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STRUCTURAL CALCULATIONS

FOR

SVA GREELEY

SHELL BUILDING

1911 59th Avenue
Greeley, CO 80634

NLI #25-001

PREPARED FOR

BATTISTA DESIGN GROUP

February 11, 2025

COLORADO LICENSED
MATTHEW W. NICHOLS
36279
02-11-25
PROFESSIONAL ENGINEER

CALCULATION INDEX

STRUCTURAL DESIGN CRITERIA

REFERENCES

A. International Building Code 2021 Edition
B. Greeley, Colorado Code Amendments
C. ASCE/SEI 7-16 Minimum Design Loads for Buildings and Other Structures
D. ACI 318-19 Building Code Requirements for Structural Concrete
E. ANSI/AISC 360-16 Specification for Structural Steel Buildings
F. ANSI/AWC NDS-2018 National Design Specification for Wood Construction
G. TMS 402-2016 Building Code for Masonry Structures
H. Geotechnical Engineering Report by Ninyo & Moore dated October 23, 2024 (Project No. 803044001).

DESIGN GRAVITY LOADS

A. LIVE LOADS
   a. Office 50 psf + 15 psf Partition Load
   b. Lobby & 1st Floor Corridors 100 psf

B. ROOF SNOW LOADS 30 psf + Drifting Requirements

C. ROOF DEAD LOADS 15 psf

WIND LOAD DESIGN CRITERIA

A. Building Risk Category II
B. Basic Wind Speed = 115 mph
C. Exposure Category C

SEISMIC LOAD DESIGN CRITERIA

A. Building Risk Category II
B. Site Class D
C. Site Spectral Response Accelerations:
   1. Ss = 0.153
   2. S1 = 0.051
   3. SDs = 0.164
   4. SD1 = 0.081
D. Seismic Importance Factor, Ie = 1.0
E. Seismic Design Category B
F. Basic Seismic Force Resisting System: Wood-framed shearwalls with wood panels
G. Response Modification Factor, R = 6.5
H. Seismic Response Coefficient, Cs = 0.0252
I. Analysis Procedure Used: Equivalent Lateral Force Procedure

MATERIALS

A. CONCRETE

a.	Footings	3,500 psi
b.	Foundation walls & pilasters:	3,500 psi ; 0.55 max w/c ratio, 5% air
c.	Interior slab-on-grade:	3,500 psi
d.	Site Trash Enclosure slab & foundation:	4,000 psi ; 0.40 max w/c ratio, 6% air
e.	Reinforcing steel, ASTM A615:	60,000 psi
f.	WWF reinforcing, ASTM A185 or A497	

B. STRUCTURAL STEEL

a.	WF shapes:	ASTM A992, Grade 50
b.	Channels, plates, angles:	ASTM A36
c.	HSS tubing:	ASTM A500, Grade B
d.	Bolts, nuts:	ASTM A325

C. WOOD

a.	Sawn Dimension lumber:	Hem Fir, No. 2 grade or better
b.	Manufactured LVL's	Per Supplier, 2.0E grade minimum
c.	Manufactured I-Joists	Per Supplier
d.	Sheathing:	APA PRP-108 or PS 1-07

D. MASONRY

a.	Masonry Units:	Medium-weight ASTM C-90
b.	Assembly Strength, f'm:	1,500 psi
c.	Grout, 28 days:	3,000 psi
d.	Mortar:	ASTM C270

FOUNDATIONS

A. FOUNDATION WALLS WITH STRIP & PAD FOOTINGS

a.	Allowable soil bearing pressure:	3,000 psf
b.	Frost Depth:	36"

ASCE 7-16 Snow Loads

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC. (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Snow Load Design Criteria

Flat Roof Snow Loads

Description : Flat Roof Snow Load

per ASCE 7-16, Chapter 7

Ground Snow Load, per Fig 7.2	30.00 psf	Roof Slope, Sec .7.3.4	5.00
Terrain Category	C (see ASCE 7-16 Section 26.7)	Roof Configuration	Monoslope
Exposure of Roof	Partially Exposed		
Ce : Exposure Factor, Table 7.3-:	1.00	pm, Minimum required	**20.00** psf
Ct : Thermal Factor	1.2 : Unheated and open air structures	pf, Calculated Snow Load per Equation 7-1	**25.20** psf
Risk Category, per Table 1.5-:	**II**	pf, Design Snow Load Max(pm min, pf calc)	**25.20** psf
Importance Factor, Is, Table 1.5-2	1.00		

Snow Drifts on Roof Projections

Description : Drift Case A

per ASCE 7-16, Chapt

Balanced Snow Load	21.00 psf	hd : windward	1.84 ft
Ground Snow Load	30.00 psf	hd : Design	1.84 ft
lu-upwind	49.00 ft	pd : Max Drift Only	32.99 psf
Height of Projection	10.00 ft	pd + Balanced	53.99 psf
Snow Density	17.90 pcf	W : Drift Width	7.37 ft
hb : Balanced	1.17 ft		
hc : Ht. of Projection - hb	8.83 ft		
hc / hb	7.52		
Importance Factor	1.00		

Description : Drift Case B

per ASCE 7-16, Chapt

Balanced Snow Load	21.00 psf	hd : windward	1.34 ft
Ground Snow Load	30.00 psf	hd : Design	1.34 ft
lu-upwind	28.00 ft	pd : Max Drift Only	23.95 psf
Height of Projection	10.00 ft	pd + Balanced	44.95 psf
Snow Density	17.90 pcf	W : Drift Width	5.35 ft
hb : Balanced	1.17 ft		
hc : Ht. of Projection - hb	8.83 ft		
hc / hb	7.52		
Importance Factor	1.00		

Description : Drift Case C

per ASCE 7-16, Chapt

Balanced Snow Load	21.00 psf	hd : windward	2.73 ft
Ground Snow Load	30.00 psf	hd : Design	2.73 ft
lu-upwind	107.00 ft	pd : Max Drift Only	48.79 psf
Height of Projection	15.00 ft	pd + Balanced	69.79 psf
Snow Density	17.90 pcf	W : Drift Width	10.90 ft
hb : Balanced	1.17 ft		
hc : Ht. of Projection - hb	13.83 ft		
hc / hb	11.79		
Importance Factor	1.00		

Description : Drift Case D

per ASCE 7-16, Chapt

Balanced Snow Load	21.00 psf	hd : windward	2.48 ft
Ground Snow Load	30.00 psf	hd : Design	2.48 ft
lu-upwind	88.00 ft	pd : Max Drift Only	44.44 psf
Height of Projection	15.00 ft	pd + Balanced	65.44 psf
Snow Density	17.90 pcf	W : Drift Width	9.93 ft
hb : Balanced	1.17 ft		
hc : Ht. of Projection - hb	13.83 ft		
hc / hb	11.79		
Importance Factor	1.00		

Snow Drifts on Roof Projections

ASCE 7-16 Snow Loads

Project File: 25-001.ec6

DESCRIPTION: Snow Load Design Criteria

Description : Drift Case E

per ASCE 7-16, Chapt

Balanced Snow Load	21.00 psf	hd : windward	1.60 ft
Ground Snow Load	30.00 psf	hd : Design	1.60 ft
lu-upwind	38.00 ft	pd : Max Drift Only	28.67 psf
Height of Projection	15.00 ft	pd + Balanced	49.67 psf
Snow Density	17.90 pcf	W : Drift Width	6.41 ft
hb : Balanced	1.17 ft		
hc : Ht. of Projection - hb	13.83 ft		
hc / hb	11.79		
Importance Factor	1.00		

ASCE 7-16 Wind Forces Chpt 28, Pt2 & Chpt 30, Pt2

Project File: 25-001.ec6

DESCRIPTION: Wind Load Design Criteria

General Design Values

Calculations per ASCE 7-16

V : Basic Wind Speed per Sect 26.5-1 or 2	**115.0**	mph	
User specified minimum design pressu	16.0	psf	
Occupancy per Table 1.5-1	II		All Buildings and other structures except those listed
Exposure Category per 26.7	Exposure C		
Topographic Factor Kzt per 26.8	1.00		

"Lambda" is interpolated between height tablular values.

Main Force Resisting System Valu

MRH : Mean Roof Height	15.0	ft
Roof Slope Angle	0 to 5	degrees

Lambda MWFRS: per Figure 26. 1.21

Component & Cladding Values

Effective Wind Area of Component & Cladc	10.0	ft^2
Roof pitch for cladding pressu	Flat/Hip/Gable Roof	
LHD : Least Horizontal Dimension	52.0	ft
a = max (0.04 * LHD, 3, min(0.10 * LHD, 0.4*MRH))	5.20	ft

Lambda Component & Cladding : per Figur 1.21

Design Wind Pressures

Horizontal Pressures . . .

Zone: A =	25.41	psf	Zone: C =	16.82	psf
Zone: B =	-16.00	psf	Zone: D =	-16.00	psf

Vertical Pressures . . .

Zone: E =	-30.49	psf	Zone: G =	-21.18	psf
Zone: F =	-17.30	psf	Zone: H =	-16.00	psf

Overhangs . . .

Zone: Eoh =	-42.71	psf	Zone: Goh =	-33.40	psf

ASCE 7-16 Section 28.5.4 Minimum Design Wind Loads requires that the load effects of the design wind pressures from Section 28.5.3 shall not be less than a minimum load defined by assuming the pressures, ps, for zones A and C equal to +16 psf, Zones B and D equal to +8 psf, while assuming ps for Zones E, F, G, and H are equal to 0 psf.

Component & Cladding Design Wind Pres:

*Design Wind Pressure = Lambda * Kzt * Ps30 pe*

Roof Pressures	Positive	Negative		Overhang Pressures	Negative	
Zone 1	16.000	-45.859	psf	Zone 1	-41.503	psf
Zone 1'	16.000	-26.378	psf	Zone 1'	-41.503	psf
Zone 2	16.000	-60.500	psf	Zone 2	-56.144	psf
Zone 2e	***	***	psf	Zone 2e	***	psf
Zone 2n	***	***	psf	Zone 2n	***	psf
Zone 2r	***	***	psf	Zone 2r	***	psf
Zone 3	16.000	-82.401	psf	Zone 3	-78.045	psf
Zone 3e	***	***	psf	Zone 3e	***	psf
Zone 3r	***	***	psf	Zone 3r	***	psf

Wall Pressures

Wall Zone 4 :	28.798	-31.218	psf
Wall Zone 5 :	28.798	-38.599	psf

'*** : There is no value in Figure 30.4-1 Tabular Values

ASCE 7-16 Seismic Base Shear

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC.

DESCRIPTION: Seismic Base Shear Analysis

Specific Description: Seismic Design Criteria

Risk Category

Calculations per ASCE 7-16

Risk Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ASCE 7-16, Page 4, Table 1.5-1*

Seismic Importance Factor = 1 — *ASCE 7-16, Page 5, Table 1.5-2*

USER DEFINED Ground Motion

ASCE 7-16 11.4.2

Max. Ground Motions, 5% Damping

S_S = 0.1530 g, 0.2 sec response
S_1 = 0.0510 g, 1.0 sec response

For the closest datapoint grid location . . .

