

Structural Calculations

For

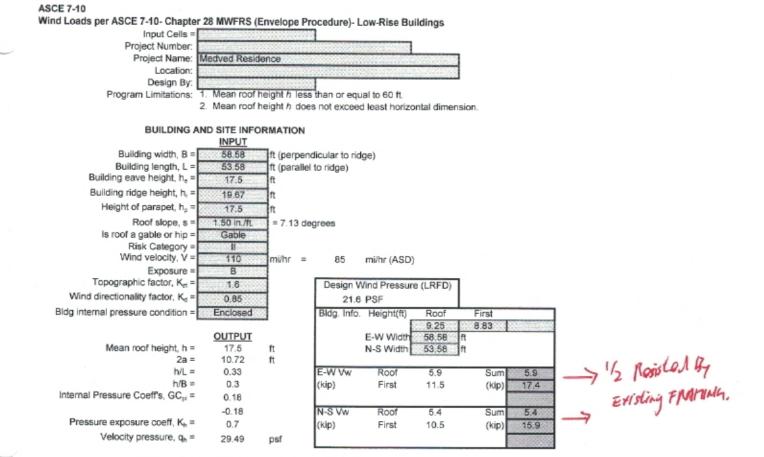
MEDVED RESIDENCE 4752 89th AVE SE MERCER ISLAND, WA 98040

> BE Project # 19125 July 28, 2020

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



PROJECT NAME	Medved Residence		
ADDRESS	4752 89th AVE SE Merce	r island V	NA 98040
PROJECT #			
DATE	7/28/2020		
BUILDING CODE	2015 International Reside		ode
	2015 International Buildi	ng Code	
	14.14	110	
WIND DESIGN	Vult =	110	MPH
	Vasd =	85	MPH
	Exposure = Kzt =	B	_
		1.6	_
	Importance Factor =	1.0	-
SEISMIC DESIGN	Ss(g) =	1.43	Sms(g) = 1.43 Sds(g) = 0.953
SEISIVIIC DESIGN	S1(g) =	0.549	SM1(g) = 0.823 $SD1(g) = 0.549$
	Seismic Design Category =	D.047	0.023 0.01(9) 0.047
	Site Class =	D	-
	Importance Factor =	1.0	_
		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
		00	F 31
	Roof Dead Load =	15	PSF
	Floor Dead Load =	15	PSF (For framing gravity design)
	Exterior Wall Dead Load =	10	PSF
Part	tition Wall Seismic Weight =	10	PSF
	Floor Seismic Weight =	10	PSF
			-
	Allowable Soil Pressure =	1500	PSF
Lateral Ea	arth (Restrained) Pressure =	50	PCF
	Passive Pressure =	300	PCF
	Coefficient of Friction =	0.4	
SCOPE OF WORK	Existing residence remod	el desiar	
	Existing residence remou	ci ucsiyi	



MAIN WIND-FORCE RESISTING SYSTEM (MWFRS) Wind Pressures for Low-Rise Buildings

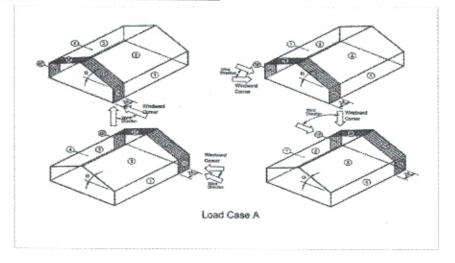
 $p = q_h[(GC_{pl}) - (GC_{pl})] \ (lb/lt^2)$ Load Case A: Winds Perpendicular to Ridge

Bldg Surface GCpf Wind Pressure (lb/ft2) LRFD ASD 0.42 12.4 74 2 -0.69 -20.4 -12.2 -0.39 3 -11.6-7 -0.31 4 -8.2 -5.5 1E 0.64 11.3 18.9 -1.07 2E -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
 - 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)
 - 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.



SHEET___OF___

Load Case B: Winds Parallel to Ridge

Bldg Surface	GCpf	Wind Pre	essure (lb/ft²)	
		LRFD	ASD	1
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	

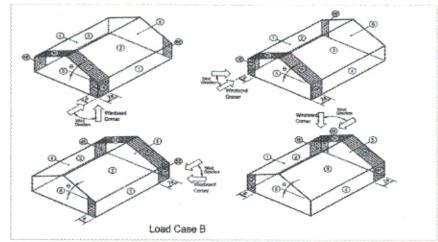
Internal pressure = +/- 5.3 psf (LRFD)

+/- 3.2 psf (ASD)

lote: 1. Sign Convention

 positive numbers denote forces toward the surface negative numbers denote forces away from the surface
 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).

 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.



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)	
$p_p = q_p$	(GCpn) (lb/	tt ²)	
Windward parapets, pp_wind =	44.2	psf (LRFD)	
Leeward parapets, pp_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)	

