BURT ENGINEERING
18530 MERIDIAN AVE N.
SHORELINE, WA 98133

Structural Calculations

For

MEDVED RESIDENCE
4752 89TH AVE SE
MERCER ISLAND, WA 98040

BE Project # 19125
July 28, 2020

2015 International Building Code
Wind: 110 MPH, Exposure B, Kzt= 1.6
Seismic: Design Category D

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PROFESSIONAL ENGINEER

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PROJECT NAME	Medved Residence
ADDRESS	4752 89th AVE SE Mercer island WA 98040
PROJECT #	
DATE	7/28/2020

BUILDING CODE

2015 International Residential Code

2015 International Building Code

WIND DESIGN

Vult =	110	MPH
Vasd =	85	MPH
Exposure =	B	
K_{ZT} =	1.6	
Importance Factor =	1.0	

SEISMIC DESIGN

S_S(g) =	1.43	S_{MS}(g) =	1.43	S_{DS}(g) =	0.953
S_1(g) =	0.549	S_{M1}(g) =	0.823	S_{D1}(g) =	0.549
Seismic Design Category =	D				
Site Class =	D				
Importance Factor =	1.0				

DESIGN LOADING

Roof Snow Load =	25	PSF	
Floor Live Load =	40	PSF	
Bedroom Live Load =	30	PSF	
Deck & Balcony Live Load =	60	PSF	
Roof Dead Load =	15	PSF	
Floor Dead Load =	15	PSF	(For framing gravity design)
Exterior Wall Dead Load =	10	PSF	
Partition Wall Seismic Weight =	10	PSF	
Floor Seismic Weight =	10	PSF	
Allowable Soil Pressure =	1500	PSF	
Lateral Earth (Restrained) Pressure =	50	PCF	
Passive Pressure =	300	PCF	
Coefficient of Friction =	0.4		

SCOPE OF WORK

Existing residence remodel design

ASCE 7-10

Wind Loads per ASCE 7-10- Chapter 28 MWFRS (Envelope Procedure)- Low-Rise Buildings

Input Cells =	
Project Number:	
Project Name:	Medved Residence
Location:	
Design By:	
Program Limitations:	1. Mean roof height *h* less than or equal to 60 ft.
	2. Mean roof height *h* does not exceed least horizontal dimension.

BUILDING AND SITE INFORMATION

	INPUT			
Building width, B =	58.58	ft (perpendicular to ridge)		
Building length, L =	53.58	ft (parallel to ridge)		
Building eave height, h_e =	17.5	ft		
Building ridge height, h_r =	19.67	ft		
Height of parapet, h_p =	17.5	ft		
Roof slope, s =	1.50 in./ft.	= 7.13 degrees		
Is roof a gable or hip =	Gable			
Risk Category =	II			
Wind velocity, V =	110	mi/hr	= 85	mi/hr (ASD)
Exposure =	B			
Topographic factor, K_{zt} =	1.6			
Wind directionality factor, K_d =	0.85			
Bldg internal pressure condition =	Enclosed			

	OUTPUT	
Mean roof height, h =	17.5	ft
2a =	10.72	ft
h/L =	0.33	
h/B =	0.3	
Internal Pressure Coeff's, GC_{pi} =	0.18	
	-0.18	
Pressure exposure coeff, K_h =	0.7	
Velocity pressure, q_h =	29.49	psf

Design Wind Pressure (LRFD)
21.6 PSF

Bldg. Info.	Height(ft)	Roof	First	
		9.25	8.83	
	E-W Width	58.58	ft	
	N-S Width	53.58	ft	
E-W Vw (kip)	Roof	5.9	Sum (kip)	5.9
	First	11.5		17.4
N-S Vw (kip)	Roof	5.4	Sum (kip)	5.4
	First	10.5		15.9

→ ½ Resisted By Existing FRAMING.
→

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)

Wind Pressures for Low-Rise Buildings

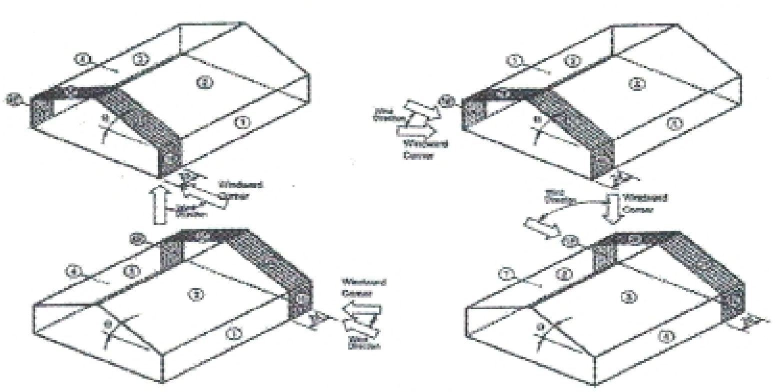
$p = q_h[(GC_{pf}) - (GC_{pi})]$ (lb/ft^2)

Load Case A: Winds Perpendicular to Ridge

Bldg Surface	GC_{pf}	Wind Pressure (lb/ft^2)	
		LRFD	ASD
1	0.42	12.4	7.4
2	-0.69	-20.4	-12.2
3	-0.39	-11.6	-7
4	-0.31	-9.2	-5.5
1E	0.64	18.9	11.3
2E	-1.07	-31.6	-19
3E	-0.55	-16.3	-9.8
4E	-0.46	-13.6	-8.2

Internal pressure = +/- 5.3 psf (LRFD)
+/- 3.2 psf (ASD)

Note: 1. Sign Convention
positive numbers denote forces toward the surface
negative numbers denote forces away from the surface
2. Minimum wind design loads shall not be less than 16 psf (LRFD) multiplied by wall area of building and 8 psf (LRFD) multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction (see Sect. C27.4.7 & Figure C27.4-1)
3. Internal pressure cancels when Zones 1 & 4 and 1E & 4E are combined, but adds or subtracts at Zones 2 & 3 and 2E & 3E that do not have directly opposing loads.



Load Case A

Load Case B: Winds Parallel to Ridge

Bldg Surface	GC_{pf}	Wind Pressure (lb/ft^2)	
		LRFD	ASD
1	-0.45	-13.3	-8
2	-0.69	-20.4	-12.2
3	-0.37	-11	-6.6
4	-0.45	-13.3	-8
5	0.4	11.8	7.1
6	-0.29	-8.6	-5.2
1E	-0.48	-14.2	-8.5
2E	-1.07	-31.6	-19
3E	-0.53	-15.7	-9.4
4E	-0.48	-14.2	-8.5
5E	0.61	18	10.8
6E	-0.43	-12.7	-7.6

Internal pressure = +/- 5.3 psf (LRFD)

+/- 3.2 psf (ASD)

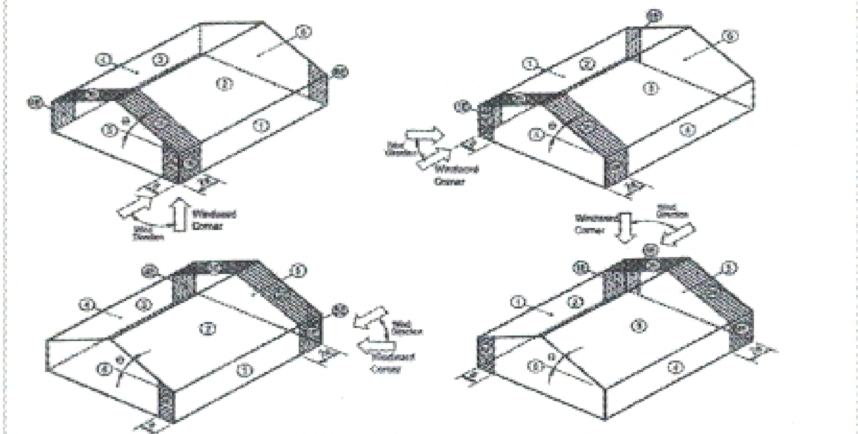
Note: 1. Sign Convention

positive numbers denote forces toward the surface

negative numbers denote forces away from the surface

2. Minimum wind design loads shall not be less than 16 psf (LRFD) multiplied by wall area of building (see Sect. C27.4.7 & Figure C27.4-1).

3. Internal pressure cancels when Zones 1 & 4 and 1E & 4E are combined, but adds or subtracts at Zones 2 & 3 and 2E & 3E that do not have directly opposing loads.



Load Case B

MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)

Wind Pressures for Parapets

Pressure exposure coeff, K_z = 0.7

Velocity pressure, q_p = 29.49 psf (LRFD)

$p_p = q_p(GC_{pn})$ (lb/ft^2)

Windward parapets, p_{p_wind} = 44.2 psf (LRFD)

Leeward parapets, p_{p_lee} = -29.5 psf (LRFD)

positive numbers signify net pressure acting toward the exterior side of the parapet
negative numbers signify net pressure acting away from the exterior side of the parapet

Wind Pressures for Roof Uplift

Roof uplift load up to 10.72 feet from exterior walls, p = -31.5 psf (LRFD)

Roof uplift load more than 10.72 feet from exterior walls, p = -20.4 psf (LRFD)

ASCE Seismic Base Shear

Lic. # : KW-06007583

DESCRIPTION: New Addition

New Addition

Risk Category

Calculations per ASCE 7-10

Risk Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ASCE 7-10, Page 2, Table 1.5-1*

Seismic Importance Factor = 1 — *ASCE 7-10, Page 5, Table 1.5-2*

USER DEFINED Ground Motion

ASCE 7-10 11.4.1

Max. Ground Motions, 5% Damping :

S_S = 1.430 g, 0.2 sec response
S_1 = 0.5490 g, 1.0 sec response

Conforms to ASCE 7 Section 12.8.1.3: Regular structure with period of 0.5 s or less, Ss limited to max of 1.5 for calculation of Cs.

Site Class, Site Coeff. and Design Category

Site Classification "D" : Shear Wave Velocity 600 to 1,200 ft/sec		=	D	*ASCE 7-10 Table 20.3-1*
Site Coefficients Fa & Fv	Fa	=	1.00	*ASCE 7-10 Table 11.4-1 & 11.4-2*
(using straight-line interpolation from table values)	Fv	=	1.50	
Maximum Considered Earthquake Acceleration	S_{MS} = Fa * Ss	=	1.430	*ASCE 7-10 Eq. 11.4-1*
	S_{M1} = Fv * S1	=	0.824	*ASCE 7-10 Eq. 11.4-2*
Design Spectral Acceleration	S_{DS} = S_{MS} * 2/3	=	0.953	*ASCE 7-10 Eq. 11.4-3*
	S_{D1} = S_{M1} * 2/3	=	0.549	*ASCE 7-10 Eq. 11.4-4*
Seismic Design Category		=	D	*ASCE 7-10 Table 11.6-1 & -2*

Resisting System

ASCE 7-10 Table 12.2-1

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

Response Modification Coefficient " R "	=	6.50	Building height Limits :	
System Overstrength Factor " Wo "	=	2.50	Category "A & B" Limit:	No Limit
Deflection Amplification Factor " Cd "	=	4.00	Category "C" Limit:	No Limit
			Category "D" Limit:	Limit = 65
			Category "E" Limit:	Limit = 65
			Category "F" Limit:	Limit = 65

NOTE! See ASCE 7-10 for all applicable footnotes.

