$

CHECK DIAPHRAGM SHEAR STRESS

$v = \frac{3020\ lb}{24\ ft} = 126\ lb/ft$

$v_{all} = \frac{360\ lb/ft}{2}(0.93) = 167\ lb/ft > 126\ lb/ft$

NORTH WALL

SEE ATTACHED FTAO SPREADSHEET PRINTOUTS

$V_{WIND} = 287$

USE W6 SHEARWALLS

$v_{all} = 339\ lb/ft > 287\ lb/ft$

STRAP FORCE = 1120 lb.

USE CS16 STRAPS TOP & BOT OF OPENINGS.

$T_{all} = 1705\ lb > 1120\ lb.$

OTF = 767 lb.

USE CS16 STRAPS AT SHEARWALL ENDS.

$T_{all} = 1705\ lb > 767\ lb.$

MIDDLE WALL

$V_{SEIS} = 378\ lb/ft$

USE W3 SHEARWALL

$v_{all} = 456\ lb/ft > 378\ lb/ft \rightarrow OK$

OTF = 2895 lb.

USE (2) CS16 STRAPS

$T_{all} = 3410\ lb > 2895\ lb \rightarrow OK$

SOUTH WALLS

$V_{SEIS} = 108\ lb/ft.$

USE W6 SHEARWALLS

LATERAL DESIGN, CONT.

$v_{all} = 242\ lb/ft > 108\ lb/ft \rightarrow OK$

OTF = 827 lb

USE CS16 STRAPS

$T_{all} = 1705\ lb > 827\ lb \rightarrow OK$

MAIN FLR

DEMANDS

$V_{NORTH,\ WIND} = 1.15\ k + 60\ lb/ft\ (13.5ft) = 1.96\ k$

$V_{MID,\ WIND} = 2.30\ k + 60\ lb/ft\ (27ft) = 3.92\ k$

$V_{SOUTH,\ WIND} = 1.96\ k$

$V_{NORTH,\ SEIS} = 0.64\ k + 3.49\ k\ (12ft)(13.5ft)/2657\ ft^2 = 0.85\ k$

$V_{MID,\ SEIS} = 3.02\ k + 3.49\ k\ (54')(28.25')/2657\ ft^2/2 = 4.02\ k$

$V_{SOUTH,\ SEIS} = 1.51\ k + 3.49\ k\ (27')(28.25')/2657\ ft^2/2 = 2.01\ k$

CHECK DIAPHRAGM SHEAR STRESS

$v = \frac{1000\ lb}{24\ ft} = 42\ lb/ft$

$v_{all} = 167\ lb/ft > 42\ lb/ft \rightarrow OK$

NORTH WALL

$v_{WIND} = 653\ lb/ft$

USE W2 SHEARWALLS

$v_{all} = \frac{1790\ lb/ft}{2}(0.93) = 832\ lb/ft > 653\ lb/ft \rightarrow OK$

OTF = 767 lb + 1960 lb = 2727 lb.

USE HDU2

$T_{all} = 3075\ lb > 2727\ lb \rightarrow OK$

MIDDLE WALL

$v_{SEIS} = 503\ lb/ft$

USE W2 SHEARWALL

$v_{all} = \frac{1280\ lb/ft}{2}(0.93) = 595\ lb/ft > 503\ lb/ft$

OTF = 2895 lb + 1759 lb = 4654 lb

USE HDU5

$T_{all} = 5645\ lb > 4654\ lb$

LATERAL DESIGN, CONT.

SOUTH WALL

$V_{SEIS} = 110$ lb/ft

USE W6 SHEARWALL

$V_{all} = 242$ lb/ft > 110 lb/ft.

OTF = 827 lb + 293 lb = 1120 lb

USE HOLD 2

Tall = 3075 lb $>$ 1120 lb $\rightarrow$ OK

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Project: MANGINI ZABOROWSKI RESIDENCE

Project #: CEH

Designer:

Date: 6/14/21

Sheet: L11

Force Transfer Around Openings Calculator

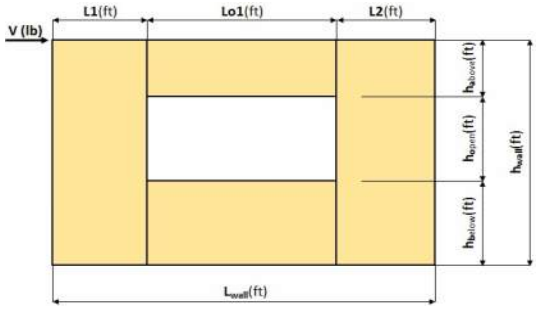
ONE OPENING

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:	2018 IBC
Designer:	CEH
Client:	CTA Design Builders, Inc.
Project:	Mangini Zamborowski Residence
Wall Line:	North Wall

Date: 6/14/2021



Shear Wall Calculation Variables

V	1150 lbf	Opening 1		Wall Pier Aspect Ratio	Adj. Factor
L1	3.00 ft	ha1	0.67 ft	P1=ho1/L1= 1.67	N/A
L2	2.00 ft	ho1	5.00 ft	P2=ho1/L2= 2.50	0.9375
h_{wall}	7.67 ft	hb1	2.00 ft		
L_{wall}	11.50 ft	Lo1	6.50 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 767 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 287 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1867 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1120 lbf

F2 = O1(L2)/(L1+L2) = 747 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.90 ft

T2 = (L2*Lo1)/(L1+L2) = 2.60 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 230 plf