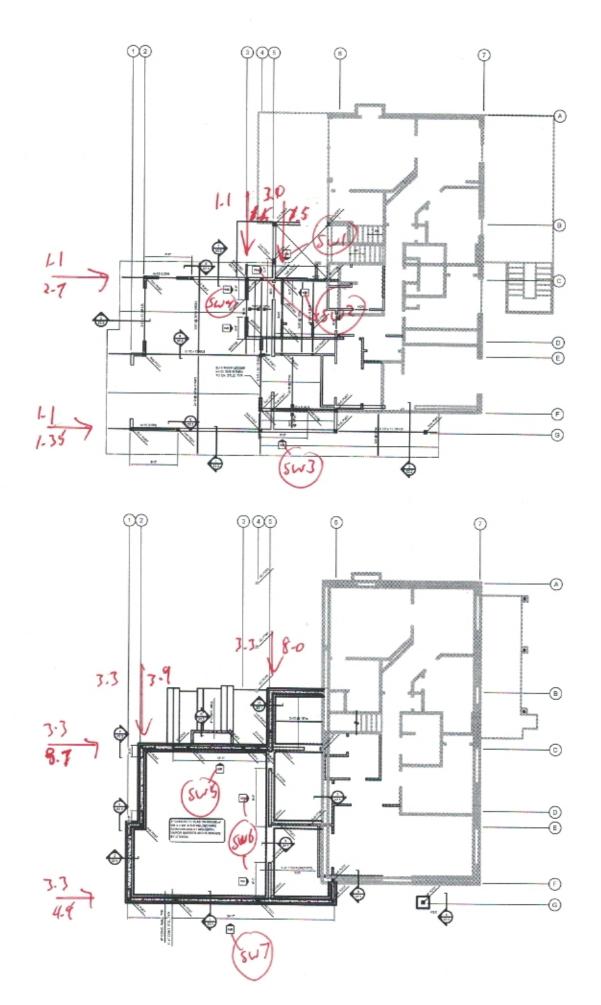
Burnel Control Burnel Account Burnel Exciting on the Wey Addition New Addition Calculations per ASCE 7- ASCE 7-10, Page 2, Table 1.5 Setimal Importance Factor = 1 VEER DEFINED Ground Motions, 9% Damping : Setimal Importance Factor = Setimal Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Setimal Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Setimal Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Setimal Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Setimation S	ASCE Seismic Base Shear					Software convrint F	NERCALC, INC. 1983-20	d Residence.ec6
Calculations per ASCE 7-10 Risk Category Calculations per ASCE 7-10 Risk Category ACCE 7-10, Page 2, Table 1.3 Site Category of Building or Other Structure: IT: * A Buildings and other structures except hose leated as Category I, III, and IV ASCE 7-10, Page 2, Table 1.3 Site Control 10.000 ASCE 7-10, Page 2, Table 1.3 Site Control 10.0000 ASCE 7-10 Table 5.3 Site Control 10.00000 ASCE 7-10 Table 20 Site Control 10.000000 Site Control 10.00000000000000000000000000000000000						Councilo coblight o		
Risk Category Calculations per ASCE 7- Risk Category of Building or Other Structure : "I" : All Buildings and other structures except those listed as Category I, III, and IV ASCE 7-10, Page 2, Table 1.5 Selamic Importance Factor = 1 ASCE 7-10, Page 3, Table 1.5 Selamic Importance Factor = 1 ASCE 7-10, Page 3, Table 1.5 USER DEFINED Ground Motion ASCE 7-10, Page 5, Table 1. ASCE 7-10, 11.4 Max, Ground Michon, SK Damping: Sign 1, 430, 0, 0.2 sec response ASCE 7-10, Table 20. Site Cates, Site Coeff. and Design Category Site Category File D Site Coefficient F. Sa Fy File 1.00 ASCE 7-10 Table 11.4.1 8.11.4 Using straight-file interpolation from table values) File 0.824 ASCE 7-10 Eq.11.4 Site Coefficient F. Sa Fy = 0.824 ASCE 7-10 Eq.11.4 Valing straight-file interpolation from table values) File 0.824 ASCE 7-10 Eq.11.4 Site Coefficient F. Sa Fy = 0.824 ASCE 7-10 Eq.11.4 Site Stamm Considered Earthquake Acceleration Site 7.5 a, 71.2 a 0.549 ASCE 7-10 Eq.11.4 Site Stamm Considered Earthquake Acceleration Site 7.5 a, 71.2 a	DESCRIPTION: New Addition							
The All Building of Other Structure : "IT : All Buildings and other structures except hose lested as Category I, III, and IV ASCE 7-10, Page 2, Table 1.5 Selemic Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Selemic Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Selemic Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Selemic Importance Factor = 1 ASCE 7-10, Page 2, Table 1.5 Selemic Importance Factor = 0.5490 g. 10 sec response ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification TD : Share Wave Model 600 to 1.200 Neec = D ASCE 7-10 Table 1.1 Site Cassification Cassification Cassification Cassification Castreaction Site Cassification Cassificati	New Addition							
The decay of stating if control decays in a constraint in the decay for decay of the decay of t	Risk Category						Calculations	per ASCE 7-1
USER DEFINED Ground Motion ASCE 7-10 114 Max. Ground Motions, 5% Damping : Sig = 1,400 g, 0.2 sector 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 Fa = 1,00 Site Class, Site Coeff, and Design Category Fa = 1,00 Residuation "D": Shear Wave Valceby 600 to 1,200 Nace Fa = 1,00 Residuation To": Shear Wave Valceby 600 to 1,200 Nace Fa = 1,00 Residuation To": Shear Wave Valceby 600 to 1,200 Nace Fa = 1,00 Residuar Strategistine interpolation from table values) Fv = 1,50 Residuar Strategistine interpolation from table values) Sum = fer * St = 0,024 Sum = fer * St = 0,024 ASCE 7-10 Eq. 11.4 Sum = fer * St = 0,0549 ASCE 7-10 Table 11.6+16 Residuar Design Category = D ASCE 7-10 Table 11.6+16 Sum = fer * St = 0,0549 Residuar System Sum = fer * St = 0,0549 Residuar System Back Residuar System Structural System Status Residuar System Status Residuar System Status Constrate Residuar System Sup = 6,50 Category Ye Limit No Limit e65 NOTE! See ASCE 7-10 for all applicable f	Risk Category of Building or Other Structure :	"II" : All Buildin	ngs and other	structur	es except thos	e listed as Category I, III, and IV	ASCE 7-10,	Page 2, Table 1.5-
USER DEFINED Ground Motion ASCE 7.10 114 Max. Ground Motion, 5% Damping: S S S S 1 0.5400 g. 1.0 sec response Conforms to ASCE 7-80 close 128.1.3. Regular structure with period of 0.5 s or less, St indiced to max of 1.5 for calculation of Cs. Site Classification D: Shear Wave Velocity 600 to 1.200 fixee Fa D ASCE 7-10 Table 11.4.8.11.4 Site Classification D: Shear Wave Velocity 600 to 1.200 fixee Fa = 1.00 ASCE 7-10 Table 11.4.8.11.4 Reside Coefficients Fa & Fv Fa = 1.00 ASCE 7-10 Table 11.4.8.11.4 ASCE 7-10 Table 11.4.8.11.4 Reside Coefficients Fa & Fv Fa = 1.00 ASCE 7-10 Table 11.4.8.11.4 Reside Coefficients Fa & Fv Fa = 0.022.4 ASCE 7-10 Table 11.4.8.11.4 Reside Stating System Sp = S M_2/3 = 0.923 ASCE 7-10 Table 11.6.1.6 Reside Stating System Sp = S M_2/3 = 0.953 ASCE 7-10 Table 11.6.1.6 Reside Stating System Basic Stating System Basic Stating Notinit Not limit Not	Seismic Importance Factor =	1					ASCE 7-10.	Page 5. Table 1.5
$\begin{array}{rcl} S_{S} & = & 1.430 \ g. 0.2 \ sec response \\ S_{1} & = & 0.5490 \ g. 1.0 \ sec response \\ Conforms to ASCE 7 \ Section 12.8.1.3. Regular structure with period of 0.5 s or less, S is limited to max of 1.5 for calculation of Cs. \\ \hline Site Class, Site Coeff. and Design Category \\ Site Classification TU: Shear Weav Valood y 600 to 1.20 Nisec = D ASCE 7.10 Table 11.4.1 & 11.4 (using streight-fine interpoletion from table values) F_{V} = 1.50 ASCE 7.10 Table 11.4.1 & 11.4 (using streight-fine interpoletion from table values) F_{V} = 1.50 ASCE 7.10 Eq. 11.4 SSCE 7.10$	USER DEFINED Ground Motion	No.	1					ASCE 7-10 11.4.
$S_{1} = 0.5490 \text{ g.} 1.0 \text{ sec response}$ Conforms to ASCE 7.5 eaction 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 Niece = D	Max. Ground Motions, 5% Damping :							
Site Class, Site Coeff. and Design Category Site Classification "D": Shear Wave Velocity 600 to 1,200 filesc = D ASCE 7-10 Table 20. Site Coefficients Fa & Fv Fa = 1,00 ASCE 7-10 Table 11.4-1 & 11.4 (using straight-file intropolation from table velues) Fv = 1,50 ASCE 7-10 Eq. 11.4 Sum Coefficients Fa & Fv S 1.430 ASCE 7-10 Eq. 11.4 Valing straight-file intropolation from table velues) Fv = 1.430 ASCE 7-10 Eq. 11.4 Sum Coefficient Fa & Fv S 0.953 ASCE 7-10 Eq. 11.4 Sum Coefficient S S $S_{D}^{-2}S_{M}^{-2}/3$ 0.953 ASCE 7-10 Eq. 11.4 Sum Coefficient S S $S_{D}^{-2}S_{M}^{-2}/3$ 0.953 ASCE 7-10 Table 11.6-1 & Resisting System Bearing Wall Systems ASCE 7-10 Table 11.6-1 & ASCE 7-10 Table 11.6-1 & System Overstraing Factor "No" * 2.50 Category "C Limit No Limit Category "C Limit No Limit Deflection Amplification Coefficient "R" * 6.50 Building height Limits : SCE 7-10 Section 12.8 Category "F' Limit Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes Category "F' Limit Limit = 65 Limit = 65								
Site Classification "D": Shear Wave Valcely 600 to 1,200 filsec = D ASCE 7-10 Table 20. Site Coefficients Fa & Fv Fa = 1.00 ASCE 7-10 Table 11.4-1 & 11.4 (using streight-line interpolation from table values) Fv = 1.50 ASCE 7-10 Table 11.4-1 & 11.4 (using streight-line interpolation from table values) Fv = 1.50 ASCE 7-10 Eq. 11.4 Str.4 ASCE 7-10 Eq. 11.4 ASCE 7-10 Eq. 25.0 Category 7 & A B' Limit No Limit Category 7' C' Limit No Limit Category 7' D' Limit Limit = 65 Category 7' C' Limit No Limit Eq. 25.0 Category 7' C' Limit No Limit Eq. 25.0 Category 7' Limit Limit = 65 Category F'	Conforms to ASCE 7 Section 12.8.1.3: Re	egular structure	with period o	f 0.5 s o	less, Ss limite	ed to max of 1.5 for calculation of	of Cs.	
Site Classification "D": Shear Wave Valcely 600 to 1,200 filsec = D ASCE 7-10 Table 20. Site Coefficients Fa & Fv Fa = 1.00 ASCE 7-10 Table 11.4-1 & 11.4 (using streight-line interpolation from table values) Fv = 1.50 ASCE 7-10 Table 11.4-1 & 11.4 (using streight-line interpolation from table values) Fv = 1.50 ASCE 7-10 Eq. 11.4 Str.4 ASCE 7-10 Eq. 11.4 ASCE 7-10 Eq. 25.0 Category 7 & A B' Limit No Limit Category 7' C' Limit No Limit Category 7' D' Limit Limit = 65 Category 7' C' Limit No Limit Eq. 25.0 Category 7' C' Limit No Limit Eq. 25.0 Category 7' Limit Limit = 65 Category F'								
(using streight-line interpolation from table values) Fv = 1.50 deximum Considered Earthquake Acceleration $S_{MS} = Fa^* Ss$ = 1.430 ASCE 7-10 Eq. 11.4 S MI = Fv^* S1 = 0.824 ASCE 7-10 Eq. 11.4 beigin Spectral Acceleration $S_{DF}^{-S} S_{MS}^{+2/3}$ = 0.953 ASCE 7-10 Eq. 11.4 s D F^-S M_1^+23 = 0.953 ASCE 7-10 Eq. 11.4 beigin Cleagony = D 4SCE 7-10 Table 11.6-1 8 Resisting System Bearing Wall Systems ASCE 7-10 Table 11.6-1 8 System Overstrangh Fador * Wo** = 2.50 Category * A B* Limit: No Limit Defection Amplification Factor * Cd** = 4.00 Category * T- Limit: No Limit System Overstrangh Fador * Wo** = 2.50 Category * T- Limit: Limit = 65 Lateral Force Procedure Category * T- Limit: Limit = 65 Limit = 65 VoTE! See ASCE 7-10 for all applicable footnotes. Category * T- Limit: Limit = 65 Limit = 65 Structure Type for Building Period Calculation: All Other Structural Systems * Category * T- Limit: Limit = 65 Limit = 65 Structure Type for Building Period sclaulation: All Other Structural Systems * Category * T- Limit: Limit = 65 Limit = 65		and the second s		=	D		ASC	E 7-10 Table 20.3
Iterimum Considered Earthquake Acceleration $S_{MS} = Fa^*Ss = 1.430$ ASCE 7-10 Eq. 11.4 Segin Spectral Acceleration $S_{MS} = Fa^*Ss = 1.430$ ASCE 7-10 Eq. 11.4 besign Spectral Acceleration $S_{DS} = S_{M}^*2/3 = 0.953$ ASCE 7-10 Eq. 11.4 besign Category = D 4SCE 7-10 Table 11.6.1 8 Resisting System Bearing Wall Systems ASCE 7-10 Table 11.6.1 8 Response Modification Coefficient * R * = 6.50 Building height Limits : System Covership for Factor * Cot * = 2.50 Category *C Limit No Limit Defection Amplification Factor * Cot * = 2.50 Category *C Limit No Limit Defection Amplification Factor * Cot * = 2.50 Category *C Limit No Limit Defection Amplification Factor * Cot * = 2.50 Category *C Limit Limit = 65 Category *F Limit Limit = 65 Category *F Limit Limit = 65 Category *F Limit Limit = 65 Lateral Force Procedure In * Height from base to highest level = 19.50 ft *x value = 0.75 *Ta * Approximate fundemental period using Eq. 12.8.7 : Ta = Ct * (nn ^ x) = 0.186 sec =	Site Coefficients Fa & Fv		Fa	=	1.00		ASCE 7-10 Ta	ble 11.4-1 & 11.4-
$S_{M}^{MS} = Fv^*S1 = 0.824$ ASCE 7-10 Eq. 11.4 Pesign Spectral Acceleration $S_{D}^{S} S_{M}^{K2} 23 = 0.953$ ASCE 7-10 Eq. 11.4 S_D = S_{M}^{K2} 23 = 0.549 ASCE 7-10 Eq. 11.4 S_D = S_{M}^{K2} 23 = 0.549 ASCE 7-10 Eq. 11.4 S_D = S_{M}^{K2} 23 = 0.549 ASCE 7-10 Eq. 11.4 S_D = S_{M}^{K2} 23 = 0.549 ASCE 7-10 Table 11.6.1 & Resisting System ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System ASCE 7-10 Table 12.2 Category 7C Limit No Limit Deflection Amplification Factor * Cd* = 4.00 Category 7C Limit Limit = 65 Category 7C Limit Category 7C Limit Limit = 65 Category 7C Limit Category 7C L	(using straight-line interpolation from table values)		Fv	=	1.50			
besign Spectral Acceleration $S_{DC}^{ers} S_{MS}^{ers} 2/3 = 0.953$ ASCE 7-10 Eq. 11.4 $S_{DC}^{ers} S_{M1}^{ers} 2/3 = 0.549$ ASCE 7-10 Eq. 11.4 $S_{DC}^{ers} S_{M1}^{ers} 2/3 = 0.549$ ASCE 7-10 Eq. 11.4 Elemic Design Category $= D$ ASCE 7-10 Table 11.6-1.8 Resisting System ASCE 7-10 Table 11.6-1.8 Resisting System ASCE 7-10 Table 11.6-1.8 Response Modification Coefficient "R" = 6.50 System Overstrength Factor "Vo" = 2.50 Category "C" Limit No Limit Deflection Amplification Factor "Cd" = 4.00 Category "C" Limit Limit = 65 ASCE 7-10 Section 12.8 Lateral Force Procedure ASCE 7-10 Section 12.8 Lateral Force Procedure Category "F" Limit Limit = 65 Category "F" Limit E Limit = 65 Category "F" Limit E Limit = 65 Lateral Force Procedure DISCE 7-10 Section 12.8 Lateral Force Procedure DISCE 7-10 Section 12.8 Lateral Force Procedure DISCE 7-10 Section 12.8 Structure Type for Building Period Calculation : All Other Structural Systems 'C1 'value = 0.20 'hn": Height from base to highest level = 19.50 ft *X" value = 0.75 'Ta "Approximate fundemental period using Eq. 12.8-7 : Ta = C1" (hn ^x) = 0.186 sec "Cs" Response Coefficient ASCE 7-10 Maps 22-12 -> 22-16 Building Period "Ta "Calculated from Approximate Method selected = 0.186 sec "Cs" Response Coefficient ASCE 7-10 Maps 22-12 -> 22-16 Building Period "Ta "Calculated from Approximate Method selected = 0.186 sec "Cs" Response Coefficient ASCE 7-10 Section 12.8.1 S bg Short Period Design Spectral Response = 0.953 From Eq. 12.8.2 Read not exceed = 0.1467 'R" "Response Modification Factor = 5.50 From Eq. 12.8.3 R2.8.4, Cs need not exceed = 0.042 User has selected ASCE 12.8.1.3.1 Regular structure	aximum Considered Earthquake Acceleration	S _{MS} =	Fa*Ss	=	1.430		AS	CE 7-10 Eq. 11.4-
$S_{D}^{=S}$ M_{s}^{S-T} 0.000 ASCE 7-10 Eq. 11.4 $S_{D}^{=S}$ M_{s}^{S-T} 0.549 ASCE 7-10 Eq. 11.4 Resisting System = D 4SCE 7-10 Table 11.2.1 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 System Overstrength Factor * R* = 6.50 Building height Limits : System Overstrength Factor * Vo* = 2.50 Category *C Limit No Limit Category *C Limit No Limit Deflection Amplification Factor * Cd* 4.00 Category *C Limit Limit = 65 Category *C Limit Limit = 65 Category *C Limit Limit = 65 Lateral Force Procedure Iter * Equivalent Lateral Force Procedure* Iter * Equivalent Lateral Force Procedure* Use ASCE 7-10 Section 12.8 Determine Building Period Use ASCE 7-10 Maps 22-12 -> 22-16 Category *C* Use ASCE 7-10 Section 12.8 Structure Type for Building Period Part (Tell * Ta* Calculated from Approximate Mathod selected = 0.186 sec *TL*: Long-period transition period per ASCE 7-10 Ma		S _{M1} =	Fv * S1	=	0.824		AS	CE 7-10 Eq. 11.4-
$S_{D}^{=S}$ M_{s}^{S-T} 0.000 ASCE 7-10 Eq. 11.4 $S_{D}^{=S}$ M_{s}^{S-T} 0.549 ASCE 7-10 Eq. 11.4 Resisting System = D 4SCE 7-10 Table 11.2.1 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems ASCE 7-10 Table 12.2 System Overstrength Factor * R* = 6.50 Building height Limits : System Overstrength Factor * Vo* = 2.50 Category *C Limit No Limit Category *C Limit No Limit Deflection Amplification Factor * Cd* 4.00 Category *C Limit Limit = 65 Category *C Limit Limit = 65 Category *C Limit Limit = 65 Lateral Force Procedure Iter * Equivalent Lateral Force Procedure* Iter * Equivalent Lateral Force Procedure* Use ASCE 7-10 Section 12.8 Determine Building Period Use ASCE 7-10 Maps 22-12 -> 22-16 Category *C* Use ASCE 7-10 Section 12.8 Structure Type for Building Period Part (Tell * Ta* Calculated from Approximate Mathod selected = 0.186 sec *TL*: Long-period transition period per ASCE 7-10 Ma	lesion Spectral Acceleration	S = S	* 2/3		0.053		24	CE 7-10 En 11 A
elsimic Design Category = D ASCE 7-10 Table 11.6-1 & ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems 13.Light-frame (wood) walls sheathed wiwood structural panels rated for shear resistance. ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems 13.Light-frame (wood) walls sheathed wiwood structural panels rated for shear resistance. Response Modification Coefficient * R * = 6.50 Building height Limits : System Overstrangin Factor * Wo* = 2.50 Category * C Limit No Limit Deflection Amplification Factor * Cd * = 4.00 Category * C' Limit No Limit Soce 7-10 Section 12.8 Deflection Amplificable footnotes. Category * C' Limit Limit = 65 Category * C' Limit Limit = 65 Lateral Force Procedure The *Equivalent Lateral Force Procedure* Is being used according to the provisions of ASCE 7-10 12.8 Determine Building Period All Other Structural Systems ''Ct 'walue = 0.75 ''T a * Calculated from Approximate Method selected = 0.186 sec * Tu* : Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec Suiding Period * Ta * Calculated from Approximate Method selected = 0.186 sec * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 Spectral Response = 0.50 From Eq. 12.8-5 & 12.		~~	11110					
ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System ASCE 7-10 Table 12.2 Basic Seismic Force Resisting System Bearing Wall Systems 13.Light-frame (wood) walls sheathed wiwood structural panels rated for shear resistance. Response Modification Coefficient 1 R * = 6.50 Building height Limits : System Overstrength Factor *Wo* = 2.50 Category *C Limit: No Limit Category *C' Limit: Limit = 65 No Limit Deflection Amplification Factor *Wo* = 2.50 Category *C' Limit: Limit = 65 ASCE 7-10 To all applicable footnotes. Category *C' Limit: Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes. Category *C' Limit: Limit = 65 ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8 Use ASCE 12.8 Structure Type for Building Period Calculation : All Other Structural Systems *C't value = 0.200 *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 aSCE 7-10 Section 12.8.1 *C * Response Coefficient Spe Sibnt Period Design Spectral Response = 0.953 * Spe Sibnt Period Disign Spectral Response = 0.953 From Eq. 12.8-2. Preliminary Cs = 0.147 * R* : Response Modification Factor = 0.50 From Eq. 12.8-3 & 12.8-4. Cs note le	eismic Design Category	Di	MI					
Bearing Wall Systems 13.Light-frame (wood) walls sheathed wiwood 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 Category *C* Limit: No Limit Deflection Amplification Factor * Cd * = 4.00 Category *C* Limit: No Limit Category *C* Limit: Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes. Category *C* Limit: Limit = 65 ASCE 7-10 Section 12.8 Lateral Force Procedure ASCE 7-10 Section 12.8 Category *C* Limit: Limit = 65 Equivalent Lateral Force Procedure The *Equivalent Lateral Force Procedure* is being used according to the provisions of ASCE 7-10 12.8 Use ASCE 12.8 Structure Type for Building Period Calculation : All Other Structural Systems *Ct * value = 0.75 *In ** Height from base to highest level = 19.50 ft use ASCE 7-10 Section 12.8 ** * value = 0.75 *Ta * Calculated from Approximate Method selected = 0.186 sec *TL*: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec ** ** ** ** ** ** ** ** ** ** ** ** **				-	D			
13.Light-frame (wood) walls sheathed wiwood structural panels rated for shear resistance. Response Modification Coefficient * R* = 6.50 Building height Limits : System Overstrength Factor * Wo* = 2.50 Category *C* Limit: No Limit Category *C* Limit: No Limit Limit = 65 Deflection Amplification Factor * Cd* = 4.00 Category *C* Limit: Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes. Category *C* Limit: Limit = 65 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 7.10 Section 12.8 Structure Type for Building Period Calculation : All Other Structural Systems * Ct * value = 0.75 * 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 Metho		Wall Systems					ASCE	7-10 Table 12.2-
System Overstrengti Factor "Wo" = 2.50 Category "C Limit: No Limit Deflection Amplification Factor "Cd" = 4.00 Category "C" Limit: No Limit Deflection Amplification Factor "Cd" = 4.00 Category "C" Limit: Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes. Category "F" Limit: Limit = 65 Lateral Force Procedure ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure ASCE 7-10 Section 12.8 Determine Building Period Use ASCE 12.8 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 .0.75 "Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.186 sec "Ct * Response Coefficient ASCE 7-10 Maps 22-12 -> 22-16 Bog Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.186 sec "Ct * Response Modification Factor = 0.50 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.475 "R *: Response Modification Factor = 0.50 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.4455 "I': Selsmic Importance Factor = 1 From Eq. 12.8-5, Con the less than = 0.042 0.4457 <td></td> <td></td> <td></td> <td>hed w/w</td> <td>ood structura</td> <td>al panels rated for shear resis</td> <td>tance.</td> <td></td>				hed w/w	ood structura	al panels rated for shear resis	tance.	
Deflection Amplification Factor * Cd * 4.00 Category *C* Limit: No Limit: NOTE! See ASCE 7-10 for all applicable footnotes. Category *C* Limit: Limit = 65 Lateral Force Procedure ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure ASCE 7-10 Section 12.8 Determine Building Period Use ASCE 7-10 Section 12.8 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 * Ct * (hn ^ x) = 0.186 sec * TL*: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec Euliding Period *Ta * Calculated from Approximate Method selected = 0.186 sec * Cs * Response Coefficient Scce 7-10 Section 12.8.1 Scce 7-10 Section 12.8.1 Scce 7-10 Section 12.8.1 * R *: Response Modification Factor = 0.953 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.425 * I*: Selsmic Importance Factor = 1 From Eq. 12.8-5 & 12.8-6, Cs not be less than = 0.042								
Detection Anglinication Factor Col 4,00 Category *D* Limit: Category *D* Limit: Limit = 65 Limit = 65 NOTE! See ASCE 7-10 for all applicable footnotes. Category *E* Limit: Limit = 65 Limit = 65 Lateral Force Procedure ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8 Use ASCE 12.8 Determine Building Period Use ASCE 12.8 Structure Type for Building Period Calculation : All Other Structural Systems *Ct* value = 0.20 * hn *: Height from base to highest level = 19.50 ft *** value = 0.75 *Ta * Approximate fundemental period using Eq. 12.8-7 : Ta = Ct* (hn ^ x) = 0.186 sec *** value = 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 *** value *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 ************************************								
Category "F" Limit: Limit = 65 Lateral Force Procedure ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure Imit: Limit: Limit: Edit Equivalent Lateral Force Procedure Imit: Limit: Limit: Edit ASCE 7-10 Section 12.8 Determine Building Period Use ASCE 12.8 Use ASCE 12.8 Use ASCE 12.8 Structure Type for Building Period Calculation : All Other Structural Systems Ta * Cit * value 19.50 ft * Ct * value = 0.020 * hn *: Height from base to highest level = 19.50 ft * x value = 0.75 Ta = Ct * (hn ^ x) = 0.186 sec * TL :: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec = 0.186 sec * TL :: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec = 0.186 sec * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 Section 12.8.1 E 0.186 sec * R :: Response Modification Factor = 0.953 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.		4.00	Categ	ory "D" L	imit:			
Lateral Force Procedure ASCE 7-10 Section 12.8 Equivalent Lateral Force Procedure The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8 Use ASCE 12.8 Determine Building Period Use ASCE 12.8 Use ASCE 12.8 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 = Ct* (hn ^ x) = 0.186 sec * TL *: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec = 0.186 sec * Cs ** Response Coefficient ASCE 7-10 Section 12.8.1 S or 12.8.1 S or 12.8.1 * R *: Response Modification Factor = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 * R *: Response Modification Factor = 0.50 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.455 * I*: Selsmic 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, Cs : Selsmic Response Coefficient = = 0.1467 <td>NOTE! See ASCE 7-10 for all applicable footnotes.</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td>	NOTE! See ASCE 7-10 for all applicable footnotes.							
The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-10 12.8 Use ASCE 12.8 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 = Ct * (hn ^ x) = 0.186 sec * * TL *: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec * 0.186 sec * 0.186 sec * Cs ** Response Coefficient ASCE 7-10 Section 12.8.1 S DS Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 * R *: Response Modification Factor = 0.50 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.455 * 1' : Selsmic 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, Cs : Selsmic Response Coefficient = = 0.1467	Lateral Force Procedure			,			ASCE 7	-10 Section 12.8.
Determine Building Period Use ASCE 12.8 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 = Ct * (hn ^ x) = 0.186 sec * TL *: Long-period transition period per ASCE 7-10 Maps 22-12 -> 22-16 6.000 sec = 0.186 sec * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 * S Des Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 * R *: Response Modification Factor = 0.953 From Eq. 12.8-3 & 12.8-4, Cs need not exceed = 0.455 * I *: Selsmic 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, Cs : Selsmic Response Coefficient = = 0.1467	Equivalent Lateral Force Procedure							
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 = 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 S DG Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 "R": Response Modification Factor = 0.953 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, Cs : Seismic Response Coefficient = = 0.1467	The "Equivalent Late	eral Force Pro	cedure" is b	eing us	ed according	to the provisions of ASCE 7	-10 12.8	
* 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 * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 S DS* Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 * R *: Response Modification Factor = 0.50 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, Cs : Seismic Response Coefficient = = 0.1467	Determine Building Period							Use ASCE 12.8-
* 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 * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 S DS* Short Period Design Spectral Response = 0.953 From Eq. 12.8-2, Preliminary Cs = 0.147 * R *: Response Modification Factor = 0.50 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, Cs : Seismic Response Coefficient = = 0.1467	Structure Type for Building Period Calculation : All C	ther Structural	Systems					
* 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 Ta = Ct* (hn ^ x) = 0.186 sec Building Period * Ta * Calculated from Approximate Method selected = 0.186 sec * Cs * Response Coefficient ASCE 7-10 Section 12.8.1 S DS Short Period Design Spectral Response = 0.953 * R *: Response Modification Factor = 0.953 * 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, Cs : Seismic Response Coefficient = = 0.1467				highest	level =	19.50 ft		
"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 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, Cs : Seismic Response Coefficient = = 0.1467	"x "value = 0.75							
Building Period " Ta " Calculated from Approximate Method selected = 0.186 sec " Cs " Response Coefficient ASCE 7-10 Section 12.8.1 S DG 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, Cs : Seismic Response Coefficient = = 0.1467				a = Ct*	hn ^ x) =	0.186 sec		
"Cs " Response Coefficient ASCE 7-10 Section 12.8.1 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 ": Selsmic 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, Cs : Selsmic Response Coefficient = = 0.1467	"TL" : Long-period transition period per ASCE 7-10 M	Maps 22-12 -> 2	2-16			6.000 sec		
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, Cs : Seismic Response Coefficient = = 0.1467			Bui	ding Per	iod " Ta " Cak	culated from Approximate Metho	d selected =	0.186 sec
"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, Cs : Seismic Response Coefficient = = 0.1467	" Cs " Response Coefficient						ASCE 7-	10 Section 12.8.1.
"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, Cs : Seismic Response Coefficient = = 0.1467		=	0.953					0.147
User has selected ASCE 12.8.1.3 : Regular structure, Cs : Seismic Response Coefficient = = 0.1467		=						
	I Seismic Importance Factor	=	1		From Eq	. 12.8-5 & 12.8-6, Cs not be les	is than =	0.042
Less than 5 Stories and with T <<= 0.5 sec, SO Ss <= 1.5 for Cs calculation					min Dear			0 4 407