Lateral Force Procedure

ASCE 7-10 Section 12.8.2

Equivalent Lateral Force Procedure

The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8

Determine Building Period

Use ASCE 12.8-7

Structure Type for Building Period Calculation : All Other Structural Systems

" Ct " value = 0.020 — " hn " : Height from base to highest level = 19.50 ft
" x " value = 0.75

" Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.186 sec
"TL" : Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 — 6.000 sec

Building Period " Ta " Calculated from Approximate Method selected = 0.186 sec

" Cs " Response Coefficient

ASCE 7-10 Section 12.8.1.1

S_{DS}: Short Period Design Spectral Response	=	0.953	From Eq. 12.8-2, Preliminary Cs	=	0.147
" R " : Response Modification Factor	=	6.50	From Eq. 12.8-3 & 12.8-4 , Cs need not exceed	=	0.455
" I " : Seismic Importance Factor	=	1	From Eq. 12.8-5 & 12.8-6, Cs not be less than	=	0.042

User has selected ASCE 12.8.1.3 : Regular structure,
Less than 5 Stories and with T <<= 0.5 sec, SO Ss <= 1.5 for Cs calculation

Cs : Seismic Response Coefficient = = 0.1467

ASCE Seismic Base Shear

Lic. # : KW-06007583

File: Medved Residence.ec6
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BURT ENGINEERING PLLC

DESCRIPTION: New Addition

Seismic Base Shear

ASCE 7-10 Section 12.8.1

Cs = 0.1467 from 12.8.1.1

W (see Sum Wi below) = 34.80 k

Seismic Base Shear V = Cs * W = 5.10 k

Vertical Distribution of Seismic Forces

ASCE 7-10 Section 12.8.3

" k " : hx exponent based on Ta = 1.00

Table of building Weights by Floor Level...

Level #	Wi : Weight	Hi : Height	(Wi * Hi^k)	Cvx	Fx=Cvx * V	Sum Story Shear	Sum Story Moment
2	6.90	18.08	124.75	0.3362	1.72	1.72	0.00
1	27.90	8.83	246.36	0.6638	3.39	5.10	15.87

Sum Wi = 34.80 k Sum Wi * Hi = 371.11 k-ft Total Base Shear = 5.10 k

Base Moment = 60.9 k-ft

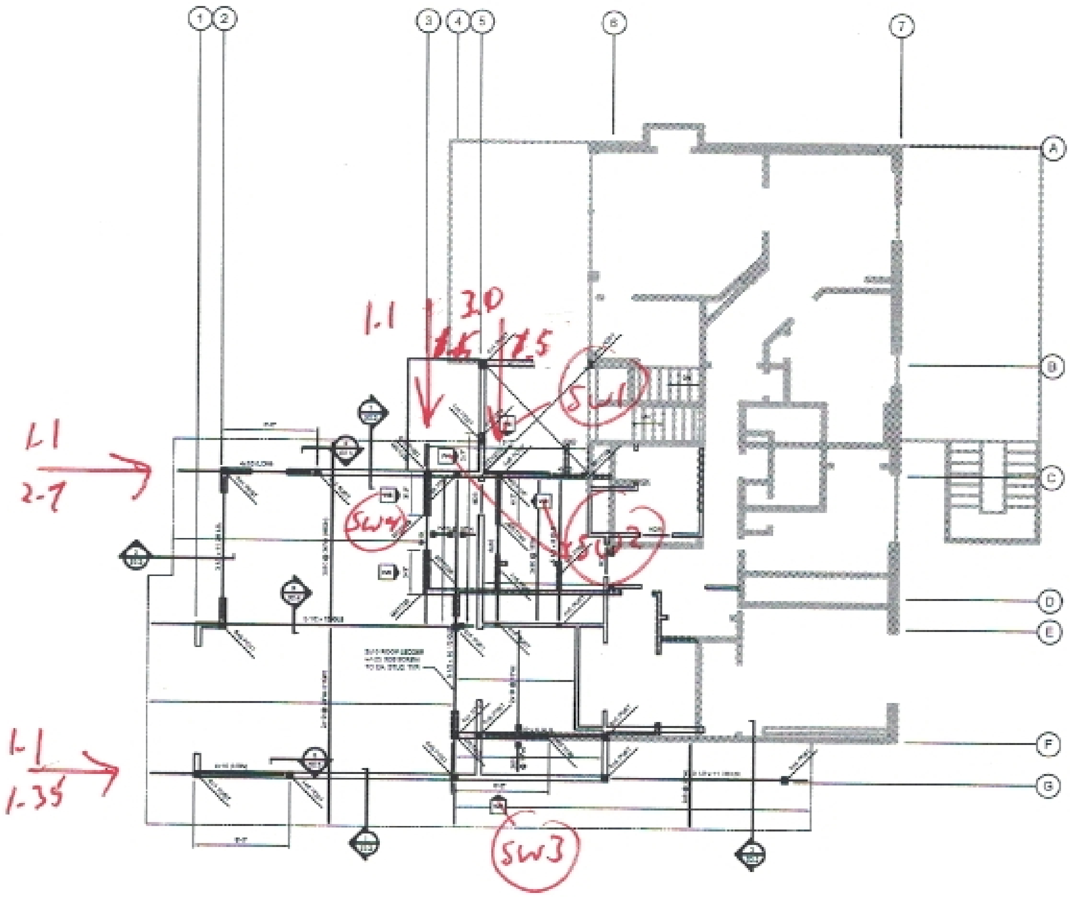
Diaphragm Forces : Seismic Design Category "B" to "F"

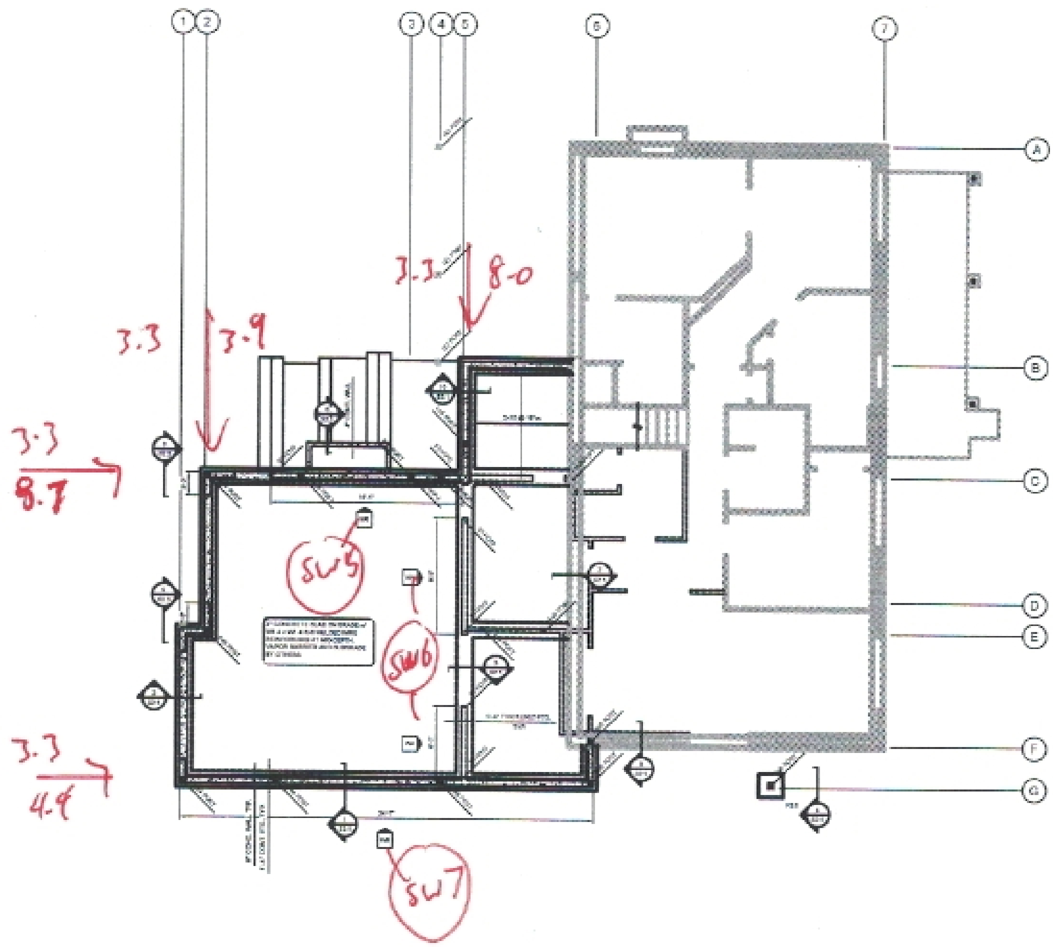
ASCE 7-10 12.10.1.1

Level #	Wi	Fi	Sum Fi	Sum Wi	Fpx : Calcd	Fpx : Min	Fpx : Max	Fpx	Dsgn. Force
2	6.90	1.72	1.72	6.90	1.72	1.32	2.63	1.72	1.72
1	27.90	3.39	5.10	34.80	4.09	5.32	10.64	5.32	5.32

Wpx . . . . Weight at level of diaphragm and other structure elements attached to it.
Fi . . . . Design Lateral Force applied at the level.
Sum Fi . . . . Sum of "Lat. Force" of current level plus all levels above
MIN Req'd Force @ Level . . . . 0.20 * S_{DS} * I * Wpx
MAX Req'd Force @ Level . . . . 0.40 * S_{DS} * I * Wpx
Fpx : Design Force @ Level . . . . Wpx * SUM(x->n) Fi / SUM(x->n) wi, x = Current level, n = Top Level

1.72 × 1.3 = 2.2 k
5.1 × 1.3 = 6.6 k.