V2 = (V/L)(T2+L2)/L2 = 230 plf

Check V1*L1+V2*L2=V? 1150 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 690 lbf

R2 = V2*L2 = 460 lbf

8. Difference corner force + resistance

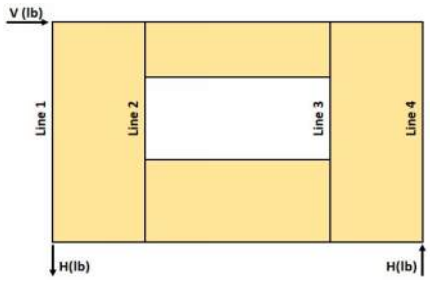
R1-F1 = -430 lbf

R2-F2 = -287 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -143 plf

vc2 = (R2-F2)/L2 = -143 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-383	1150	767 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	767	-383	1150	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		-383	1150	767 lbf

Design Summary

Req. Sheathing Capacity	287 plf	**4-Term Deflection**	0.365 in.	**3-Term Deflection**	0.411 in.
Req. Strap Force	1120 lbf	**4-Term Story Drift %**	0.016 %	**3-Term Story Drift %**	0.018 %
Req. HD Force (H)	767 lbf		See Page 2		See Page 3

Project Information

Code:	2018 IBC	**Date:**	6/14/2021
Designer:	CEH		
Client:	CTA Design Builders, Inc.		
Project:	Mangini Zamborowski Residence		
Wall Line:	North Wall		

Shear Wall Deflection Calculation Variables

Sheathing:

Plywood	Sheathing Material
15/32	Performance Category
APA Rated Sheathing	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	Douglas Fir-Larch #2	
E:	1.60E+06	(psi)
	Qty	Stud Size
Dimensions:	2	2x6
A:	16.5	($in.^2$)
A Override:		($in.^2$)

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	3075	3075	(lbf)
HD Deflection:	0.088	0.088	(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:	15/32	15/32	15/32	15/32	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	230	230	230	230	(plf)
$v_{strength}$:	329	329	329	329	(plf)
E:	1.60E+06	1.60E+06	1.60E+06	1.60E+06	(psi)
h:	7.67	5.67	5.67	7.67	(ft)
A:	16.5	16.5	16.5	16.5	($in.^2$)
Gt:	27,000	27,000	27,000	27,000	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
Vn:	164	164	164	164	(plf)
e:	0.0185	0.0185	0.0185	0.0185	(in.)
b:	3.00	3.00	2.00	2.00	(ft)
HD Capacity:	3075	3075	3075	3075	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(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
0.015	0.093	0.107	0.184	0.006	0.069	0.079	0.101
		Sum	0.399			Sum	0.255
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
0.009	0.069	0.079	0.151	0.022	0.093	0.107	0.277
		Sum	0.308			Sum	0.499

Total Defl.	
0.365	(in.)
0.0159	%drift

Project Information

Code:	2018 IBC	**Date:**	6/14/2021
Designer:	CEH		
Client:	CTA Design Builders, Inc.		
Project:	Mangini Zamborowski Residence		
Wall Line:	North Wall		

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:	15/32	15/32	15/32	15/32	
Nail:	8d common	8d common	8d common	8d common	
v_{asd}:	230	230	230	230	(plf)
$v_{strength}$:	329	329	329	329	(plf)
E:	1.60E+06	1.60E+06	1.60E+06	1.60E+06	(psi)
h:	7.67	5.67	5.67	7.67	(ft)
A:	16.5	16.5	16.5	16.5	($in.^2$)
Ga:	10.0	10.0	10.0	10.0	(kips/in.)
b:	3.00	3.00	2.00	2.00	(ft)
HD Capacity:	3075	3075	3075	3075	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(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
0.015	0.252	0.184	0.006	0.186	0.101
	Sum	0.451		Sum	0.293
Pier 2 (left)			Pier 2 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.009	0.186	0.151	0.022	0.252	0.277
	Sum	0.347		Sum	0.551

Total Defl.	
0.411	(in.)
0.0178	%drift

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

CS/CMST/CMSTC

Coiled Straps

CMSTC provides coined nail slots for lower profile when installed with 0.148" x 3¼" sinkers; it can be cut to length. CS are continuous utility straps which can be cut to length on the jobsite. Packaged in lightweight (about 40 lb.) cartons.

Finish: Galvanized. Some products available in ZMAX® coating; see Corrosion Information, pp. 13–15.

Installation: • Use all specified fasteners; see General Notes.

- Wood shrinkage after strap installation across horizontal wood members may cause strap to buckle outward.
- Refer to the applicable code for minimum nail penetration and minimum wood edge and end distances.
- The table shows the maximum allowable loads and the nails required to obtain them. Fewer nails may be used; reduce the allowable load as shown in the Straps and Ties General Notes on pp. 260–261.
- For lap slice and alternate nailing information, refer to p. 268.
- The cut length of the strap shall be equal to twice the "End Length" noted in the table plus the clear span dimension.
- CMST only — Use every other round hole if the wood tends to split. Use round and triangle holes for comparable MST loads, providing wood does not tend to split.
- CS straps are available in 25' lengths; order CS14-R, CS16-R or CS20-R.
- For stainless steel, order CS16SS-R.