Latitude = 0.000 deg North
Longitude = 0.000 deg West

Site Class, Site Coeff. and Design Category

Classification: "D" : Shear Wave Velocity 600 to 1,200 ft/sec		=	**D** (Based on Testing)	*ASCE 7-16 Table 20.3-1*
Site Coefficients Fa & Fv	Fa	=	1.60	*ASCE 7-16 Table 11.4-1 & 11.4-2*
(using straight-line interpolation from table val	Fv	=	2.40	
Maximum Considered Earthquake Accelerat	S_{MS} = Fa * Ss	=	0.245	*ASCE 7-16 Eq. 11.4-1*
	S_{M1} = Fv * S1	=	0.122	*ASCE 7-16 Eq. 11.4-2*
Design Spectral Acceleration	S_{DS} = S_{MS} * 2/3	=	0.163	*ASCE 7-16 Eq. 11.4-3*
	S_{D1} = S_{M1} * 2/3	=	0.082	*ASCE 7-16 Eq. 11.4-4*
Seismic Design Category		=	**B**	*\SCE 7-16 Table 11.6-1 & -2*

Resisting System

ASCE 7-16 Table 12.2-1

Basic Seismic Force Resisting System . . . **Bearing Wall Systems**
15.Light-frame (wood) walls sheathed w/wood structural panels rated for shear resistance.

Response Modification Coefficient " F	=	6.50
System Overstrength Factor " Wo "	=	3.00
Deflection Amplification Factor " Cd "	=	4.00

NOTE! See ASCE 7-16 for all applicable footno

Building height Limits :

Category "A & B" Limit:	No Limit
Category "C" Limit:	No Limit
Category "D" Limit:	Limit = 65
Category "E" Limit:	Limit = 65
Category "F" Limit:	Limit = 65

Lateral Force Procedure

ASCE 7-16 Section 12.8.2

Equivalent Lateral Force Procedure

The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-16 12.8

Determine Building Period

Use ASCE 12.8-7

Structure Type for Building Period CalculaAll Other Structural Systems

" Ct " value = 0.020 " hn " : Height from base to highest level 14.0 ft
" x " value = 0.75

" Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.145 sec
"TL" : Long-period transition period per ASCE 7-16 Maps 22-14 -> 22-17 4.000 sec

Building Period " Ta " Calculated from Approximate Method sel= 0.145

" Cs " Response Coefficient

ASCE 7-16 Section 12.8.1.1

S_{DS} : Short Period Design Spectral Response	=	0.163	From Eq. 12.8-2, Preliminary Cs	=	0.025
" R " : Response Modification Factor	=	6.50	From Eq. 12.8-3 & 12.8-4 , Cs need not excee	=	0.087
" I " : Seismic Importance Factor	=	1	From Eq. 12.8-5 & 12.8-6, Cs not be less than	=	0.010

Cs : Seismic Response Coefficient = = **0.0251**

Steel Beam

Project File: 25-001.ec6

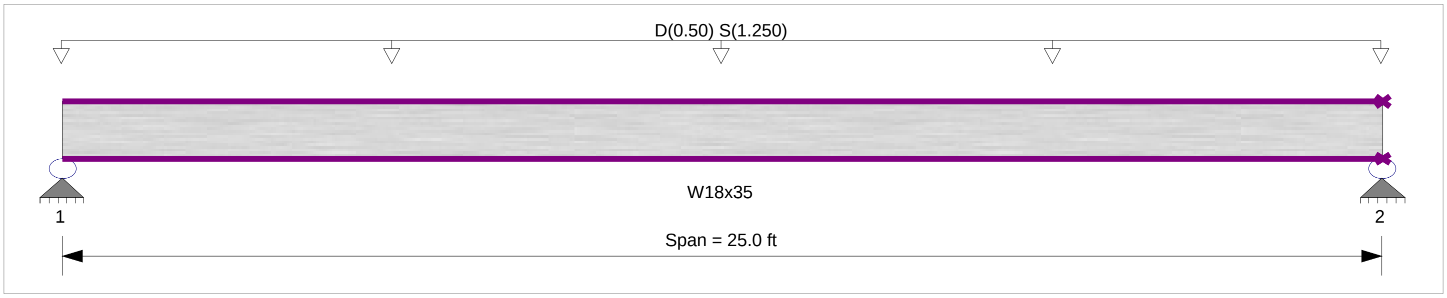
DESCRIPTION: WF Roof Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.050 ksf, Tributary Width = 25.0 ft, (Roof Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.828** : 1	Maximum Shear Stress Ratio =	**0.207** : 1
Section used for this span	**W18x35**	Section used for this span	**W18x35**
Mu : Applied	206.406 k-ft	Vu : Applied	33.025 k
Mn * Phi : Allowable	249.375 k-ft	Vn * Phi : Allowable	159.30 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.745 in	Ratio =	402 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.066 in	Ratio =	282 >=240.	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240.0	n/a

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	1.0656	12.571		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	22.313	22.313
Max Upward from Load Combinations	22.313	22.313
Max Upward from Load Cases	15.625	15.625
D Only	6.688	6.688
+D+S	22.313	22.313
+D+0.750S	18.406	18.406
+0.60D	4.013	4.013
S Only	15.625	15.625

Steel Column

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC. (c) ENERCALC, LLC 1982-2025

DESCRIPTION: HSS Interior Column

Code References

Calculations per AISC 360-16, IBC 2021
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS5x5x1/4**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 46.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 16 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 16 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 16 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 249.920 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 16.0 ft, D = 12.50, S = 31.250 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7138** : 1
Load Combination +1.20D+1.60S
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu 65.30 k
0.9 * Pn 91.485 k
Mu-x 0.0 k-ft
0.9 * Mn-x : 26.255 k-ft
Mu-y 0.0 k-ft
0.9 * Mn-y : 26.255 k-ft

PASS Maximum Shear Stress Ratio **0.0** : 1
Load Combination 0.0
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.0 k
Vn * Phi : Allowable 0.0 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum Load Deflections . . .
Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.195	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft
+1.20D	0.167	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft
+1.20D+0.50S	0.338	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft
+1.20D+1.60S	0.714	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft
+1.20D+0.70S	0.406	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft
+0.90D	0.125	PASS	0.00 ft	1.00	1.00	99.48	99.48	0.000	PASS	0.00 ft

Steel Beam

Project File: 25-001.ec6

DESCRIPTION: Central Tower WF Beam A

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, S = 0.070 ksf, Tributary Width = 2.0 ft, (Main Roof Load)
Uniform Load : D = 0.020, S = 0.030 ksf, Tributary Width = 11.0 ft, (Tower Roof Load)
Uniform Load : D = 0.20 k/ft, Tributary Width = 1.0 ft, (Tower Wall Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.634 : 1	Maximum Shear Stress Ratio =	0.152 : 1
Section used for this span	**W14x22**	Section used for this span	**W14x22**
Mu : Applied	78.892 k-ft	Vu : Applied	14.344 k
Mn * Phi : Allowable	124.500 k-ft	Vn * Phi : Allowable	94.530 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.431 in	Ratio =	612 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.853 in	Ratio =	309 >=240.	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240.0	n/a

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.8533	11.063		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	10.230	10.230
Max Upward from Load Combinations	10.230	10.230
Max Upward from Load Cases	5.170	5.170
D Only	5.060	5.060
+D+S	10.230	10.230
+D+0.750S	8.938	8.938
+0.60D	3.036	3.036
S Only	5.170	5.170

Steel Beam

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC. (c) ENERCALC, LLC 1982-2025

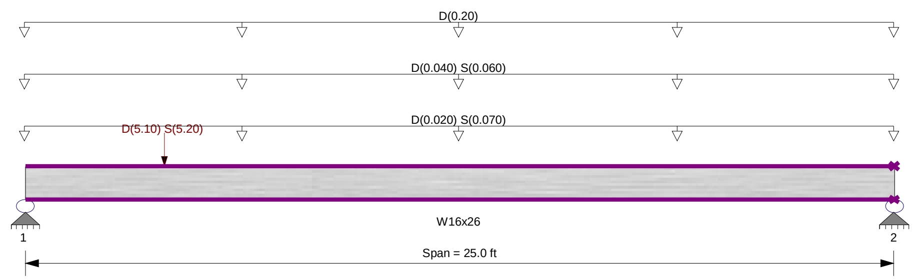
DESCRIPTION: Central Tower WF Beam B

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, S = 0.070 ksf, Tributary Width = 1.0 ft, (Main Roof Load)
Uniform Load : D = 0.020, S = 0.030 ksf, Tributary Width = 2.0 ft, (Tower Roof Load)
Uniform Load : D = 0.20 k/ft, Tributary Width = 1.0 ft, (Tower Wall Load)
Point Load : D = 5.10, S = 5.20 k @ 4.0 ft, (Tower Beam A Reaction)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.450 : 1	Maximum Shear Stress Ratio =	0.176 : 1
Section used for this span	W16x26	Section used for this span	W16x26
Mu : Applied	74.638 k-ft	Vu : Applied	18.630 k
Mn * Phi : Allowable	165.750 k-ft	Vn * Phi : Allowable	105.975 k
Load Combination	+1.20D+1.60S	Load Combination	+1.20D+1.60S
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.290 in Ratio =	1,035 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in Ratio =	0 <360	n/a
Max Downward Total Deflection	0.707 in Ratio =	424 >=240.	Span: 1 : +D+S
Max Upward Total Deflection	0 in Ratio =	0 <240.0	n/a

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+S	0.7069	11.786		0.0000	0.000

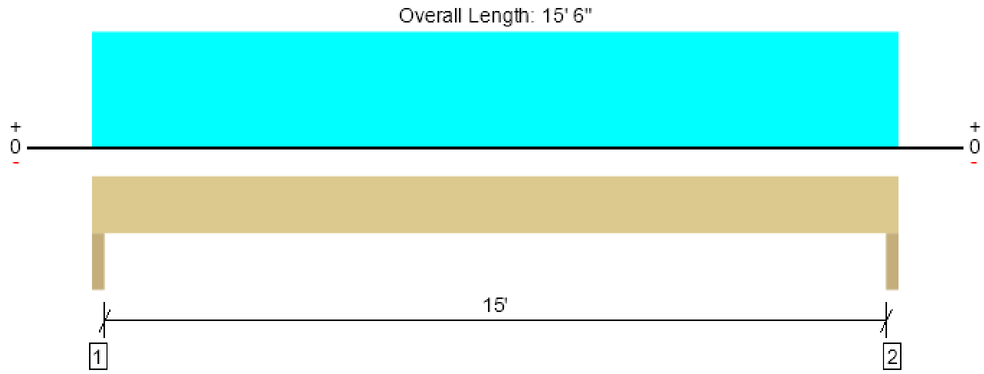
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	13.527	6.523
Max Upward from Load Combinations	13.527	6.523
Max Upward from Load Cases	7.534	4.066
D Only	7.534	4.066
+D+S	13.527	6.523
+D+0.750S	12.029	5.909
+0.60D	4.520	2.440
S Only	5.993	2.457

Level, LVL at Exterior Tower Wall

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5669 @ 1 1/2"	11419 (3.00")	Passed (50%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	4632 @ 1' 5"	16060	Passed (29%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	21263 @ 7' 9"	41846	Passed (51%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.182 @ 7' 9"	0.508	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.404 @ 7' 9"	0.762	Passed (L/453)	--	1.0 D + 1.0 S (All Spans)

