McGraw Structural Engineering, LLC
1118 Enstad Ln
Silverton, OR
503.884.2178

JOB TITLE	Alev Residence		
	Garage Structural Repair		
JOB NO.	22-108	**SHEET NO.**	
CALCULATED BY	RDM	**DATE**	5/4/23
CHECKED BY	RDM	**DATE**	5/5/23

CS2021 Ver 2022-04-02 www.struware.com

STRUCTURAL CALCULATIONS

FOR

Alev Residence

6848 SE 33rd St, Mercer Island, WA

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Code Search

Code: International Building Code 2021

Occupancy:

Occupancy Group = R Residential

Risk Category & Importance Factors:

Risk Category = II

Wind factor = 1.00
Snow factor = 1.00
Seismic factor = 1.00

Type of Construction:

Fire Rating:

Roof = 0.0 hr
Floor = 0.0 hr

Building Geometry:

Roof angle (θ)	0.13 / 12	0.6 deg
Building length	30.0 ft	
Least width	20.0 ft	
Mean Roof Ht (h)	8.5 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	

Live Loads:

Roof
0 to 200 sf: 20 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12 psf

Decks (1.5 times live load) 60 psf

Floor:

Typical Floor	40 psf
Partitions	N/A
Partitions	N/A
Partitions	N/A
Partitions	N/A
Storage areas above ceilings	20 psf

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Wind Loads :

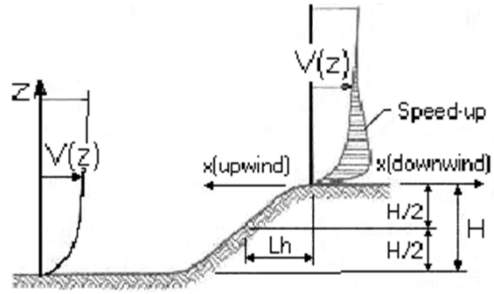
ASCE 7- 16

Ultimate Wind Speed	98 mph
Nominal Wind Speed	75.9 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.849
Kh case 2	0.849
Type of roof	Monoslope

Topographic Factor (Kzt)

Topography	3D Axisym Hill
Hill Height (H)	150.0 ft
Half Hill Length (Lh)	820.0 ft
Actual H/Lh =	0.18
Use H/Lh =	0.00
Modified Lh =	820.0 ft
From top of crest: x =	0.0 ft
Bldg up/down wind?	downwind

H/Lh<0.2
∴ Kzt=1.0



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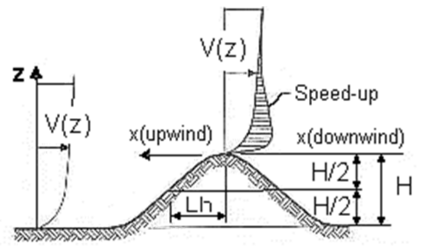

2D RIDGE or 3D AXISYMMETRICAL HILL

H/Lh= 0.00	K_1 = 0.000
x/Lh = 0.00	K_2 = 1.000
z/Lh = 0.02	K_3 = 0.929

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^2 = 1.00$

Gust Effect Factor

h =	8.5 ft
B =	20.0 ft
/z (0.6h) =	15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.43 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	427.1 ft
Q =	0.95
I_z =	0.23
G =	**0.90** use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	1.0 Hz				
Damping ratio (β) =	5				
/b =	0.65				
/α =	0.15				
Vz =	82.8				
N_1 =	5.16				
R_n =	0.050				
R_h =	0.747	η =	0.472	h =	.5 ft
R_B =	0.539	η =	1.112		
R_L =	0.163	η =	5.583		
g_R =	4.189				
R =	0.049				
Gf =	0.898				

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Enclosure Classification

Test for Enclosed Building: Ao < 0.01Ag or 4 sf, whichever is smaller

Test for Open Building: All walls are at least 80% open.
Ao ≥ 0.8Ag

Test for Partially Enclosed Building: Predominately open on one side only

	Input			Test	
Ao	112.0	sf	Ao ≥ 1.1Aoi	NO	
Ag	200.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	153.0	sf	Aoi / Agi ≤ 0.20	YES	Building is NOT Partially Enclosed
Agi	1280.0	sf			

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
- Ao ≥ 1.1Aoi
- Ao > smaller of 4' or 0.01 Ag
- Aoi / Agi ≤ 0.20

Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Test for Partially Open Building: A building that does not qualify as open, enclosed or partially enclosed.
(This type building will have same wind pressures as an enclosed building.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog):		0 sf
Unpartitioned internal volume (Vi) :		0 cf
	Ri =	1.00

Ground Elevation Factor (Ke)

Grd level above sea level =	0.0 ft			Ke =	1.0000
Constant =	0.00256	Adj Constant =	0.00256		

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Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) =	0.85	Edge Strip (a) =	3.0 ft
Base pressure (qh) =	**17.7 psf**	End Zone (2a) =	6.0 ft
GCpi =	+/-0.18	Zone 2 length =	10.0 ft

Wind Pressure Coefficients

	CASE A			CASE B		
		θ = 0.6 deg				
Surface	GCpf	w/-GCpi	w/+GCpi	GCpf	w/-GCpi	w/+GCpi
1	0.40	0.58	0.22	-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87
3	-0.37	-0.19	-0.55	-0.37	-0.19	-0.55
4	-0.29	-0.11	-0.47	-0.45	-0.27	-0.63
5				0.40	0.58	0.22
6				-0.29	-0.11	-0.47
1E	0.61	0.79	0.43	-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25
3E	-0.53	-0.35	-0.71	-0.53	-0.35	-0.71
4E	-0.43	-0.25	-0.61	-0.48	-0.30	-0.66
5E				0.61	0.79	0.43
6E				-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

Surface	Case A w/-GCpi	Case A w/+GCpi	Case B w/-GCpi	Case B w/+GCpi
1	10.3	3.9	-4.8	-11.2
2	-9.0	-15.4	-9.0	-15.4
3	-3.4	-9.8	-3.4	-9.8
4	-2.0	-8.3	-4.8	-11.2
5			10.3	3.9
6			-2.0	-8.3
1E	14.0	7.6	-5.3	-11.7
2E	-15.8	-22.2	-15.8	-22.2
3E	-6.2	-12.6	-6.2	-12.6
4E	-4.4	-10.8	-5.3	-11.7
5E			14.0	7.6
6E			-4.4	-10.8

Parapet

Windward parapet = 0.0 psf (GCpn = +1.5)
Leeward parapet = 0.0 psf (GCpn = -1.0)

Windward roof overhangs = 12.4 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

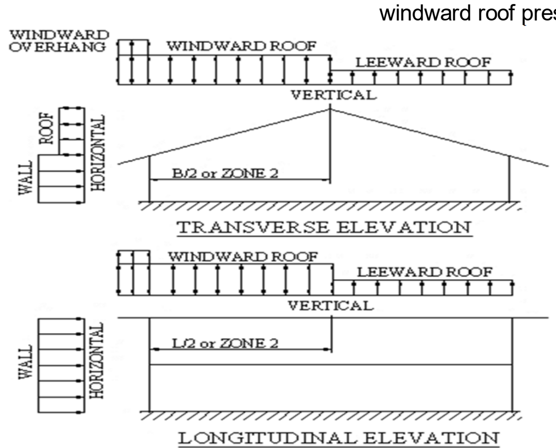
Interior Zone: Wall	12.2 psf	
Roof	-5.7 psf	**
End Zone: Wall	18.4 psf	
Roof	-9.6 psf	**

Longitudinal direction (parallel to L)

Interior Zone: Wall	12.2 psf
End Zone: Wall	18.4 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



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Snow Loads : ASCE 7- 16

Nominal Snow Forces

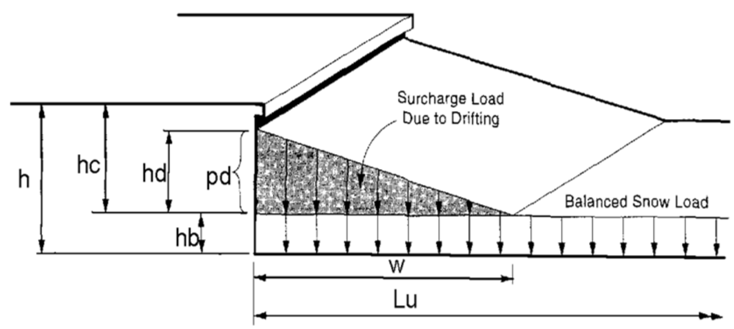
Roof slope		=	0.6 deg
Horiz. eave to ridge dist (W)		=	20.0 ft
Roof length parallel to ridge (L)		=	30.0 ft
Type of Roof			Monoslope
Ground Snow Load	Pg	=	25.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	17.5 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**17.5 psf**
Rain on Snow Surcharge Angle			0.40 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	17.5 psf
Minimum Snow Load	Pm	=	20.0 psf
Uniform Roof Design Snow Load		=	**20.0 psf** use 25.0

Near ground level surface balanced snow load = **25.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Windward Snow Drifts 1 - Against walls, parapets, etc

Up or downwind fetch	lu	=	65.0 ft
Projection height	h	=	12.0 ft
Projection width/length	lp	=	20.0 ft
Snow density	g	=	17.3 pcf
Balanced snow height	hb	=	1.01 ft
	hd	=	2.03 ft
	hc	=	10.99 ft
hc/hb >0.2 = 10.8	**Therefore, design for drift**		
Drift height (hd)		=	2.03 ft
Drift width	w	=	8.12 ft
Surcharge load:	pd = γ*hd	=	**35.0 psf**
Balanced Snow load:		=	17.5 psf
			52.5 psf



Windward Snow Drifts 2 - Against walls, parapets, etc

Up or downwind fetch	lu	=	0.0 ft
Projection height	h	=	0.0 ft
Projection width/length	lp	=	0.0 ft
Snow density	g	=	17.3 pcf
Balanced snow height	hb	=	1.01 ft
	hd	=	1.00 ft
	hc	=	-1.01 ft
hc/hb <0.2 = -1.0	**lp <15', drift not req'd**		
Drift height (hc)		=	0.00 ft
Drift width	w	=	-8.12 ft
Surcharge load:	pd = γ*hd	=	**0.0 psf**
Balanced Snow load:		=	17.5 psf
			17.5 psf

Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

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Seismic Loads: IBC 2021 — Strength Level Forces

Risk Category : II
Importance Factor (Ie) : 1.00
Site Class : D - code default

Ss (0.2 sec) = 141.00 %g
S1 (1.0 sec) = 49.10 %g

A site specific ground motion analysis is required for seismically isolated structures or with damping systems, see ASCE7 11.4.8
Site specific ground motion analysis performed:

Fa =	1.200	Sms =	1.692	S_{DS} =	1.128	Design Category =	D
Fv =	1.809	Sm1 =	0.888	S_{D1} =	0.592	Design Category =	D