Codes: See p. 12 for Code Reference Key Chart

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

	Model No.	Total L	Ga.	DF/SP Fasteners (in.)	DF/SP End Length	SPF/HF Fasteners (in.)	SPF/HF End Length	Allowable Tension Loads (160)	Code Ref.
	CMST12	40'	12	(74) 0.162 x 2½	33"	(84) 0.162 x 2½	38"	9,215	IBC, FL, LA
				(86) 0.148 x 2½	39"	(98) 0.148 x 2½	44"	9,215	
	CMST14	52½'	14	(56) 0.162 x 2½	26"	(66) 0.162 x 2½	30"	6,475	
				(66) 0.148 x 2½	30"	(76) 0.148 x 2½	34"	6,475	
	CMSTC16	54'	16	(50) 0.148 x 3¼	20"	(58) 0.148 x 3¼	25"	4,690	
	CS14	100'	14	(26) 0.148 x 2½	15"	(30) 0.148 x 2½	16"	2,490	
				(30) 0.131 x 2½	16"	(36) 0.131 x 2½	19"	2,490	
SS	CS16	150'	16	(20) 0.148 x 2½	11"	(22) 0.148 x 2½	13"	1,705	
				(22) 0.131 x 2½	13"	(26) 0.131 x 2½	15"	1,705	
	CS20	250'	20	(12) 0.148 x 2½	7"	(14) 0.148 x 2½	9"	1,030	
				(14) 0.131 x 2½	9"	(16) 0.131 x 2½	9"	1,030	

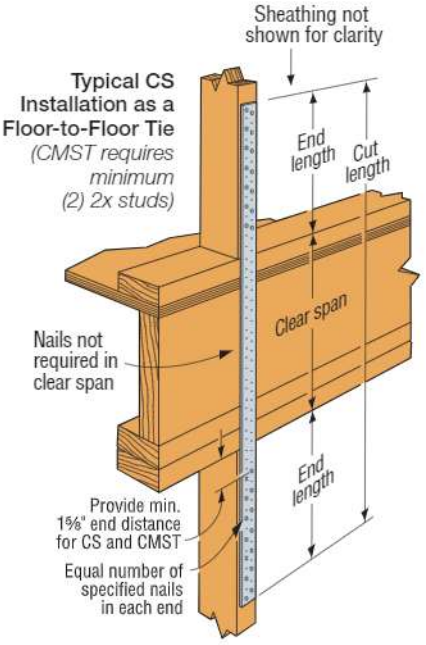
1. See pp. 260–261 for Straps and Ties General Notes.
2. Calculate the connector value for a reduced number of nails as follows:

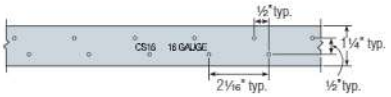
$$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

Example: CMSTC16 in DF/SP with 40 nails total.
(Half of the nails in each member being connected)

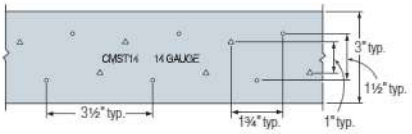
$$\text{Allowable Load} = \frac{\text{40 Nails (Used)}}{\text{50 Nails (Table)}} \times 4{,}690 \text{ lb.} = 3{,}752 \text{ lb.}$$

3. See page 268 for alternate nailing and lap splice information.
4. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

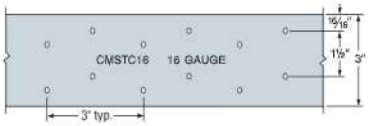




CS16 Hole Pattern
(all other CS straps similar)

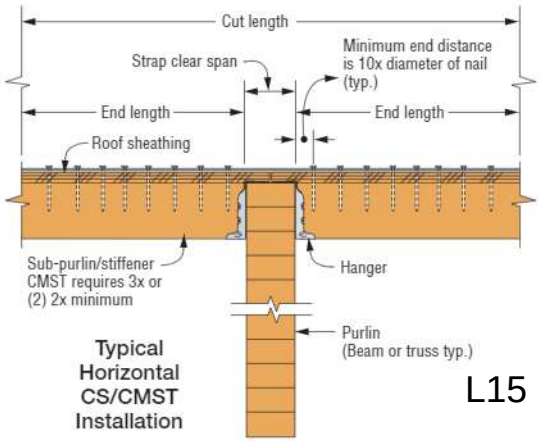


CMST14 Hole Pattern
(CMST12 similar)