Member Length : 15' 6"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.50"	3111	2558	5669	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	3111	2558	5669	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 6" o/c	
Bottom Edge (Lu)	15' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

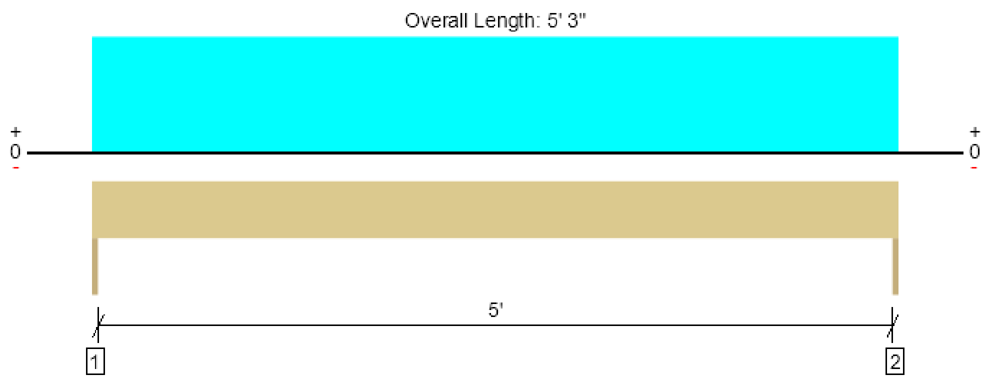
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 15' 6"	N/A	21.5	--	
1 - Uniform (PLF)	0 to 15' 6"	N/A	380.0	330.0	Tower Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Matt Nichols Next Level, Inc. (303) 260-9456 matt@nlengineers.com	

Level, Header H1

3 piece(s) 2 x 10 HF No.2



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2325 @ 0	2734 (1.50")	Passed (85%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1531 @ 10 3/4"	4787	Passed (32%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3051 @ 2' 7 1/2"	5750	Passed (53%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.028 @ 2' 7 1/2"	0.175	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.039 @ 2' 7 1/2"	0.262	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 5' 3"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	684	1641	2325	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	684	1641	2325	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 3" o/c	
Bottom Edge (Lu)	5' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 5' 3"	N/A	10.6	--	
1 - Uniform (PSF)	0 to 5' 3"	12' 6"	20.0	50.0	Roof Load

Weyerhaeuser Notes

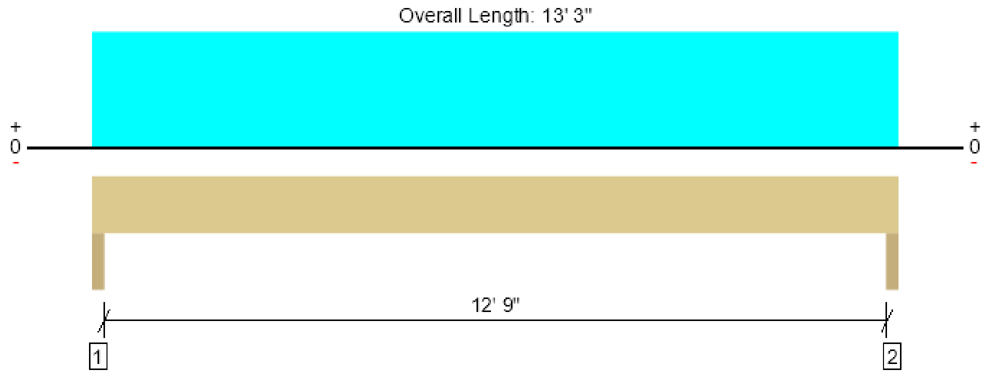
ForteWEB Software Operator	Job Notes
Matt Nichols Next Level, Inc. (303) 260-9456 matt@nlengineers.com	

Weyerhaeuser

2/10/2025 6:48:21 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 25-001 - SVA Greeley

Level, Header H2

3 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4095 @ 1 1/2"	11419 (3.00")	Passed (36%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3329 @ 1' 2 7/8"	13622	Passed (24%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	13059 @ 6' 7 1/2"	30788	Passed (42%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.172 @ 6' 7 1/2"	0.433	Passed (L/907)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.295 @ 6' 7 1/2"	0.650	Passed (L/528)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Member Length : 13' 3"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.50"	1710	2385	4095	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	1710	2385	4095	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 3" o/c	
Bottom Edge (Lu)	13' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 13' 3"	N/A	18.2	--	
1 - Uniform (PSF)	0 to 13' 3"	12'	20.0	30.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Matt Nichols Next Level, Inc. (303) 260-9456 matt@nlengineers.com	

Weyerhaeuser

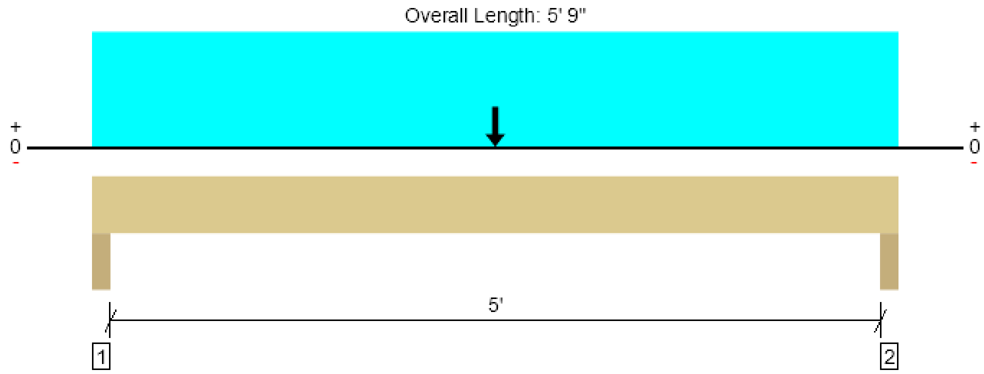
1/14/2025 6:08:31 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 25-001 - SVA Greeley

MEMBER REPORT

PASSED

Level, Header H3

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	11402 @ 3"	17128 (4.50")	Passed (67%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	11153 @ 1' 6 1/2"	16060	Passed (69%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	29267 @ 2' 10 1/2"	41846	Passed (70%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.067 @ 2' 10 1/2"	0.175	Passed (L/936)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.094 @ 2' 10 1/2"	0.262	Passed (L/667)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Member Length : 5' 9"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	4.50"	4.50"	3.00"	3302	8100	11402	None
2 - Trimmer - HF	4.50"	4.50"	3.00"	3302	8100	11402	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 9" o/c	
Bottom Edge (Lu)	5' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 5' 9"	N/A	21.5	--	
1 - Uniform (PSF)	0 to 5' 9"	2'	20.0	50.0	Roof Load
2 - Point (lb)	2' 10 1/2"	N/A	6250	15625	WF Roof Beam Rxn

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Matt Nichols Next Level, Inc. (303) 260-9456 matt@nlengineers.com	

Steel Beam

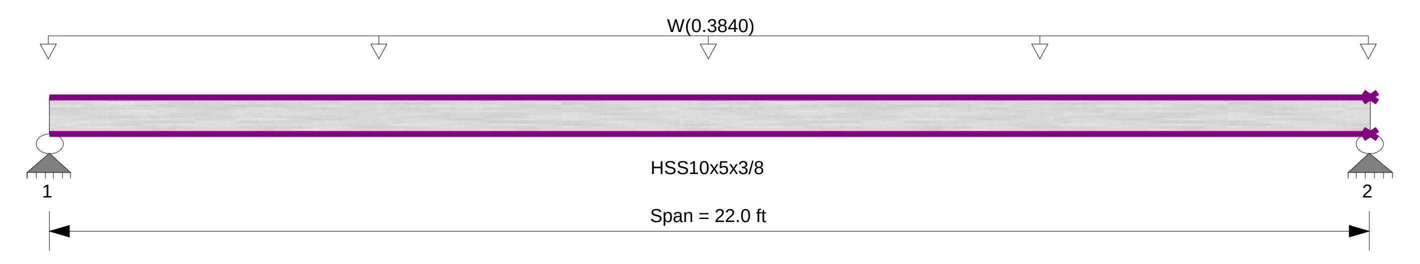
DESCRIPTION: Central Tower HSS Horiz. Girt - Horiz Wind Load

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Load Resistance Factor Design	Fy : Steel Yield :	46.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.0320 ksf, Tributary Width = 12.0 ft, (Horiz. Wind Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.360 : 1	Maximum Shear Stress Ratio =	0.062 : 1
Section used for this span	HSS10x5x3/8	Section used for this span	HSS10x5x3/8
Mu : Applied	23.232 k-ft	Vu : Applied	4.224 k
Mn * Phi : Allowable	64.515 k-ft	Vn * Phi : Allowable	68.538 k
Load Combination	W Only	Load Combination	W Only
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	1.036 in	Ratio =	255 >=240.	Span: 1 : +0.60W
Max Upward Total Deflection	0 in	Ratio =	0 <240.0	n/a

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+0.60W	1.0361	11.063		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.224	4.224
Max Upward from Load Combinations	2.534	2.534
Max Upward from Load Cases	4.224	4.224
+0.60W	2.534	2.534
+0.450W	1.901	1.901
W Only	4.224	4.224

General Footing

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC. (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Interior Column Footing

Code References

Calculations per ACI 318-19, IBC 2021
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	3.0 ksf
Soil Density	=	120.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	1.670 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

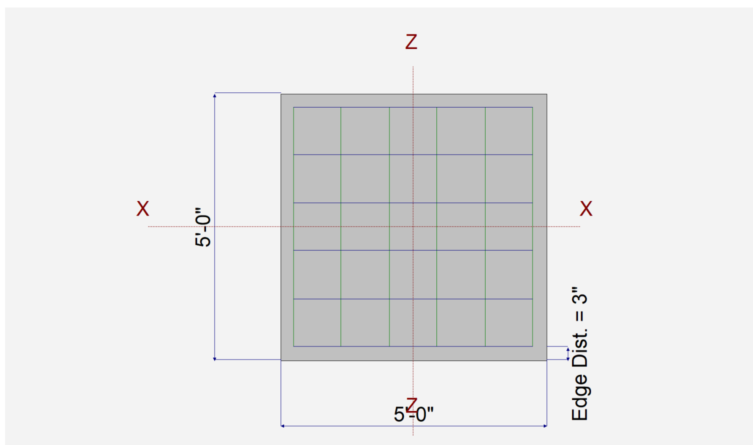
Width parallel to X-X Axis	=	5.0 ft
Length parallel to Z-Z Axis	=	5.0 ft
Footing Thickness	=	16.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	11.0 in
pz : parallel to Z-Z Axis	=	11.0 in
Height	=	in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.250 in



Reinforcing

Bars parallel to X-X Axis

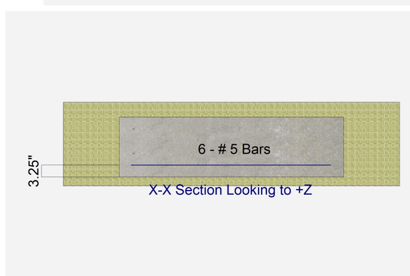
Number of Bars	=	6.0
Reinforcing Bar Size	=	# 5

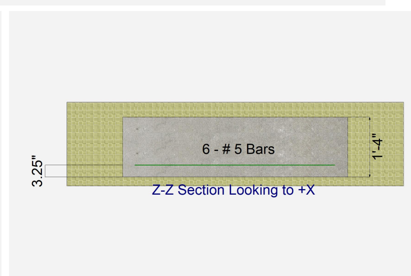
Bars parallel to Z-Z Axis

Number of Bars	=	6.0
Reinforcing Bar Size	=	# 5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	12.50			31.250				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC.