Seismic Design Category = **D**
Redundancy Coefficient ρ = 1.00 Error p should be 1.3 due to plan irregularity
Number of Stories: 1
Structure Type: Light Frame
Horizontal Struct Irregularities: 1b) Extreme Torsional Irregularity — See ASCE7 Sect 12.3.3.4 & 12.8.4.3
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Bearing Wall Systems**
Seismic resisting system: **Light frame (wood) walls with structural wood shear panels**
System Structural Height Limit: **65 ft**
Actual Structural Height (hn) = 8.5 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 6.5
Over-Strength Factor (Ωo) = 3
Deflection Amplification Factor (Cd) = 4
S_{DS} = 1.128
S_{D1} = 0.592

Seismic Load Effect (E) = Eh +/-Ev = $\rho\, Q_E \pm 0.2 S_{DS} D$ = Qe +/- 0.226D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E \pm 0.2 S_{DS} D$ = 3Qe +/- 0.226D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) = 0.020 Cu = 1.40
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.100 sec x= 0.75 Tmax = CuTa = 0.139 sec
User calculated fundamental period = 0.100 s T = 0.100 sec
Long Period Transition Period (TL) = ASCE7 map = 16 sec

Seismic response coef. (Cs) = SdsI/R = 0.174 ASCE7 11.4.8 exception 2 equations used
but not less than Cs = 0.044SdsI = 0.050
USE Cs = 0.174

Design Base Shear V = 0.174W
See ASCE7 Sect 12.3.3.4 for 25% connection increase

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

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Roof Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Roofing	Single ply	x 1.0	1.0	0.7
Decking	5/8" plywood/OSB	x 1.0	2.2	1.8
Decking	5/8" plywood/OSB	x 1.0	2.2	1.8
Misc.	Misc.	x 1.0	0.5	0.0
		x 1.0	0.0	0.0
Framing	Wood 2x @24"	x 1.5	3.8	2.3
		x 1.0	0.0	0.0
		x 6.0	0.0	0.0
			0.0	0.0
	Actual Dead Load	○	9.7	○ 6.6
	Use this DL instead	●	10.0	● 10.0
	Live Load		20.0	0.0
	Snow Load		25.0	0.0
	Ultimate Wind (zone 2 - 100sf)		16.0	-19.2
ASD Loading	D + S		35.0	-
	D + 0.75(0.6*W + S)		36.0	-
	0.6*D + 0.6*W		-	-5.5
LRFD Loading	1.2D + 1.6 S + 0.5W		60.0	-
	1.2D + 1.0W + 0.5S		40.5	-
	0.9D + 1.0W		-	-10.2

Roof Live Load Reduction Roof angle 0.13 / 12 0.6 deg

0 to 200 sf: 20.0 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12.0 psf

	300 sf	18.0 psf
	400 sf	16.0 psf
	500 sf	14.0 psf
User Input:	450 sf	15.0 psf

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Wall Design Load #1

Items	Description	Multiple	psf (max)	psf (min)
Insulation	Rock Wool per 1" thk	x 5.50	1.1	1.1
Sheathing	1/2" gypsum board	x 1.00	2.2	2.0
Insulation	Urethane Foam w/ skin per i	x 2.00	1.0	1.0
Misc.	Misc.	x 1.00	0.5	0.0
Wall Covering	14 ga steel	x 0.50	1.8	1.7
		x 1.00	0.0	0.0
		x 1.00	0.0	0.0
			0.0	0.0
			0.0	0.0
			0.0	0.0
	Actual Dead Load		6.6	5.8
	Use this DL instead		8.0	5.0

Wall Design Load #2

Items	Description	Multiple	psf (max)	psf (min)
Framing			0.0	0.0
			0.0	0.0
veneer			0.0	0.0
		x 1.00	0.0	0.0
			0.0	0.0
		x 1.00	0.0	0.0
		x 1.00	0.0	0.0
		x 1.00	0.0	0.0
	Actual Dead Load		0.0	0.0
	Use this DL instead		0.0	0.0

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CODE SUMMARY

Code: International Building Code 2021

Live Loads:

Roof	0 to 200 sf:	20 psf
	200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
	over 600 sf:	12 psf
Decks (1.5 times live load)		60 psf
Typical Floor		40 psf
Partitions		N/A
Partitions		N/A
Partitions		N/A
Partitions		N/A
Storage areas above ceilings		20 psf

Dead Loads:

Floor	0.0 psf
Roof	10.0 psf

Roof Snow Loads:

Design Uniform Roof Snow load		=	25.0 psf
Flat Roof Snow Load	Pf	=	17.5 psf
Balanced Snow Load	Ps	=	17.5 psf
Ground Snow Load	Pg	=	25.0 psf
Importance Factor	I	=	1.00
Snow Exposure Factor	Ce	=	1.00
Thermal Factor	Ct	=	1.00
Sloped-roof Factor	Cs	=	1.00
Drift Surcharge load	Pd	=	
Width of Snow Drift	w	=	

Earthquake Design Data:

Risk Category		=	II
Importance Factor	I	=	1.00
Mapped spectral response acceleratio	Ss	=	141.00
	S1	=	49.10
Site Class		=	code default
Spectral Response Coef.	Sds	=	1.128
	Sd1	=	0.592
Seismic Design Category		=	D
Basic Structural System		=	Bearing Wall Systems
Seismic Resisting System		=	Light frame (wood) walls with structural wood shear panels
Seismic Response Coef.	Cs	=	0.174
Response Modification Factor	R	=	6.5
Analysis Procedure		=	Equivalent Lateral-Force Analysis

Rain Design Data:

Rain intensity	*i*	=	0.00 in/hr
Rain Load	R	=	0.0 psf

Wind Design Data:

Ultimate Design Wind Speed	98 mph
Nominal Design Wind Speed	75.91 mph
Risk Category	II
Mean Roof Ht (h)	8.5 ft
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure Coef.	+/-0.18
Directionality (Kd)	0.85

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DN: C=US, E=ryanmcgrawse@gmail.com, O="MSE, LLC", CN="Ryan McGraw, PE, SE"
Reason: I am the author of this document.
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:32-07'00'

McGraw Structural Engineering, LLC
1118 Enstad Ln
Silverton, OR
503.884.2178

JOB TITLE	Alev Residence		
	Garage Structural Repair		
JOB NO.	22-108	**SHEET NO.**	
CALCULATED BY	RDM	**DATE**	4/22/22
CHECKED BY	RDM	**DATE**	4/23/22

www.struware.com

Component and Cladding Ultimate Wind Pressures

Roof

	Surface Pressure (psf)							
Area	10 sf	20 sf	50 sf	100 sf	200 sf	350 sf	500 sf	1000 sf
Negative Zone 1	-33.4	-31.2	-28.2	-26.0	-23.8	-22.1	-20.9	-20.9
Negative Zone 1'	-19.2	-19.2	-19.2	-19.2	-16.5	-16.0	-16.0	-16.0
Negative Zone 2	-44.0	-41.2	-37.4	-34.6	-31.8	-29.5	-28.0	-28.0
Negative Zone 3	-60.0	-54.3	-46.8	-41.2	-35.5	-30.9	-28.0	-28.0
Positive All Zones	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1&1'	-30.2	-29.6	-28.9	-28.4	-23.8	-20.1	-17.7	-17.7
Overhang Zone 2	-40.8	-37.0	-32.0	-28.3	-24.5	-21.5	-19.5	-19.5
Overhang Zone 3	-56.8	-50.2	-41.4	-34.8	-28.2	-22.9	-19.5	-19.5

Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 3.2 psf)

Parapet

		Solid Parapet Pressure (psf)					
	Area	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf
CASE A:	Zone 2 :	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0
CASE B:	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0

Wall

	Surface Pressure (psf)			
Area	10 sf	100 sf	200 sf	500 sf
Negative Zone 4	-20.8	-17.9	-17.1	-16.0
Negative Zone 5	-25.5	-19.9	-18.2	-16.0
Positive Zone 4 & 5	19.2	16.3	16.0	16.0

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This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

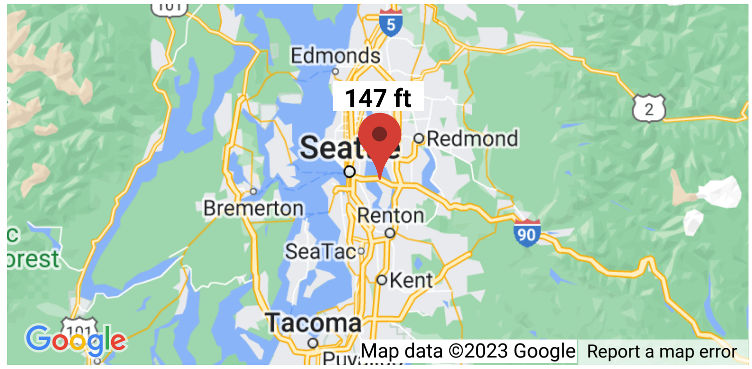
Address: 6848 SE 33rd St, Mercer Island, WA 98040, USA

Coordinates: 47.5814047, -122.2468911

Elevation: 147 ft

Timestamp: 2023-05-03T17:48:47.532Z

Hazard Type: Snow



ASCE 7-16

Ground Snow Load 16 lb/sqft

The reported ground snow load applies at the query location of 147 feet up to a maximum elevation of 320 feet with a tolerance of 100 feet.

ASCE 7-10

Ground Snow Load 15 lb/sqft

The reported ground snow load applies at the query location of 147 feet up to a maximum elevation of 400 feet.

ASCE 7-05

Ground Snow Load 15 lb/sqft

The reported ground snow load applies at the query location of 147 feet up to a maximum elevation of 400 feet.