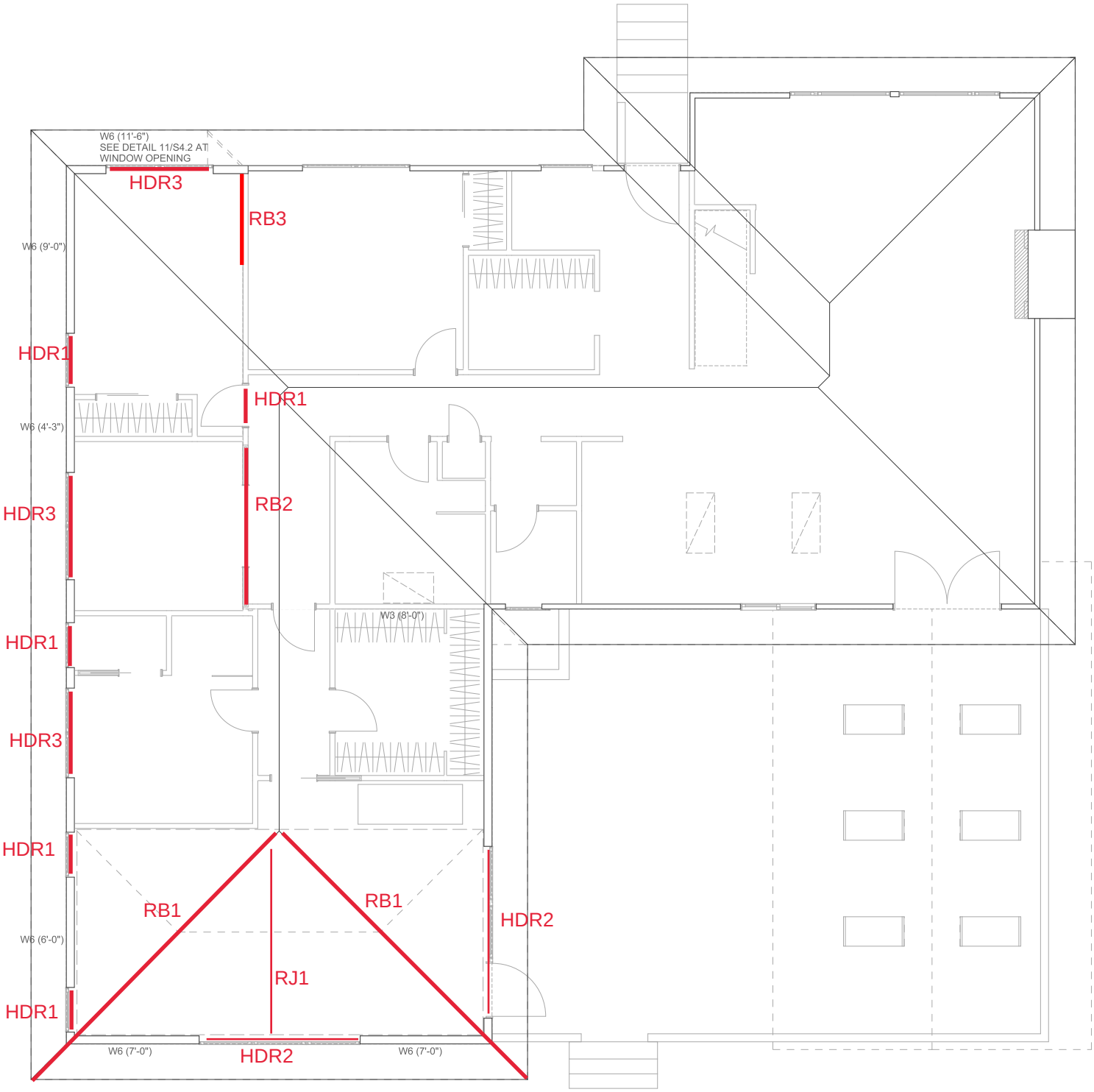
CMSTC16 Hole Pattern

Gauge stamped on part for easy identification



Typical Horizontal CS/CMST Installation

ROOF FRAMING KEY PLAN



F1

ROOF FRAMING

RJ1

L = 12 ft

DL = 15 psf
S = 25psf
w = (15psf + 25psf) 2 ft = 80 lb/ft.

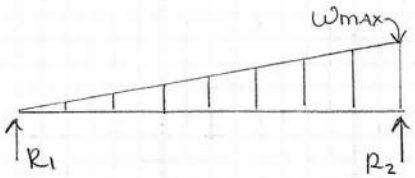
USE 2x10 @ 24" O.C.

M = 1410 lb·ft f_b = 808 psi
R = 480 lb. f_v = 52 psi

C_D = 1.15
C_F = 1.1
C_r = 1.15
F_b' = 1.15 (1.1)(1.15) 850 psi = 1236 psi
F_v' = 1.15(150 psi) = 172 psi

RB1

L = 16.5 ft



DL = 15psf
S = 25psf
w_{max} = (15psf + 25psf)(12ft + 11.67ft)/2
= 473 lb.

USE 6x10

M = 8260 lb·ft f_b = 1200 psi
R_1 = 1300 lb f_v = 75 psi
R_2 = 2600 lb

Δ_{TOT} = 0.63in = L/315
Δ_S = 0.63in (25/10) = 0.39in = L/504

HDR1

L = 3ft

DL = 15psf
S = 25 psf
w = (15psf + 25psf) 24ft/2 = 480 lb/ft

USE (2) 2x8

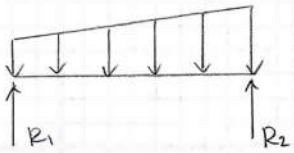
M = 540 lb·ft f_b = 247 psi
R = 720 lb. f_v = 50 psi

C_D = 1.15
C_F = 1.2
F_b' = 1.15(1.2) 850 psi = 1173 psi
F_v' = 1.15 (150 psi) = 172 psi

Δ_{TOT} = 0.01in = L/5097

HDR2

L = 10 ft



DL = 15 psf
DL_{WALL} = 20psf
S = 25psf

w_{MIN} = (15psf + 25psf)(0.5ft + 2.5ft)
+ 20psf (1ft)
= 140 lb/ft
w_{MAX} = (15psf + 25psf)(11.67ft/2 + 2.5ft)
+ 20psf (1ft)
= 353 lb./ft

USE 4x8, MIN

M = 3100 lb·ft f_b = 1210 psi
R_1 = 1060 lb. f_v = 83 psi
R_2 = 1410 lb.