DESCRIPTION: Interior Column Footing

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6607	Soil Bearing	1.982 ksf	3.0 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2612	Z Flexure (+X)	5.415 k-ft/ft	20.733 k-ft/ft	+1.20D+1.60S
PASS	0.2612	Z Flexure (-X)	5.415 k-ft/ft	20.733 k-ft/ft	+1.20D+1.60S
PASS	0.2612	X Flexure (+Z)	5.415 k-ft/ft	20.733 k-ft/ft	+1.20D+1.60S
PASS	0.2612	X Flexure (-Z)	5.415 k-ft/ft	20.733 k-ft/ft	+1.20D+1.60S
PASS	0.3843	1-way Shear (+X)	16.983 psi	44.191 psi	+1.20D+1.60S
PASS	0.3843	1-way Shear (-X)	16.983 psi	44.191 psi	+1.20D+1.60S
PASS	0.3843	1-way Shear (+Z)	16.983 psi	44.191 psi	+1.20D+1.60S
PASS	0.3843	1-way Shear (-Z)	16.983 psi	44.191 psi	+1.20D+1.60S
PASS	0.2742	2-way Punching	45.049 psi	164.317 psi	+1.20D+1.60S

TYPICAL PERIMETER FTG.

- CONSIDER MAX. WALL LOADING

TOWER ROOF = $(20_D + 30_S) \times 23'/2 = 230_D + 345_S$ PLF

MAIN ROOF = $(20_D + 50_S) \times 25'/2 = 250_D + 625_S$ PLF

BLDG. WALL = $15_D \times 30' = 450_D$ PLF

FNDN. WALL = 150 PCF × 8"/12"/1 × 3' = 300_D PLF

$$\omega_{TOT.} = 1{,}230_D + 970_S = 2{,}200 \text{ PLF}$$

FOR ALLOW. SOIL BRG. = 3,000 PSF

& 2'-0" FTG. WIDTH,

$$P_{SOIL} = \frac{2{,}200 \text{ PLF}}{2'} = 1{,}100 \text{ PSF} < 3{,}000 \text{ PSF}$$

OK//

CHECK W.F. × ROOF BEAM BRG. AT N & S WALLS

- CONSIDER BM RXN. = $(20_D + 60_S) \times 25' \times 25'/2 = 6.25^K_D + 15.6^K_S$
- CONSIDER LOAD SPREAD OVER 6' LENGTH OF FTG.

$$\Rightarrow P_{SOIL} = (6.25^K + 15.6^K) \times \frac{1}{2'} \times \frac{1}{6'} = 1.82^{KSF} < 3^{KSF}$$

- FOR ADD'L ADJACENT WALL LOAD,

$$w = 15_D \times 30' = 450 \text{ PLF} \times \frac{1}{2'} \times \frac{1}{6'} = 37.5 \text{ PSF}$$

$$\Rightarrow 1.82 + 0.375 = 2.23 \text{ KSF}$$

OK//

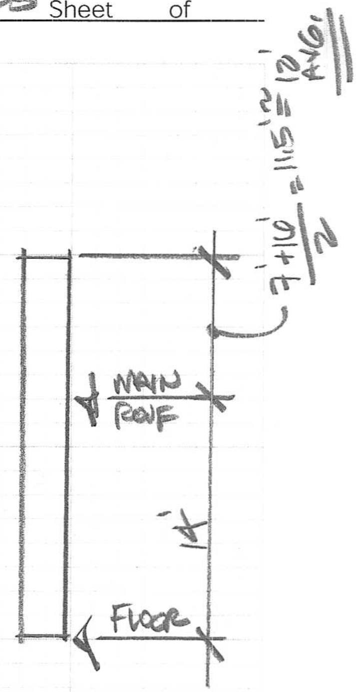
LATERAL ANALYSIS & DESIGN

LATERAL WIND LOADS

$V = 115$ MPH EXP. C $z = 5.2'$

$P_A = 26$ psf $P_C = 17$ psf (ULT.)

EFFECTIVE WALL TRIB. HT. $= \frac{14'}{2} + 12' = \underline{\underline{19'}}$



$w_A = 26$ psf $\times 19' = 494$ plf (ULT.)

$w_c = 17$ psf $\times 19' = 323$ plf (ULT.)

FOR INTERIOR SHEARWALLS

MAX ROOF TRIB. WIDTH $= 55'/2 = 27.5'$

$V_{w,max} = 323$ plf $\times 27.5' = 8{,}883^{\#}$ (ULT.)

$\times 0.6 = \underline{\underline{5.33^k}}$ (ASD)

FOR $L_{w,min} = 13'$

& $h_w = 14'$

$v_w = 5.33^k \times \frac{1}{13'} = 410$ plf

15/32" VALUE FOR STUDS 16" o.c. & SHEATHING INSTALLED HORIZ.

FOR 7/16" SHEATHING, 8d NAILS @ 4" o.c., $v_0 = \frac{1065}{2} = 532$ plf

$\Rightarrow$ FOR HEM-FIR FRMG., $[1-(0.5-0.43)] = 0.93$

& $v_w = 532 \times 0.93 = 495$ plf

OK

HOLDOWN FORCE

$$T_w = \frac{5.33^k \times 14'}{13'} = 5.74^k$$

FOR SIMPSON HDU8-SDS2.5 w/ 6x4 POST, $T_a = 5.82^k$ OK

FOR SSTB28 ON 8" CONC. WALL, $T_a = 6.33^k$ OK

FOR PAB7 w/ 5½" EMBED & 8½" MIN. SIDE CLEARANCE IN FOOTING, $T_a = 7.12k$ OK

EXTERIOR SHEARWALLS

EAST ELEVATION

$V_w = 494\ \text{PLF} \times 5.2' + 323\ \text{PLF} \times (25' - 5.2') = 8964^{\#}$ (ULT.)

$\times 0.6 = 5.38^k$ (ASD)

FOR (2) 13' LONG SHEARWALLS,

$V_{/WALL} = \underline{2.69}^k$; $v = \dfrac{2.69^k}{13'} = 207\ \text{PLF}$

$T = \dfrac{2.69 \times 14'}{13'} = 2.90^k$

7/16" SHEATHING W/ 8d NAILS @ 4" O.C. OK

HDU5-SDS2.5 W/ SSTB24 HOLDOWN ANCHOR OK//

WEST ELEVATION

- USE FTAO SHEARWALLS, PER THE FOLLOWING CALC SHEETS.
- WALL CONSTRUCTION PER EAST ELEV. OK//

NORTH & SOUTH ELEVATION

- $V_w = 494\ \text{PLF} \times 5.2' + 323\ \text{PLF} \times \left(\frac{25'}{2} - 5.2'\right) = 4{,}927^{\#}$ ULT.

  $\times 0.6 = \underline{2.96}^k$
- USE FTAO SHEARWALLS, PER THE FOLLOWING CALC SHEETS.
- WALL CONSTRUCTION PER EAST ELEV. OK//

CHECK LATERAL SEISMIC LOADS

W_{ROOF} = 20psf (D) x (152' x 51' + 23'x23' x2)
= 176.2^K

W_{WALLS} = 15psf x (14'/2 + 12') x (152' + 51') x 2
= 115.7^K

$W_{TOT.}$ = 291.9^K

$V_{E(TOT.)}$ = (291.9)(0.0251)
= 7.33^K (ULT.)

x0.7 = 5.13^K (ASD) → BUILDING TOTAL

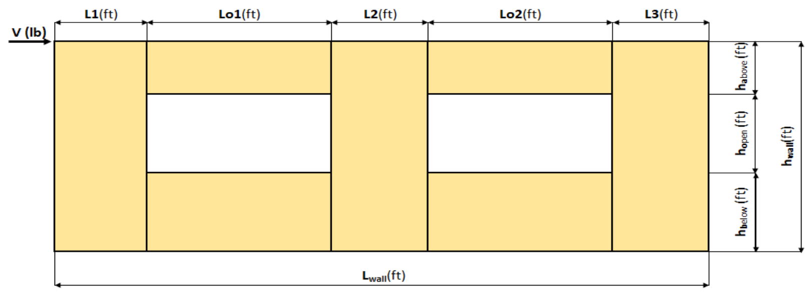
<< WIND LOADS

∴ WIND LOADS CONTROL DESIGN

Force Transfer Around Openings Calculator
TWO OPENINGS
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 ap[illegible] 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:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 1 - North End		



Shear Wall Calculation Variables

V	2690 lbf		Opening 1		Opening 2		
L1	3.42 ft	h_a1	4.33 ft	h_a2	4.33 ft		
L2	5.42 ft	h_o1	7.00 ft	h_o2	7.00 ft		
L3	4.00 ft	h_b1	2.67 ft	h_b2	2.67 ft		
h_{wall}	14.00 ft	Lo1	5.00 ft	Lo2	5.00 ft		
L_{wall}	22.84 ft						

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	2.05	0.994
$P2=h_o/L2=$	1.29	N/A
$P3=h_o/L3=$	1.75	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1649 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1)$ =	236 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2)$ =	236 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1178 lbf
Second opening: O2 = va2 x (Lo2) =	1178 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	456 lbf
F2 = O1(L2)/(L1+L2) =	722 lbf
F3 = O2(L2)/(L2+L3) =	678 lbf
F4 = O2(L3)/(L2+L3) =	500 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.93 ft
T2 = (L2*Lo1)/(L1+L2) =	3.07 ft
T3 = (L2*Lo2)/(L2+L3) =	2.88 ft
T4 = (L3*Lo2)/(L2+L3) =	2.12 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	184 plf
v2 = (V/L)(T2+L2+T3)/L2 =	247 plf
v3 = (V/L)(T4+L3)/L3 =	180 plf
Check v1*L1+v2*L2+v3*L3=V?	2690 lbf **OK**

7. Resistance to corner forces

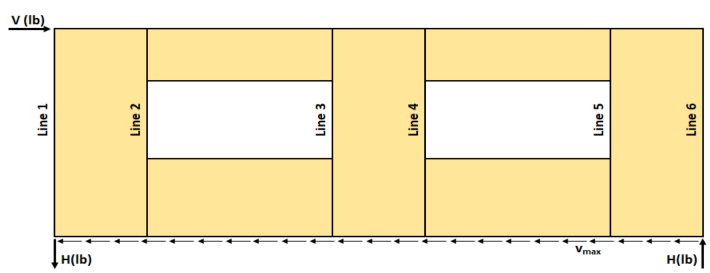
R1 = v1*L1 =	631 lbf
R2 = v2*L2 =	1338 lbf
R3 = v3*L3 =	721 lbf

8. Difference corner force + resistance

R1-F1 =	175 lbf
R2-F2-F3 =	-62 lbf
R3-F4 =	221 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	51 plf
vc2 = (R2-F2-F3)/L2 =	-11 plf
vc3 = (R3-F4)/L3 =	55 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		358	1291	1649 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	1649	358	1291	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	-79	1728	1649	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	1649	1728	-79	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	1649	387	1262	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H$?		387	1262	1649 lbf

Design Summary*

Req. Sheathing Capacity	247 plf	**4-Term Deflection**	0.272 in.	**3-Term Deflection**	0.358 in.
Req. Strap Force	722 lbf	**4-Term Story Drift %**	0.002 %	**3-Term Story Drift %**	0.002 %
Req. HD Force	1649 lbf				
Req. Shear Wall Anchorage Force	118 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 1 - North End		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Sheathing Type: 15/32 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: Hem Fir
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d:

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	4	4	(in.)
HD Capacity:	4340	4340	(lbf)
HD Deflection:	0.115	0.115	(in.)