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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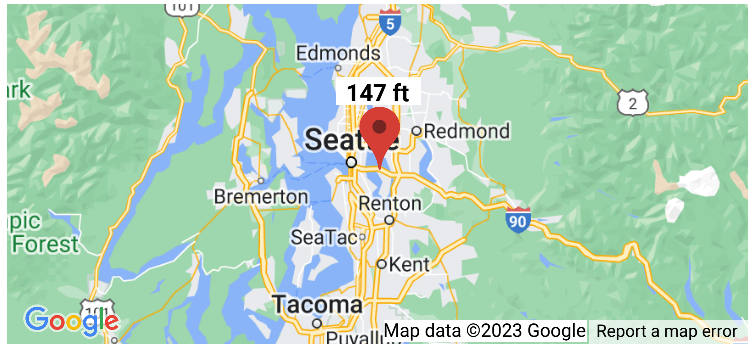
ATC Hazards by Location

Search Information

Address:	6848 SE 33rd St, Mercer Island, WA 98040, USA
Coordinates:	47.5814047, -122.2468911
Elevation:	147 ft
Timestamp:	2023-05-03T17:49:30.131Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default



Basic Parameters

Name	Value	Description
S_S	1.41	MCE_R ground motion (period=0.2s)
S_1	0.491	MCE_R ground motion (period=1.0s)
S_{MS}	1.692	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.128	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.603	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.724	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.41	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.563	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

SsD	3.359	Factored deterministic acceleration value (0.2s)
S1RT	0.491	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.547	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.359	Factored deterministic acceleration value (1.0s)
PGAd	1.154	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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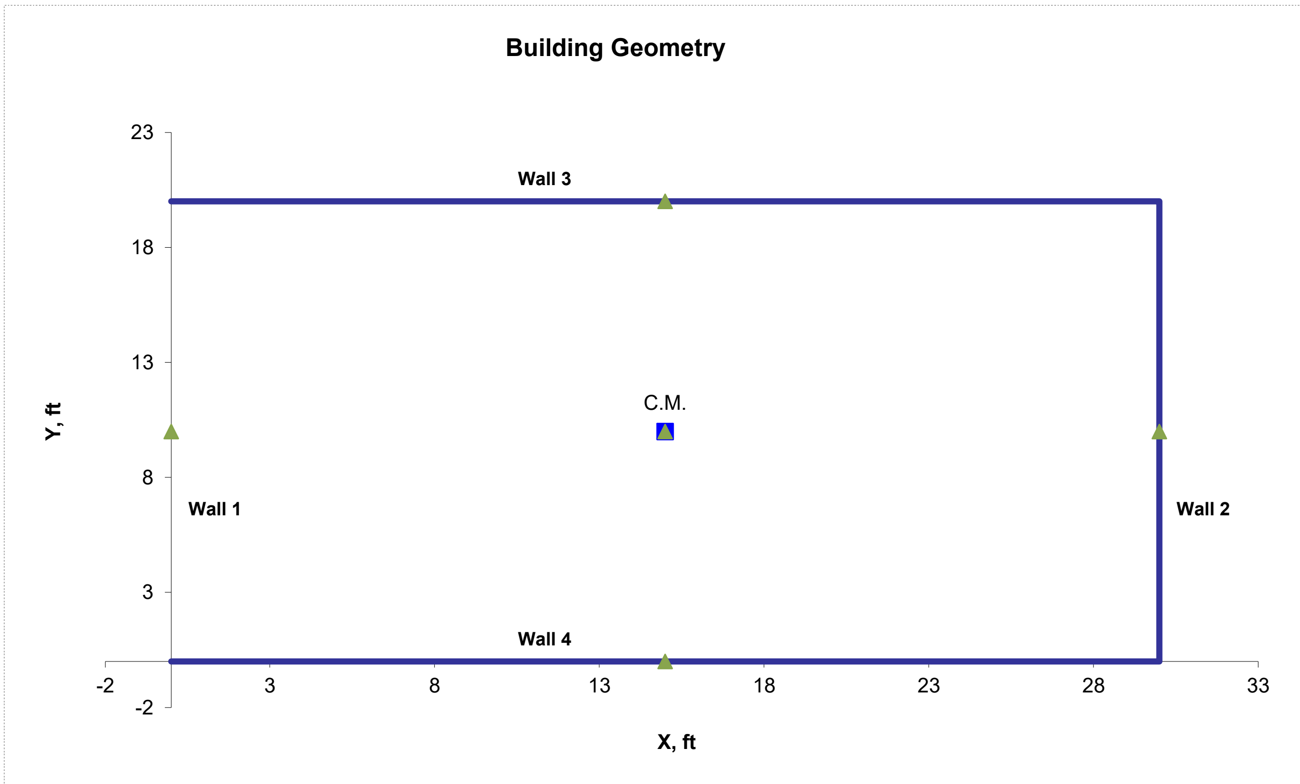
MCGRAW STRUCTURAL ENGINEERING, LLC

Project Name:	Alev Residence	**Date:**	5/4/2023
Client Name:	Alev Family	**Designer:**	Ryan
Job NO:	22-108	**Checker:**	Ryan

CENTER OF MASS ANALYSIS

GEOMETRY

BUILDING LENGTH, "x"	x	=	30	ft
BUILDING WIDTH, "y"	y	=	20	ft
FLOOR HEIGHT	h	=	8.5	ft



	m_i (kips)	x_i (feet)	y_i (feet)
1	1.3	15	0
2	0.9	0	10
3	0.85	30	10
4	1.3	15	20
5	8.316	15	10
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
19			
20			
Σ	13	75	50
	-		-
	-	-	

	Σm	x_{bar}	y_{bar}
CM	12.6	15	10

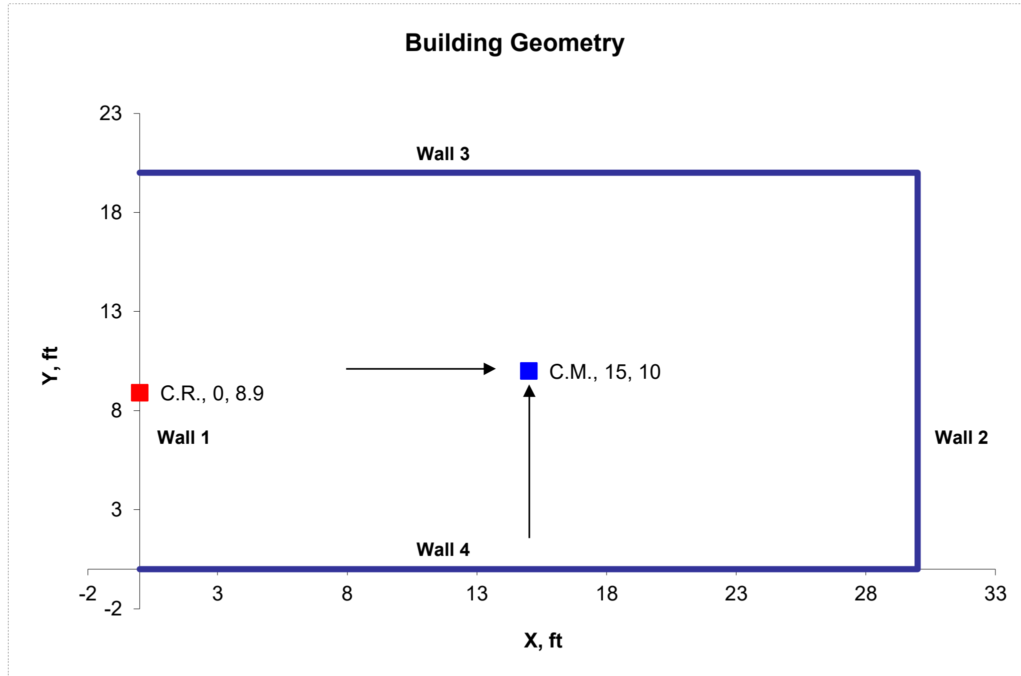
MCGRAW STRUCTURAL ENGINEERING, LLC

Project Name: Alev Residence
Client Name: Alev Family
Job NO: 22-108
Date: 5/5/2023
Designer: Ryan
Checker: Ryan

RIGID DIAPHRAGM ANALYSIS (WALL RIGIDITIES BASED ON LENGTH OF WALL)

GEOMETRY

BUILDING LENGTH, "x"	x	=	30	ft
BUILDING WIDTH, "y"	y	=	20	ft
FLOOR HEIGHT	h	=	8.5	ft
SHEAR PERP TO LENGTH "x"	v_y	=	2840	#
SHEAR PERP TO WIDTH "y"	v_x	=	2840	#
Σ (R 1)	R_1	=	16.83	
Σ (R 2)	R_2	=	0	
Σ (R 1 & 2)	$R_y = R_1+R_2$	=	16.83	
Σ (R 3)	R_3	=	24	
Σ (R 4)	R_4	=	30	
Σ (R 3 & 4)	$R_x = R_3+R_4$	=	54	
CENTER OF MASS, m_x	m_x	=	15	ft
CENTER OF MASS, m_y	m_y	=	10	ft
CENTER OF RIGIDITY, r_x	r_x	=	0	ft
CENTER OF RIGIDITY, r_y	r_y	=	8.889	ft
ACC. ESSENTRICITY, e_{ACCx}	e_{ACCx}	=	16.5	ft
ACC. ESSENTRICITY, e_{ACCy}	e_{ACCy}	=	2.111	ft



Line	Stiffness (ft) k	Distance (ft) r	$k(r)$	$J = k(r^2)$		$\frac{F_x(e_y')k(r)}{\Sigma k(r^2)}$	$\frac{F_y(e_x')k(r)}{\Sigma k(r^2)}$
1	16.833	0.0	0.0	0.0	0.000	0.0	0.0
2	0	30.0	0.0	0.0	0.000	0.0	0.0
3	24	11.1	266.7	2963.0	0.050	299.8	2343.0
4	30	8.9	266.7	2370.4	0.050	299.8	2343.0
			ΣJ =	5333.3			

WIND DESIGN

$V = 98$ mph $\qquad h_{AVG} = \left[8'\text{-}7" + 8'\text{-}4\tfrac{1}{2}"\right]\frac{1}{2} \cong 8.5$ FT

Exp : C

$K_d = 0.85$

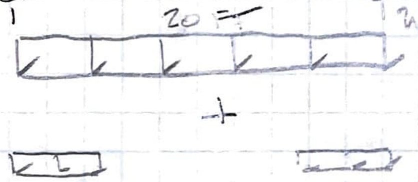
$K_z = K_h = 0.85$

$K_{zt} = 1.0$ (MERCER ISLAND MAPS)

$q_h = 0.00256(1.0)(.85)(.85)(98)^2 = 17.8$ PSF

$a = 3$ FT, $2a = 6$ FT

Ⓐ 20 FT Ⓑ



6 FT $\qquad$ 6 FT

$W_{INT} = q_h \times \frac{h}{2}\,(GC_p \pm GC_{p,i}) = 52$ PLF

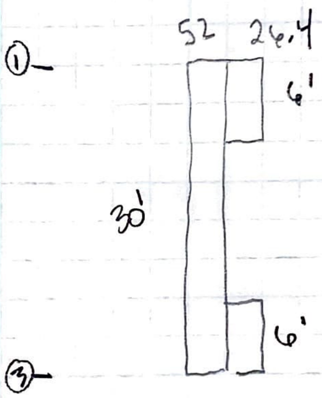
1 & 4 FROM FIG 28.3-1
$= 0.49$

$W_{EDGE} = q_h \times \frac{h}{2}\,(GC_{pE}$ 1E & 4E$) = 26.4$ PLF

$\underbrace{\qquad}_{(1.04-0.69) = 0.35}$

$V_A = (26.4)6 + 52(10) = 678$# WIND MWFRS

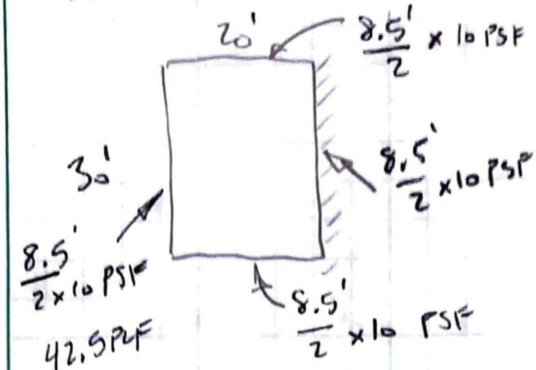
$V_B = V_A = 678$# WIND MWFRS



$V_1 = (26.4)6 + 52(15) = 938.4$# MWFRS

$V_3 = V_1 = 938.4$# MWFRS

■ SEISMIC HORIZ DIST.