1
2
3
4
5
6
7
A
B
C
D
E
F
G
1.1
3.0
1.5
7.5
1.1
2-7
1.1
1.35
SW1
SW2
SW3
SW4
3/4" ROOF LEDGER


1
2
3
4
5
6
7
A
B
C
D
E
F
G
3.3
8-0
3.3
3.9
3.3
8.7
3.3
4.4
SW5
SW6
SW7

Wood Shear Wall Design

	SW#	Length b(ft)	Height h(ft)	Vseismic (LRFD)(kips)	Vwind (LRFD)(kips)	Aspect Ratio h/b	h/b>2?	Total Design V (ASD)		SW Design	SW Uplift (ASD)	Wall Holdown	Foundation Holdown
Wind	SW1	13.58'	9.25'	1.10 k	2.70 k	0.68	N	1.62 k	0.12 klf	W6	.73 k	Not Reqd'	
		WALL											
Seisc.	SW2	7.83'	9.25'	3.0 k	1.50 k	1.18	N	2.10 k	0.27 klf	W4	2.26 k	MSTC40	
		WALL											
Wind	SW3	8.17'	9.25'	1.10 k	1.35 k	1.13	N	.81 k	0.10 klf	W6	.69 k	Not Reqd'	
		BEAM											
Wind	SW4	7.50'	9.25'	1.10 k	1.50 k	1.23	N	.90 k	0.12 klf	W6	.90 k	Not Reqd'	
		BEAM											
Wind	SW5	16.08'	8.83'	3.30 k	8.70 k	0.55	N	5.22 k	0.32 klf	W6	2.44 k		STHD10
Wind	SW6	15.92'	8.83'	3.30 k	8.0 k	0.55	N	4.80 k	0.30 klf	W6	2.24 k		STHD10
Wind	SW7	34.58'	8.83'	3.30 k	4.40 k	0.26	N	2.64 k	0.08 klf	W6	-.24 k		

SW #	Vs,all (ASD) (kip/ft)	Vw,all (ASD) (kip/ft)
W6	0.26	0.37
W4	0.38	0.53
W3	0.49	0.69
2W6	0.52	0.73
2W4	0.76	1.07
2W3	0.98	1.37
2W2	1.28	1.79

Wall HD	Tall (ASD)(kips)
MSTC28	1.54
MSTC40	3.08
MSTC52	4.62
MSTC66	5.86
MST72	6.73
CMST12x84"	9.215
2xMSTC66	11.72
2xMST72	13.46
2xCMST12x84"	18.43
HD19(SPC.)	19.07

FTG HD	Tall (ASD) (kips)
STHD10	3.40
STHD14	3.82
HDU4	4.57
HDU5	5.65
HDU8	6.97
HDU11	9.34
HDU14	10.77
HD12	12.67
HDU14(SPC.)	14.44
HD12(SPC.)	15.51
HD19	16.77
HD19(SPC.)	19.07

*Holdown not required for uplift less than 1 Kips(ASD)

→ NOTE, NO CHANGES TO SW SHEATHING & SW HOLDOWNS DUE TO WIND Kzt= 1.6.

APA - HAS IMPROVED PORTAL FRAME STRENGTH.
(APA TT-100F).

Table 1. Recommended Allowable Design Values for APA Portal Frame Used on a Rigid-Base

Minimum Width (in.)	Maximum Height (ft)	Allowable Design (ASD) Values per Frame Segment		Load Factor
		Shear[e,f] (lbf)	Deflection (in.)	
16	8	850	0.33	3.09
	10	625	0.44	2.97
24	8	1,675	0.38	2.88
	10	1,125	0.51	3.42

Foundation for Wind or Seismic Loading[a,b,c,d]

(a) Design values are based on the use of Douglas-fir or Southern pine framing. For other species of framing, multiply the above shear design value by the specific gravity adjustment factor = (1 – (0.5 – SG)), where SG = specific gravity of the actual framing. This adjustment shall not be greater than 1.0.

(b) For construction as shown in Figure 1.

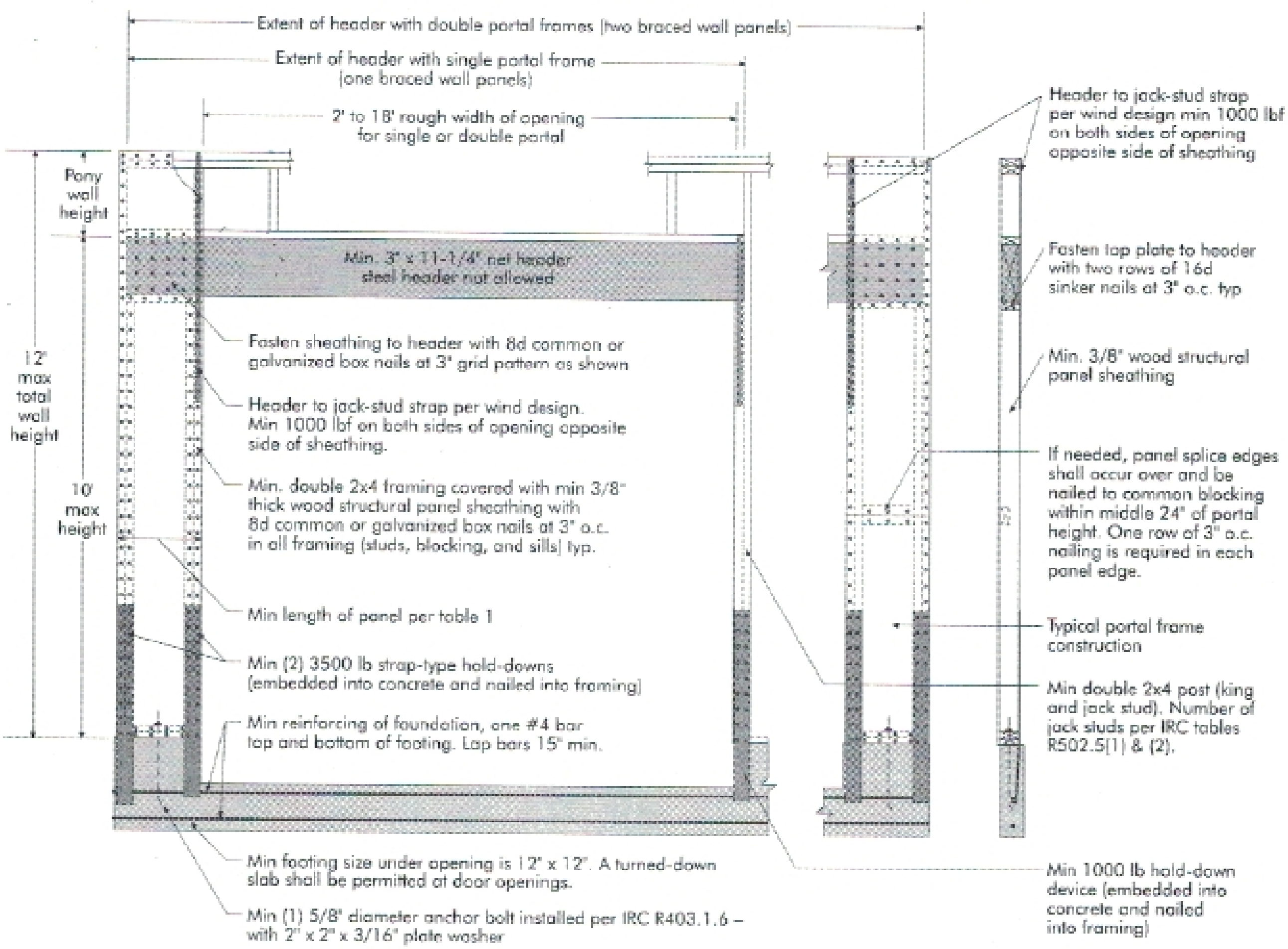
(c) Values are for a single portal-frame segment (one vertical leg and a portion of the header). For multiple portal-frame segments, the allowable shear design values are permitted to be multiplied by the number of frame segments (e.g., two = 2x, three = 3x, etc.).

(d) Interpolation of design values for heights between 8 and 10 feet, and for portal widths between 16 and 24 inches, is permitted.

(e) The allowable shear design value is permitted to be multiplied by a factor of 1.4 for wind design.

(f) If story drift is not a design consideration, the tabulated design shear values are permitted to be multiplied by a factor of 1.15. This factor is permitted to be used cumulatively with the wind-design adjustment factor in Footnote (e) above.

Figure 1. Construction Details for APA Portal-Frame Design with Hold Downs



PORTAL FRAME DESIGN: @ GRID 2.

REQUIRED V_s = 3.3 k (LRFD) = 2.31 k (ASD)

V_w = 3.9 k (LRFD) = 2.34k (ASD).

ALLOWABLE SHEAR PER FRAME : $V_{s,all}$ = 1.675 k (ASD)

$V_{w,all}$ = 1.675k · 1.4 (ASD)

= 2.34 k.

2× $V_{s,all}$ = 3.35 k > 2.31 k. ok

2× $V_{w,all}$ = 4.68 k > 2.34k. ok

DESCRIPTION: 4'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	4.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Design Summary

Wall Stability Ratios

Overturning	=	2.44 OK
Sliding	=	1.35 OK
Slab Resists All Sliding !		
Total Bearing Load	=	1,394 lbs
...resultant ecc.	=	5.08 in
Soil Pressure @ Toe	=	975 psf OK
Soil Pressure @ Heel	=	39 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,170 psf
ACI Factored @ Heel	=	46 psf
Footing Shear @ Toe	=	2.4 psi OK
Footing Shear @ Heel	=	6.7 psi OK
Allowable	=	75.0 psi

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	528.8 lbs
less 100% Passive Force	= -	156.3 lbs
less 100% Friction Force	= -	550.0 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	79.2 lbs NG

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	4.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 4	# 4
Rebar Spacing	in =	18.00	18.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.000	0.210
Total Force @ Section	lbs =	0.0	570.0
Moment....Actual	ft-l =	0.0	767.0
Moment.....Allowable	ft-l =	3,655.6	3,655.6
Shear.....Actual	psi =	0.0	7.6
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.25	6.25
Lap splice if above	in =	18.72	18.72
Lap splice if below	in =	18.72	3.60
Hook embed into footing	in =	18.72	3.60
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

DESCRIPTION: 4'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Footing Dimensions & Strengths

Toe Width	=	1.00 ft
Heel Width	=	1.75
Total Footing Width	=	2.75
Footing Thickness	=	12.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 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	=	1,170	46 psf
Mu' : Upward	=	517	0 ft-lb
Mu' : Downward	=	123	415 ft-lb
Mu: Design	=	394	415 ft-lb
Actual 1-Way Shear	=	2.41	6.73 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 4 @ 18.00 in	
Heel Reinforcing	=	# 4 @ 18.00 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: Slab Resists Sliding - No Force on Key

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	562.5	1.67	937.5
Surcharge over Heel	=			
Toe Active Pressure	=	-33.8	0.50	-16.9
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	528.8	**O.T.M.** =	920.6

Resisting/Overturning Ratio = **2.44**
Vertical Loads used for Soil Pressure = 1,394.2 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	476.7	2.21	1,052.6
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	55.0	0.50	27.5
Surcharge Over Toe	=			
Stem Weight(s)	=	450.0	1.33	600.0
Earth @ Stem Transitions	=			
Footing Weight	=	412.5	1.38	567.2
Key Weight	=		2.50	
Vert. Component	=			
Total =		1,394.2 lbs	**R.M. =**	2,247.3

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

DESCRIPTION: 6'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	6.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Design Summary

Wall Stability Ratios

Overturning	=	2.42	OK
Sliding	=	1.10	OK
Slab Resists All Sliding !			
Total Bearing Load	=	2,543 lbs	
...resultant ecc.	=	6.87 in	
Soil Pressure @ Toe	=	1,182 psf	OK
Soil Pressure @ Heel	=	90 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,418 psf	
ACI Factored @ Heel	=	107 psf	
Footing Shear @ Toe	=	7.9 psi	OK
Footing Shear @ Heel	=	15.6 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	1,068.8 lbs	
less 100% Passive Force	= -	156.3 lbs	
less 100% Friction Force	= -	1,010.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	429.9 lbs	NG