Four-Term Equation Deflection Check

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

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	184	184	247	247	180	180	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	4	4	(in.)
V_n:	61	61	82	82	60	60	(plf)
e_n:	0.0011	0.0011	0.0028	0.0028	0.0011	0.0011	(in.)
b:	3.42	3.42	5.42	5.42	4.00	4.00	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(in.)

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

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
0.055	0.031	0.012	0.280	0.029	0.025	0.010	0.183
		Sum	0.378			Sum	0.247
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
0.025	0.034	0.023	0.155	0.025	0.034	0.023	0.155
		Sum	0.237			Sum	0.237
Pier 3 (left)				**Pier 3 (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	0.024	0.009	0.153	0.046	0.030	0.011	0.234
		Sum	0.211			Sum	0.322

Total Defl.	
0.272	(in.)
0.0016	%drift

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 1 - North End		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Wood End Post Values:

Sheathing Type:	15/32 OSB	Species:	Hem Fir	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

					Pier 1	Pier 3	
G_t Override:				Nail Spacing:	4	4	(in.)
G_a Override:		C_d:		HD Capacity:	4340	4340	(lbf)
				HD Deflection:	0.115	0.115	(in.)

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	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	184	184	247	247	180	180	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	19.0	19.0	19.0	19.0	19.0	19.0	(kips/in.)
b:	3.42	3.42	5.42	5.42	4.00	4.00	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(in.)

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

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
0.055	0.136	0.280	0.029	0.110	0.183
	Sum	0.471		Sum	0.323
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.025	0.147	0.155	0.025	0.147	0.155
	Sum	0.327		Sum	0.327
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.024	0.108	0.153	0.046	0.133	0.234
	Sum	0.285		Sum	0.413

Total Defl.	
0.358	(in.)
0.0021	%drift

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

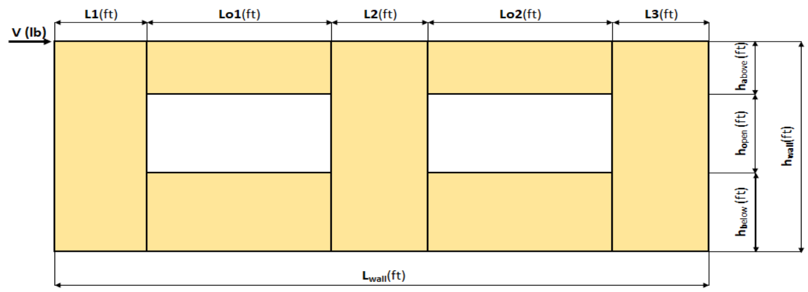
Force Transfer Around Openings Calculator

TWO OPENINGS

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 ap[…] 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:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 2 - South End		



Shear Wall Calculation Variables

V	2690 lbf	Opening 1		Opening 2		
L1	5.42 ft	h_a1	4.33 ft	h_a2	4.33 ft	
L2	5.42 ft	h_o1	7.00 ft	h_o2	7.00 ft	
L3	3.42 ft	h_b1	2.67 ft	h_b2	2.67 ft	
h_{wall}	14.00 ft	Lo1	5.00 ft	Lo2	5.00 ft	
L_{wall}	24.26 ft					

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	1.29	N/A
$P2=h_o/L2=$	1.29	N/A
$P3=h_o/L3=$	2.05	0.994

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1552 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	222 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	222 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1109 lbf
Second opening: O2 = va2 x (Lo2) =	1109 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	554 lbf
F2 = O1(L2)/(L1+L2) =	554 lbf
F3 = O2(L2)/(L2+L3) =	680 lbf
F4 = O2(L3)/(L2+L3) =	429 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.50 ft
T2 = (L2*Lo1)/(L1+L2) =	2.50 ft
T3 = (L2*Lo2)/(L2+L3) =	3.07 ft
T4 = (L3*Lo2)/(L2+L3) =	1.93 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	162 plf
v2 = (V/L)(T2+L2+T3)/L2 =	225 plf
v3 = (V/L)(T4+L3)/L3 =	174 plf
Check v1*L1+v2*L2+v3*L3=V?	2690 lbf **OK**

7. Resistance to corner forces

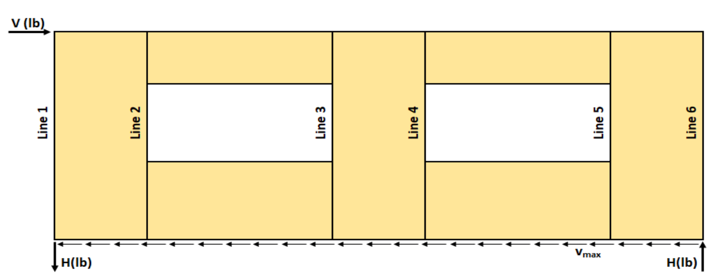
R1 = v1*L1 =	878 lbf
R2 = v2*L2 =	1218 lbf
R3 = v3*L3 =	594 lbf

8. Difference corner force + resistance

R1-F1 =	324 lbf
R2-F2-F3 =	-16 lbf
R3-F4 =	165 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	60 plf
vc2 = (R2-F2-F3)/L2 =	-3 plf
vc3 = (R3-F4)/L3 =	48 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		418	1134	1552 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	1552	418	1134	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	-21	1573	1552	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	1552	1573	-21	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	1552	337	1215	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H$?		337	1215	1552 lbf

Design Summary*

Req. Sheathing Capacity	225 plf	**4-Term Deflection**	0.230 in.	**3-Term Deflection**	0.310 in.
Req. Strap Force	680 lbf	**4-Term Story Drift %**	0.001 %	**3-Term Story Drift %**	0.002 %
Req. HD Force	1552 lbf				
Req. Shear Wall Anchorage Force	111 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 2 - South End		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Sheathing Type: 15/32 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: Hem Fir
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d:

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	4	4	(in.)
HD Capacity:	4340	4340	(lbf)
HD Deflection:	0.115	0.115	(in.)

Four-Term Equation Deflection Check

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

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	162	162	225	225	174	174	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	4	4	(in.)
V_n:	54	54	75	75	58	58	(plf)
e_n:	0.0008	0.0008	0.0021	0.0021	0.0010	0.0010	(in.)
b:	5.42	5.42	5.42	5.42	3.42	3.42	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(in.)

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

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
0.031	0.027	0.008	0.155	0.016	0.022	0.007	0.102
		Sum	0.221			Sum	0.146
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	0.030	0.018	0.141	0.022	0.030	0.018	0.141
		Sum	0.212			Sum	0.212
Pier 3 (left)				**Pier 3 (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
0.028	0.024	0.008	0.173	0.052	0.029	0.010	0.264
		Sum	0.232			Sum	0.355

Total Defl.	
0.230	(in.)
0.0014	%drift

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	West Wall 2 - South End		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Wood End Post Values:

Sheathing Type:	15/32 OSB	Species:	Hem Fir	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

		Pier 1	Pier 3	
G_t Override:				
G_a Override:				
C_d:				
Nail Spacing:		4	4	(in.)
HD Capacity:		4340	4340	(lbf)
HD Deflection:		0.115	0.115	(in.)

Three-Term Equation Deflection Check

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

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	162	162	225	225	174	174	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	19.0	19.0	19.0	19.0	19.0	19.0	(kips/in.)
b:	5.42	5.42	5.42	5.42	3.42	3.42	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(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
0.031	0.119	0.155	0.016	0.097	0.102
	Sum	0.305		Sum	0.215
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	0.134	0.141	0.022	0.134	0.141
	Sum	0.298		Sum	0.298
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.028	0.104	0.173	0.052	0.128	0.264
	Sum	0.304		Sum	0.443

Total Defl.	
0.310	(in.)
0.0018	%drift

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

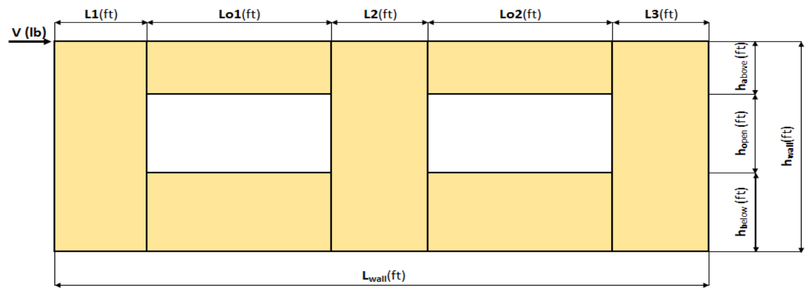
Force Transfer Around Openings Calculator

TWO OPENINGS

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 ap[illegible] 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:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	North & South Walls		



Shear Wall Calculation Variables

V	2960 lbf		Opening 1		Opening 2		
L1	6.75 ft	h_a1	4.33 ft	h_a2	4.33 ft		
L2	5.75 ft	h_o1	7.00 ft	h_o2	7.00 ft		
L3	4.75 ft	h_b1	2.67 ft	h_b2	2.67 ft		
h_{wall}	14.00 ft	Lo1	3.00 ft	Lo2	5.00 ft		
L_{wall}	25.25 ft						

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	1.04	N/A
$P2=h_o/L2=$	1.22	N/A
$P3=h_o/L3=$	1.47	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1641 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1)$ =	234 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2)$ =	234 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	703 lbf
Second opening: O2 = va2 x (Lo2) =	1172 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	380 lbf
F2 = O1(L2)/(L1+L2) =	324 lbf
F3 = O2(L2)/(L2+L3) =	642 lbf
F4 = O2(L3)/(L2+L3) =	530 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.62 ft
T2 = (L2*Lo1)/(L1+L2) =	1.38 ft
T3 = (L2*Lo2)/(L2+L3) =	2.74 ft
T4 = (L3*Lo2)/(L2+L3) =	2.26 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	145 plf
v2 = (V/L)(T2+L2+T3)/L2 =	201 plf
v3 = (V/L)(T4+L3)/L3 =	173 plf
Check v1*L1+v2*L2+v3*L3=V?	2960 lbf **OK**