W_{TOT} WALLS $= 42.5\,PLF\,(20' \times 2 + 30' \times 2) = 4250^{\#}$

W_{TOT} ROOF/DECK $= (21)(33)(12\,PSF) = 8316^{\#}$

$W_{TOT} = 12{,}000^{\#} \pm$

$$C_S = \frac{S_{DS} I_E}{R} = \frac{(1.128)(1.0)}{6.5}$$

$= 0.17$

$F_p = C_S W_{TOT} = 0.17(W_{TOT})$

$= 2136^{\#}$

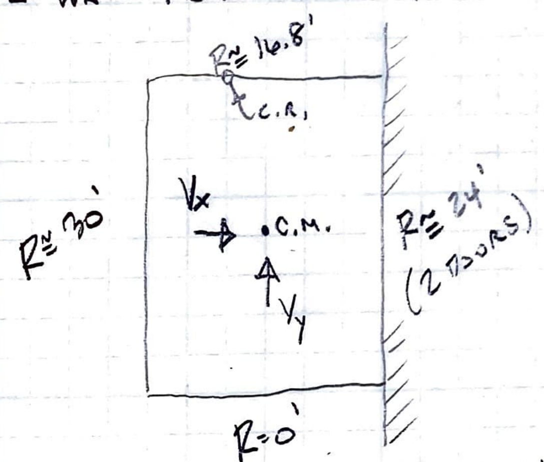
$F_{px_{MIN}} = 0.2 S_{DS} I_E w_{px}$

$= 0.2(0.17)(1.0)(12.6^k) = \underline{\underline{2.84^k}}$

GOVERNS

$0.7 F_p = 2.0^k$

■ WALL RIGIDITIES ARE APPROX PROPORTIONATE TO SHEARWALL CONTRIBUTING LENGTHS:



$F_{R=24} = 1650^{\#}$ (TORSION ONLY)

$F_{R=30} = 1650^{\#}$ (TORSION ONLY)

$\Sigma M_{C.R} = 0;$ [REF. RIGID DIAPH SPREADSHEET]

$$V_x(x_{CM} - x_{CR}) = \underbrace{(2840)}_{F_p}\underbrace{(0.7)}_{\text{LOAD COMB.}}\underbrace{(16.5)}_{e + e_{acc}} = \Sigma M_R$$

COUPLED REACTION $= \dfrac{\Sigma M_R}{20'} = 1640^{\#} = V_{REQD}$ ALONG GRIDS A & B

$l_{DIAPH\ A\,\&\,B} = 30'$; ∴ $v_{DIAPH\ REQD\ TORS} = \dfrac{1640}{30} \cong 55\,PLF$, USE ANY CASE 5/8" STRUCTURAL I w/ 8d NAILS 6" EDGE 12" FIELD

$v_{DIAPH\ GRID\ 1} = \frac{2840}{20'}(0.7) = 99.4$ PLF $\therefore$ USE ANY CASE 5/8" STRUCTURAL 1 W/ 8d NAILS @ 6" EDGE, 12" FIELD

* DIAPH ASPECT RATIO FOR TORSION $= \frac{l'}{w'} = \frac{30'}{20'}$ WHICH MATCHES MAX LIMIT (1.5) FROM NDS SDPWS § 4.2.5.2 PART 2

& SHEATHED IN ACCORDANCE WITH § 4.2.7.1. § 4.2.7.1 REQUIRES "WOOD STRUCTURAL PANEL SHEATHING USED FOR DIAPH THAT ARE PART OF THE LFRS SHALL BE APPLIED DIRECTLY TO THE FRAMING MEMBERS & BLOCKING."

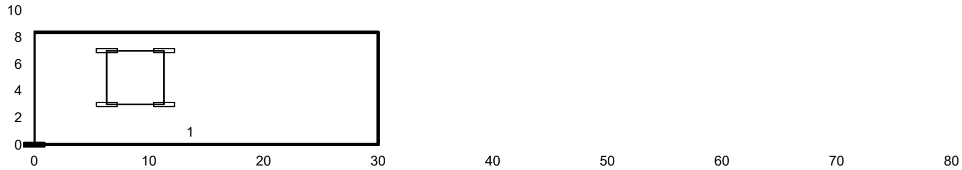
* DETAILING SHOWN ON S-SHEETS SHOWS THE USE OF EDGE & BOUNDARY NAILING TO BLOCKING & THUS AN H1 CLIP DIRECTLY TO TOP PL OF SHEARWALLS. H1 HAS 190# PER CLIP WHICH EXCEEDS WORST-CASE V_{DIAPH} OF 99.4 PLF.

Project Name :	Alev Residence Garage/Deck Repair				
Client Name :	Alev Family				
Wall Location :	Grid A			DESIGN BY :	RDM
Job # :	22-108	Date :	5/5/2023	REVIEW BY :	RDM

FTAO ShearWall Design Based on SDPWS, Latest Edition

Wall Geometries & Loading Parameters

Total length of wall	30.00	ft	Unfactored Wind wall shear, V_W =	**680**	#	S_{DS} =	1.128	g
Wall height	8.38	ft	Unfactored Seismic wall shear, V_E=	**2343**	#	q_h =	17.8	psf
Gable height	0.00	ft	Max shear for APA sheathing =	2343	#			
Int. wall height	8.38	ft	Net DL multiplier =	0.60	(wind loads adusted below)			
Trib. roof width	6.00	ft	Net DL multiplier =	0.70	(seismic loads adusted below)			
			DF studs:	1.00	fastener factor			



Panel openings	Width ft	Start ft	End ft	Height ft	Head ht ft
1	0.00	30.00	30.00	0	7
2	0.00	30.00	30.00	0	7
3	0.00	30.00	30.00	0	7
4	0.00	30.00	30.00	0	7
5	0.00	30.00	30.00	0	7
6	0.00	30.00	30.00	0	7
7	0.00	30.00	30.00	0	7
Total	0.00	ft of wall not useable for shear			

DL Load Combinations	Wind Combination Nom.	Wind Combination Adj.		Seismic Combination Nom.	Seismic Combination Adj.	
P(uplift) =	-15.5	-15.5	psf	-15.5	0.0	psf
Roof+Flr DL =	12.0	7.2	psf	12.0	8.4	psf
Roof rim/other =	2.5	1.50	plf	2.5	1.75	plf
Net roof load =		137.6	plf		52.2	plf
Ext. wall DL =	10	6.0	psf	10	7.0	psf
Int. wall DL =	10	6.0	psf	10	7.0	psf

Mid-panel windows	Width ft	Start ft	End ft	Height ft	Head ht ft	Strap?	Width ft	Strap Load lb	
1	5.00	6.33	11.33	4.00	7.00	yes	5.00	195	<1750#, OK
2	0.00	0.00	0.00	0.00	0.00	no	0.00		
3	0.00	0.00	0.00	0.00	0.00	no	0.00		
4	0.00	0.00	0.00	0.00	0.00	no	0.00		
5	0.00	0.00	0.00	0.00	0.00	no	0.00		
						Total	5.00	ft of integral windows	

Wind Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd SDS Screws
1	30.00	0.00	yes	30.00	680	45	1508	0	0	**-586**	none
2	0.00	30.00	no	0.00				0			
3	0.00	30.00	no	0.00				0			
4	0.00	30.00	no	0.00				0			
5	0.00	30.00	no	0.00				0			
6	0.00	30.00	no	0.00				0			
7	0.00	30.00	no	0.00				0			
8	0.00	30.00	no	0.00				0			
Total	30.00		Net width =	30.00	ft of designed shear panels						

Overturning loads delivered to shearwalls with less than 500# of capacity are commonly considered to have a certain amount of redundancy in the system.

Seismic Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd 10d nails
1	30.00	0.00	yes	30.00	2343	53	1759	0	0	**-252**	none
2	0.00	30.00	no	0.00				0			
3	0.00	30.00	no	0.00				0			
4	0.00	30.00	no	0.00				0			
5	0.00	30.00	no	0.00				0			
6	0.00	30.00	no	0.00				0			
7	0.00	30.00	no	0.00				0			
8	0.00	30.00	no	0.00				0			
Total	30.00		Net width =	30.00	ft of designed shear panels						

Net width = 25.00 ft available after subtracting out all openings/unused panels/strapped windows.

Unit shear above/below strapped windows =	78	#/ft <	280	#/ft
Unit shear next to strapped windows =	94	#/ft <	280	#/ft

Install **S6** wall w/ **8d NAILS (.131")**
at 6"" edge pattern
at 12" in field

Ryan McGraw, PE, SE
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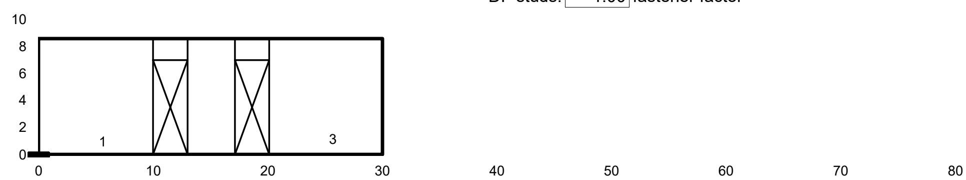
DN: C=US, E=ryanmcgrawse@gmail.com, O="MSE, LLC", CN="Ryan McGraw, PE, SE"
Reason: I am the author of this document.
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:35-07'00'

Project Name :	Alev Residence Garage/Deck Repair		
Client Name :	Alev Family		
Wall Location :	Grid B	DESIGN BY :	RDM
Job # :	22-108 Date : 5/5/2023	REVIEW BY :	RDM

FTAO ShearWall Design Based on SDPWS, Latest Edition

Wall Geometries & Loading Parameters

Total length of wall	30.00	ft	Jnfactored Wind wall shear, V_W =	**680**	#	S_{DS} =	1.128	g
Wall height	8.58	ft	ıfactored Seismic wall shear, V_E=	**2343**	#	q_h =	17.8	psf
Gable height	0.00	ft	Max shear for APA sheathing =	2343	#			
Int. wall height	8.58	ft	Net DL multiplier =	0.60	(wind loads adusted below)			
Trib. roof width	5.00	ft	Net DL multiplier =	0.70	(seismic loads adusted below)			
			DF studs:	1.00	fastener factor			