C_D = 1.15
C_F = 1.3
F_b' = 1.15(1.3)(900 psi) = 1345 psi
F_v' = 1.15 (180psi) = 207 psi

Δ_{TOT} = 0.31in = L/384 → OK

HDR3

L = 6 ft

DL = 15 psf
DL_{WALL} = 20psf
S = 25 psf
w = (15psf + 25psf) 28.25'/2 + 20psf (1ft) = 585 lb/ft

ROOF FRAMING, CONT.

HDR3, CONT.

USE 4x8

$M = 2633$ lb·ft $f_b = 1030$ lb.

$R = 1755$ lb. $f_v = 104$ psi

$C_D = 1.15$

$C_F = 1.3$

$F_b' = 1345$ psi
$F_v' = 207$ psi
} SEE HDR2

$\Delta_{TOT} = 0.10$ in $= L/751 \rightarrow$ OK

RB2

$L = 10$ ft

$DL = 15$ psf

$S = 25$ psf

$w = (15\text{psf} + 25\text{psf})\ 28.25\text{ft}/2 = 565$ lb

USE 3½"x9½" LSL, MIH

$M = 7063$ lb·ft $M_{all} = 10{,}420$ lb·ft

$R = 2825$ lb $V_{all} = 6870$ lb.

$\Delta_{TOT} = 0.33$ in $= L/366$

RB3

$L = 5.33$ ft

$DL = 15$ psf

$S = 25$ psf

$w = 565$ ft (SEE RB2)

USE 4x8, MIH

$M = 2006$ lb·ft $f_b = 785$ psi

$R = 1506$ lb. $f_v = 89$ psi

$C_D = 1.15$

$C_F = 1.3$

$F_b' = 1.15\ (1.3)\ 900$ psi $= 1345$ psi

$F_v' = 1.15\ (180$ psi$) = 207$ psi

$\Delta_{TOT} = 0.06$ in $= L/1109 \rightarrow$ OK

BEAMBOY V2.2 REPORT

RB1

BEAM PROPERTIES

Beam Length = 16.5 ft.
Moment of Inertia = 393 in^4
Modulus of Elasticity = 1600000 psi
Distance From Neutral Axis to Furthest Fiber = 4.75 in.

LOAD CONFIGURATION

Point Loads

Distributed Loads

Moments

Supports

Simple support; 0 ft., Reaction=1300 lb.
Simple support; 16.5 ft., Reaction=2600 lb.

MAXIMUM VALUES

Maximum Bending Moment = 8260 lb.-ft. at x=9.53 ft.
Maximum Bending Stress = 1200 psi at x=9.53 ft.
Maximum Deflection = -0.628 in. at x=8.57 ft.
Maximum Slope = 0.62 degrees at x=16.5 ft.

6/14/2021

BEAMBOY V2.2 REPORT

HDR2

BEAM PROPERTIES

Beam Length = 10 ft.
Moment of Inertia = 111 in^4
Modulus of Elasticity = 1600000 psi
Distance From Neutral Axis to Furthest Fiber = 3.62 in.

LOAD CONFIGURATION

Point Loads

Distributed Loads

Start=140 lb./ft., x=0 ft.; End=353 lb./ft., x=10 ft.

Moments

Supports

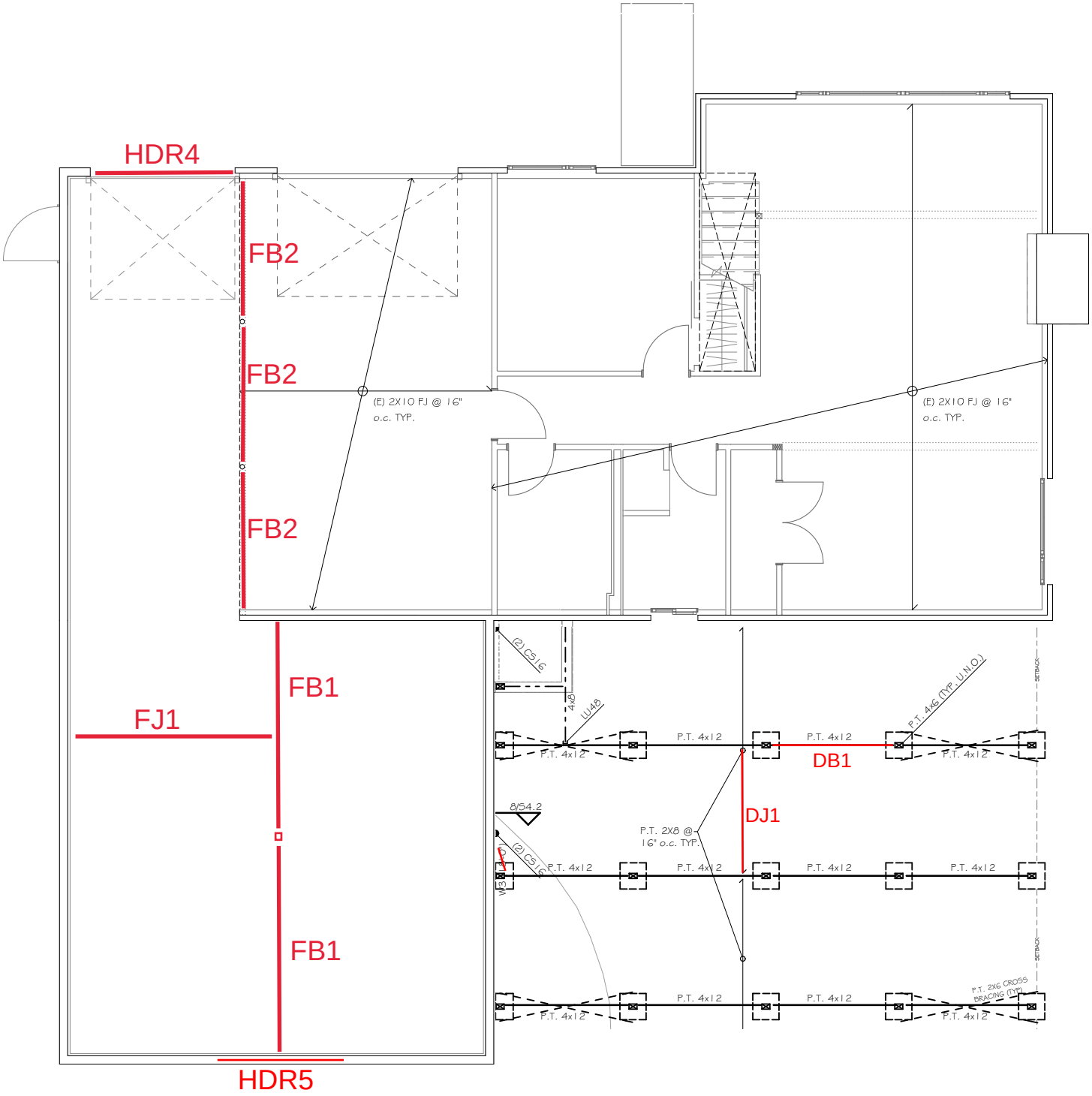
Simple support; 0 ft., Reaction=1060 lb.
Simple support; 10 ft., Reaction=1410 lb.