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem Stem OK	2nd Stem OK
Design Height Above Ftg	ft =	6.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 4	# 4
Rebar Spacing	in =	18.00	18.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.000	0.709
Total Force @ Section	lbs =	0.0	1,290.0
Moment....Actual	ft-# =	0.0	2,591.0
Moment.....Allowable	ft-# =	3,655.6	3,655.6
Shear.....Actual	psi =	0.0	17.2
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.25	6.25
Lap splice if above	in =	18.72	18.72
Lap splice if below	in =	18.72	3.60
Hook embed into footing	in =	18.72	3.60
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

DESCRIPTION: 6'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Footing Dimensions & Strengths

Toe Width	=	1.50 ft
Heel Width	=	2.50
Total Footing Width	=	4.00
Footing Thickness	=	12.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 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	=	1,418	107 psf
Mu' : Upward	=	1,411	0 ft-lb
Mu' : Downward	=	277	1,634 ft-lb
Mu: Design	=	1,134	1,634 ft-lb
Actual 1-Way Shear	=	7.92	15.63 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 4 @ 18.00 in	
Heel Reinforcing	=	# 4 @ 18.00 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: Slab Resists Sliding - No Force on Key

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	1,102.5	2.33	2,572.5
Surcharge over Heel	=			
Toe Active Pressure	=	-33.8	0.50	-16.9
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	1,068.8	**O.T.M.** =	2,555.6

Resisting/Overturning Ratio = **2.42**
Vertical Loads used for Soil Pressure = 2,542.5 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	1,210.0	3.08	3,730.8
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	82.5	0.75	61.9
Surcharge Over Toe	=			
Stem Weight(s)	=	650.0	1.83	1,191.7
Earth @ Stem Transitions	=			
Footing Weight	=	600.0	2.00	1,200.0
Key Weight	=		2.50	
Vert. Component	=			
Total =		2,542.5 lbs	**R.M. =**	6,184.4

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

DESCRIPTION: 8'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	8.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	135.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 plf
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	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
Sliding	=	0.78	OK
Slab Resists All Sliding !			
Total Bearing Load	=	4,078 lbs	
...resultant ecc.	=	14.23 in	
Soil Pressure @ Toe	=	2,069 psf	NG
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Exceeds Allowable!			
ACI Factored @ Toe	=	2,483 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	22.9 psi	OK
Footing Shear @ Heel	=	29.7 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	2,285.8 lbs	
less 100% Passive Force	= -	156.3 lbs	
less 100% Friction Force	= -	1,630.0 lbs	
Added Force Req'd	=	498.2 lbs	NG
....for 1.5 : 1 Stability	=	1,641.1 lbs	NG

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	8.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 4	# 5
Rebar Spacing	in =	18.00	8.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.000	0.760
Total Force @ Section	lbs =	0.0	3,004.9
Moment....Actual	ft-l =	0.0	8,970.6
Moment.....Allowable	ft-l =	3,655.6	11,799.2
Shear.....Actual	psi =	0.0	40.5
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.25	6.19
Lap splice if above	in =	18.72	23.40
Lap splice if below	in =	18.72	4.67
Hook embed into footing	in =	18.72	4.67
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

DESCRIPTION: 8'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Footing Dimensions & Strengths

Toe Width	=	2.00 ft
Heel Width	=	3.00
Total Footing Width	=	5.00
Footing Thickness	=	12.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 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	=	2,483	0 psf
Mu' : Upward	=	4,126	0 ft-lb
Mu' : Downward	=	492	3,953 ft-lb
Mu: Design	=	3,634	3,953 ft-lb
Actual 1-Way Shear	=	22.90	29.72 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 4 @ 13.25 in	
Heel Reinforcing	=	# 4 @ 11.75 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 13.25 in, #5@ 20.50 in, #6@ 29.00 in, #7@ 39.25 in, #8@ 48.25 in, #9@ 4
Heel: #4@ 11.75 in, #5@ 18.25 in, #6@ 25.75 in, #7@ 35.25 in, #8@ 46.25 in, #9@ 4
Key: Slab Resists Sliding - No Force on Key

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	1,822.5	3.00	5,467.5
Surcharge over Heel	=	497.0	4.50	2,236.7
Toe Active Pressure	=	-33.8	0.50	-16.9
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	2,285.8	**O.T.M.** =	7,687.3

Resisting/Overturning Ratio = **1.70**
Vertical Loads used for Soil Pressure = 4,078.3 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	2,053.3	3.83	7,871.1
Sloped Soil Over Heel	=			
Surcharge Over Heel	=	315.0	3.83	1,207.5
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	110.0	1.00	110.0
Surcharge Over Toe	=			
Stem Weight(s)	=	850.0	2.33	1,983.3
Earth @ Stem Transitions	=			
Footing Weight	=	750.0	2.50	1,875.0
Key Weight	=		2.50	
Vert. Component	=			
Total =		4,078.3 lbs	**R.M. =**	13,046.9

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

Cantilevered Retaining Wall

Lic. # : KW-06007583

DESCRIPTION: 10'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Calculations per ACI 318-14, ACI 530-11, IBC 2015, CBC 2016, ASCE 7-10

Criteria

Retained Height	=	10.00 ft
Wall height above soil	=	0.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft

Vertical component of active
Lateral soil pressure options:
NOT USED for Soil Pressure.
NOT USED for Sliding Resistance.
NOT USED for Overturning Resistance.

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	45.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.400
Soil height to ignore for passive pressure	=	12.00 in

Design Summary

Wall Stability Ratios

Overturning	=	2.50	OK
Sliding	=	0.89	OK
Slab Resists All Sliding !			
Total Bearing Load	=	5,584 lbs	
...resultant ecc.	=	8.41 in	
Soil Pressure @ Toe	=	1,277 psf	OK
Soil Pressure @ Heel	=	318 psf	OK
Allowable	=	1,500 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,532 psf	
ACI Factored @ Heel	=	382 psf	
Footing Shear @ Toe	=	21.2 psi	OK
Footing Shear @ Heel	=	31.4 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	2,764.0 lbs	
less 100% Passive Force	= -	222.2 lbs	
less 100% Friction Force	= -	2,233.8 lbs	
Added Force Req'd	=	308.1 lbs	NG
....for 1.5 : 1 Stability	=	1,690.0 lbs	NG

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Top Stem	2nd
		Stem OK	Stem OK
Design Height Above Ftg	ft =	10.50	0.00
Wall Material Above "Ht"	=	Concrete	Concrete
Thickness	in =	8.00	8.00
Rebar Size	=	# 4	# 5
Rebar Spacing	in =	18.00	7.00
Rebar Placed at	=	Edge	Edge
Design Data			
fb/FB + fa/Fa	=	0.000	0.902
Total Force @ Section	lbs =	0.0	3,594.0
Moment....Actual	ft-l =	0.0	11,999.0
Moment.....Allowable	ft-l =	3,655.6	13,297.3
Shear.....Actual	psi =	0.0	48.4
Shear.....Allowable	psi =	75.0	75.0
Wall Weight	psf =	100.0	100.0
Rebar Depth 'd'	in =	6.25	6.19
Lap splice if above	in =	18.72	23.40
Lap splice if below	in =	18.72	6.30
Hook embed into footing	in =	18.72	6.30
Concrete Data			
f'c	psi =	2,500.0	2,500.0
Fy	psi =	60,000.0	60,000.0

DESCRIPTION: 10'-0" Cantilevered Retaining Wall (No Surcharge) (S.O.G. Restrain)

Footing Dimensions & Strengths

Toe Width	=	3.50 ft
Heel Width	=	3.50
Total Footing Width	=	7.00
Footing Thickness	=	14.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 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	=	1,532	382 psf
Mu' : Upward	=	8,212	0 ft-lb
Mu' : Downward	=	1,691	6,141 ft-lb
Mu: Design	=	6,521	6,141 ft-lb
Actual 1-Way Shear	=	21.22	31.41 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	# 4 @ 10.75 in	
Heel Reinforcing	=	# 4 @ 9.75 in	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: #4@ 10.75 in, #5@ 16.50 in, #6@ 23.50 in, #7@ 31.75 in, #8@ 42.00 in, #9@ 4
Heel: #4@ 9.75 in, #5@ 15.00 in, #6@ 21.50 in, #7@ 29.00 in, #8@ 38.25 in, #9@ 48
Key: Slab Resists Sliding - No Force on Key

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	2,805.6	3.72	10,443.2
Surcharge over Heel	=			
Toe Active Pressure	=	-41.7	0.56	-23.1
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	2,764.0	**O.T.M.** =	10,420.0

Resisting/Overturning Ratio = **2.50**

Vertical Loads used for Soil Pressure = 5,584.2 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-lb
Soil Over Heel	=	3,116.7	5.58	17,401.4
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	192.5	1.75	336.9
Surcharge Over Toe	=			
Stem Weight(s)	=	1,050.0	3.83	4,025.0
Earth @ Stem Transitions	=			
Footing Weight	=	1,225.0	3.50	4,287.5
Key Weight	=		2.50	
Vert. Component	=			
Total =		5,584.2 lbs	**R.M. =**	26,050.8

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

SPANDREL BEAM DESIGN @ ENTRANCE.

WIND C & C = 18.2 PSF (ASD). MAX.

→ TRIB. HT = (9'3 + 8'6) / 2 = 9' → 18.2 PSF · 9' = 164 PLF.
= 273 PLF (LRFD)

BEAM SPAN = 8'6

USE 4x6.

ASCE 7-10

Wind Loads per ASCE 7-10- Chapter 30 - Components & Cladding - h ≤ 60'-0"

Input Cells =	
Project Number:	
Project Name:	Medved Residence
Location:	
Design By:	
Program Limitations:	1. Building must be a low-rise building or 2. Building mean roof height does not exceed 60 feet. 3. Building is enclosed or partially enclosed.