Panel openings	Width ft	Start ft	End ft	Height ft	Head ht ft
1	3.00	10.00	13.00	7	7
2	3.00	17.13	20.13	7	7
3	0.00	30.00	30.00	0	7
4	0.00	30.00	30.00	0	7
5	0.00	30.00	30.00	0	7
6	0.00	30.00	30.00	0	7
7	0.00	30.00	30.00	0	7
Total	6.00	ft of wall not useable for shear			

DL Load Combinations	Wind Combination Nom.	Adj.		Seismic Combination Nom.	Adj.	
P(uplift) =	-15.5	-15.5	psf	-15.5	0.0	psf
Roof+Flr DL =	12.0	7.2	psf	12.0	8.4	psf
Roof rim/other =	2.5	1.50	plf	2.5	1.75	plf
Net roof load =		114.9	plf		43.8	plf
Ext. wall DL =	10	6.0	psf	10	7.0	psf
Int. wall DL =	10	6.0	psf	10	7.0	psf

Mid-panel windows	Width ft	Start ft	End ft	Height ft	Head ht ft	Strap?	Width ft	Strap Load lb
1	0.00	0.00	0.00	0.00	0.00	no	0.00	
2	0.00	0.00	0.00	0.00	0.00	no	0.00	
3	0.00	0.00	0.00	0.00	0.00	no	0.00	
4	0.00	0.00	0.00	0.00	0.00	no	0.00	
5	0.00	0.00	0.00	0.00	0.00	no	0.00	
						Total	0.00	ft of integral windows

Wind Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd SDS Screws
1	10.00	0.00	yes	10.00	342	15	1200	0	0	**-314**	none
2	4.13	13.00	no	0.00				0			
3	9.88	20.13	yes	9.88	338	15	1185	0	0	**-306**	none
4	0.00	30.00	no	0.00				0			
5	0.00	30.00	no	0.00				0			
6	0.00	30.00	no	0.00				0			
7	0.00	30.00	no	0.00				0			
8	0.00	30.00	no	0.00				0			
Total	24.00		Net width =	19.88	ft of designed shear panels						

Overturning loads delivered to shearwalls with less than 500# of capacity are commonly considered to have a certain amount of redundancy in the system.

Seismic Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd 10d nails
1	10.00	0.00	yes	10.00	1179	18	1400	0	0	**303**	1.2
2	4.13	13.00	no	0.00				0			
3	9.88	20.13	yes	9.88	1164	17	1383	0	0	**312**	1.2
4	0.00	30.00	no	0.00				0			
5	0.00	30.00	no	0.00				0			
6	0.00	30.00	no	0.00				0			
7	0.00	30.00	no	0.00				0			
8	0.00	30.00	no	0.00				0			
Total	24.00		Net width =	19.88	ft of designed shear panels						

Net width = 19.88 ft available after subtracting out all openings/unused panels/strapped windows.

Unit shear above/below strapped windows = 118 #/ft < 280 #/ft

Unit shear next to strapped windows = 118 #/ft < 280 #/ft

Install **S6** wall w/ **8d NAILS (.131")**
at 6"" edge pattern
at 12" in field

Ryan McGraw, PE, SE

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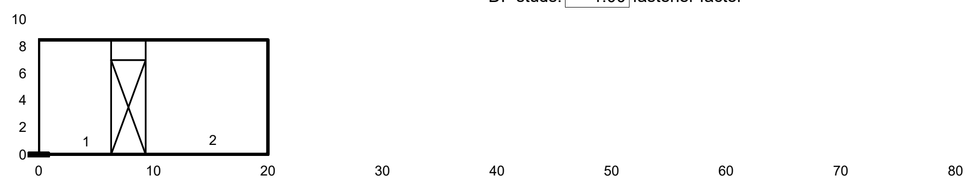
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:35-07'00'

Project Name :	Alev Residence Garage/Deck Repair		
Client Name :	Alev Family		
Wall Location :	Grid 1	DESIGN BY :	RDM
Job # :	22-108 Date : 5/5/2023	REVIEW BY :	RDM

FTAO ShearWall Design Based on SDPWS, Latest Edition

Wall Geometries & Loading Parameters

Total length of wall	20.00	ft	Unfactored Wind wall shear, V_W =	**1880**	#	S_{DS} =	1.128	g
Wall height	8.50	ft	Unfactored Seismic wall shear, V_E=	**2840**	#	q_h =	17.8	psf
Gable height	0.00	ft	Max shear for APA sheathing =	2840	#			
Int. wall height	8.50	ft	Net DL multiplier =	0.60	(wind loads adusted below)			
Trib. roof width	1.67	ft	Net DL multiplier =	0.70	(seismic loads adusted below)			
			DF studs:	1.00	fastener factor			



Panel openings	Width ft	Start ft	End ft	Height ft	Head ht ft
1	3.00	6.33	9.33	7	7
2	0.00	20.00	20.00	0	7
3	0.00	20.00	20.00	0	7
4	0.00	20.00	20.00	0	7
5	0.00	20.00	20.00	0	7
6	0.00	20.00	20.00	0	7
7	0.00	20.00	20.00	0	7
Total	3.00	ft of wall not useable for shear			

DL Load Combinations	Wind Combination Nom.	Adj.		Seismic Combination Nom.	Adj.	
P(uplift) =	-15.5	-15.5	psf	-15.5	0.0	psf
Roof+Flr DL =	12.0	7.2	psf	12.0	8.4	psf
Roof rim/other =	2.5	1.50	plf	2.5	1.75	plf
Net roof load =		39.3	plf		15.8	plf
Ext. wall DL =	10	6.0	psf	10	7.0	psf
Int. wall DL =	10	6.0	psf	10	7.0	psf

Mid-panel windows	Width ft	Start ft	End ft	Height ft	Head ht ft	Strap?	Width ft	Strap Load lb
1	0.00	0.00	0.00	0.00	0.00	no	0.00	
2	0.00	0.00	0.00	0.00	0.00	no	0.00	
3	0.00	0.00	0.00	0.00	0.00	no	0.00	
4	0.00	0.00	0.00	0.00	0.00	no	0.00	
5	0.00	0.00	0.00	0.00	0.00	no	0.00	
						Total	0.00	ft of integral windows

Wind Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd SDS Screws
1	6.33	0.00	yes	6.33	700	10	323	0	0	**774**	3.1
2	10.67	9.33	yes	10.67	1180	16	544	0	0	**660**	2.6
3	0.00	20.00	no	0.00				0			
4	0.00	20.00	no	0.00				0			
5	0.00	20.00	no	0.00				0			
6	0.00	20.00	no	0.00				0			
7	0.00	20.00	no	0.00				0			
8	0.00	20.00	no	0.00				0			
Total	17.00	Net width =		17.00	ft of designed shear panels						

Overturning loads delivered to shearwalls with less than 500# of capacity are commonly considered to have a certain amount of redundancy in the system.

Seismic Overturning Calculation

Panels able to take shear	Width ft	Start ft	Use for shear? ft	Width ft	Panel shear lb	Roof Effect lb	Wall DL lb	LF of Int. wall	Int. wall lb	HD load lb	Req'd 10d nails
1	6.33	0.00	yes	6.33	1058	11	377	0	0	**1226**	4.9
2	10.67	9.33	yes	10.67	1782	19	635	0	0	**1093**	4.4
3	0.00	20.00	no	0.00				0			
4	0.00	20.00	no	0.00				0			
5	0.00	20.00	no	0.00				0			
6	0.00	20.00	no	0.00				0			
7	0.00	20.00	no	0.00				0			
8	0.00	20.00	no	0.00				0			
Total	17.00	Net width =		17.00	ft of designed shear panels						

Net width = 17.00 ft available after subtracting out all openings/unused panels/strapped windows.

Unit shear above/below strapped windows = 167 #/ft < 280 #/ft

Unit shear next to strapped windows = 167 #/ft < 280 #/ft

Install **S6** wall w/ **8d NAILS (.131")**
at 6"" edge pattern
at 12" in field

Ryan McGraw, PE, SE

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SIMPSON Strong-Tie®	Anchor Designer™ Software Version 3.0.7947.2

Company:	MSE, LLC	Date:	4/29/2023
Engineer:	RDM	Page:	1/5
Project:	Alev Garage Post-Installed Anchorage		
Address:	1118 Enstad Lane, Silverton, OR 97381		
Phone:	503-884-2178		
E-mail:	ryanmcgrawse@gmail.com		

1.Project information

Customer company: Alev Family Residence
Customer contact name: Zeynep Alev
Customer e-mail: zeynep_alev@hotmail.com
Comment:

Project description: SW Holdowns in Existing Concrete
Location: Mercer Island, WA
Fastening description: HDU2 w/Set-XP 0.625 AB

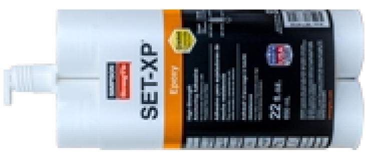
2. Input Data & Anchor Parameters

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): 5.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.75
c_{ac} (inch): 7.58
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Uncracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: Yes
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

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



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SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 3.0.7947.2

Company:	MSE, LLC	Date:	4/29/2023
Engineer:	RDM	Page:	2/5
Project:	Alev Garage Post-Installed Anchorage		
Address:	1118 Enstad Lane, Silverton, OR 97381		
Phone:	503-884-2178		
E-mail:	ryanmcgrawse@gmail.com		

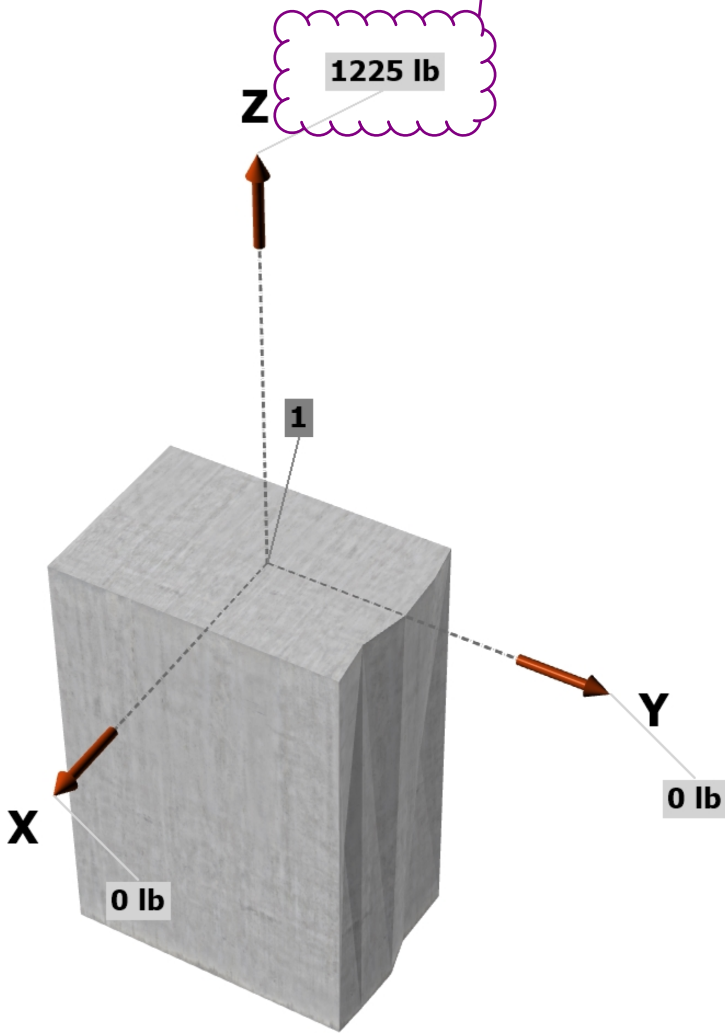
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Max load required from HDU2 with post-installed SET-XP. See detail 8/S01.