ASCE Seismic Base Shear Lic: #: KW-06007583

File: Medved Residence.ec6 Software copyright ENERCALC, INC. 1983-2020, Build: 12.20.5.31 BURT ENGINEERING PLLC

DESCRIPTION: New Addition

Cs =	0.1467 from	12.8.1.1			W	see Sum	Wibelow) =	34.80	k	
					Seismic Base	e Shear N	/= Cs*W =	5.10	k	
/ertical Distri	bution of Seis	mic Forces							ASCE	7-10 Section 12.8.3
' k " : hx exponen			.00							
The restored and the second seco	leights by Floor Lev									
Level #	Wi : We	eight	Hi : Heig	ht	(Wi * Hi^k)	Cvx	Fx=Cvx * V	Sum Story	Shear	Sum Story Momen
2		6.90	18.0	8	124.75	0.3362	1.72	1	1.72)	0.00
1		27.90	8.8	3	246.36	0.6638	3.39	1	5.10	15.87
		34.80 k	0.0111.1	Wi*Hi =	371.11 k-ft		Total Base Shear =	0.1	ok	
Diaphragm Fo	orces : Seismic	Design Ca	tegory "B" t	o "F"				Base N	foment = AS	60.9 k-ft SCE 7-10 12.10.1.1
Diaphragm Fo	orces : Seismic Wi	Fi	tegory "B" t Sum Fi	o "F" Sum Wi	Fpx : Calcd	Fr	px:Min Fp	Base M		SCE 7-10 12.10.1.1
		The second s			Fpx : Calco 1.72	F	ox : Min Fp 1.32		AS Fpx	SCE 7-10 12.10.1.1 Dsgn. Force
Level # 2 1	Wi	Fi 1.72 3.39	Sum Fi 1.72 5.10	Sum Wi 6.90 34.80	1.72		1.32 5.32	x : Max	AS Fpx 1.7	SCE 7-10 12.10.1.1 Dsgn. Force