BUILDING AND SITE INFORMATION

	INPUT			
Building width, B =	58.58	ft		
Building length, L =	53.58	ft		
Building eave height, h_e =	17.5	ft		
Building ridge height, h_r =	19.67	ft		
Height of parapet, h_p =	17.5	ft		
Roof slope, s =	1.50 in./ft.	= 7.13 degrees		
Is roof a gable or hip =	Gable			
Risk Category =	II			
Wind velocity, V =	110	mi/hr	= 85	mi/hr (ASD)
Exposure =	B			
Topographic factor, K_{zt} =	1.6			
Wind directionality factor, K_d =	0.85			
Bldg internal pressure condition =	Enclosed			
Wall Effective Wind Area, EWA =	200	ft^2		
Roof Effective Wind Area, EWA =	200	ft^2		

	OUTPUT	
Mean roof height, h =	17.5	ft
a =	5.36	ft
Internal Pressure Coeff's, GC_{pi} =	0.18	
	-0.18	
Pressure exposure coeff, K_h =	0.7	
Velocity pressure, q_h =	29.49	psf

COMPONENTS AND CLADDING (C&C)

$p = q_h[(GC_p) - (GC_{pi})]$ (lb/ft^2)

Wind Pressures for Walls with h ≤ 60'

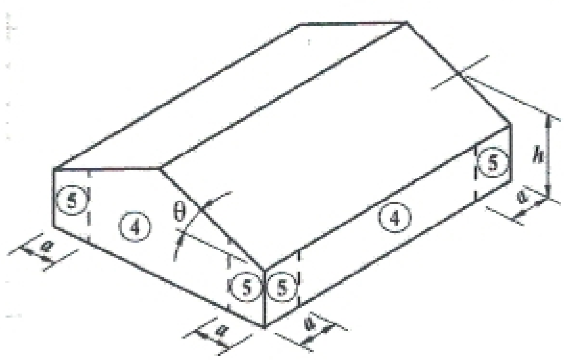
**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

Wind Pressures Acting Toward Surface

Bldg Surface 4 & 5 =	25.7	psf (LRFD) = 15.4 psf (ASD)

Wind Pressures Acting Away From Surface

Bldg Surface 4 =	-28.4	psf (LRFD) = -17 psf (ASD)
Bldg Surface 5 =	-30.3	psf (LRFD) = -18.2 psf (ASD)



Wind Pressures for Roofs (gable roofs, & hip roofs) with h ≤ 60'

**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

Wind Pressures Acting Toward Surface

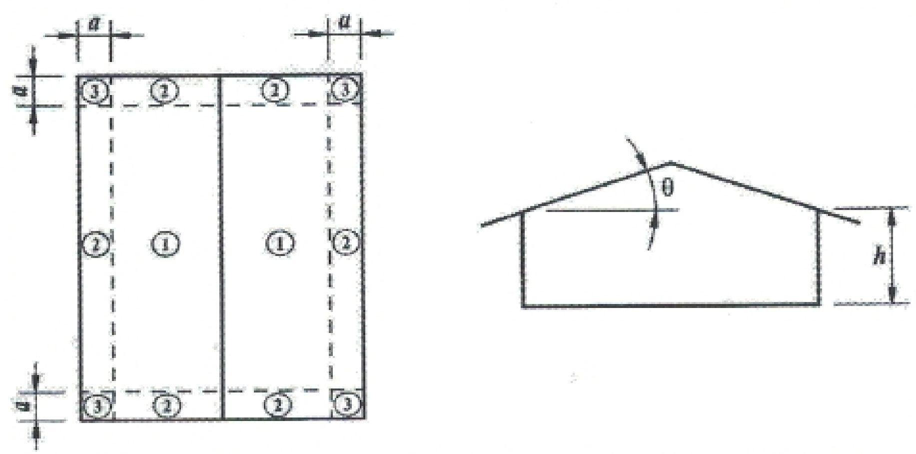
Roof Surface 1, 2 & 3 =	14.2	psf (LRFD) = 8.5 psf (ASD)

Wind Pressures Acting Away From Surface

Roof Surface 1 =	-28.9	psf (LRFD) = -17.3 psf (ASD)
Roof Surface 2 =	-40.7	psf (LRFD) = -24.4 psf (ASD)
Roof Surface 3 =	-64.3	psf (LRFD) = -38.6 psf (ASD)

Wind Pressures Acting Away From Surface (Overhang)

Roof Surface 1 =	N.A.	psf (LRFD)
Roof Surface 2 =	-64.9	psf (LRFD) = -38.9 psf (ASD)
Roof Surface 3 =	-73.7	psf (LRFD) = -44.2 psf (ASD)



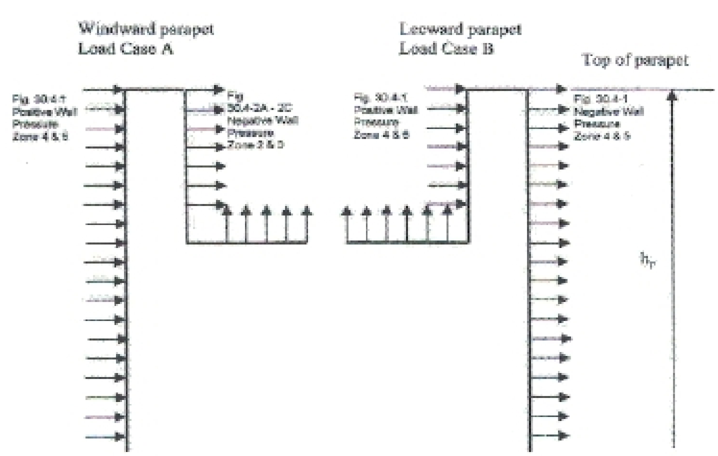
5. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with θ ≤ 7°, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_p in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 30.4-1.

Wind Pressures for Parapets (gable roofs, & hip roofs) with h ≤ 60'

**Note: Design wind pressures shall not be less than a net pressure of 16psf (ult) acting in either direction normal to the surface.*

$$p_p = q_p[(GC_p) - (GC_{pi})] \ (\text{lb/ft}^2)$$

Velocity pressure, q_p =	29.49	psf
Parapet Load Case A		
Wind load @ corner, p_p =	79.4	psf (LRFD) = 47.6 psf (ASD)
Wind load not @ corner, p_p =	55.8	psf (LRFD) = 33.5 psf (ASD)
Parapet Load Case B		
Wind load @ corner, p_p =	45.4	psf (LRFD) = 27.2 psf (ASD)
Wind load not @ corner, p_p =	43.5	psf (LRFD) = 26.1 psf (ASD)



Wood Beam

File: Medved Residence.ec6
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BURT ENGINEERING PLLC

DESCRIPTION: Spandrel Beam at Entrance

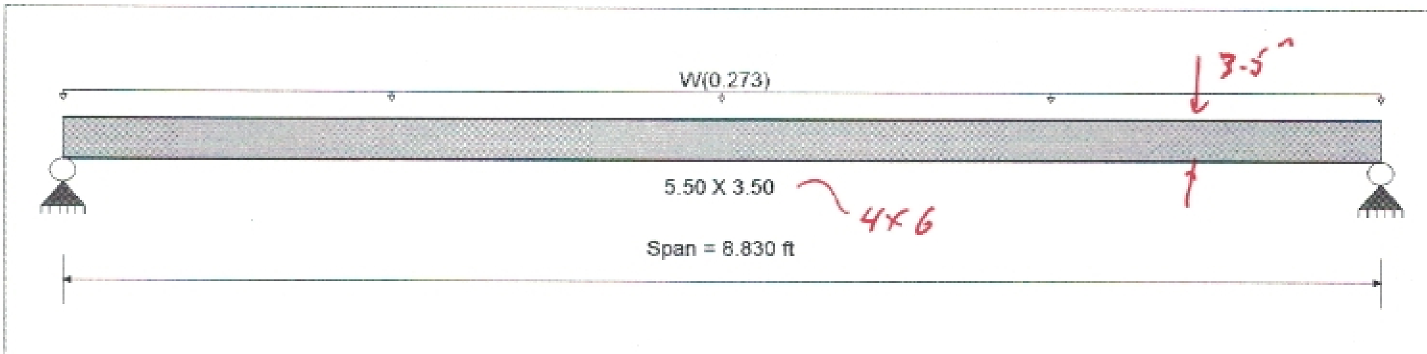
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Uniform Load : W = 0.2730 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.911: 1**	Maximum Shear Stress Ratio	=	**0.183 : 1**
Section used for this span		**5.50 X 3.50**	Section used for this span		**5.50 X 3.50**
	=	1,706.00psi		=	52.65 psi
	=	1,872.00psi		=	288.00 psi
Load Combination		+D+0.60W+H	Load Combination		+D+0.60W+H
Location of maximum on span	=	4.415ft	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	1.195 in	Ratio =	88 >=60
Max Upward Transient Deflection	0.000 in	Ratio =	0 <60
Max Downward Total Deflection	0.717 in	Ratio =	147 >=60
Max Upward Total Deflection	0.000 in	Ratio =	0 <60

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 8.798 ft	1			0.90	1.300	1.00	1.00	1.00	1.00	1.00			1053.00	0.00	0.00	162.00
Length = 0.03223 ft	1			0.90	1.300	1.00	1.00	1.00	1.00	1.00			1053.00	0.00	0.00	162.00
+D+L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.00	1.300	1.00	1.00	1.00	1.00	1.00			1170.00	0.00	0.00	180.00
Length = 0.03223 ft	1			1.00	1.300	1.00	1.00	1.00	1.00	1.00			1170.00	0.00	0.00	180.00
+D+Lr+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.25	1.300	1.00	1.00	1.00	1.00	1.00			1462.50	0.00	0.00	225.00
Length = 0.03223 ft	1			1.25	1.300	1.00	1.00	1.00	1.00	1.00			1462.50	0.00	0.00	225.00
+D+S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.15	1.300	1.00	1.00	1.00	1.00	1.00			1345.50	0.00	0.00	207.00
Length = 0.03223 ft	1			1.15	1.300	1.00	1.00	1.00	1.00	1.00			1345.50	0.00	0.00	207.00
+D+0.750Lr+0.750L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.25	1.300	1.00	1.00	1.00	1.00	1.00			1462.50	0.00	0.00	225.00

Wood Beam

DESCRIPTION: Spandrel Beam at Entrance

Load Combination Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 0.03223 ft	1			1.25	1.300	1.00	1.00	1.00	1.00	1.00			1462.50	0.00	0.00	225.00
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.15	1.300	1.00	1.00	1.00	1.00	1.00			1345.50	0.00	0.00	207.00
Length = 0.03223 ft	1			1.15	1.300	1.00	1.00	1.00	1.00	1.00			1345.50	0.00	0.00	207.00
+D+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1	0.911	0.183	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.60	1,706.00	1872.00	0.68	52.65	288.00
Length = 0.03223 ft	1	0.013	0.183	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.02	24.81	1872.00	0.68	52.65	288.00
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00
Length = 0.03223 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00
+D+0.750Lr+0.750L+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1	0.683	0.137	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.20	1,279.50	1872.00	0.51	39.49	288.00
Length = 0.03223 ft	1	0.010	0.137	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.02	18.61	1872.00	0.51	39.49	288.00
+D+0.750L+0.750S+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1	0.683	0.137	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.20	1,279.50	1872.00	0.51	39.49	288.00
Length = 0.03223 ft	1	0.010	0.137	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.02	18.61	1872.00	0.51	39.49	288.00
+D+0.750L+0.750S+0.5250E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00
Length = 0.03223 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1	0.911	0.183	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.60	1,706.00	1872.00	0.68	52.65	288.00
Length = 0.03223 ft	1	0.013	0.183	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.02	24.81	1872.00	0.68	52.65	288.00
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.798 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00
Length = 0.03223 ft	1			1.60	1.300	1.00	1.00	1.00	1.00	1.00			1872.00	0.00	0.00	288.00