Strength level loads:

N_{ua} [lb]: 1225
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



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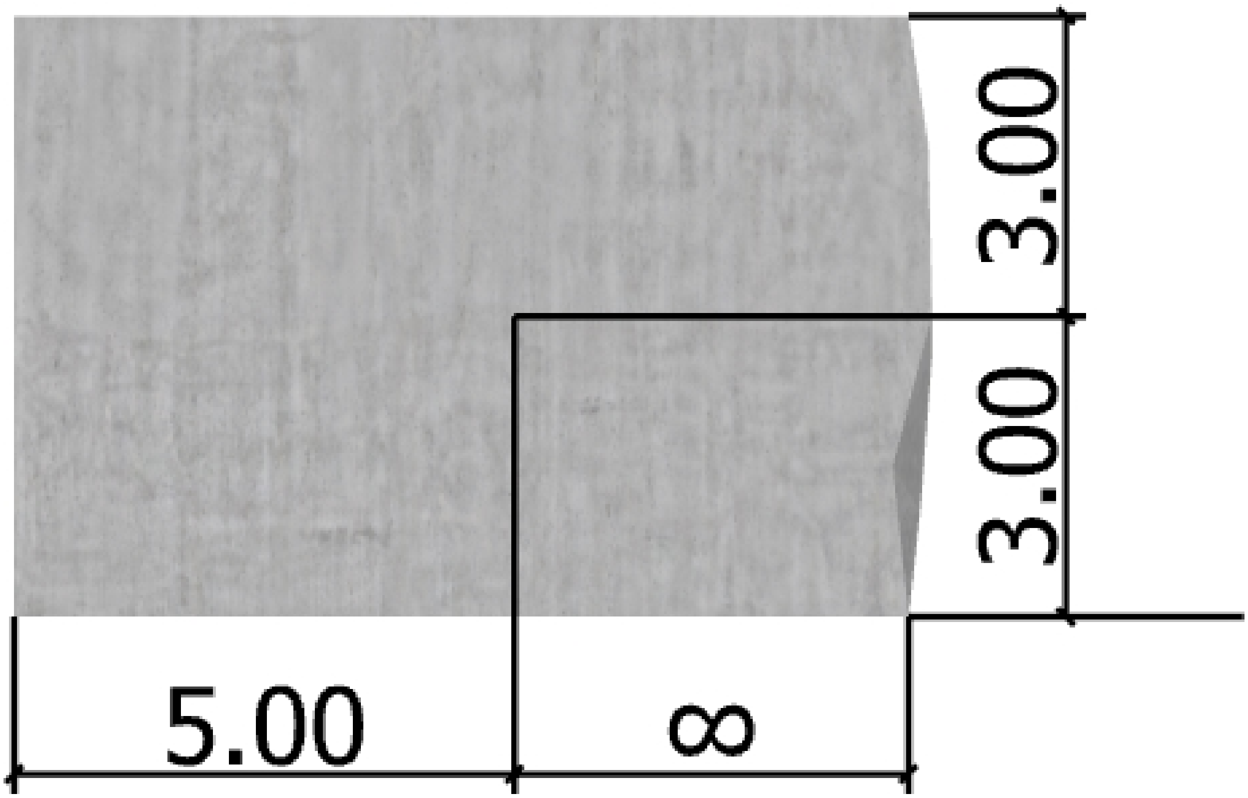
Company:	MSE, LLC	Date:	4/29/2023
Engineer:	RDM	Page:	3/5
Project:	Alev Garage Post-Installed Anchorage		
Address:	1118 Enstad Lane, Silverton, OR 97381		
Phone:	503-884-2178		
E-mail:	ryanmcgrawse@gmail.com		

<Figure 2>



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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1225.0	0.0	0.0	0.0
Sum	1225.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1225
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

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

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

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	3.333	7303

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
60.00	100.00	3.00	0.880	1.00	0.989	7303	0.65	1860

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

$\tau_{k,uncr} = \tau_{k,uncr} f_{short\text{-}term} K_{sat} \alpha_{N.seis}$

$\tau_{k,uncr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	$\tau_{k,uncr}$ (psi)
1060	1.72	1.00	1.00	1823

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

λ_a	τ_{uncr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1823	0.63	5.000	17899

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
78.28	258.98	8.05	3.00	0.812	1.000	17899	0.65	2141

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11. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1225	9833	0.12	Pass
Concrete breakout	**1225**	**1860**	**0.66**	**Pass (Governs)**
Adhesive	1225	2141	0.57	Pass

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 5.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	1225	15732	7.8%	

Concrete	Factored Load, N_{ua} (lb)	Nominal Strength, N_n (lb)	Ratio	
Concrete breakout	**1225**	**3815**	**32.1%**	**Governs**
Adhesive	1225	4392	27.9%	

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

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.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

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

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

Ryan McGraw, PE, SE

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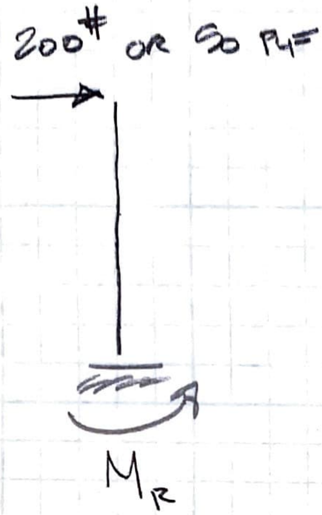
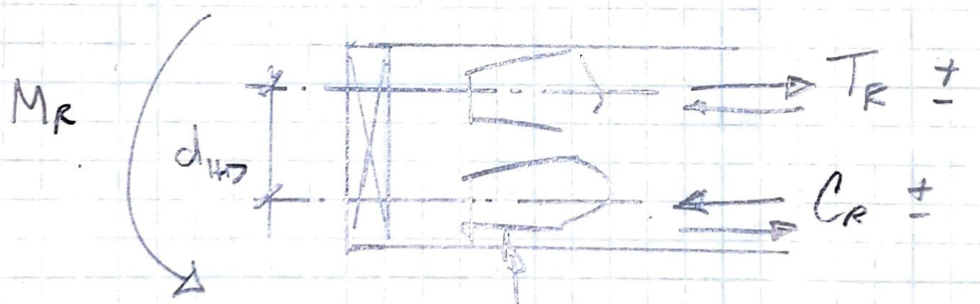
R312.1 GUARDRAIL ANCHORAGE CHECK



$M_R = 200^{\#} (40'' \text{ or } 3'\text{-}4'') = 667^{\#}$

$T_R = \frac{M_R}{d_{HD}} = \frac{667^{\#}}{4''_{min}} \approx 2000^{\#}$

$\therefore$ TRY DTT2Z, $T_{CAP} \approx 2180^{\#}$



USE DTT2Z @ MIN 4" SEPARATION

FORTEWEB®

JOB SUMMARY REPORT

22-108 Method Hardscapes

Roof			
Member Name	**Results**	**Current Solution**	**Comments**
Floor: Joist	Passed	2 piece(s) 1 3/4" x 5 1/2" 2.0E Microllam® LVL @ 32" OC	
Floor: Joist	Passed	1 piece(s) 2 x 8 DF No.2 @ 16" OC	

Ryan McGraw, PE, SE

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ForteWEB Software Operator	Job Notes
RYAN D MCGRAW McGraw Structural Engineering, LLC (503) 884-2178 ryanmcgrawse@gmail.com	

Weyerhaeuser

5/5/2023 2:56:56 PM UTC

ForteWEB v3.5

File Name: 22-108 Method Hardscapes

FORTEWEB

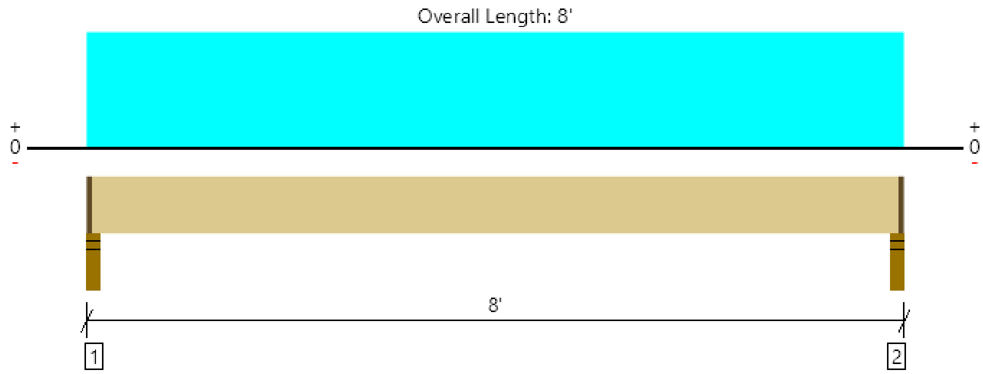
MEMBER REPORT

PASSED

Roof, Floor: Joist

2 piece(s) 1 3/4" x 5 1/2" 2.0E Microllam® LVL @ 32" OC

Direct bearing on (2) 1 3/4" x 5 1/2" LVL (2.0E) or (3) 1 1/2" x 5 1/2" LVL (2.0E) is equivalent by observation.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	727 @ 2 1/2"	4922 (2.25")	Passed (15%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	607 @ 9"	3658	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1342 @ 4'	4251	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.130 @ 4'	0.253	Passed (L/702)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.151 @ 4'	0.379	Passed (L/602)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	42	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Span rating of 1' 8" o.c. for selected sheathing is less than design on center spacing of 2' 8" o.c. for product.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- No composite action between deck and joist was considered in analysis.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.50"	107	640	747	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.50"	107	640	747	1 1/4" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 10" o/c	
Bottom Edge (Lu)	7' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 8'	32"	10.0	60.0	Default Load

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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5/5/2023 2:56:56 PM UTC
ForteWEB v3.5, Engine: V8.2.5.1, Data: V8.1.3.6
File Name: 22-108 Method Hardscapes