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 Desi	gn V (ASD)	SW Design	SW Uplift (ASD)	Wall Holdown	Foundation Holdown
Wind	SW1	13.58' WALL	9.25'	1.10 k	2.70 k	0.68	N	1.62 k	0.12 klf	<u>W6</u>	.73 k	Not Regd'	
Seisc.	SWZ	7.83' WALL	9.25'	3.0 k	1.50 k	1.18	N	2.10 k	0.27 klf	<u>W4</u>	2.26 k	MSTC40	
Wind	SW3	8.17' BEAM	9.25'	1.10 k	1.35 k	1.13	N	.81 k	0.10 kif	<u>W6</u>	.69 k	Not Regd'	
Wind	SW4	7.50' BEAM	9.25'	1.10 k	1.50 k	1.23	N	.90 k	0.12 klf	<u>W6</u>	.90 k	Not Regd	
Wind	SW5		8.83'	3,30 k	8.70 k	0.55	N	5.22 k	0.32 kif	<u>W6</u>	2.44 k		STHD10
Wind	6W2	15.92'	8.83'	3.30 k	8.0 k	0.55	N	4.80 k	0.30 klf	<u>W6</u>	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	<u>W6</u>	24 k		
_													

SW #	Vs,all (ASD) (kip/ft)	Vw,all (ASD) (kip/ft)	Wall HD	Tall (ASD)(kips)	FTG HD	Tall (ASD) (kips)	-7 NOTEr NOC
W6	0.26	0.37	MSTC28	1.54	STHD10	3.40	To SU.
W4	0.38	0.53	MSTC40	3.08	STHD14	3.82	
W3	0.49	0.69	MSTC52	4.62	HDU4	4.57	SW HOLI
2W6	0.52	0.73	MSTC66	5.86	HDUS	5.65	
2W4	0.76	1.07	MST72	6.73	HDU8	6.97	To WI
2W3	0.98	1.37	CMST12x84"	9.215	HDU11	9.34	-
2W2	1.28	1.79	2xMSTC66	11.72	HDU14	10.77	
1			2xMST72	13.46	HD12	12.67	
1			2xCMST12x84"	18.43	HDU14(SPC.)	14.44	
1			HD19(SPC.)	19.07	HD12(SPC.)	15.51	
1					HD19	16.77	
					HD19(SPC.)	19.07	*Holdown not required for uplift less

CHANGES SHERTHING & DOWNS DUE ILID Kets 16.

ss than 1 Kips(ASD)

AMA - HAS PAMPROVED PORTAL ERAME STRENGTH. (AMA TT-1007).

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

Minimum Width	Maximum Height	Allowable Design (ASD)	/alues per Frame Segment	
(in.)	(ft)	Shear ^(e,f) (lbf)	Deflection (in.)	Load Factor
16	8	850	0.33	3.09
.0	10	625	0.44	2.97
24	8	1,675	0.38	2.88
2.4	10	1,125	0.51	3.42

Foundation for Wind or Seismic Loading^(a,b,c,d)

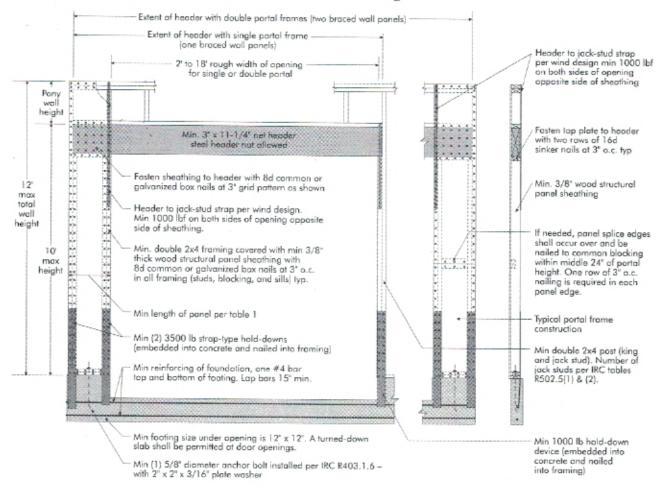
(a) Design values are based on the use of Douglas-fir or Southern pine framing. For other species of framing, multiply the above shear design value by the specific gravity adjustment factor = [1 - [0.5 - SG]), where SG = specific gravity of the actual framing. This adjustment shall not be greater than 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: Q GAMD(2) NERMINER VS = 3.3 k (LMED) = 2-31 k (ASD) VW = 3.9 k (LMPA) = 2.344 (ASD). ALOUPSIE SHEAR PER ENAME : VScall = 1675 k (ASA) Vu, all = 1.613/ . 1.4 (ASO) = 2-34 k. 2+ Usrall: 3.35k 7 2.31k. ok 2× Vugall, = 468 k 7 2.346 ok

Lic. # : KW-06007583

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

File: Retaining Wall Design (1500PSF).ec6 Software copyright ENERCALC, INC. 1983-2020, Build:12.20.5.31 BURT ENGINEERING PLLC

Criteria		
	-	4.00.6
Retained Height		4.00 ft
Wall height above soil Slope Behind Wall	-	0.50 ft
Height of Soil over Toe	-	0.00:1
•	_	6.00 in
Water height over heel	=	0.0 ft
Vertical component of act Lateral soil pressure optic	IVe IVe	
NOT USED for Soil I		9
NOT USED for Slidir		
NOT USED for Over		
Design Summary		
Wall Stability Ratios		
Overturning	=	2.44 OK
Sliding	=	1.35 OK
Slab Resists All Slidin	g !	
Total Bearing Load	=	1,394 lbs 5.08 in
resultant euc.	-	0.00 III
Soil Pressure @ Toe	=	975 psf OK
Soil Pressure @ Heel	=	39 psf OK
Allowable Soil Pressure Less	Than A	1,500 psf
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 Resis	sts All S	liding !
Lateral Sliding Force	=	528.8 lbs
less 100% Passive Force less 100% Friction Force		156.3 lbs 550.0 lbs
		0.0 lbs OK
Added Force Req'd for 1.5 : 1 Stability	=	79.2 lbs NG
,	-	79.2 IUS ING
Load Factors		1 000
Live Load		1.200
Earth, H		1.600
Wind, W		1.600
Seismic, E		1.000
eventino, E		1.000

Soil Data				Calculations (per ACI 318-14,	ACI 530-11, IBC 2015,
Allow Soil Bearing	=	1,500.0	psf			CBC 2016, ASCE 7-10
Equivalent Fluid Pressure Me	thod					
Heel Active Pressure	=	45.0	psf/ft			
Toe Active Pressure	=		psf/ft			
Passive Pressure	=	250.0				
Soil Density, Heel	=	110.00				
Soil Density, Toe	=	110.00				
Friction Coeff btwn Ftg & Soil		0.400	poi			
Soil height to ignore		0.100				
for passive pressure	=	12.00 ir	ı			
Stem Construction			Top Stem	2nd		
Design Height Above	Eta		Stem OK	Stern OK		
		ft =	4.50	0.00		
Wall Material Above ' Thickness	H	=	Concrete	Concrete		
Rebar Size		in =	8.00	8.00		
		=	# 4	# 4		
Rebar Spacing		in =	18.00	18.00		
Rebar Placed at Design Data		=	Edge	Edge		
fb/FB + fa/Fa		=	0.000	0.210		
Total Force @ Sectio	n	lbs =	0.0	570.0		
MomentActual		ft-l =	0.0	767.0		
MomentAllowable		ft-l =	3,655.6	3,655.6		
ShearActual		psi =	0.0	7.6		
ShearAllowable		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 foot	ting	in =	18.72	3.60		
Concrete Data						
fc		psi =	2,500.0	2.500.0		
Fy		psi =	60,000.0	60,000.0		

Lic. # : KW-06007583

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

Footing Dimensions 8	Strengt	hs
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
fc = 2,500 psi Footing Concrete Density	Fy =	60,000 psi 150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00) @B	tm.= 3.00 ir

Footing Design Res	ults	1]
		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 Size	s & 3	Spacings	
Toe: Not reg'd, Mi	u < 5	S*Fr	

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

Summary of Overturning & Resisting Forces & Moments

562.5 -33.8	1.67 0.50	937.5 -16.9
-33.8	0.50	-16.9
-33.8	0.50	-16.9
528.8	0.T.M. =	920.6
		528.8 O.T.M. =

	Force Ibs	ESISTING Distance ft	Moment ft-lb
=	476.7	2.21	1,052.6
=			
=			
=			
=			
=			
=	55.0	0.50	27.5
=			
=	450.0	1.33	600.0
=			
=	412.5	1.38	567.2
=		2.50	
=			
=	1,394.2	bs R.M. =	2,247.3
		Force lbs = 476.7 = = = = = 55.0 = 450.0 = 412.5 = =	lbs ft = 476.7 2.21 = - - = - - = 55.0 0.50 = - - = 450.0 1.33 = 412.5 1.38 = 2.50 -

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

File: Retaining Wall Design (1500PSF):ec6 Software copyright ENERCALC, INC. 1983-2020, Build 12 20.5.31

BURT ENGINEERING PLLC

Lic. # : KW-06007583

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

File: Retaining Wall Design (1500PSF).ec6 Software copyright ENERCALC, INC. 1983-2020, Build:12:20:5:31 BURT ENGINEERING PLLC

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 act Lateral soil pressure optio NOT USED for Soil F NOT USED for Slidin NOT USED for Overt	ns: ressur g Resi:	stance.
Design Summary		
Wall Stability Ratios		
Overturning	=	2.42 OK
Sliding Slab Resists All Slidin	g !	1.10 OK
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 Soil Pressure Less	= Than A	1,500 psf
ACI Factored @ Toe	=	1,418 psf
ACI Factored @ Heel	=	107 psf
Footing Shear @ Toe	=	7.9 psi OK
Footing Shear @ Heel Allowable	-	15.6 psi OK 75.0 psi
Sliding Calcs Slab Resis		
Lateral Sliding Force	=	1,068.8 lbs
less 100% Passive Force	= .	156.3 lbs
less 100% Friction Force	= •	1,010.0 lbs
Added Force Reg'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

Soil Data			Calculations	per ACI 318-14,	ACI 530-11, IBC 2015,
Allow Soil Bearing =	1,500.0	psf			CBC 2016, ASCE 7-10
Equivalent Fluid Pressure Metho	bd				
Heel Active Pressure =	45.0	psf/ft			
Toe Active Pressure =					
Passive Pressure =	250.0	psf/ft			
Soil Density, Heel =	110.00	DCf			
Soil Density, Toe =					
riction Coeff btwn Ftg & Soil =		001			
Soil height to ignore for passive pressure =		ı			
Stem Construction		Top Stem	2nd		
Design Unight About D	-	Stem OK	Stem OK		
Design Height Above Fi Wall Material Above "H		6.50	0.00		
Thickness		Concrete 8.00	Concrete		
Rebar Size	in = =	# 4	8.00 # 4		
Rebar Spacing	in =	18.00	18.00		
Rebar Placed at	=	Edge	Edge		
Design Data		2090	2090		
fb/FB + fa/Fa	=	0.000	0.709		
Total Force @ Section	lbs =	0.0	1,290.0		
MomentActual	ft-I =	0.0	2,591.0		
MomentAllowable	ft-I =	3,655.6	3,655.6		
ShearActual	psi =	0.0	17.2		
ShearAllowable	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 footin	g in =	18.72	3.60		
Concrete Data					
fc	psi =	2,500.0	2,500.0		
Fy	psi =	60,000.0	60,000.0		

Lic. # : KW-06007583

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

Footing Dimensions &	Strength	IS
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
fc = 2,500 psi	Fy =	60,000 psi 150.00 pcf
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover@Top 2.00	@ Bt	m.= 3.00 ir

Footing Design Res	sults	i]
		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	=		
Other Acceptable Size			

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

Item		Force Ibs	VERTURNING 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

		Force Ibs	ESISTING 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	bs 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.