7. Resistance to corner forces

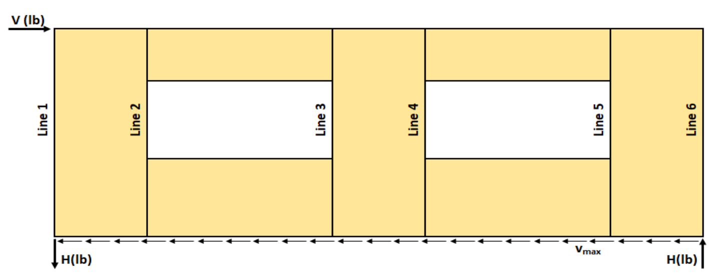
R1 = v1*L1 =	981 lbf
R2 = v2*L2 =	1157 lbf
R3 = v3*L3 =	822 lbf

8. Difference corner force + resistance

R1-F1 =	601 lbf
R2-F2-F3 =	191 lbf
R3-F4 =	292 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	89 plf
vc2 = (R2-F2-F3)/L2 =	33 plf
vc3 = (R3-F4)/L3 =	61 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		624	1018	1641 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	1641	624	1018	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	233	1408	1641	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	1641	1408	233	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	1641	430	1211	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H$?		430	1211	1641 lbf

Design Summary*

Req. Sheathing Capacity	234 plf	**4-Term Deflection**	0.161 in.	**3-Term Deflection**	0.232 in.
Req. Strap Force	642 lbf	**4-Term Story Drift %**	0.001 %	**3-Term Story Drift %**	0.001 %
Req. HD Force	1641 lbf				
Req. Shear Wall Anchorage Force	117 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	North & South Walls		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Sheathing Type: 15/32 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: Hem Fir
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d:

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	4	4	(in.)
HD Capacity:	4340	4340	(lbf)
HD Deflection:	0.115	0.115	(in.)

Four-Term Equation Deflection Check

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

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	132	132	183	183	157	157	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	4	4	(in.)
V_n:	44	44	61	61	52	52	(plf)
e_n:	0.0004	0.0004	0.0011	0.0011	0.0007	0.0007	(in.)
b:	6.75	6.75	5.75	5.75	4.75	4.75	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(in.)

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

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
0.020	0.022	0.004	0.102	0.011	0.018	0.004	0.067
		Sum	0.148			Sum	0.099
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
0.017	0.025	0.009	0.108	0.017	0.025	0.009	0.108
		Sum	0.160			Sum	0.160
Pier 3 (left)				**Pier 3 (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
0.018	0.021	0.006	0.113	0.034	0.026	0.007	0.172
		Sum	0.158			Sum	0.240

Total Defl.	
0.161	(in.)
0.0010	%drift

Project Information

Code:	2021 IBC	**Date:**	1/31/2025
Designer:	MWN		
Client:	Battista Design		
Project:	SVA Greeley		
Wall Line:	North & South Walls		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2690 (lbf)

Wood End Post Values:

Sheathing Type:	15/32 OSB	Species:	Hem Fir	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

			Pier 1	Pier 3
G_t Override:		Nail Spacing:	4	4 (in.)
G_a Override:	C_d:	HD Capacity:	4340	4340 (lbf)
		HD Deflection:	0.115	0.115 (in.)

Three-Term Equation Deflection Check

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

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	132	132	183	183	157	157	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	14.00	11.33	11.33	11.33	11.33	14.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	19.0	19.0	19.0	19.0	19.0	19.0	(kips/in.)
b:	6.75	6.75	5.75	5.75	4.75	4.75	(ft)
HD Capacity:	4340	4340	4340	4340	4340	4340	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	0.115	0.115	(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
0.020	0.097	0.102	0.011	0.079	0.067
	Sum	0.219		Sum	0.156
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.017	0.109	0.108	0.017	0.109	0.108
	Sum	0.234		Sum	0.234
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.018	0.094	0.113	0.034	0.116	0.172
	Sum	0.224		Sum	0.322

Total Defl.	
0.232	(in.)
0.0014	%drift

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

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3]

Wood-based Panels (Excluding Plywood for G_a)[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration in Framing (in.)	Fastener Type & Size	A SEISMIC Panel Edge Fastener Spacing (in.)								B WIND Panel Edge Fastener Spacing (in.)			
				6		4		3		2		6	4	3	2
				v_s (plf)	G_a (kips/in.)	v_s (plf)	G_a (kips/in.)	v_s (plf)	G_a (kips/in.)	v_s (plf)	G_a (kips/in.)	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
Wood Structural Panels - Structural I[4,5]			Nail (common or galvanized box)												
	5/16	1-1/4	6d	400	13.0	600	18.0	780	23.0	1020	35.0	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19.0	720	24.0	920	30.0	1220	43.0	645	1010	1290	1710
	7/16[2]			510	16.0	790	21.0	1010	27.0	1340	40.0	715	1105	1415	1875
	15/32			560	14.0	860	18.0	1100	24.0	1460	37.0	785	1205	1540	2045
	15/32	1-1/2	10d	680	22.0	1020	29.0	1330	36.0	1740	51.0	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13.0	540	18.0	700	24.0	900	37.0	505	755	980	1260
	3/8			400	11.0	600	15.0	780	20.0	1020	32.0	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17.0	640	25.0	820	31.0	1060	45.0	615	895	1150	1485
	7/16[2]			480	15.0	700	22.0	900	28.0	1170	42.0	670	980	1260	1640
	15/32			520	13.0	760	19.0	980	25.0	1280	39.0	730	1065	1370	1790
	15/32	1-1/2	10d	620	22.0	920	30.0	1200	37.0	1540	52.0	870	1290	1680	2155
	19/32			680	19.0	1020	26.0	1330	33.0	1740	48.0	950	1430	1860	2435
Plywood Siding			Nail (galvanized casing)												
	5/16	1-1/4	6d	280	13.0	420	16.0	550	17.0	720	21.0	390	590	770	1010
	3/8	1-3/8	8d	320	16.0	480	18.0	620	20.0	820	22.0	450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")			Nail (common or galvanized box)												
	3/8		6d	240	15.0	360	17.0	460	19.0	600	22.0	335	505	645	840
	3/8		8d	260	18.0	380	20.0	480	21.0	630	23.0	365	530	670	880
	1/2			280	18.0	420	20.0	540	22.0	700	24.0	390	590	755	980
	1/2		10d	370	21.0	550	23.0	720	24.0	920	25.0	520	770	1010	1290
	5/8			400	21.0	610	23.0	790	24.0	1040	26.0	560	855	1105	1455
Fiberboard Sheathing - Structural			Nail (common or galvanized roofing)												
	1/2		8d common or 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)			340	4.0	460	5.0	520	5.5		475	645	730
	25/32		8d common or 11 ga. galv. roofing nail (0.120" x 1-3/4" long x 7/16" head)			360	4.0	480	5.0	540	5.5		505	670	755

1. Nominal unit shear capacities shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls.
2. Shears are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
3. For framing grades other than Douglas Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1 – (0.5 – G)], where G = Specific Gravity of the framing lumber from the *NDS*. The Specific Gravity Adjustment Factor shall not be greater than 1.
4. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with OSB panels. When plywood panels are used, G_a values shall be determined in accordance with Appendix A.
5. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.

Holdowns (cont.)

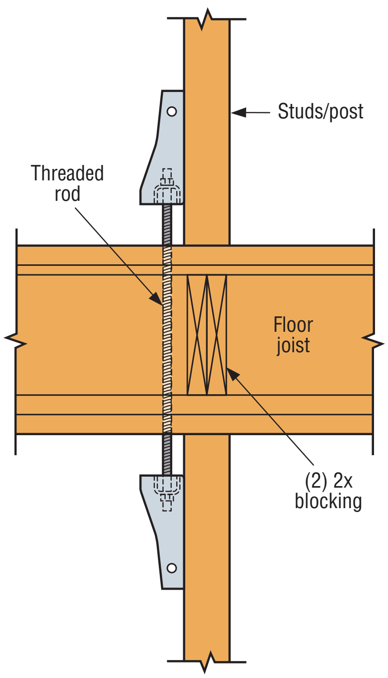
These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

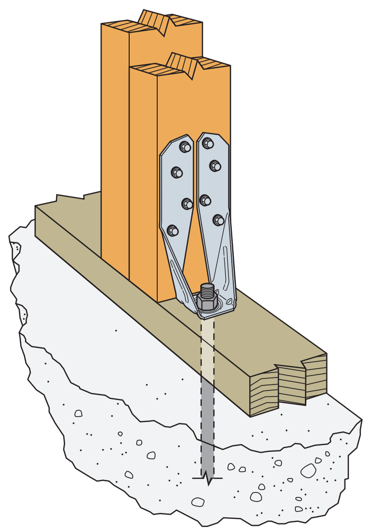
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)			Code Ref.
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	⅜	(6) #9 x 1½" SD	1½ x 3½	840	840	0.17	IBC®, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
DTT2Z	14	3¼	6 15/16	1⅝	13/16	3/16	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. **Fasteners:** Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. See pp. 23–24 for fastener information.



Typical HDU Tie Between Floors



Typical DTT2Z Installation

Pre-Assembled Anchor Bolt (cont.)

PAB Anchor Bolt – Anchorage Solutions

Design Criteria	Diameter (in.)	Anchor Bolt	2,500 psi Concrete				3,000 psi Concrete			
			Dimensions (in.)		Tension Load		Dimensions (in.)		Tension Load	
			d_e	F	ASD	LRFD	d_e	F	ASD	LRFD
Wind	½	PAB4	4½	7	4,270	6,405	4	6	4,270	6,405
	⅝	PAB5	4	6	4,030	6,720	4	6	4,415	7,360
			6	9	6,675	10,010	5½	8½	6,675	10,010
	¾	PAB6	5½	8½	6,500	10,835	5	7½	6,175	10,290
			7½	11½	9,610	14,415	7	10½	9,610	14,415
	⅞	PAB7	6	9	7,405	12,345	5½	8½	7,120	11,870
			9	13½	13,080	19,620	8½	13	13,080	19,620
		PAB7H	9	13½	13,610	22,680	8½	13	13,680	22,805
			14	21	27,060	40,590	13½	20½	27,060	40,590
	1	PAB8	8	12	11,405	19,005	7½	11½	11,340	18,900
			10½	16	17,080	25,565	10	15	17,080	25,560
		PAB8H	10½	16	17,150	28,580	10	15	17,460	29,100
			16½	25	35,345	53,015	15½	23½	35,345	53,015
	1⅛	PAB9	9	13½	13,610	22,680	8	12	12,495	20,820
			12½	19	21,620	32,430	12	18	21,620	32,430
	1¼	PAB10	14	21	26,690	40,035	13½	20½	26,690	40,035
Seismic	½	PAB4	5	7½	4,270	6,405	4½	7	4,270	6,405
	⅝	PAB5	6½	10	6,675	10,010	6	9	6,675	10,010
	¾	PAB6	7½	11½	9,060	12,940	7	10½	8,945	12,780
			8	12	9,610	14,415	7½	11½	9,610	14,415
	⅞	PAB7	9	13½	11,905	17,010	8½	13	11,970	17,100
			10	15	13,080	19,620	9½	14½	13,080	19,620
		PAB7H	14½	22	25,350	36,215	13½	20½	24,650	35,215
			15½	23½	27,060	40,590	14½	22	27,060	40,590
	1	PAB8	11	16½	15,996	22,850	10½	16	16,435	23,480
			11½	17½	17,080	25,625	11	16½	17,080	25,625
		PAB8H	17	25½	33,045	47,205	16	24	32,720	46,740
			18	27	35,345	53,015	17	25½	35,345	53,015
	1⅛	PAB9	12½	19	19,795	28,275	12	18	20,255	28,940
			13½	20½	21,620	32,430	12½	19	21,620	32,430
	1¼	PAB10	14½	22	25,350	36,215	14	21	26,190	37,415
			15	22½	26,690	40,035	14½	22	26,690	40,035