MAXIMUM VALUES

Maximum Bending Moment = 3100 lb.-ft. at x=5.36 ft.
Maximum Bending Stress = 1210 psi at x=5.36 ft.
Maximum Deflection = -0.312 in. at x=5.08 ft.
Maximum Slope = 0.49 degrees at x=10 ft.

6/14/2021

1ST FLOOR FRAMING KEY PLAN



1ST FLR FRAMING

FJ1

L = 12 ft

DL = 15 psf
LL = 40 psf
w = (15psf + 40psf) (16/12ft) = 73 lb/ft

USE 2x10 @ 16" O.C.

M = 1314 lb·ft fb = 737 psi
R = 438 lb. fv = 47 psi

CF = 1.1
Cr = 1.15
Fb' = 1.1 (1.15) (850 psi) = 1075 psi
Fv' = 150 psi

Δ_{TOT} = 0.26in = L/544 → OK

FB1

L = 12 ft

DL = 15psf
LL = 40 psf
w = (15psf + 40psf) 24ft/2 = 660 lb/ft

USE 5 1/4" x 9 1/2" PSL

M = 11,880 lb·ft Mau = 19,585 lb·ft
R = 3960 lb Vau = 9645 lb

Δ_{TOT} = 0.41in = L/351
Δ_{LL} = 0.3in = L/482

HDR4

L = 8 ft

DL = 15psf
DL_{WALL} = 20psf
LL = 40 psf
S = 25psf

w_D = 15psf (3ft + 8/12ft) + 20psf (9.5ft)
= 245 lb/ft.

w_L = 40psf (8/12 ft) = 27 lb/ft
w_S = 25psf (3ft) = 75 lb/ft

w_{MAX} = 245 lb/ft + 0.75 (27 lb/ft + 75 lb/ft)
= 322 lb/ft

USE 4x8

M = 2576 lb·ft fb = 1008 psi
R = 1288 lb. fv = 76 psi

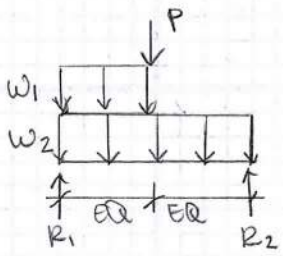
C_D = 1.15
CF = 1.3
Fb' = 1.15 (1.3) (900psi) = 1345 psi
Fv' = 1.15 (180 psi) = 207 psi

Δ_{TOT} = 0.17in = L/575

FB2

L = 8.5 ft

DL = 15 psf
L = 40psf
S = 25psf



$w_{1,D}$ = 15psf (24ft/2) = 180 lb/ft
$w_{1,S}$ = 25psf (24ft/2) = 300 lb/ft

$w_{2,D}$ = 180 lb/ft
$w_{2,L}$ = 40psf (24ft/2) = 480 lb/ft

P_D = 565 lb/ft (13')/2 (15/40) = 1377 lb.
P_S = 565 lb/ft (13'/2) (25/40) = 2295 lb.

w_1 = 180 lb/ft + 0.75 (300 lb/ft) = 405 lb/ft
w_2 = 180 lb/ft + 0.75 (480 lb/ft) = 540 lb/ft
P = 1377 lb + 0.75 (2295 lb) = 3098 lb.

USE 5 1/4" x 9 1/2" PSL

M = 13,300 lb·ft Mau = 19,585 lb·ft
R_1 = 5130 lb. Vau = 9,645 lb.
R_2 = 4270 lb.

Δ_{TOT} = 0.21 in = L/490 → OK

1ST FLR FRAMING, CONT.

DJ1

L = 7.33 ft

D = 15 psf
L = 60psf
S = 25 psf
w = 15psf + 0.75(60psf + 25psf) = 79 psf

USE P.T. 2x8 @ 16" O.C.