File: Retaining Wall Design (1500PSF).ec6 Software copyright ENERCALC, INC. 1983-2020, Build: 12.20.5.31 BURT ENGINEERING PLLC

Lic. # : KW-06007583

Load Factors Dead Load

Live Load

Earth, H

Wind, W

Seismic, E

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

1.200

1.600

1.600

1.600

1.000

Criteria Calculations per ACI 318-14, ACI 530-11, IBC 2015, Soil Data CBC 2016, ASCE 7-10 Retained Height 8.00 ft Allow Soil Bearing = = 1,500.0 psf Wall height above soil 0.50 ft Equivalent Fluid Pressure Method = Slope Behind Wall = 0.00:1Heel Active Pressure -45.0 psf/ft 6.00 in Height of Soil over Toe Toe Active Pressure = 30.0 psf/ft -Water height over heel = 0.0 ft Passive Pressure 250.0 psf/ft Vertical component of active Soil Density, Heel 110.00 pcf -Lateral soil pressure options: Soil Density, Toe 110.00 pcf = NOT USED for Soil Pressure. Friction Coeff btwn Ftg & Soil 0.400 = NOT USED for Sliding Resistance. Soil height to ignore NOT USED for Overturning Resistance. 12.00 in for passive pressure = Surcharge Loads Lateral Load Applied to Stem Adjacent Footing Load Lateral Load -0.0 plf Surcharge Over Heel = 135.0 Used To Resist Sliding & Overturning 135.0 psf 0.0 lbs Adjacent Footing Load 2 ...Height to Top 0.00 ft = Footing Width = 0.00 ft Surcharge Over Toe 0.0 psf 0.00 in 0.00 ft Eccentricity = ...Height to Bottom = Used for Sliding & Overturning Wall to Ftg CL Dist 0.00 ft = Footing Type Line Load Axial Load Applied to Stem Base Above/Below Soil = 0.0 ft Axial Dead Load 0.0 lbs = at Back of Wall Wind on Exposed Stem 0.0 psf Axial Live Load 0.0 lbs = = Poisson's Ratio 0.300 = Axial Load Eccentricity 0.0 in -Top Stem 2nd Stem Construction Design Summary Stem OK Stem OK Wall Stability Ratios Design Height Above Ftg ft = 8.50 0.00 Overturning -1.70 OK Wall Material Above "Ht" = Concrete Concrete 0.78 OK Sliding = Thickness in = 8.00 8.00 Slab Resists All Sliding Rebar Size # 4 # = 5 Total Bearing Load 4,078 lbs Rebar Spacing 18.00 8.00 = in = ...resultant ecc. 14.23 in = Rebar Placed at -Edge Edge Design Data Soil Pressure @ Toe 2,069 psf NG -0.000 0.760 fb/FB + fa/Fa -Soil Pressure @ Heel = 0 psf OK Total Force @ Section lbs = 0.0 3.004.9 Allowable = 1,0 Soil Pressure Exceeds Allowable 1,500 psf Moment....Actual ft-| = 0.0 8.970.6 Moment.....Allowable ft-l = 3.655.6 11.799.2 ACI Factored @ Toe = 2.483 psf Shear Actual psi = 40.5 ACI Factored @ Heel 0.0 = 0 psf Shear.....Allowable 75.0 75.0 psi = Footing Shear @ Toe = 22.9 psi OK 100.0 100.0 Wall Weight psf = Footing Shear @ Heel 29.7 psi OK = Rebar Depth 'd' in = 6.25 6.19 Allowable 75.0 psi = Lap splice if above in = 18.72 23.40 Sliding Calcs Slab Resists All Sliding ! Lap splice if below in = 18.72 4.67 Lateral Sliding Force 2.285.8 lbs = 4.67 Hook embed into footing in ≃ 18.72 less 100% Passive Force 156.3 lbs -Concrete Data less 100% Friction Force = 1,630.0 lbs 2.500.0 2.500.0 fc psi = 498.2 lbs NG Added Force Reg'd = Fy psi = 60.000.0 60.000.0 1,641.1 lbs NG for 1.5 : 1 Stability =

File: Retaining Wall Design (1500PSF).ec6

BURT ENGINEERING PLLC

Software copyright ENERCALC, INC. 1983-2020, Build 12:20:5:31

Lic. # : KW-06007583

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			
fc = 2,500 psi	Fy =	60,000 psi			
Footing Concrete Density	=	150.00 pcf			
Min. As %	=	0.0018			
Cover@Top 2.00	@ Bt	m.= 3.00 ir			

Footing Design Res	ults	;	
		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 Accentable Sizes	2 2 0	Snacinge	

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

Item		0 Force Ibs	VERTURNING Distance ft	 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	0.T.M. =	7,687.3
Resisting/Overturning Vertical Loads used		oil Pressure	= 4,078.3	1.70

		Force Ibs	ESISTING 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	=			
Stern 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	=			
Tota	=	4.078.3	bs 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.

File: Retaining Wall Design (1500PSF).ec6 Software copyright ENERCALC, INC. 1983-2020, Build:12.20.5.31 BURT ENGINEERING PLLC

Lic. # : KW-06007583

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

Ele: Relaining Wall Design (1500PSF).co6 Software copyright ENERCALC, INC. 1983-2028, Build: 12.20.5.31 BURT ENGINEERING PLLC

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 acti Lateral soil pressure optio NOT USED for Soil P NOT USED for Slidin NOT USED for Overti	ns: ressur g Resi	stance.
Design Summary		
Wall Stability Ratios Overturning Sliding	=	2.50 OK 0.89 OK
Slab Resists All Sliding		0.05 OK
Total Bearing Load resultant ecc.	=	5,584 lbs 8.41 in
Soil Pressure @ Toe Soil Pressure @ Heel	=	1,277 psf OK 318 psf OK 1,500 psf
Allowable Soil Pressure Less		Allowable
ACI Factored @ Toe ACI Factored @ Heel	=	1,532 psf 382 psf
Footing Shear @ Toe	=	21.2 psi OK
Footing Shear @ Heel	=	31.4 psi OK
Allowable	=	75.0 psi
Sliding Calcs Slab Resis		0
Lateral Sliding Force less 100% Passive Force		2,764.0 lbs 222.2 lbs
less 100% Friction Force	= -	2,230.0 lbs
Added Force Reg'd	=	308.1 lbs NG
for 1.5 : 1 Stability	=	1,690.0 lbs NG
Load Factors Dead Load Live Load Earth, H Wind, W		1.200 1.600 1.600 1.600
Seismic, E		1.000

Soil Data				Calculations	per ACI 318-14,		
Allow Soil Bearing	=	1,500.0	psf			CBC 2016	ASCE 7-10
Equivalent Fluid Pressure Me	thod						
Heel Active Pressure	=	45.0	osf/ft				
Toe Active Pressure	=	30.0					
Passive Pressure	=	250.0	osf/ft				
Soil Density, Heel	=	110.00	ocf				
Soil Density, Toe	=	110.00					
Friction Coeff btwn Ftg & Soil	=	0.400					
Soil height to ignore for passive pressure	=	12.00 in	1				
Stem Construction			Top Stem	2nd			
Design Height About	Eta	-	Stem OK	Stem OK			
Design Height Above		ft =	10.50	0.00			
Wall Material Above Thickness	нι	= in =	Concrete 8.00	Concrete 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	n	lbs =	0.0	3,594.0			
MomentActual		ft-I =	0.0	11,999.0			
MomentAllowable	1	ft-I =	3,655.6	13,297.3			
ShearActual		psi =	0.0	48.4			
ShearAllowable		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 foo	ting	in =	18.72	6.30			
Concrete Data			0 000 0	0 500 0			
fc		psi =	2,500.0	2,500.0			
Fy		psi =	60,000.0	60,000.0			

Lic. # : KW-06007583

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

Footing Dimensions &	Strengt	hs
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
fc = 2,500 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ B	tm.= 3.00 in

Footing Design Results			
		Toe	Heel
Factored Pressure	=	1,532	382 psf
Mu': Upward	\simeq	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 Assessable Circ	- 0 1		

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

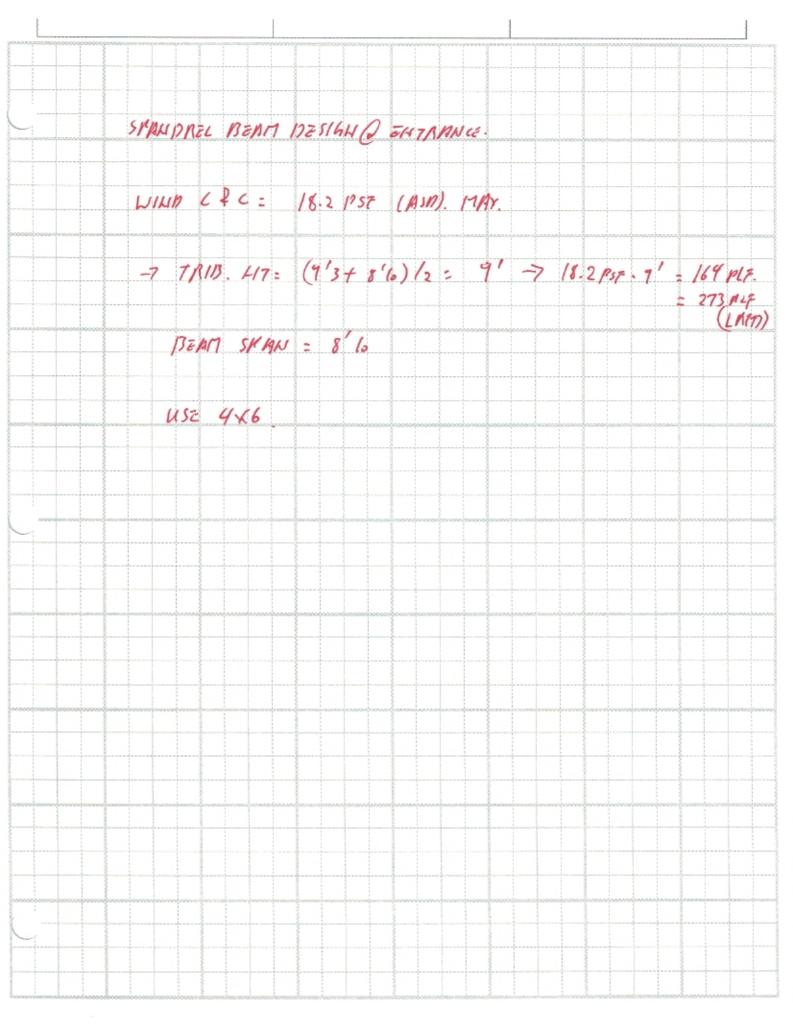
Item		O Force Ibs	VERTURNING Distance ft	Moment ft-lb
Heel Active Pressure	=	2,805.6	3.72	10,443.2
Surcharge over Heel	=			
Toe Active Pressure	=	-41.7	0.56	-23.1
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=			
Total	=	2,764.0	0.T.M. =	10,420.0
Resisting/Overturning Vertical Loads used				2.50 Ibs

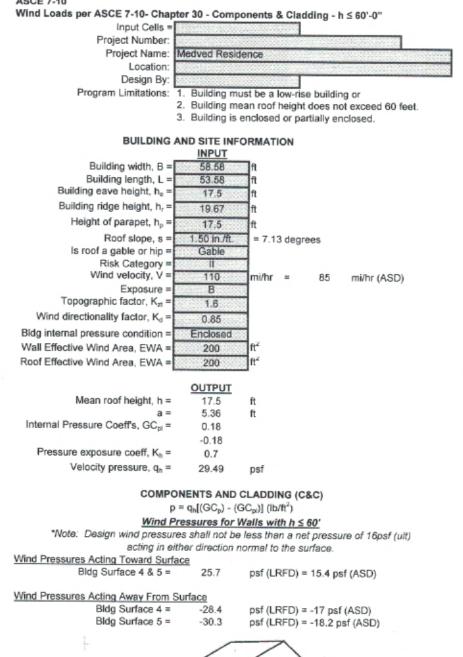
		Force Ibs	SISTING Distance ft	Moment ft-lb
Soil Over Heel	=	3,116.7	5.58	17,401.4
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
Axial Live Load on Stem	=			
Soil Over Toe	=	192.5	1.75	336.9
Surcharge Over Toe	=			
Stem Weight(s)	=	1,050.0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

File: Retaining Wall Design (1500PSF).ec6

Software copyright ENERCALC, INC. 1963-2020, Build 12 20.5.31 BURT ENGINEERING PLLC

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





4) 4)

ASCE 7-10

Wind Pressures for Roofs (gable roofs, & hip roofs) with $h \le 60'$

*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

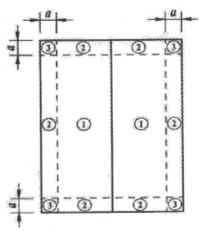
Roof Surface 1 =	-28.9	psf (LRFD) = -17.3 psf (ASD)
Roof Surface 2 =	-40.7	psf (LRFD) = -24.4 psf (ASD)
Roof Surface 3 =	-64.3	psf (LRFD) = -38.6 psf (ASD)

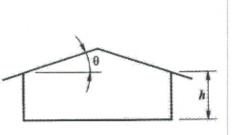
Wind Pressures Acting Away From Surface (Overhang) Roof Surface 1 = N.A. psf (L

Roof Surface 2 =

Roof Surface 3 =

N.A.	psf (LRFD)
-64.9	psf (LRFD) = -38.9 psf (ASD)
-73.7	psf (LRFD) = -44.2 psf (ASD)





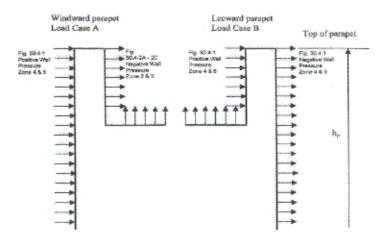
5. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with 0 ≤ 7°, the negative values of GC₂ in Zone 3 shall be equal to those for Zone 2 and positive values of GC₂ in Zones 2 and 3 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 30.4-1.