Overall Maximum Deflections

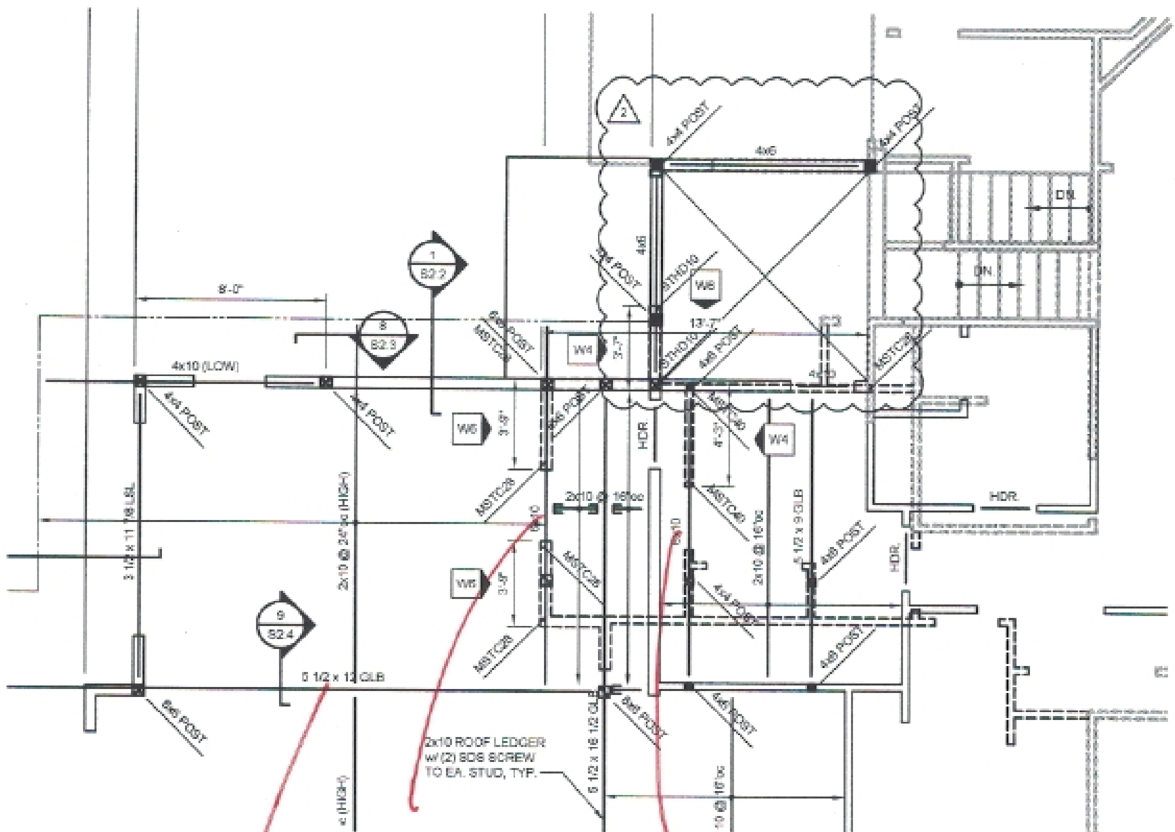
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	1.1946	4.447		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.205	1.205
Overall MINimum	1.205	1.205
+D+0.60W+H	0.723	0.723
+D+0.750Lr+0.750L+0.450W+H	0.542	0.542
+D+0.750L+0.750S+0.450W+H	0.542	0.542
+0.60D+0.60W+0.60H	0.723	0.723
W Only	1.205	1.205
H Only		

SW DISCONTINUED ON BEAMS CHECK.



Check Beam 1

Check Beam 2, SW @ 3'11 $V_s = 4.35k$
$(w/\ \Omega_0 = 2.5)$

Check Beam 3

SW @ 3'3
10' $V_s = 1.42k$ $(w/\ \Omega_0 = 2.5)$

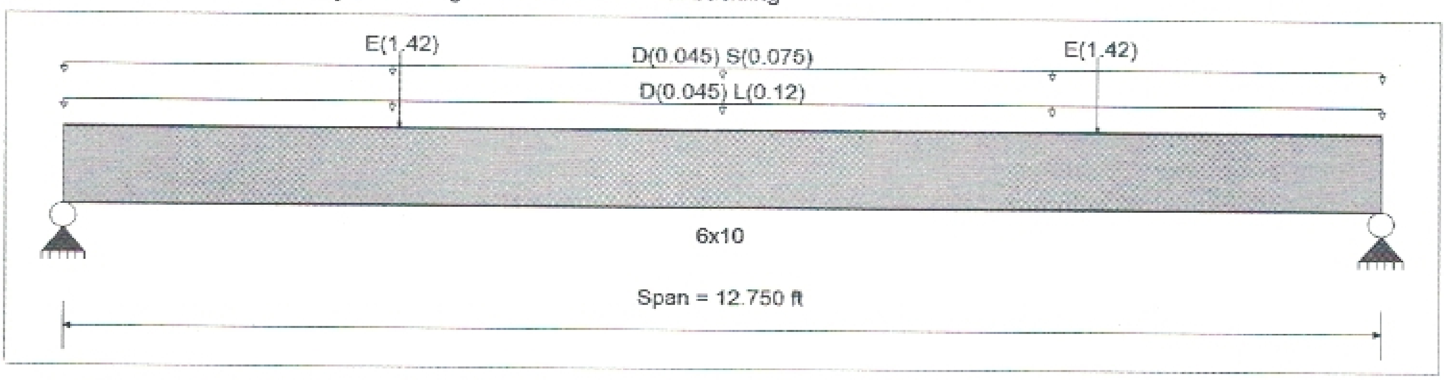
DESCRIPTION: Beam 1 with SW loading

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: DouglasFir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 3.0 ft, (floor load)
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 3.0 ft
Point Load : E = 1.420 k @ 3.250 ft
Point Load : E = 1.420 k @ 10.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.732** 1	Maximum Shear Stress Ratio	=	**0.215 : 1**
Section used for this span		**6x10**	Section used for this span		**6x10**
	=	1,054.37psi		=	61.93 psi
	=	1,440.00psi		=	288.00 psi
Load Combination		+D+0.750L+0.750S+0.5250E+H	Load Combination		+D+0.750L+0.750S+0.5250E+H
Location of maximum on span	=	6.235ft	Location of maximum on span	=	11.959 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.221 in	Ratio =	691 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.352 in	Ratio =	435 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.369	0.100	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.06	298.65	810.00	0.57	16.24	162.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.725	0.197	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.50	652.35	900.00	1.24	35.48	180.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.265	0.072	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.06	298.65	1125.00	0.57	16.24	225.00
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.502	0.137	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.58	519.72	1035.00	0.98	28.27	207.00
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.501	0.136	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.89	563.93	1125.00	1.07	30.67	225.00

DESCRIPTION: Beam 1 with SW loading

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.705	0.192	1.15	1.000	1.00	1.00	1.00	1.00	1.00	5.03	729.72	1035.00	1.38	39.69	207.00
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.207	0.056	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.06	298.65	1440.00	0.57	16.24	288.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.509	0.159	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.05	732.28	1440.00	1.60	45.90	288.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.392	0.106	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.89	563.93	1440.00	1.07	30.67	288.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.507	0.138	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.03	729.72	1440.00	1.38	39.69	288.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.732	0.215	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.27	1,054.37	1440.00	2.16	61.93	288.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.124	0.034	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.24	179.19	1440.00	0.34	9.75	288.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.426	0.137	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.23	613.55	1440.00	1.37	39.40	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E+H	1	0.3517	6.375		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.295	2.353
Overall MINimum	1.364	1.476
+D+H	0.646	0.646
+D+L+H	1.411	1.411
+D+Lr+H	0.646	0.646
+D+S+H	1.124	1.124
+D+0.750Lr+0.750L+H	1.220	1.220
+D+0.750L+0.750S+H	1.578	1.578
+D+0.60W+H	0.646	0.646
+D+0.70E+H	1.601	1.679
+D+0.750Lr+0.750L+0.450W+H	1.220	1.220
+D+0.750L+0.750S+0.450W+H	1.578	1.578
+D+0.750L+0.750S+0.5250E+H	2.295	2.353
+0.60D+0.60W+0.60H	0.388	0.388
+0.60D+0.70E+0.60H	1.343	1.421
D Only	0.646	0.646
L Only	0.765	0.765
S Only	0.478	0.478
E Only	1.364	1.476
H Only		

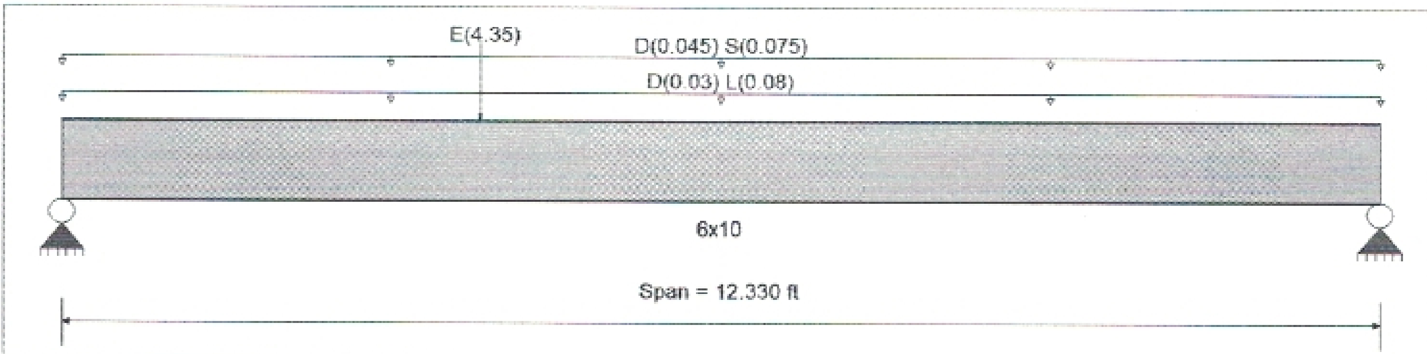
DESCRIPTION: Beam 2 with SW loading

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: DouglasFir-Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, L = 0.040 ksf, Tributary Width = 2.0 ft, (floor load)
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 3.0 ft
Point Load : E = 4.350 k @ 3.920 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.962 1	Maximum Shear Stress Ratio	=	0.264 : 1
Section used for this span		6x10	Section used for this span		6x10
	=	1,385.70 psi		=	76.12 psi
	=	1,440.00 psi		=	288.00 psi
Load Combination		+D+0.70E+H	Load Combination		+D+0.750L+0.750S+0.5250E+H
Location of maximum on span	=	3.915 ft	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	0.392 in	Ratio =	377 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.373 in	Ratio =	396 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.294	0.083	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.64	237.95	810.00	0.47	13.38	162.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.509	0.143	1.00	1.000	1.00	1.00	1.00	1.00	1.00	3.16	458.47	900.00	0.90	25.78	180.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.212	0.059	1.25	1.000	1.00	1.00	1.00	1.00	1.00	1.64	237.95	1125.00	0.47	13.38	225.00
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.430	0.121	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.07	444.69	1035.00	0.87	25.01	207.00
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.359	0.101	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.78	403.34	1125.00	0.79	22.68	225.00
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