M = 531 lb·ft f_b = 208 psi
R = 290 lb. f_v = 17 psi

C_D = 1.15 C_F = 1.2
C_i = 0.8 C_r = 1.15
F_b' = 1.15 (1.2) (0.8) (1.15) 850 psi = 1079 psi
F_v' = 1.15 (0.8) 150 psi = 138 psi

DB1

L = 7.25 ft

D = 15psf
L = 60 psf
S = 25 psf
w = 79 psf (15 ft /2) = 593 lb/ft

USE P.T. 4x12

M = 3896 lb·ft f_b = 633 psi
R = 2150 lb. f_v = 82 psi

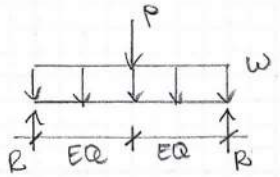
C_D = 1.15 C_i = 0.8
C_F = 1.1
F_b' = 1.15(1.1) (0.8) 900 psi = 910 psi
F_v' = 1.15 (0.8) 180 psi = 165 psi

Δ_{TOT} = 0.06 in = L/1568

HDR5

L = 4.67 ft

DL = 15psf
DL_{WALL} = 20psf



L = 40 psf

w = 20 psf (3.67 ft) = 73 lb/ft
P = (15 psf + 40 psf)(23.25'/2)(12'/2) = 3836 lb.

USE 3 1/2" x 9 1/2" LSL, M1H

M = 4670 lb·ft M_{all} = 10,420 lb·ft
R = 2090 lb. V_{all} = 6870 lb.

Δ_{TOT} = 0.04 in = L/1465 → OK

BEAMBOY V2.2 REPORT

FB2

BEAM PROPERTIES

Beam Length = 8.5 ft.
Moment of Inertia = 375 in^4
Modulus of Elasticity = 2000000 psi
Distance From Neutral Axis to Furthest Fiber = 4.75 in.

LOAD CONFIGURATION

Point Loads

3100 lb., x=4.25 ft.

Distributed Loads

Start=540 lb./ft., x=0 ft.; End=540 lb./ft., x=8.5 ft.
Start=405 lb./ft., x=0 ft.; End=405 lb./ft., x=4.25 ft.

Moments

Supports

Simple support; 0 ft., Reaction=5130 lb.
Simple support; 8.5 ft., Reaction=4270 lb.

MAXIMUM VALUES

Maximum Bending Moment = 13300 lb.-ft. at x=4.25 ft.
Maximum Bending Stress = 2020 psi at x=4.25 ft.
Maximum Deflection = -0.208 in. at x=4.2 ft.
Maximum Slope = -0.37 degrees at x=0.00085 ft.

3/13/2022

BEAMBOY V2.2 REPORT

HDR5

BEAM PROPERTIES

Beam Length = 4.67 ft.
Moment of Inertia = 250 in^4
Modulus of Elasticity = 1550000 psi
Distance From Neutral Axis to Furthest Fiber = 4.75 in.

LOAD CONFIGURATION

Point Loads

3840 lb., x=2.33 ft.

Distributed Loads

Start=73 lb./ft., x=0 ft.; End=73 lb./ft., x=4.67 ft.

Moments

Supports

Simple support; 0 ft., Reaction=2090 lb.
Simple support; 4.67 ft., Reaction=2090 lb.

MAXIMUM VALUES

Maximum Bending Moment = 4670 lb.-ft. at x=2.33 ft.
Maximum Bending Stress = 1070 psi at x=2.33 ft.
Maximum Deflection = -0.0382 in. at x=2.33 ft.
Maximum Slope = -0.118 degrees at x=0.000467 ft.

3/13/2022

Allowable Design Properties(1) (100% Load Duration)

Grade	Width	Design Property	Depth											
			4⅜"	5½"	5½" Plank Orientation	7¼"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	20"
TimberStrand® LSL														
1.3E	3½"	Moment (ft-lbs)	1,735	2,685	1,780	4,550								
		Shear (lbs)	4,340	5,455	1,925	7,190								
		Moment of Inertia (in.⁴)	24	49	20	111								
		Weight (plf)	4.5	5.6	5.6	7.4								
1.55E	1¾"	Moment (ft-lbs)						5,210		7,975	10,920	14,090		
		Shear (lbs)						3,435		4,295	5,065	5,785		
		Moment of Inertia (in.⁴)						125		244	400	597		
		Weight (plf)						5.2		6.5	7.7	8.8		
	3½"	Moment (ft-lbs)						10,420		15,955	21,840	28,180		
		Shear (lbs)						6,870		8,590	10,125	11,575		
		Moment of Inertia (in.⁴)						250		488	800	1,195		
		Weight (plf)						10.4		13	15.3	17.5		
Microllam® LVL														
2.0E	1¾"	Moment (ft-lbs)		2,125		3,555	5,600	5,885	8,070	8,925	12,130	15,555	19,375	23,580
		Shear (lbs)		1,830		2,410	3,075	3,160	3,740	3,950	4,655	5,320	5,985	6,650
		Moment of Inertia (in.⁴)		24		56	115	125	208	244	400	597	851	1,167
		Weight (plf)		2.8		3.7	4.7	4.8	5.7	6.1	7.1	8.2	9.2	10.2
Parallam® PSL														
2.0E	3½"	Moment (ft-lbs)					12,415	13,055	17,970	19,900	27,160	34,955	43,665	
		Shear (lbs)					6,260	6,430	7,615	8,035	9,475	10,825	12,180	
		Moment of Inertia (in.⁴)					231	250	415	488	800	1,195	1,701	
		Weight (plf)					10.1	10.4	12.3	13.0	15.3	17.5	19.7	
	5¼"	Moment (ft-lbs)					18,625	19,585	26,955	29,855	40,740	52,430	65,495	
		Shear (lbs)					9,390	9,645	11,420	12,055	14,210	16,240	18,270	
		Moment of Inertia (in.⁴)					346	375	623	733	1,201	1,792	2,552	
		Weight (plf)					15.2	15.6	18.5	19.5	23.0	26.3	29.5	
	7"	Moment (ft-lbs)					24,830	26,115	35,940	39,805	54,325	69,905	87,325	
		Shear (lbs)					12,520	12,855	15,225	16,070	18,945	21,655	24,360	
		Moment of Inertia (in.⁴)					462	500	831	977	1,601	2,389	3,402	
		Weight (plf)					20.2	20.8	24.6	26.0	30.6	35.0	39.4	