<u>Wind Pressures for Parapets (gable roofs, & hip roofs) with $h \le 60'$ </u> *Note: Design wind pressures shall not be less than a net pressure of 16psf (ult)

acting in either direction normal to the surface.

$p_n = 0$	-I(GC_) -	(GC)]	(lb/	ft ²)	

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



ood Beam		Software	copyright ENERCALC, INC. 198	
ESCRIPTION: Spandrel Beam at Entrance			BURI	ENGINEERING
CODE REFERENCES				
alculations per NDS 2018, IBC 2018, CBC 2019, A oad Combination Set : IBC 2018	SCE 7-16			
Material Properties				
Analysis Method : Allowable Stress Design Load Combination IBC 2018	Fb + Fb - Fc - Pril	900.0 psi 900.0 psi 1,350.0 psi	E : Modulus of Elastic Ebend- xx Eminbend - xx	city 1,600.0ksi 580.0ksi
Wood Species : Douglas Fir - Larch Wood Grade : No.2 Beam Bracing : Beam is Fully Braced against latera	Fc - Perp Fv Ft	625.0 psi 180.0 psi 575.0 psi	Density	31.210pcf
Beam Brasing . Beam for any Braded against later	al-torsional buckling			
\$	W(0.273)		÷ 13	-5
	5.50 X 3.50 🦯	4×6	1	2
	Span = 8.830 ft			

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 N	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 Deflec	tion	1.195 in Ratio	= 88>=60		
Max Upward Transient Deflection	1	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 St	ress Ratios								Mom	ent Values	;		Shear Va	lues
Segment Length	Span #	Μ	٧	Cd	C F/V	Ci	Cr	Сm	C t	CL_	М	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 Lic. # : KW-06007583

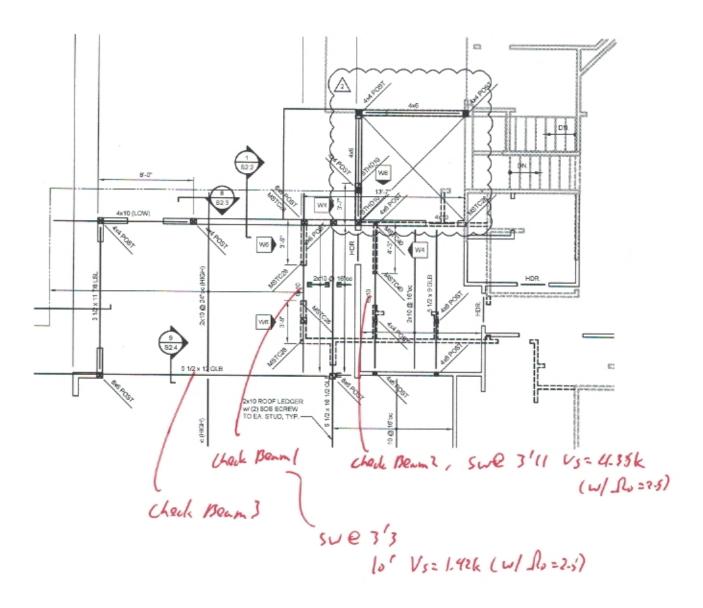
File: Medved Residence.ec6 Software copyright ENERCALC, INC. 1963-2020, Build:12.20.5.31 BURT ENGINEERING PLLC

DESCRIPTION: Spandrel Beam at Entrance

Load Combination		Max Stres	s Ratios								Mor	ment Values			Shear Va	lues
Segment Length	Span #	M	٧	Cd	C F/V	Ci	Cr	Cm	C t	cL _	М	fb	F'b	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	225.0
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	207.0
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	207.0
+D+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+D+0.750Lr+0.750L+0.4	50W+H				1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+D+0.750L+0.750S+0.4	50W+H				1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+D+0.750L+0.750S+0.5	250E+H				1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.0
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	288.0
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	288.0
Overall Maxim	um De	eflectio	ons													
Load Combination		1	Span	Max. "-"	Defi	Locatio	on in Spar	1	Load Co	mbinatio	n		Max. "+	Defl I	location in	Span
W Only			1	1.1	1946		4.447						0.0	0000	0.	000
Vertical React	tions						Sup	port no	tation : F	ar left is	#1		Values in F	IPS		
Load Combination					Support	rt1 S	upport 2									
Overall MAXimum					1.1	205	1.205									
Overall MINimum					1.3	205	1.205									
+D+0.60W+H					0.3	723	0.723									
+D+0.750Lr+0.750L+	0.450W++	1				542	0.542									
+D+0.750L+0.750S+6	0.450W+H					542	0.542									
+0.60D+0.60W+0.60H	н				0.1	723	0.723									
W Only						205	1.205									
H Only																

W Only H Only

SW DISCONTINUED ON BEAMS CHECK.



DESCRIPTION: Beam 1 with SW loading			BUK	T ENGINEERING P
CODE REFERENCES				
Calculations per NDS 2018, IBC 2018, CBC 2019, A .oad Combination Set : IBC 2018	SCE 7-16			
Material Properties				
Analysis Method : Allowable Stress Design Load Combination JBC 2018 Wood Species : DouglasFir-Larch Wood Grade : No.2	Fb + Fb - Fc - Prll Fc - Perp Fv	900.0 psi 900.0 psi 1,350.0 psi 625.0 psi 180.0 psi	E : Modulus of Elasti Ebend- xx Eminbend - xx	city 1,600.0ksi 580.0ksi
Beam Bracing : Beam is Fully Braced against latera	Et	575.0 psi	Density	31.210pcf
E(1,42)	D(0.045) S(0.075)		E(1,42)	
P P	D(0.045) L(0.12)		÷	¢
	6x10			
•	Span = 12.750 ft			
Applied Loads	Service	loads entered. Loa	ad Factors will be appl	lied for calculatio
learn self weight calculated and added to loads Uniform Load: D = 0.0150, L = 0.040 ksf, Tributary Width =				

				P1 11	Design OK
Maximum Bending Stress Ratio Section used for this span) =	0.732 1 6x10	Maximum Shear Stress Ratio Section used for this span	=	0.215 : 1 6x10
	=	1,054.37 psi		=	61.93 psi
	=	1,440.00psi		=	288.00 psi
Load Combination Location of maximum on span Span # where maximum occurs	+D+0.750L+0.750 = =)S+0.5250E+H 6.235ft Span # 1	Load Combination Location of maximum on span Span # where maximum occurs	+D+0.750L+0.7 = =	50S+0.5250E+H 11.959 ft Span # 1
Maximum Deflection Max Downward Transient Deflect Max Upward Transient Deflect Max Downward Total Deflection Max Upward Total Deflection	lion	0.221 in Ration 0.000 in Ration 0.352 in Ration 0.000 in Ration	p = 0 <360 p = 435 >= 240		

Maximum Forces & Stresses for Load Combinations

(

Load Combination		Max Stress	s Ratios								Mom	ent Values			Shear Va	alues
Segment Length	Span #	М	V	Cd	C F/V	Ci	Cr	Cm	C t	CL _	M	fb	F'b	V	fv	F'v
+D+H						The 's dd see							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	2.00	200100	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		002.00	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

Wood Beam

File: Medved Residence.ec6 Software copyright ENERCALC, INC. 1983-2020, Build 12 20.5.31 BURT ENGINEERING PLLC

Lic. # : KW-06007583

DESCRIPTION: Beam 1 with SW loading

Load Combination		Max Stres	s Ratios								Mor	nent Values			Shear Va	lues
Segment Length	Span #	М	V	Cd	C F/V	Ci	Cr	Cm	C t	CL	М	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		and the second second second	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+0.4	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+0.4	50W+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+0.5	250E+H				1.000	1.00	1.00	1.00	1.00	1.00	0.00	T LOTT L	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	1.101	1004101	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	1.24	170.10	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 Maxin		floatio	-									210.00				

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defi	Location in Span	Load Combination	Max. "+" Defl	Location in Spar
+D+0.750L+0.750S+0.5250E+H	1	0.3517	6.375		0.0000	0.000
Vertical Reactions			Suppo	ort notation : Far left is #1	Values in KIPS	
Load Combination		Suppor	t 1 Support 2			
Overall MAXimum		2.2	95 2.353			
Overall MINimum		1.3	64 1.476			
+D+H		0.6	46 0.646			
+D+L+H		1.4	11 1.411			
+D+Lr+H		0.6	46 0.646			
+D+S+H		1.1	24 1.124			
+D+0.750Lr+0.750L+H		1.2	20 1.220			
+D+0.750L+0.750S+H		1.5	78 1.578			
+D+0.60W+H		0.6	46 0.646			
+D+0.70E+H		1.6	01 1.679			
+D+0.750Lr+0.750L+0.450W+H		1.2	20 1.220			
+D+0.750L+0.750S+0.450W+H		1.5	78 1.578			
+D+0.750L+0.750S+0.5250E+H		2.2	95 2.353			
+0.60D+0.60W+0.60H		0.3	88 0.388			
+0.60D+0.70E+0.60H		1.3	43 1.421			
D Only		0.6	46 0.646			
L Only		0.7	65 0.765			
S Only		0.4	78 0.478			
E Only		1.3	64 1.476			
H Only						

Wood Beam			Coffusion	popyright ENERCALC, INC. 1983-	ved Residence.ec6
Lic. # : KW-06007583			Sorwaren		NGINEERING PLL
DESCRIPTION: Beam 2 with SW loading					
CODE REFERENCES					
Calculations per NDS 2018, IBC 2018, CBC Load Combination Set : IBC 2018	C 2019, ASCE 7-16				
Material Properties					
Analysis Method : Allowable Stress Design Load Combination JBC 2018		Fb + Fb -	900.0 psi 900.0 psi	E : Modulus of Elasticit Ebend- xx	y 1,600.0ksi
Wood Species : DouglasFir-Larch		Fc - Prll Fc - Perp	1,350.0 psi 625.0 psi	Eminbend - xx	580.0ksi
Wood Grade : No.2 Beam Bracing : Beam is Fully Braced aga	inst lateral-torsional bu	Fv Ft uckling	180.0 psi 575.0 psi	Density	31.210 pcf
+		s(0.075)		\$	\$
÷	D(0.03	3) L(0.08)		÷	*
	6	x10			
	0	10.000.0			· · · · · ·
	Span =				
•	opan	12.550 1			
Applied Loads			oads entered. Loa	ad Factors will be applie	d for calculations
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Trib Point Load : E = 4.350 k @ 3.920 ft	tary Width = 2.0 ft, (floor loa	Service	loads entered. Loa		
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Trib Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft	Service ad)	loads entered. Loa		esign OK
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10	Service ad) Maximum She		=	esign OK 0.264 : 1 6x10
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Trib Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio =	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10 1,385.70 psi	Service ad) Maximum She	ar Stress Ratio	= n =	esign OK 0.264 : 1 6x10 76.12 psi
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Trib Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = = Load Combination	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10 1,385.70psi 1,440.00psi +D+0.70E+H	Service ad) Maximum She Section u Load Comb	ar Stress Ratio sed for this spar	= n	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span =	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10 1,385.70psi 1,440.00psi	Service ad) Maximum She Section u Load Comb Location of	ar Stress Ratio sed for this spar ination maximum on span	n = = +D+0.750L+0.750S+ =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H 0.000 ft
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span = Span # where maximum occurs = Maximum Deflection	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10 1,385.70 psi 1,440.00 psi +D+0.70E+H 3.915ft Span # 1	Service ad) Maximum Shea Section u Load Comb Location of Span # whe	ar Stress Ratio sed for this span ination maximum on span are maximum occurs	n = = +D+0.750L+0.750S+ =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span = Span # where maximum occurs = Maximum Deflection Max Downward Transient Deflection	tary Width = 2.0 ft, (floor loa sutary Width = 3.0 ft 0.962 1 6x10 1,385.70 psi 1,440.00 psi +D+0.70E+H 3,915ft Span # 1 0.392 in Ratio	Service ad) Maximum Shea Section u Load Comb Location of Span # whe = 377 >=3(ar Stress Ratio sed for this span ination maximum on span are maximum occurs	n = = +D+0.750L+0.750S+ =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H 0.000 ft
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span = Span # where maximum occurs = Maximum Deflection	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 0.962 1 6x10 1,385.70 psi 1,440.00 psi +D+0.70E+H 3.915ft Span # 1	Service ad) Maximum Shea Section u Load Comb Location of Span # whe = 377 >=30 = 0 <360 = 396 >=24	ar Stress Ratio sed for this span ination maximum on span are maximum occurs 50	n = = +D+0.750L+0.750S+ =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H 0.000 ft
Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Trib Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span = Span # where maximum occurs = Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Downward Total Deflection	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 6x10 1,385.70psi 1,440.00psi +D+0.70E+H 3.915ft Span # 1 0.392 in Ratio 0.000 in Ratio 0.000 in Ratio	Service ad) Maximum Shea Section u Load Comb Location of Span # whe = 377 >=30 = 0 <360 = 396 >=24	ar Stress Ratio sed for this span ination maximum on span are maximum occurs 50	n = = +D+0.750L+0.750S+ =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H 0.000 ft
Beam self weight calculated and added to loads Uniform Load : D = 0.0150, L = 0.040 ksf, Tribu Uniform Load : D = 0.0150, S = 0.0250 ksf, Tribu Point Load : E = 4.350 k @ 3.920 ft DESIGN SUMMARY Maximum Bending Stress Ratio = Section used for this span = Load Combination Location of maximum on span = Span # where maximum occurs = Maximum Deflection Max Downward Transient Deflection Max Upward Transient Deflection Max Upward Total Deflection Max Stress Ratios	tary Width = 2.0 ft, (floor loa outary Width = 3.0 ft 6x10 1,385.70psi 1,440.00psi +D+0.70E+H 3.915ft Span # 1 0.392 in Ratio 0.000 in Ratio 0.000 in Ratio	Service ad) Maximum She Section u Load Comb Location of Span # whe = 377 >=3(= 0 <36(= 396 >=2/ = 0 <24(ar Stress Ratio sed for this span ination maximum on span are maximum occurs 50 0 10 10 10 10 10 10	D = = +D+0.750L+0.750S+ = s =	esign OK 0.264 : 1 6x10 76.12 psi 288.00 psi 0.5250E+H 0.000 ft