1. Anchorage designs conform to ACI 318-14 and assume cracked concrete with no supplementary reinforcement.
2. Seismic indicates Seismic Design Category C-F and designs comply with ACI 318-19, Section 17.10.5.3. Per Section 1613 of the 2012/2015/2018/2021 IBC®, detached one- and two-family dwellings in SDC C may use wind values.
3. Wind includes Seismic Design Category A and B.
4. Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) by designer. The registered design professional may specify alternative embedment, footing size, and anchor bolt.
5. Where tension loads are governed by anchor steel, the design provisions from AISC 360 are used to determine the tensile steel limit. LRFD values are calculated by multiplying the nominal AISC steel capacity by a 0.75 phi factor, and allowable values are calculated by dividing the AISC nominal capacity by a 2.0 omega factor.
6. Where tension loads are governed by ACI 318 concrete limit, the Allowable Stress Design (ASD) values are obtained by multiplying Load Resistance Factor Design (LRFD) capacities by 0.7 for seismic and by 0.6 for wind.

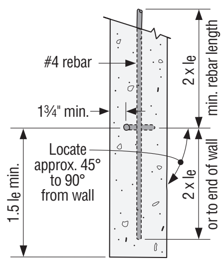
Anchor Bolt

These products are available with additional corrosion protection. For more information, see p. 16.

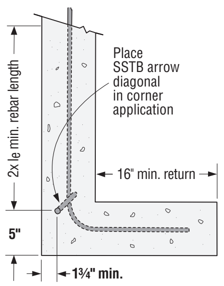
SSTB Bolts at Stemwall

Model No.	Dimensions (in.)				Allowable Tension Loads						Code Ref.
	Stemwall Width	Diameter	Length	Min. Embed. (l_e)	Wind and SDC A&B			SDC C–F			
					Midwall	Corner	End Wall[6]	Midwall	Corner	End Wall[6]	
SSTB16	6	⅝	17⅝ (16L = 19⅝)	12⅝	3,465	3,465	3,465	2,550	2,550	2,550	IBC®, FL, LA
SSTB20	6	⅝	21⅝ (20L = 24⅝)	16⅝	4,145	3,880	3,880	3,145	2,960	2,960	
SSTB24	6	⅝	25⅝ (24L = 28⅛)	20⅝	4,825	4,295	4,295	3,740	3,325	3,325	
SSTB28	8	⅞	29⅞ (28L = 32⅞)	24⅞	9,505	8,360	7,310	8,315	7,315	6,395	
SSTB34	8	⅞	34⅞	28⅞	9,505	8,360	7,310	8,315	7,315	6,395	
SSTB36	8	⅞	36⅞	28⅞	9,505	8,360	7,310	8,315	7,315	6,395	

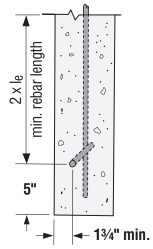
1. Rebar is required at the top of stem wall foundations, but is not required for slab-on-grade edge and garage curb, or stem wall garage front installations.
2. Minimum end distances for SSTB bolts are as shown in graphics.
3. To obtain LRFD values, multiply ASD seismic load values by 1.43 and wind load values by 1.67.
4. Per Section 1613 of the IBC, detached one- and two-story dwellings in SDC C may use "Wind and SDC A&B" allowable loads.
5. Midwall loads apply when anchor is 1.5 l_e or greater from the end. For bolts acting in tension simultaneously, the minimum bolt center-to-center spacing is 3 l_e.
6. SSTB28, SSTB34 and SSTB36 with 3⅞" end distance allowable loads are 6,330 lb. (Wind and SDC A&B) and 5,550 lb. (SDC C–F).



Midwall

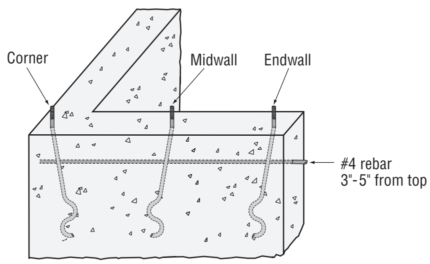


Corner



End Wall

Stemwall Plan Views

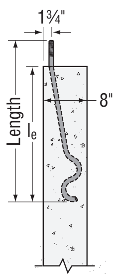


Perspective View

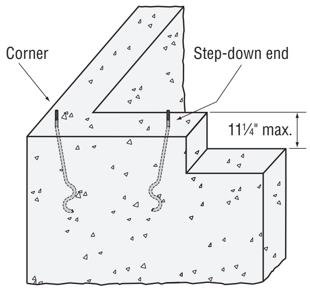
SSTB Bolts at Stemwall: Garage Front

Model No.	Dimensions (in.)				Allowable Tension Loads				Code Ref.
	Stemwall Width	Diameter	Length	Min. Embed. (le)	Wind and SDC A&B		SDC C–F		
					Step-Down End	Corner	Step-Down End	Corner	
SSTB28	8	⅞	29⅞	24⅞	6,735	6,765	5,895	5,920	IBC, FL, LA

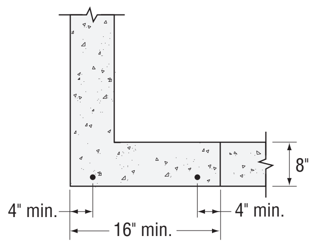
1. Rebar is required at the top of stem wall foundations, but is not required for slab-on-grade edge and garage curb, or stem wall garage front installations.
2. Minimum end distances for SSTB bolts are as shown in graphics.
3. To obtain LRFD values, multiply ASD seismic load values by 1.43 and wind load values by 1.67.
4. Per Section 1613 of the IBC, detached one- and two-story dwellings in SDC C may use "Wind and SDC A&B" allowable loads.
5. Midwall loads apply when anchor is 1.5 l_e or greater from the end. For bolts acting in tension simultaneously, the minimum bolt center-to-center spacing is 3 l_e.



Stemwall Garage Front



Perspective View



Plan View

Strap Ties (cont.)

Codes: See p. 13 for Code Reference Key Chart

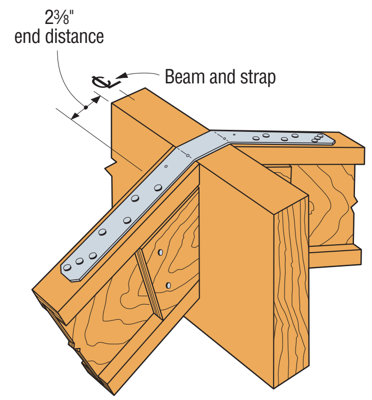
These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

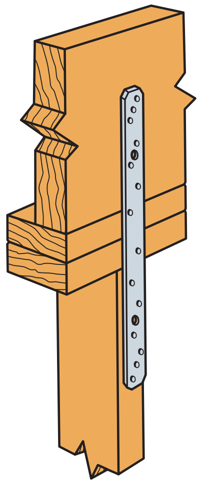
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

	Model No.	Ga.	Dimensions (in.) W	Dimensions (in.) L	Fasteners (Total) (in.)	DF/SP Allowable Tension Loads (160)	SPF/HF Allowable Tension Loads (160)	Code Ref.
	ST2115	20	¾	16 5/16	(10) 0.162 x 2½	660	660	IBC®, FL, LA
	LSTA9		1¼	9	(8) 0.148 x 2½	740	635	
	LSTA12		1¼	12	(10) 0.148 x 2½	925	795	
	LSTA15		1¼	15	(12) 0.148 x 2½	1,110	955	
	LSTA18		1¼	18	(14) 0.148 x 2½	1,235	1,115	
	LSTA21		1¼	21	(16) 0.148 x 2½	1,235	1,235	
	LSTA24		1¼	24	(18) 0.148 x 2½	1,235	1,235	
	LSTA30	18	1¼	30	(22) 0.148 x 2½	1,640	1,640	
	LSTA36		1¼	36	(24) 0.148 x 2½	1,640	1,640	
	MSTA9		1¼	9	(8) 0.148 x 2½	750	650	
SS	MSTA12		1¼	12	(10) 0.148 x 2½	940	810	
	MSTA15		1¼	15	(12) 0.148 x 2½	1,130	970	
SS	MSTA18		1¼	18	(14) 0.148 x 2½	1,315	1,135	
	MSTA21		1¼	21	(16) 0.148 x 2½	1,505	1,295	
SS	MSTA24		1¼	24	(18) 0.148 x 2½	1,640	1,460	
	MSTA30	16	1¼	30	(22) 0.148 x 2½	2,050	1,825	
SS	MSTA36		1¼	36	(26) 0.148 x 2½	2,050	2,050	
	MSTA49		1¼	49	(26) 0.148 x 2½	2,020	2,020	
	ST9		1¼	9	(8) 0.162 x 2½	885	765	
	ST12		1¼	11 5/8	(10) 0.162 x 2½	1,105	955	
	ST18		1¼	17¾	(14) 0.162 x 2½	1,420	1,335	
	ST22		1¼	21 5/8	(18) 0.162 x 2½	1,420	1,420	
	HRS6	12	1 3/8	6	(6) 0.148 x 2½	605	530	
	HRS8		1 3/8	8	(10) 0.148 x 2½	1,010	880	
	HRS12		1 3/8	12	(14) 0.148 x 2½	1,415	1,230	
	ST292	20	2 1/16	9 5/16	(12) 0.162 x 2½	1,260	1,120	
	ST2122		2 1/16	12 13/16	(16) 0.162 x 2½	1,530	1,510	
	ST2215		2 1/16	16 5/16	(20) 0.162 x 2½	1,875	1,875	
	ST6215	16	2 1/16	16 5/16	(20) 0.162 x 2½	2,090	1,910	
	ST6224		2 1/16	23 5/16	(28) 0.162 x 2½	2,535	2,535	
	ST6236	14	2 1/16	33 13/16	(40) 0.162 x 2½	3,845	3,845	
	MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380	
	MSTI36		2 1/16	36	(36) 0.148 x 1½	3,800	3,295	
	MSTI48		2 1/16	48	(48) 0.148 x 1½	5,070	4,390	
	MSTI60		2 1/16	60	(60) 0.148 x 1½	5,070	5,070	
	MSTI72		2 1/16	72	(72) 0.148 x 1½	5,070	5,070	
	HTP37Z	16	3	7	(20) 0.148 x 1½	900	690	
	MSTC28		3	28¼	(36) 0.148 x 3¼	3,460	2,990	
	MSTC40		3	40¼	(52) 0.148 x 3¼	4,735	4,315	
	MSTC52		3	52¼	(62) 0.148 x 3¼	4,735	4,735	
	MSTC66	14	3	65¾	(68) 0.148 x 3¼	5,850	5,850	
	MSTC78		3	77¾	(76) 0.148 x 3¼	5,850	5,850	
	HRS416Z	12	3¼	16	(16) ¼ x 1½ SDS	2,835	2,305	—
	LSTI49	18	3¾	49	(32) 0.148 x 1½	2,970	2,560	IBC, FL, LA
	LSTI73		3¾	73	(48) 0.148 x 1½	4,205	3,840	

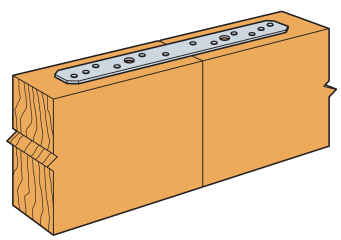
1. See pp. 276–277 for Straps and Ties General Notes.
2. **Fasteners:** Nail dimensions are listed diameter by length. SDS screws are Simpson Strong-Tie Strong-Drive SDS Heavy-Duty Connector screws. See pp. 23–24 for fastener information.