DESCRIPTION: Beam 2 with SW loading

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 12.330 ft	1	0.540	0.152	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.85	558.40	1035.00	1.09	31.40	207.00
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.165	0.046	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.64	237.95	1440.00	0.47	13.38	288.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.962	0.253	1.60	1.000	1.00	1.00	1.00	1.00	1.00	9.55	1,385.70	1440.00	2.54	73.01	288.00
+D-0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.676	0.188	1.60	1.000	1.00	1.00	1.00	1.00	1.00	6.71	973.18	1440.00	1.88	54.05	288.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.280	0.079	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.78	403.34	1440.00	0.79	22.68	288.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.388	0.109	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.85	558.40	1440.00	1.09	31.40	288.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.950	0.264	1.60	1.000	1.00	1.00	1.00	1.00	1.00	9.44	1,368.60	1440.00	2.65	76.12	288.00
+D+0.750L+0.750S-0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.278	0.117	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.76	400.56	1440.00	1.17	33.67	288.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.099	0.028	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.98	142.77	1440.00	0.28	8.03	288.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.905	0.235	1.60	1.000	1.00	1.00	1.00	1.00	1.00	8.98	1,303.19	1440.00	2.36	67.65	288.00
+0.60D-0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.330 ft	1	0.733	0.195	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.28	1,055.68	1440.00	1.96	56.28	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
E Only	1	0.3920	5.625		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.967	1.975
Overall MINimum	2.967	1.383
+D+H	0.532	0.532
+D+L+H	1.025	1.025
+D+Lr+H	0.532	0.532
+D+S+H	0.995	0.995
+D+0.750Lr+0.750L+H	0.902	0.902
+D+0.750L+0.750S+H	1.249	1.249
+D+0.60W+H	0.532	0.532
+D+0.70E+H	2.609	1.500
+D+0.750Lr+0.750L+0.450W+H	0.902	0.902
+D+0.750L+0.750S+0.450W+H	1.249	1.249
+D+0.750L+0.750S+0.5250E+H	2.807	1.975
+0.60D+0.60W+0.60H	0.319	0.319
+0.60D+0.70E+0.60H	2.396	1.287
D Only	0.532	0.532
L Only	0.493	0.493
S Only	0.462	0.462
E Only	2.967	1.383
H Only		

DESCRIPTION: Beam 3 with SW loading

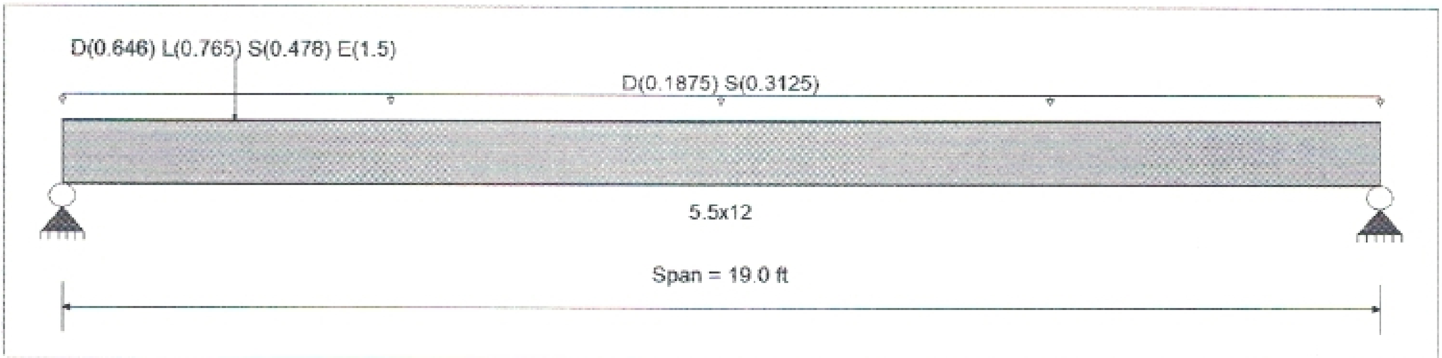
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species	: DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade	: 24F - V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 12.50 ft
Point Load : D = 0.6460, L = 0.7650, S = 0.4780, E = 1.50 k @ 2.50 ft, (beam 1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.811: 1**	Maximum Shear Stress Ratio	=	**0.400 : 1**
Section used for this span		**5.5x12**	Section used for this span		**5.5x12**
	=	2,239.48 psi		=	121.88 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	9.223 ft	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	0.679 in	Ratio =	335 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	1.140 in	Ratio =	200 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.418	0.217	0.90	1.000	1.00	1.00	1.00	1.00	1.00	9.93	902.89	2160.00	2.28	51.87	238.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.415	0.253	1.00	1.000	1.00	1.00	1.00	1.00	1.00	10.96	995.96	2400.00	2.95	66.97	265.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.301	0.157	1.25	1.000	1.00	1.00	1.00	1.00	1.00	9.93	902.89	3000.00	2.28	51.87	331.25
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.811	0.400	1.15	1.000	1.00	1.00	1.00	1.00	1.00	24.63	2,239.48	2760.00	5.36	121.88	304.75
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.324	0.191	1.25	1.000	1.00	1.00	1.00	1.00	1.00	10.69	972.26	3000.00	2.78	63.19	331.25
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.715	0.380	1.15	1.000	1.00	1.00	1.00	1.00	1.00	21.70	1,973.17	2760.00	5.09	115.70	304.75

DESCRIPTION: Beam 3 with SW loading

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.235	0.122	1.60	1.000	1.00	1.00	1.00	1.00	1.00	9.93	902.89	3840.00	2.28	51.87	424.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.269	0.171	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.35	1,031.80	3840.00	3.19	72.59	424.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.253	0.149	1.60	1.000	1.00	1.00	1.00	1.00	1.00	10.69	972.26	3840.00	2.78	63.19	424.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.514	0.273	1.60	1.000	1.00	1.00	1.00	1.00	1.00	21.70	1,973.17	3840.00	5.09	115.70	424.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.539	0.310	1.60	1.000	1.00	1.00	1.00	1.00	1.00	22.75	2,068.25	3840.00	5.77	131.24	424.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.141	0.073	1.60	1.000	1.00	1.00	1.00	1.00	1.00	5.96	541.74	3840.00	1.37	31.12	424.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 19.0 ft	1	0.175	0.122	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.41	673.50	3840.00	2.28	51.85	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	1.1398	9.431		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	6.198	5.034
Overall MINimum	1.303	0.197
+D+H	2.478	2.002
+D+L+H	3.142	2.103
+D+Lr+H	2.478	2.002
+D+S+H	5.862	5.034
+D+0.750Lr+0.750L+H	2.976	2.078
+D+0.750L+0.750S+H	5.514	4.351
+D+0.60W+H	2.478	2.002
+D+0.70E+H	3.390	2.140
+D+0.750Lr+0.750L+0.450W+H	2.976	2.078
+D+0.750L+0.750S+0.450W+H	5.514	4.351
+D+0.750L+0.750S+0.5250E+H	6.198	4.455
+0.60D+0.60W+0.60H	1.487	1.201
+0.60D+0.70E+0.60H	2.399	1.339
D Only	2.478	2.002
L Only	0.664	0.101
S Only	3.384	3.032
E Only	1.303	0.197
H Only		

POST - INSTALLED ANCHOR DESIGN.

→ FOR HOLDOWN @ GRID 6.

HDU4 (9") → REQUIRED TENSION = 110 #· Ω_0=2.5
= 280 #.

PLEASE NOTE TENSION DEMAND IS VERY SMALL DUE TO LONG
SW LENGTH AND LOW SEISMIC FORCE.

→ CHECK SIMPSON SET-XP. w/ 5/8" ϕ BOLT (EMBED 9").

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 9.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 12.13
c_{ac} (inch): 12.33
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

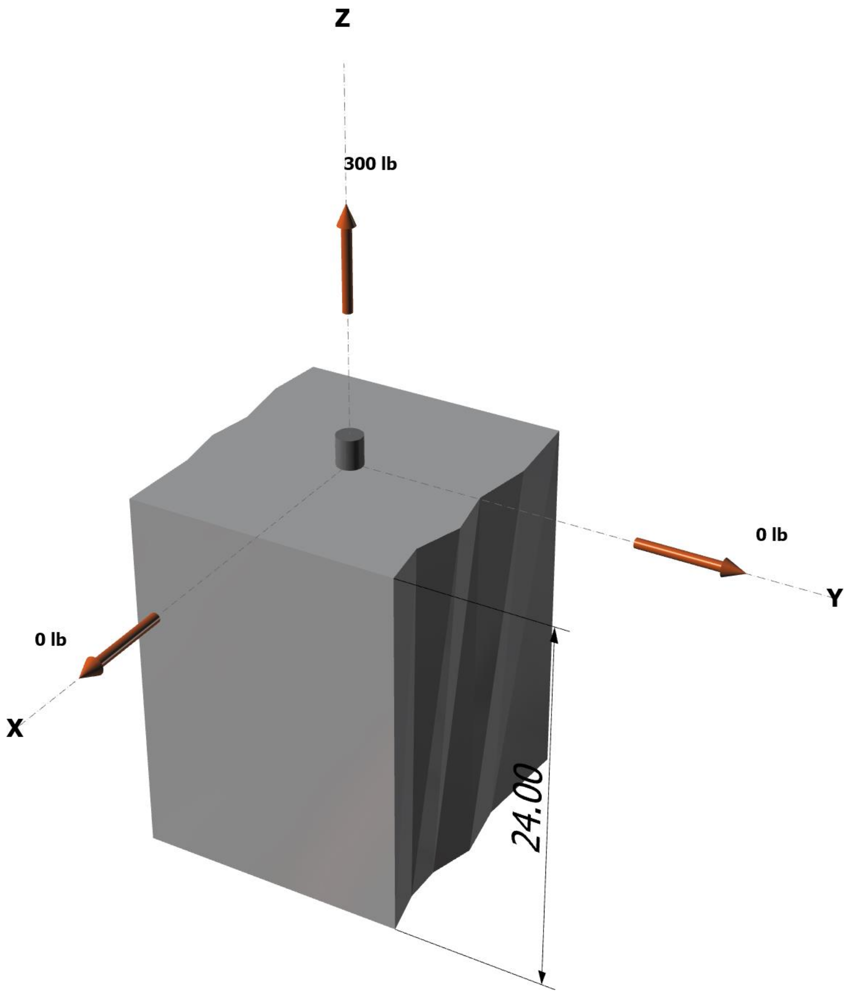
Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Yes
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