Segment Length	Span #	M	V	Cd	C F/V	CI	Cr	Сm	с _t	CL	M	fb	F'b	٧	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
D 011002 011000 11					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0

Wood Beam Lic. # : KW-06007583

File: Medved Residence.ec6 Software copyright ENERCALC, INC. 1983-2020, Build: 12 20:5 31 BURT ENGINEERING PLLC

DESCRIPTION: Beam 2 with SW loading

Load Combination		Max Stres	s Ratios								Mor	ment Values			Shear Va	alues
Segment Length	Span #	М	V	Cd	C FN	Ci	Cr	Cm	Ct	CL_	М	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		201.00	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	0100	1000110	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+0.45	50W+H				1.000	1.00	1.00	1.00	1.00	1.00	0.7 1	010.10	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+0.45	OW+H				1.000	1.00	1.00	1.00	1.00	1.00	2.10	400.04	0.00	0.00	0.00	200.00
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+0.52	250E+H				1.000	1.00	1.00	1.00	1.00	1.00	0.00	330.40	0.00	0.00	0.00	200.00
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-0.52	50E+H				1.000	1.00	1.00	1.00	1.00	1.00	0.44	1,000.00	0.00	0.00	0.00	200.00
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		
+0.60D+0.60W+0.60H		01210	0.111		1.000	1.00	1.00	1.00	1.00	1.00	2.70	400.00	0.00		33.67	288.00
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		0.00	0.00	0.00
+0.60D+0.70E+0.60H		0.000	0.020	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.90	142.17	1440.00	0.28	8.03	288.00
Length = 12.330 ft	1	0.905	0.235	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.00	4 000 40	0.00	0.00	0.00	0.00
+0.60D-0.70E+0.60H		0.000	0.200	1.00	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
Length = 12.330 ft	1	0.733	0.195	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.00	1.055.00	0.00	0.00	0.00	0.00
Overall Maxim				1.00	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 Lo	cation in Span	Load Combination	Max. "+" Defl	Location in Spa
E Only	1	0.3920	5.625		0.0000	0.000
Vertical Reactions			Suppo	rt 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						

Nood Beam					Software o	File: M popyright ENERCALC, INC. 19	edved Residence.ec6 33-2020 Build 12 20 5 31
ic. # : KW-06007583							FENGINEERING PLL
DESCRIPTION: Beam 3 with SW	loading						
CODE REFERENCES							
alculations per NDS 2018, IBC oad Combination Set : IBC 2018 Material Properties		2019, ASCE 7-1	6				
Analysis Method : Allowable Stres Load Combination IBC 2018	s Design		Fb + Fb - Fc -		2,400.0 psi 1,850.0 psi 1,650.0 psi	E : Modulus of Elasti Ebend- xx Eminbend - xx	<i>city</i> 1,800.0 ksi 950.0 ksi
Wood Species : DF/DF Wood Grade : 24F - V4				Регр	650.0 psi 265.0 psi 1,100.0 psi	Ebend- yy Eminbend - yy Density	1,600.0ksi 850.0ksi 31.210pcf
Beam Bracing : Beam is Fully B	lraced agai	inst lateral-torsion		9	1,100.0 pai	Density	51.210 pci
D(0.646) L(0.765) S(0.478) E(1	1.5)						
6	9	D(0.	1875) S(0.3	3125)		\$	\$
6			5.5x12				
A			5.5X12				
4		S	span = 19.0	ft			
							1
				· · · ·	loade entered 1 o	ad Factors will be app	lied for calculations
Applied Loads				Service	ioaus entered. Lo		neu tor carculations
	250 ksf, Trib			Service	loads entered. Lo		
Beam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY	0250 ksf, Trib 0, S = 0.4780	0, E = 1.50 k @ 2.50	ft, (beam 1)				Design OK
eam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY laximum Bending Stress Ratio	250 ksf, Trib	0, E = 1.50 k @ 2.50 0.811: 1	ft, (beam 1) Maxin	num She	ear Stress Ratio	=	Design OK 0.400 : 1
earn self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY	0250 ksf, Trib 0, S = 0.4780	0, E = 1.50 k @ 2.50 0.811: 1 5.5x12	ft, (beam 1) Maxin	num She		=	Design OK 0.400 : 1 5.5x12
eam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY aximum Bending Stress Ratio	0250 ksf, Trib 0, S = 0.4780 =	0, E = 1.50 k @ 2.50 0.811: 1	ft, (beam 1) Maxin	num She	ear Stress Ratio	= n	Design OK 0.400 : 1
ieam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY laximum Bending Stress Ratio Section used for this span	0250 ksf, Trib 0, S = 0.4780 = =	0.811: 1 5.5x12 2,239.48psi 2,760.00psi +D+S+H	ft, (beam 1) Maxin	num She	ear Stress Ratio used for this spa	= n =	Design OK 0.400 : 1 5.5x12 121.88 psi
Beam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY laximum Bending Stress Ratio Section used for this span Load Combination Location of maximum on span	0250 ksf, Trib 0, S = 0.4780 = =	0, E = 1.50 k @ 2.50 0.811: 1 5.5x12 2,239.48 psi 2,760.00 psi +D+S+H 9.223ft	ft, (beam 1) Maxin	num She Section (Load Comi	ear Stress Ratio used for this spa bination f maximum on span	= n = =	Design OK 0.400 : 1 5.5x12 121.88 psi 304.75 psi +D+S+H 0.000 ft
keam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY laximum Bending Stress Ratio Section used for this span Load Combination Location of maximum on span Span # where maximum occurs)250 ksf, Trib 0, S = 0.4780 = = = =	0.811: 1 5.5x12 2,239.48psi 2,760.00psi +D+S+H	ft, (beam 1) Maxin	num She Section (Load Comi	ear Stress Ratio used for this spa bination	= n = =	Design OK 0.400 : 1 5.5x12 121.88 psi 304.75 psi +D+S+H
Beam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY laximum Bending Stress Ratio Section used for this span Load Combination Location of maximum on span)250 ksf, Trib 0, S = 0.4780 = = = = =	0, E = 1.50 k @ 2.50 0.811: 1 5.5x12 2,239.48 psi 2,760.00 psi +D+S+H 9.223 ft Span # 1	ft, (beam 1) Maxin	num She Section (Load Comi Location of Span # wh	ear Stress Ratio used for this spa bination f maximum on span ere maximum occur	= n = =	Design OK 0.400 : 1 5.5x12 121.88 psi 304.75 psi +D+S+H 0.000 ft
Beam self weight calculated and added Uniform Load : D = 0.0150, S = 0.0 Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY Maximum Bending Stress Ratio Section used for this span Load Combination Location of maximum on span Span # where maximum occurs Maximum Deflection)250 ksf, Trib 0, S = 0.4780 = = = = = ction	0, E = 1.50 k @ 2.50 0.811: 1 5.5x12 2,239.48 psi 2,760.00 psi +D+S+H 9.223ft	ft, (beam 1) Maxin 1 1 2 Ratio =	num She Section (Load Comi	ear Stress Ratio used for this spa bination f maximum on span ere maximum occur 240	= n = =	Design OK 0.400 : 1 5.5x12 121.88 psi 304.75 psi +D+S+H 0.000 ft
Point Load : D = 0.6460, L = 0.7650 DESIGN SUMMARY Maximum Bending Stress Ratio Section used for this span Load Combination Location of maximum on span Span # where maximum occurs Maximum Deflection Max Downward Transient Deflect)250 ksf, Trib 0, S = 0.4780 = = = = = = ction	0, E = 1.50 k @ 2.50 0.811: 1 5.5x12 2,239.48 psi 2,760.00 psi +D+S+H 9.223 ft Span # 1 0.679 in	ft, (beam 1) Maxin S I I Ratio = Ratio =	num She Section o Load Comi Location o Span # wh 335 >=2	ear Stress Ratio used for this spa bination f maximum on span ere maximum occur 240	= n = =	Design OK 0.400 : 1 5.5x12 121.88 psi 304.75 psi +D+S+H 0.000 ft

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress	s Ratios								Mor	nent Values			Shear Va	lues
Segment Length	Span #	M	V	Cd	C F/V	Ci	Cr	Сm	C t	CL_	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

Wood Beam

File: Medved Residence.cc6 Software capyright ENERCALC, INC. 1983-2020, Build: 12,20,5,31 BURT ENGINEERING PLLC

DESCRIPTION: Beam 3 with SW loading

Load Combination		Max Stres	s Ratios								Mor	ment Values			Shear Va	lues
Segment Length	Span #	M	٧	Сd	C F/V	Ci	Cr	Cm	C t	CL	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+0.4	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+0.5	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 Maxim	num De	flectio	ns													

Load Combination Max. "-" Defl Load Combination Span Location in Span Max. "+" Defl Location in Span +D+S+H 1 1.1398 9.431 0.0000 0.000 Support notation : Far left is #1 Vertical Reactions 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 PUCKON NESTAN. -> FOR HOLDOWAI Q GINIB (6.) HIDH 4 (9) -7 RERWIREN TENSION = 110 #1. D. = 2.5 = 280 \$1. PLEPSE LISTE TENSION NEMAND IS VERY SMALL PLE TO LONG SW LENGTH PNP LOW SOISMIK EARCE. -7 MECK SIMPSON SET-XA. 41 5/8 9 BULT CEMBER 9"].

SIMPSON

Strong-I

Anchor Designer™ Software Version 2.9.7376.1

Company:	Date:	7/28/2020
Engineer:	Page:	1/5
Project:		
Address:		
Phone:		
E-mail:		

1.Project information

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

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

Recommended Anchor

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



Project description: Location: Fastening description:

Base Material

Concrete: Normal-weight Concrete thickness, h (inch): 24.00 State: Cracked Compressive strength, f° (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

SIMPSON

Strong-Tie

Anchor Designer™ Software Version 2.9.7376.1

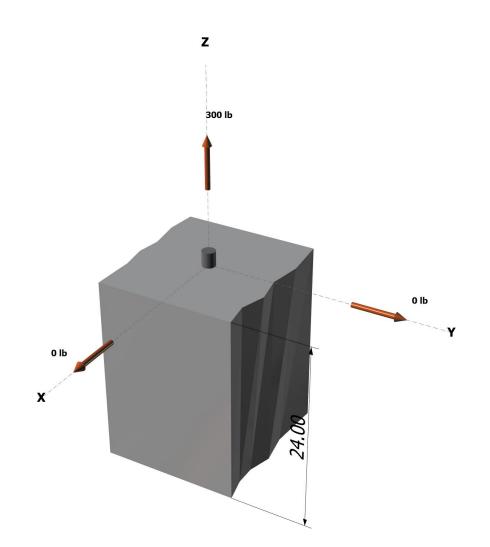
Company:	Date:	7/28/2020
Engineer:	Page:	2/5
Project:		
Address:		
Phone:		
E-mail:		

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
Vuax	[lb]	0
Vuay	[lb]:	0



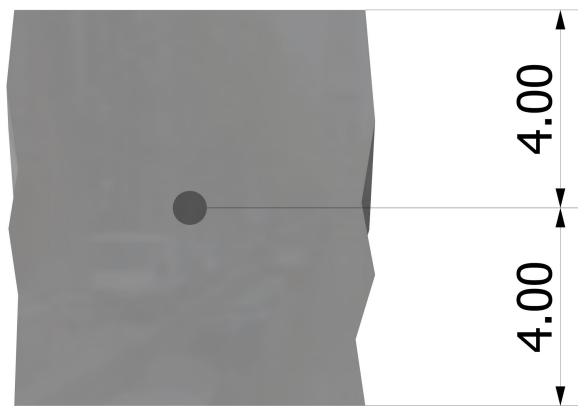




Anchor Designer™ Software Version 2.9.7376.1

Company:	Da	te:	7/28/2020
Engineer:	Pa	ge:	3/5
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