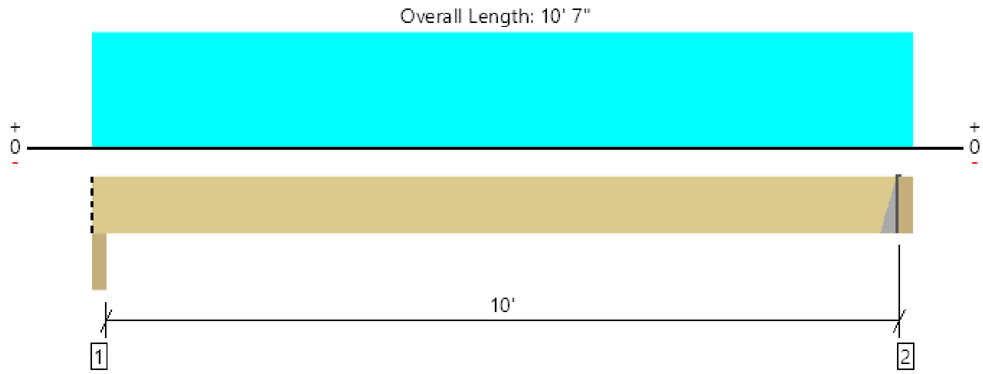
FORTEWEB

MEMBER REPORT

PASSED

Roof, Floor: Joist

1 piece(s) 2 x 8 DF No.2 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	496 @ 10' 3 1/2"	1406 (1.50")	Passed (35%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	414 @ 9' 8 1/4"	1305	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1186 @ 5' 3"	1360	Passed (87%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.259 @ 5' 3"	0.336	Passed (L/466)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.300 @ 5' 3"	0.504	Passed (L/403)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Beam - DF	3.50"	3.50"	1.50"	70	420	175	516	Blocking
2 - Hanger on 7 1/4" DF beam	3.50"	Hanger¹	1.50"	71	427	178	524	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' o/c	
Bottom Edge (Lu)	10' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Top Mount Hanger	THA29	2.25"	4-10d	6-10d	4-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 10' 7"	16"	10.0	60.0	25.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Ryan McGraw, PE, SE

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Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:38-07'00'

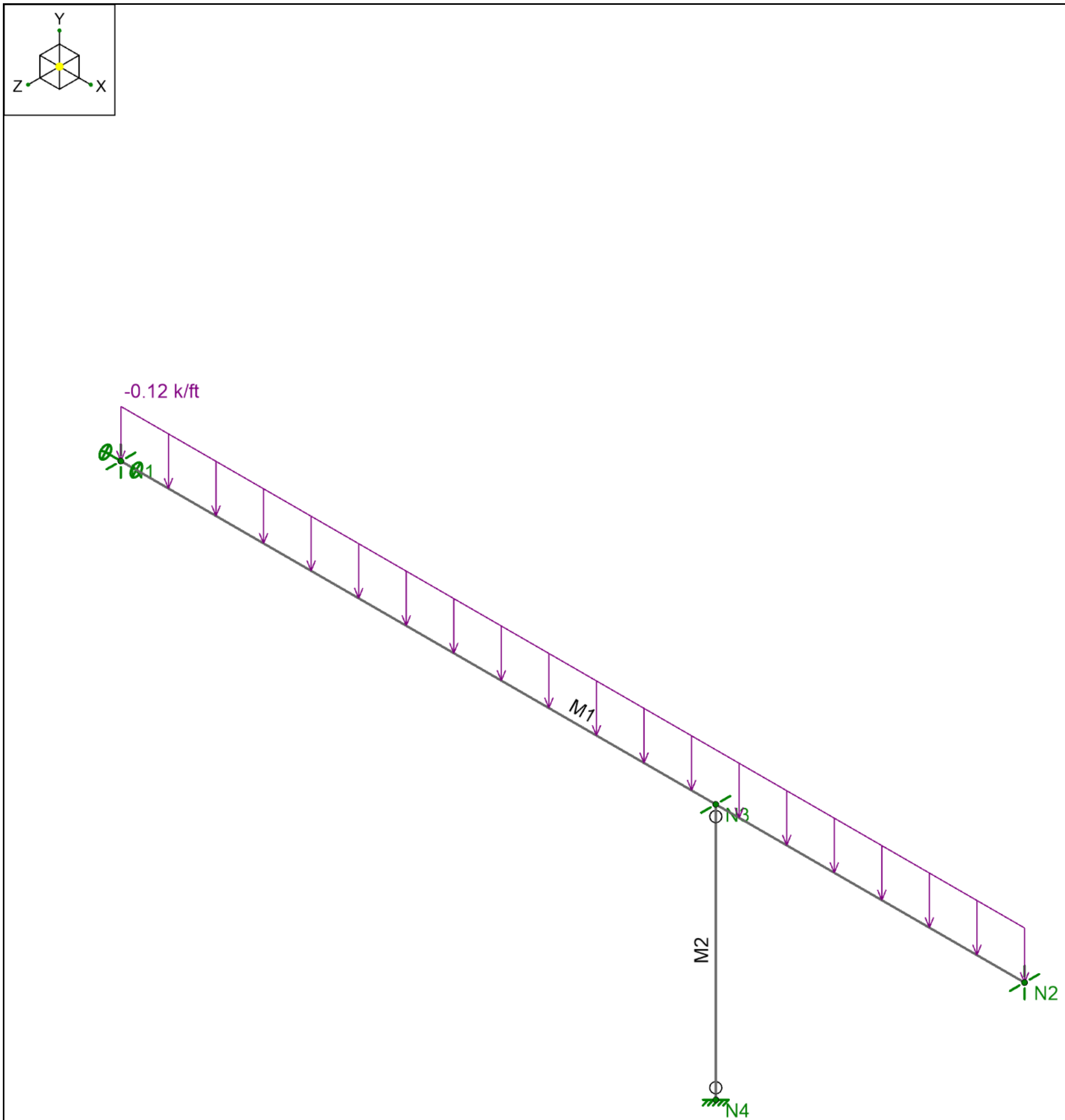
ForteWEB Software Operator	Job Notes
RYAN D MCGRAW McGraw Structural Engineering, LLC (503) 884-2178 ryanmcgrawse@gmail.com	

Weyerhaeuser

5/5/2023 2:56:56 PM UTC

ForteWEB v3.5, Engine: V8.2.5.1, Data: V8.1.3.6

File Name: 22-108 Method Hardscapes



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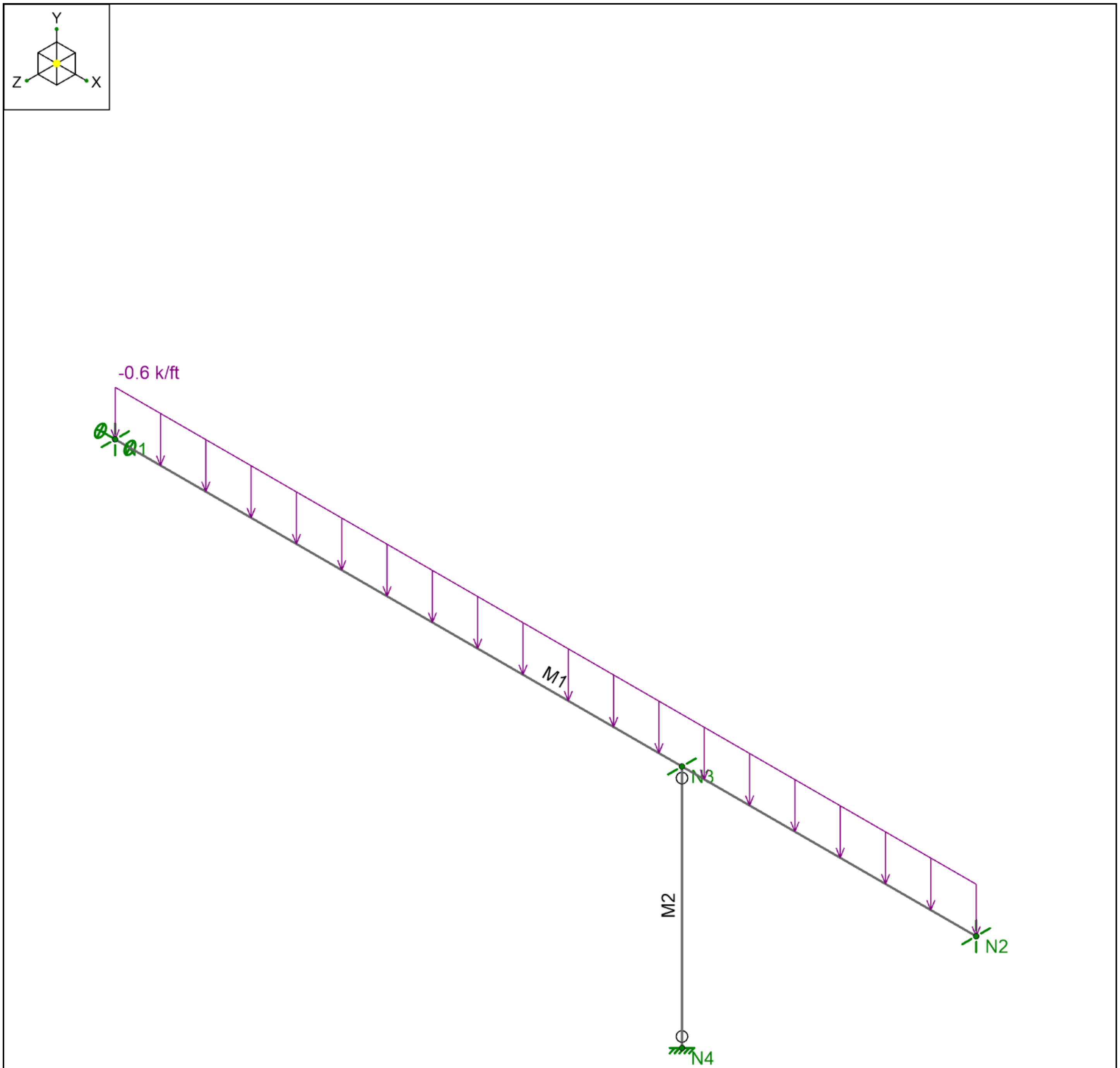
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:38-07'00'