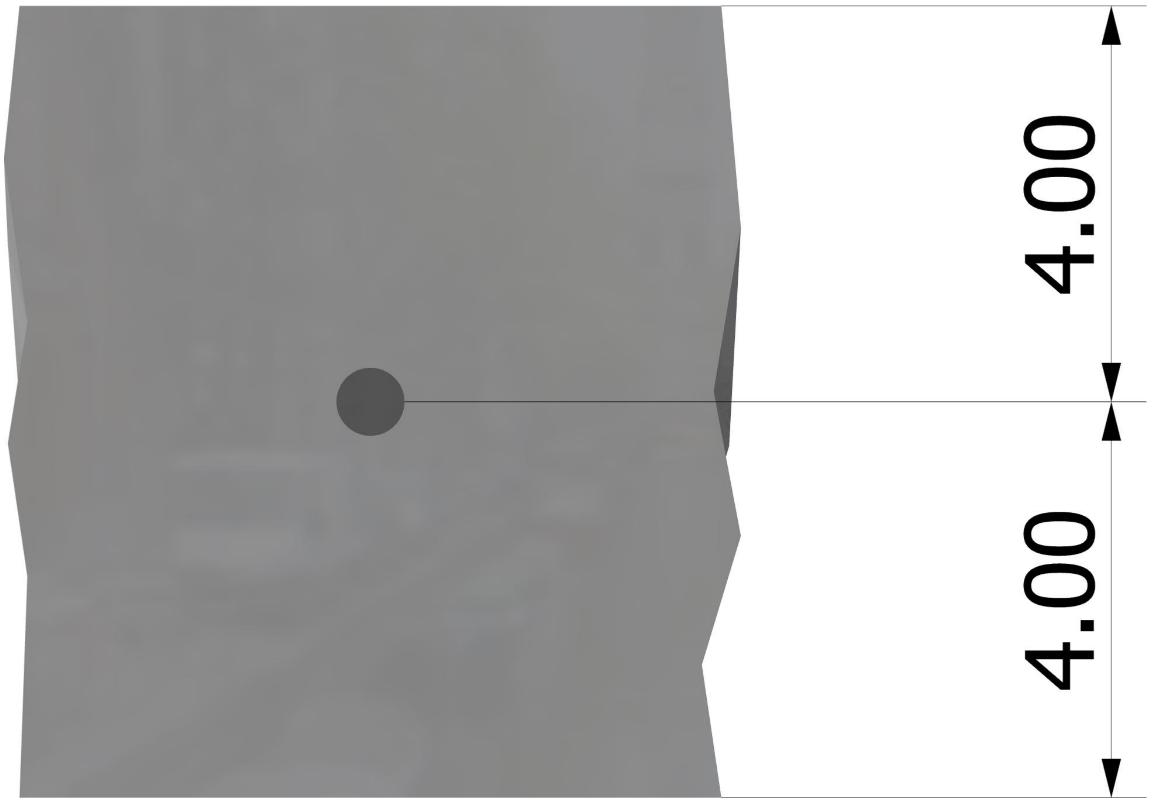
Strength level loads:

N_{ua} [lb]: 300
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	300.0	0.0	0.0	0.0
Sum	300.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 300
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	9.000	22950

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
216.00	729.00	4.00	0.789	1.00	1.000	22950	0.65	3487

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
435	1.00	1.00	435

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	435	0.63	9.000	7687

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
98.16	150.57	6.14	4.00	0.896	1.000	7687	0.55	2469

$\phi N_{sust} = 0.55 \phi N_{ba}$ (Eq. 17.3.1.2)

ϕ	N_{ba}(lb)	ϕN_{sust} (lb)
0.55	7687	2325

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	300	9833	0.03	Pass
Concrete breakout	300	3487	0.09	Pass
Adhesive	300	2469	0.12	Pass
Adhesive (sustained)	**300**	**2325**	**0.13**	**Pass (Governs)**

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 9.000 inch meets the selected design criteria.

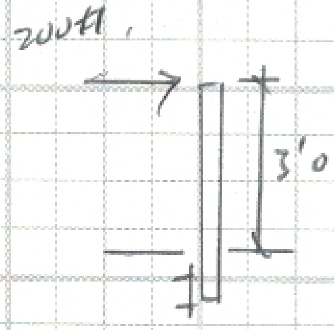
12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

RAILING ATTACHMENT DESIGN CHECK.



MOMENT @ SUPPORT = 600 #-ft.

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.000
Effective Embedment depth, h_{ef} (inch): 2.190
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 4.67
c_{ac} (inch): 3.31
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 5.00 x 5.00 x 0.25

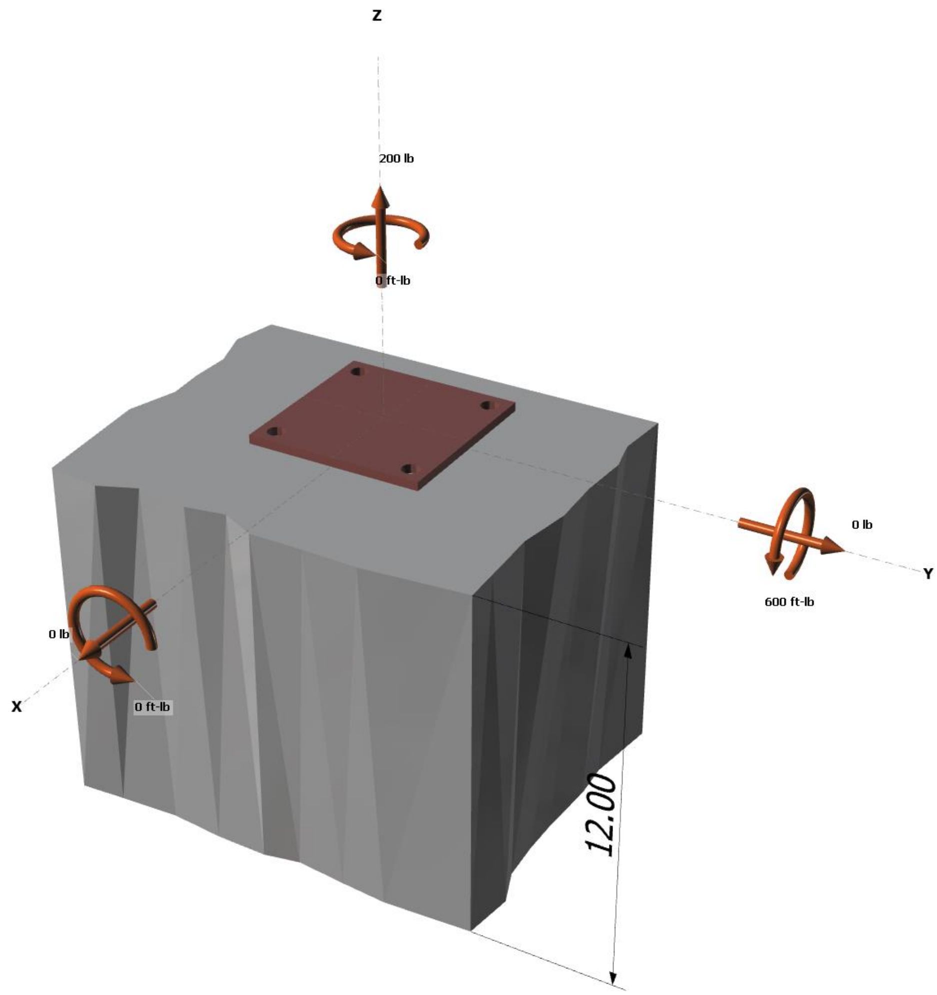
Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:3" (76mm)
Code Report: ICC-ES ESR-2713

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

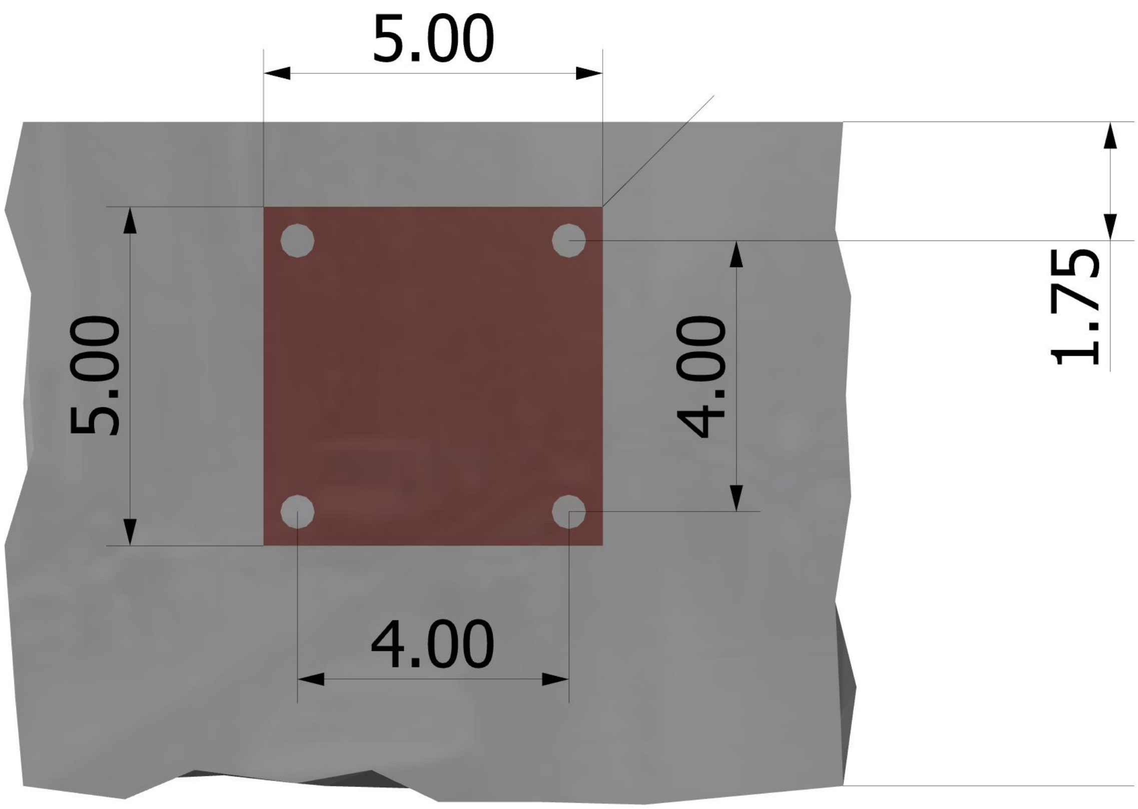
Strength level loads:

N_{ua} [lb]: 200
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 600
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

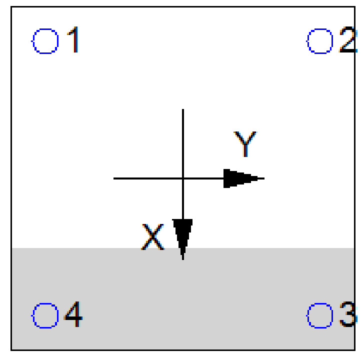


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	948.9	0.0	0.0	0.0
2	948.9	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0
Sum	1897.8	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.11
Maximum concrete compression stress (psi): 460
Resultant tension force (lb): 1898
Resultant compression force (lb): 1698
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
10890	0.65	7079

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	2.190	2755

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
53.22	43.16	1.75	1.000	0.860	1.00	1.000	2755	0.65	1898

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} \lambda_a N_p (f'_c / 2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	ϕN_{pn} (lb)
1.0	1.00	2212	2500	0.50	0.65	1438

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	949	7079	0.13	Pass
Concrete breakout	**1898**	**1898**	**1.00**	**Pass (Governs)**
Pullout	949	1438	0.66	Pass

3/8"Ø Titen HD, hnom:3" (76mm) meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer’s product literature for hole cleaning and installation instructions.