Anchor Designer™	Company:	Date:	7/28/2020
	Engineer:	Page:	4/5
ong-Tie Software	Project:		
Version 2.9.7376.1	Address:		
U.	Phone:		
	E-mail:		

3. Resulting Anchor Forces

SI

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)

Nb = Kcla√t c	h _{ef} ^{1.5} (Eq. 17.4.2	.2a)						
Kc	λa	f' _c (psi)	<i>h_{ef}</i> (in)	N _b (lb)				
17.0	1.00	2500	9.000	22950				
$\phi N_{cb} = \phi \left(A_{N} \right)$	c / A _{Nco}) Ψ _{ed,N} Ψ _{c,N}	𝖓 _{cp,N} N _b (Sec. 1	7.3.1 & 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
$\tau_{k,cr} = \tau_{k,cr} f_{show}$		K		(N				
$\tau_{k,cr} = \tau_{k,cr} f_{show}$	rt-termKsat							
τ _{k,cr} (psi)	f _{short-term}	Ksat		τ _{k,cr} (psi)				
<i>τ_{k,cr}</i> (psi) 435	1.00	1.0		<i>τ_{k,cr}</i> (psi) 435				
435		1.0		. ,				
435 $N_{ba} = \lambda_{a} \tau_{cr} \pi c$	1.00	1.0		. ,				
$\frac{435}{N_{ba}} = \lambda_{a} \tau_{cr} \pi \sigma_{cr}$ λ_{a}	1.00 d _a h _{ef} (Eq. 17.4.5.	2)	0	435				
$\frac{435}{N_{ba}} = \lambda_{a} \tau_{cr} \pi \sigma_{cr}$ λ_{a} 1.00	1.00 d _a h _{ef} (Eq. 17.4.5. τ _{cr} (psi)	1.0 2) <i>d_a</i> (in) 0.63	0 <i>h_{ef}</i> (in) 9.000	435 <i>N_{ba}</i> (lb) 7687				
$\frac{435}{N_{ba}} = \lambda_{a} \tau_{cr} \pi \sigma_{cr}$ λ_{a} 1.00	1.00 d _a h _{ef} (Eq. 17.4.5. <i>τ_{cr}</i> (psi) 435	1.0 2) <i>d_a</i> (in) 0.63	0 <i>h_{ef}</i> (in) 9.000	435 <i>N_{ba}</i> (lb) 7687	 Ψ _{ср.Na}	N _{ba} (lb)	φ	<i>∳N₅</i> (Ib)
$\frac{435}{N_{ba}} = \lambda_{a} \tau_{cr} \pi \alpha$ $\frac{\lambda_{a}}{1.00}$ $\phi N_{a} = \phi (A_{Na})$	1.00 d _a h _{ef} (Eq. 17.4.5. <i>τ_{cr}</i> (psi) 435 / Α _{Na0}) Ψ _{ed,Na} Ψ _{cp} ,	1.0 2) da (in) 0.63 NaNba (Sec. 17.3	0 <i>h_{ef}</i> (in) 9.000 8.1 & Eq. 17.4.	435 <i>N_{ba}</i> (lb) 7687 5.1a)	<u> </u>	<i>N_{ba}</i> (lb) 7687	φ 0.55	<i>∳N₂</i> (Ib) 2469
435 $N_{ba} = \lambda_{a} \tau_{cr} \pi c$ λ_{a} 1.00 $\phi N_{a} = \phi (A_{Na})$ $A_{Na} (in^{2})$ 98.16	1.00 d _a h _{ef} (Eq. 17.4.5. τ _{cr} (psi) 435 / Α _{Na0} Ψ _{ed,Na} Ψ _{cp,} Α _{Na0} (in ²)	1.0 2) <u>da (in)</u> 0.63 NaNba (Sec. 17.3 <u>CNa (in)</u> 6.14	0 h _{ef} (in) 9.000 3.1 & Eq. 17.4. c _{a,min} (in)	435 N _{ba} (lb) 7687 5.1a) <i>Ψ_{ed,Na}</i>		. ,		, , ,
435 $N_{ba} = \lambda_{a} \tau_{cr} \pi \alpha$ λ_{a} 1.00 $\phi N_{a} = \phi (A_{Na}$ $A_{Na} (in^{2})$ 98.16	1.00 d _a h _{ef} (Eq. 17.4.5. τ _{cr} (psi) 435 / Α _{Νa0} (Ψ _{ed,Na} Ψ _{cp,} Α _{Νa0} (in ²) 150.57	1.0 2) <u>da (in)</u> 0.63 NaNba (Sec. 17.3 <u>CNa (in)</u> 6.14	0 h _{ef} (in) 9.000 3.1 & Eq. 17.4. c _{a,min} (in)	435 N _{ba} (lb) 7687 5.1a) <i>Ψ_{ed,Na}</i>		. ,		, , ,



Anchor Designer™ Software Version 2.9.7376.1

Company:	Date:	7/28/2020
Engineer:	Page:	5/5
Project:	-	
Address:		
Phone:		
E-mail:		

11. Results

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

Tension	Factored Load, Nua (Ib)	Design Strength, øNn (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)

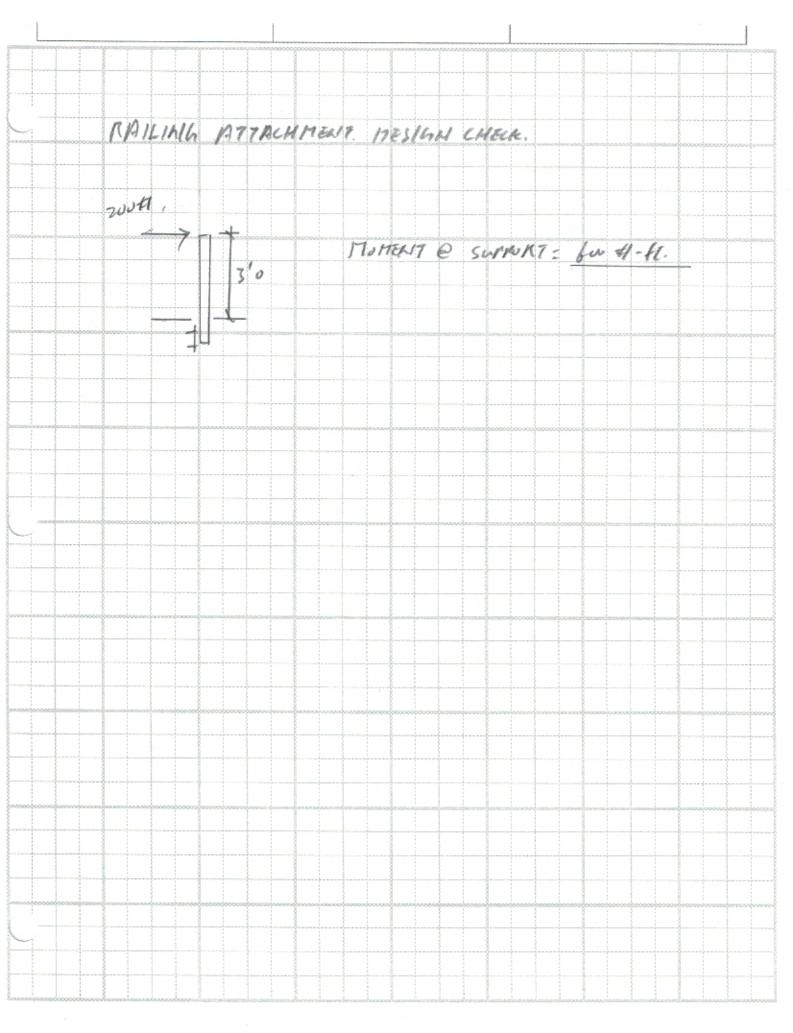
SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 9.000 inch meets the selected design criteria.

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.



SIMPSON

Strong-Tie

Anchor Designer™ Software Version 2.9.7376.2

Company:	Date:	8/19/2020
Engineer:	Page:	1/5
Project:		
Address:		
Phone:		
E-mail:		

1.Project information

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

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, hef (inch): 2.190 Code report: ICC-ES ESR-2713 Anchor category: 1 Anchor ductility: No hmin (inch): 4.67 c_{ac} (inch): 3.31 C_{min} (inch): 1.75 S_{min} (inch): 3.00

Recommended Anchor

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



Project description: Location: Fastening description:

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

SIMPSON

Strong-Tie

Anchor Designer™ Software Version 2.9.7376.2

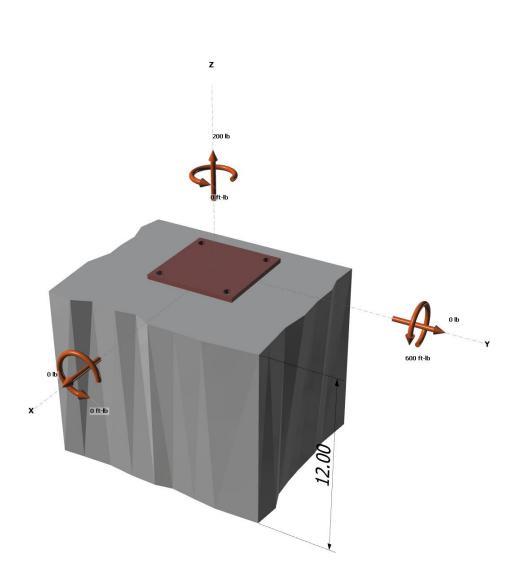
Company:	Date:	8/19/2020
Engineer:	Page:	2/5
Project:		-
Address:		
Phone:		
E-mail:		

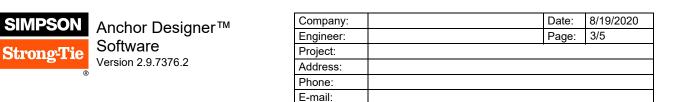
Load and Geometry Load factor source: ACI 318 Section 5.3 Load combination: not set Seismic design: No Anchors subjected to sustained tension: Not applicable Apply entire shear load at front row: No Anchors only resisting wind and/or seismic loads: No

Strength level loads:

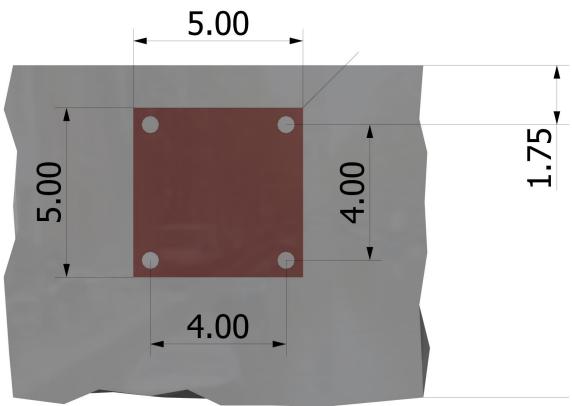
N_{ua} [lb]: 200 V_{uax} [lb]: 0 V_{uay} [lb]: 0 M_{ux} [ft-lb]: 0 M_{uy} [ft-lb]: 600 Muz [ft-lb]: 0

<Figure 1>





<Figure 2>



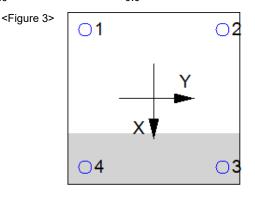
SON	Anchor Designer™ Software Version 2.9.7376.2	Company:	Date:	8/19/2020
		Engineer:	Page:	4/5
g-Tie		Project:		
		Address:		
w.		Phone:		
		E-mail:		

3. Resulting Anchor Forces

MIPOT

Anchor	Tension load, N _{ua} (lb)	Shear load x, V _{uax} (lb)	Shear load y, V _{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (Ib)
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



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)

$N_b = k_c \lambda_a \sqrt{f}$	<i>che</i> f ^{1.5} (Eq. 17.	4.2.2a)							
Kc	λa	<i>f'c</i> (psi)	<i>h</i> ef (in)	Nb (I	b)				
17.0	1.00	2500	2.190	275	5				
$\phi N_{cbg} = \phi (A)$	Nc / ANco) Yec,N	$\mathcal{Y}_{ed,N} \mathcal{Y}_{c,N} \mathcal{Y}_{cp,N} \mathcal{N}_{b}$	(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
	-	Anchor in Tens 0) ⁿ (Sec. 17.3.4	•		ort)				
$\Psi_{c,P}$	λa	N _p (lb)	f′₀ (psi)	n		ϕ	$\phi N_{ m pn}$ (Ib)		
1.0	1.00	2212	2500	0.50		0.65	1438	_	



Anchor Designer™ Software Version 2.9.7376.2

Company:	Date:	8/19/2020
Engineer:	Page:	5/5
Project:		
Address:		
Phone:		
E-mail:		

<u>11. Results</u>

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

Tension	Factored Load, N _{ua} (Ib)	Design Strength, øNn (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.