Typical LSTA Installation
(hanger not shown)
Bend strap one time only, max. 12/12 joist pitch.



Typical LSTA18 Installation



Typical MSTA15 Installation

Straps and Ties

General Footing

Project File: 25-001.ec6

LIC# : KW-06017807, Build:20.25.02.04 NEXT LEVEL, INC.

DESCRIPTION: Interior Shearwall Footing

Code References

Calculations per ACI 318-19, IBC 2021
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.0 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	4.0 ksf
Soil Density	=	120.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	250.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	3.50 ft
Allow press. increase per foot of depth	=	ksf
when footing base is below	=	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

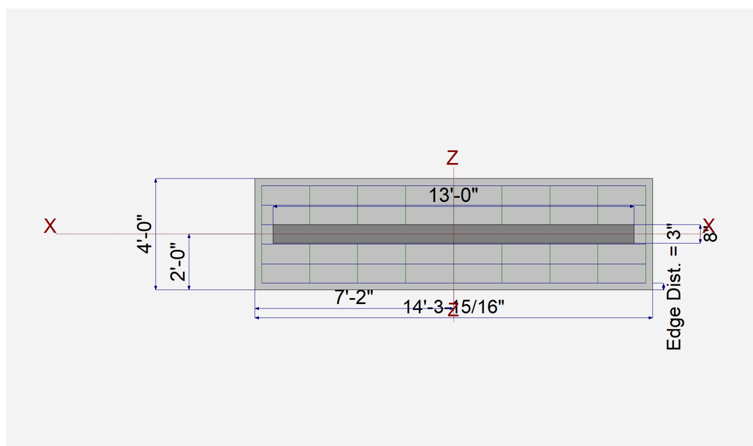
Width parallel to X-X Axis	=	14.330 ft
Length parallel to Z-Z Axis	=	4.0 ft
Footing Thickness	=	16.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	156.0 in
pz : parallel to Z-Z Axis	=	8.0 in
Height	=	18.0 in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



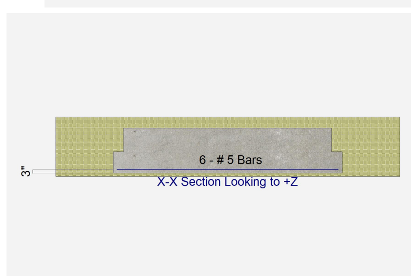
Reinforcing

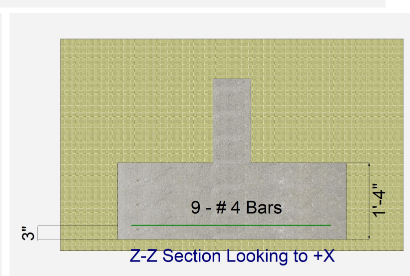
Bars parallel to X-X Axis	=	
Number of Bars		6.0
Reinforcing Bar Size	=	# 5
Bars parallel to Z-Z Axis		
Number of Bars	=	9.0
Reinforcing Bar Size	=	# 4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

	Bars along Z-Z Axis
# Bars required within zone	43.6 %
# Bars required on each side of zone	56.4 %





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=								k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=					124.40			k-ft
V-x	=					8.883			k
V-z	=								k

General Footing

DESCRIPTION: Interior Shearwall Footing

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6688	Soil Bearing	2.675 ksf	4.0 ksf	+0.60D+0.60W about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	1.137	Overturning - Z-Z	89.741 k-ft	102.023 k-ft	+0.60D+0.60W
PASS	1.510	Sliding - X-X	5.330 k	8.049 k	+0.60D+0.60W
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.09699	Z Flexure (+X)	2.546 k-ft/ft	26.249 k-ft/ft	+0.90D+W
PASS	0.004555	Z Flexure (-X)	0.1196 k-ft/ft	26.249 k-ft/ft	+1.20D+0.50W
PASS	0.01050	X Flexure (+Z)	0.07643 k-ft/ft	7.279 k-ft/ft	+1.40D
PASS	0.01050	X Flexure (-Z)	0.07643 k-ft/ft	7.279 k-ft/ft	+1.40D
PASS	n/a	1-way Shear (+X)	0.0 psi	47.296 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	0.006924	1-way Shear (+Z)	0.2117 psi	30.574 psi	+1.40D
PASS	0.006924	1-way Shear (-Z)	0.2117 psi	30.574 psi	+1.40D
PASS	n/a	2-way Punching	1.663 psi	47.296 psi	+1.20D+W

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

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Topic: CMU Trash Enclosure Wall
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Design of a Reinforced Masonry Wall with Out-of-Plane Loads

Material and Construction Data

8 in. CMU, Partial grout, running bond
Wall Weight = 44.96 psf
Type S Masonry cement / Air-entrained PCL Mortar, Coarse Grout
CMU Density = 115.0 pcf
f'_m = 2,792 psi (Specified)
$E_m = 900f'_m$ = 2,513,000 psi

Wall Design Details

Thickness = 7.625 in.
Height = 72.00 in. (Cantilevered Wall, Effective height = 2H)
x = 3.813 in.
#5 Bars, Fy = 60,000
Reinforcement Spacing = 32.00 in. On-Center
Effective Width = 32.00 in.

Wall Design Section Properties

A_o = 45.98 in^2 per foot width
S_o = 90.15 in^3 per foot width
I_o = 343.7 in^4 per foot width
r_o = 2.733 in

Wall Average Section Properties

A_{avg} = 53.66 in^2 per foot width
I_{avg} = 360.5 in^4 per foot width
r_{avg} = 2.592 in

Specified Load Components (Cantilevered Wall)

Load	P (lb)	e (in)	W1(psf)	W2 (psf)	L (lb/ft)	h1 (in)	h2 (in)
Dead	0	0	0	0	0	0	72
Live	0	0	0	0	0	0	72
Soil	0	0	0	0	0	0	72
Fluid	0	0	0	0	0	0	72
Wind	0	0	31	31	0	0	72
Seismic	0	0	0	0	0	0	72
Roof	0	0	0	0	0	0	72
Rain	0	0	0	0	0	0	72
Snow	0	0	0	0	0	0	72

f_1 = 1.000

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f_2 = 1.000
f_3 = 0.
f_4 = 0.

Controlling Load Cases

Section Forces at the Location of Controlling Flexure and Axial Load

Critical Load Combination: 57 (0.9D + 1.6H + 1.0W)
x/H = 0. from bottom of wall
Moment utilization ratio = 0.279
V_u = 186.0 lb/ft
M_{Lu} = -6,696 lb-in/ft
P_u = 242.8 lb/ft at e_n = 0. in
$Delta_u$ = -0.0101 in
$M_{Tu} = M_{Lu} + P_u e_n + P_u(Delta_u)$ = -6,698 lb-in/ft
Ultimate Moment Capacity = 23,980 lb-in/ft (1,998 lb-ft/ft) at this axial load
Ultimate Shear Capacity = 4,421 lb/ft at this location

The wall is adequate for these critical section forces.

Section Forces at the Location of Controlling Shearing Force

Critical Load Combination: 57 (0.9D + 1.6H + 1.0W)
x/H = 0. from bottom of wall
V_u = 186.0 lb/ft
M_{Lu} = -6,696 lb-in./ft
P_u = 242.8 lb/ft at e_n = 0. in
$Delta_u$ = -0.0101 in
$M_{Tu} = M_{Lu} + P_u e_n + P_u(Delta_u)$ = -6,698 lb-in/ft
Ultimate Moment Capacity = 23,980 lb-in/ft at this axial load
Ultimate Shear Capacity = 4,421 lb/ft at this location

The wall is adequate for these critical section forces.

These were found to be load cases that controlled the design.
The flexural, shear and axial forces shown are those occurring
at the critical section for the case controlled by flexure and
at the critical section for the case controlled by shear.

Design Calculations: Section with Controlling Bending Moment and Axial Load
P-Delta method used for secondary moment effects

Section Design Forces Used
P_u = 242.8 lb/ft at e = 0. in (Computed from Loads)

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V_u = 186.0 lb/ft (Computed from Loads)
M_{Lu} = -6,696 lb-in./ft (Computed from Loads)

Wall Axial Design Data
$P_{u\ max}$ = 62,290 lbs/ft

Maximum Permitted Axial Load for Combination D+0.75L+0.525Q_e = 24,570 lbs/ft
P at the bottom of the wall for this combination 269.75 lbs/ft

Wall Flexural Design Data
A_s Required = 0.0757 sq.in. in each grouted cell
A_s Provided = 0.310 sq.in.
Rho_{min} = 0.

Wall Flexural Capacity with Specified Steel
a = 0.270 in.
d = 3.813 in.
M_n = 26,640 lb-in/ft
øM_n = 0.900(26,640) = 23,980 lb-in/ft (1,998 lb-ft/ft)

f_r = 76.50 psi
$M_{cr} = (f_r + P_u/A_o)S_o$ = 7,372 lb-in/ft
I_n = 343.7 in^4/ft
I_{cr} = 16.92 in^4/ft in positive moment regions
I_{cr} = 16.92 in^4/ft in negative moment regions

Shear Design Data
$V_{n\ max}$ = 9,717 lbs/ft
V_{nm} = 5,526 lbs/ft
øV_n is the lesser of:
ø$V_{n\ max}$ = 0.800(9,717) = 7,773 lbs/ft
ø$V_{\ max}$ = 0.800(5,526 + 0.) = 4,421 lbs/ft CONTROLS (Note: V_{ns} = 0 for walls with out-of-plane loading.)

Service Load Deflection Limitations
$Delta_{max}$ = 0.007(2H) = 0.007(2)(72.00) = 1.008 in
Note: This is a code interpretation given that the effective length of a cantilever is twice that of a simple beam, which is the basis of the code provision.
Maximum service load deflection = -0.006030 in
Load combination with maximum deflection: D + 0.6W

Development and Splice Lengths for Longitudinal Reinforcement
K = 3.5000 in.
Required Development Length: l_d = 16.48 in.

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Required Lap Splice Length: = 16.48 in.

Some codes may require epoxy-coated reinforcement to have longer development and splice lengths.