(1) For product in beam orientation, unless otherwise noted.

Some sizes may not be available in your region.

PRODUCT STORAGE

Protect product from sun and water

CAUTION:
Wrap is slippery when wet or icy

Align stickers (2x3 or larger) directly over support blocks

Use support blocks (6x6 or larger) at 10' on-center to keep bundles out of mud and water

This Wall in File: H:\Retaining Wall Calculations\Retaining Wall - Slab on Grade\cantilever retainin

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Cantilevered Retaining Wall

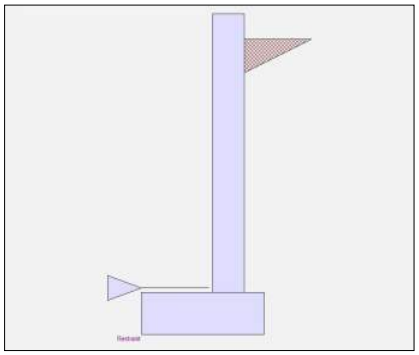
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	5.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	0.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	1.70	OK
Slab Resists All Sliding !			
Total Bearing Load	=	1,102 lbs	
...resultant ecc.	=	6.66 in	
Soil Pressure @ Toe	=	998 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,397 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	9.4 psi	OK
Footing Shear @ Heel	=	3.5 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	595.5 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

	Bottom
	Bar Lap/Emb
Design Height Above Ftg ft =	0.00
Wall Material Above "Ht" =	Concrete
Design Method =	LRFD
Thickness =	8.00
Rebar Size =	# 4
Rebar Spacing =	12.00
Rebar Placed at =	Edge

Design Data

fb/FB + fa/Fa =	**0.216**	
Total Force @ Section		
Service Level lbs =		
Strength Level lbs =	700.0	
Moment....Actual		
Service Level ft-# =		
Strength Level ft-# =	1,166.7	
Moment.....Allowable =	5,412.6	
Shear.....Actual		
Service Level psi =		
Strength Level psi =	9.3	
Shear.....Allowable psi =	75.0	
Anet (Masonry) in2 =		
Rebar Depth 'd' in =	6.25	

Masonry Data

f'm psi =		
Fs psi =		
Solid Grouting =		
Modular Ratio 'n' =		
Wall Weight psf =	100.0	
Short Term Factor =		
Equiv. Solid Thick. =		
Masonry Block Type =	Medium Weight	
Masonry Design Method =	ASD	

Concrete Data

f'c psi =	2,500.0	
Fy psi =	60,000.0	

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0437 in2/ft		
(4/3) * As :	0.0583 in2/ft	Min Stem T&S Reinf Area 1.056 in2	
200bd/fy : 200(12)(6.25)/60000 :	0.25 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.2 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.8467 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Dimensions & Strengths

Toe Width	=	1.50 ft
Heel Width	=	1.08
Total Footing Width	=	2.58
Footing Thickness	=	10.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,397	0 psf
Mu' : Upward	=	1,216	0 ft-#
Mu' : Downward	=	169	70 ft-#
Mu: Design	=	1,047	70 ft-#
Actual 1-Way Shear	=	9.42	3.51 psi
Allow 1-Way Shear	=	40.00	40.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Heel: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Key: No key defined

Min footing T&S reinf Area 0.56 in2
Min footing T&S reinf Area per foot 0.22 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 11.11 in	#4@ 22.22 in
#5@ 17.22 in	#5@ 34.44 in
#6@ 24.44 in	#6@ 48.89 in

Summary of Overturning & Resisting Forces & Moments

	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure =	595.5	1.94	1,157.9	Soil Over Heel =	229.0	2.37	543.8
Surcharge over Heel =				Sloped Soil Over Heel =			
Surcharge Over Toe =				Surcharge Over Heel =			
Adjacent Footing Load =				Adjacent Footing Load =			
Added Lateral Load =				Axial Dead Load on Stem =			
Load @ Stem Above Soil =				* Axial Live Load on Stem =			
=				Soil Over Toe =			
				Surcharge Over Toe =			
Total	595.5	**O.T.M.**	1,157.9	Stem Weight(s) =	550.0	1.83	1,008.3
=		=		Earth @ Stem Transitions =			
				Footing Weight =	322.9	1.29	417.0
				Key Weight =			
				Vert. Component =			
				Total =	1,101.9 lbs	**R.M.=**	1,969.1

Resisting/Overturning Ratio = 1.70
Vertical Loads used for Soil Pressure = 1,101.9 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.059	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.