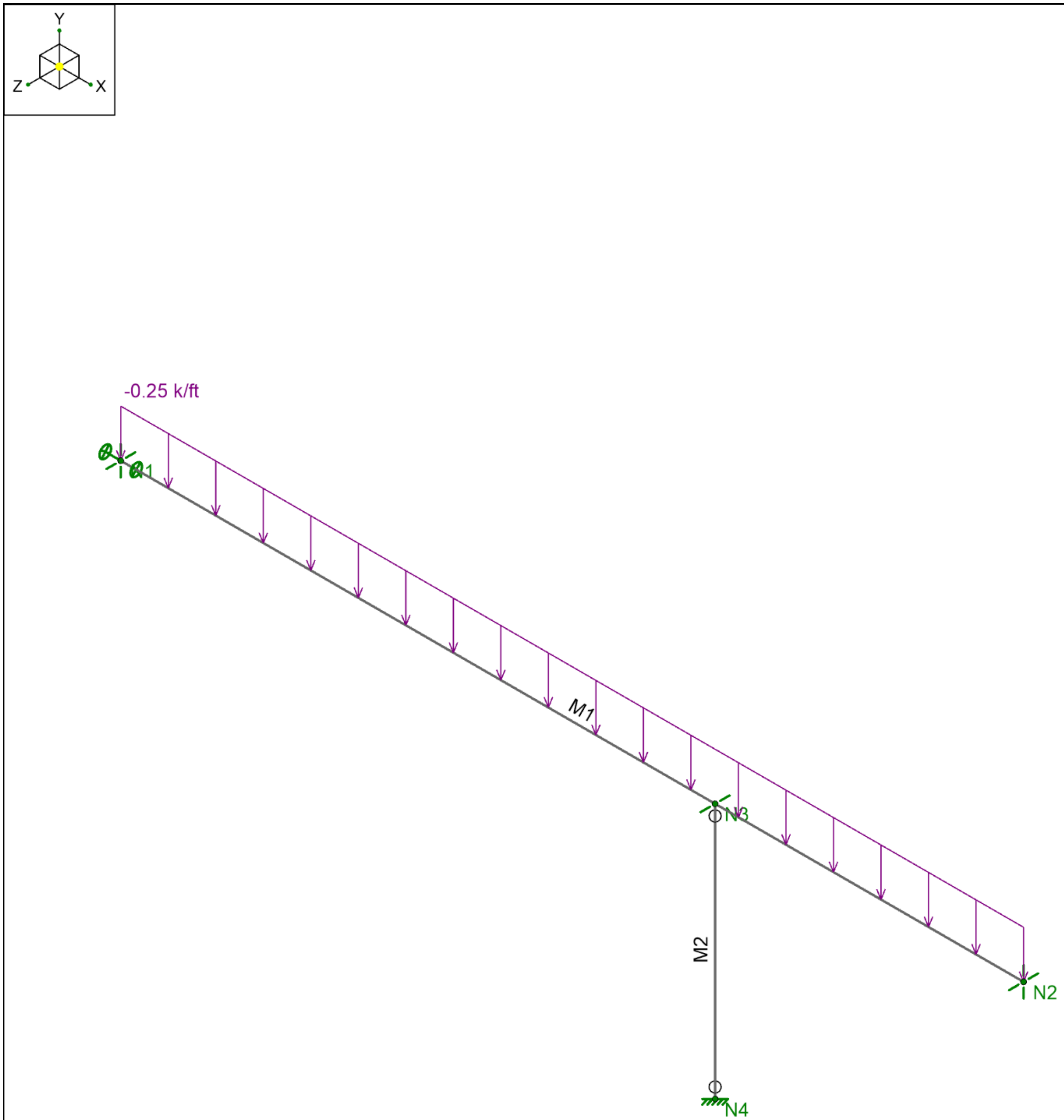
Loads: BLC 1, D
Envelope Only Solution

MCGRAW STRUCTURAL ENGINEERING, LLC	MSE LLC	Alev Garage Stl Bm	SK-1
	RDM		May 05, 2023 at 07:05 AM
	22-108		22-108 Steel Bm.r3d



Loads: BLC 2, L
Envelope Only Solution

MCGRAW STRUCTURAL ENGINEERING, LLC	MSE LLC	Alev Garage Stl Bm	SK-2
	RDM		May 05, 2023 at 07:07 AM
	22-108		22-108 Steel Bm.r3d



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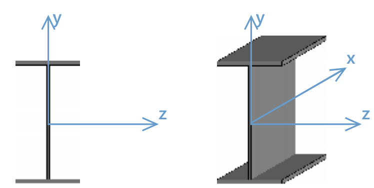
Date: 2023.05.08 08:07:39-07'00'

Loads: BLC 3, S
Envelope Only Solution

MCGRAW STRUCTURAL ENGINEERING, LLC	MSE LLC	Alev Garage Stl Bm	SK-3
	RDM		May 05, 2023 at 07:08 AM
	22-108		22-108 Steel Bm.r3d

Detail Report: M1

Load Combination: Envelope **Code check:** 0.245 (LC 7)



Input Data

Shape:	W12X30	**I Node:**	N1
Member Type:	Beam	**J Node:**	N2
Length (ft):	30	**I Release:**	Fixed
Material Type:	Hot Rolled Steel	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	AISC 15th (360-16): ASD	**T/C Only:**	Both Way

Material Properties

Material:	A992	**Therm. Coeff. (/1E5 F):**	0.65	**Fu (ksi):**	65
E (ksi):	29000	**Density (k/ft^3):**	0.49	**Rt:**	1.1
G (ksi):	11154	**Fy (ksi):**	50		
Nu:	0.3	**Ry:**	1.1		

Shape Properties

d (in):	12.3	**Area (in^2):**	8.79	**rT (in):**	1.73
bf (in):	6.52	**Zyy (in^3):**	9.56	**J (in^4):**	0.457
tf (in):	0.44	**Zzz (in^3):**	43.1	**kdet (in):**	1.125
tw (in):	0.26	**Cw (in^6):**	720	**kdes (in):**	0.74
Iyy (in^4):	20.3	**Wno (in^2):**	19.3		
Izz (in^4):	238	**Sw (in^4):**	13.9		

Design Properties

Lb y-y (ft):	2	**Ky-y:**	1	**Seismic DR:**	None
Lb z-z (ft):	2	**Kz-z:**	1	**Max Defl Ratio:**	L/828
Lcomp top:	Lbyy	**y sway:**	No	**Max Defl Location:**	9.062
Lcomp bot:	Lbyy	**z sway:**	No	**Span:**	1
Ltorque (ft):	30	**Function:**	Lateral	**τb:**	1

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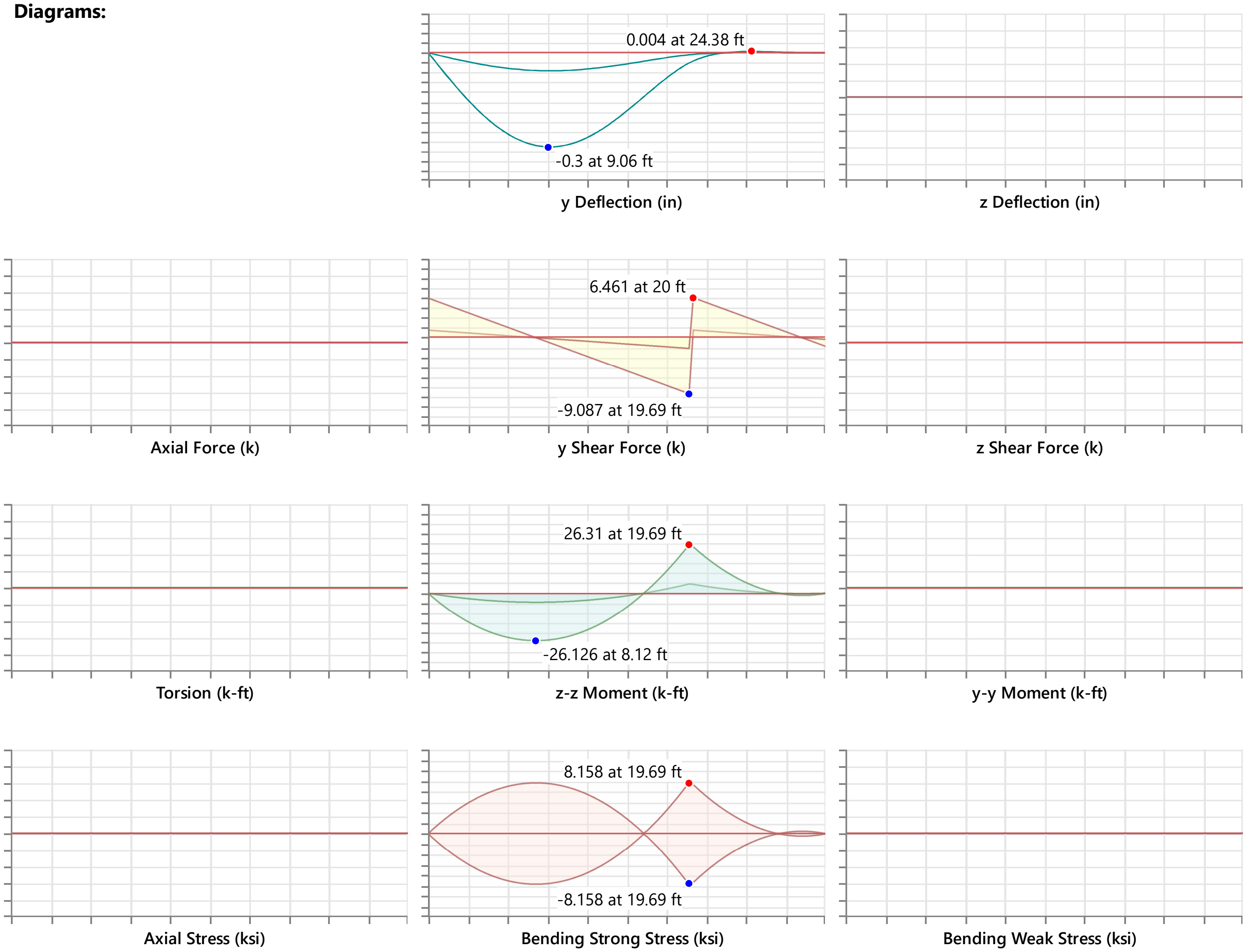
Diagrams:



AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	7	-	-	-	-
Applied Loading - Shear + Torsion	7	-	-	-	-
Axial Tension Analysis	7	0 k	263.174 k	-	-
Axial Compression Analysis	7	0 k	117.273 k	-	-
Flexural Analysis (Strong Axis)	7	26.31 k-ft	107.535 k-ft	-	-
Flexural Analysis (Weak Axis)	7	0 k-ft	23.852 k-ft	-	-
Shear Analysis (Major Axis y)	7	9.087 k	63.96 k	**0.142**	**PASS**
Shear Analysis (Minor Axis z)	7	0 k	103.071 k	**0**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	7	-	-	**0.245**	**PASS**

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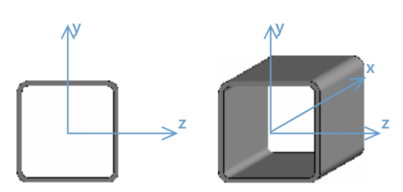
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:40-07'00'

Company	: MSE LLC	5/5/2023
Designer	: RDM	7:30:29 AM
Job Number	: 22-108	Checked By :
Model Name	: Alev Garage Stl Bm	

Detail Report: M2

Load Combination: Envelope **Code check:** 0.379 (LC 7)



Input Data

Shape:	HSS3.5X3.5X3	**I Node:**	N4
Member Type:	Column	**J Node:**	N3
Length (ft):	8.5	**I Release:**	BenPIN
Material Type:	Hot Rolled Steel	**J Release:**	BenPIN
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	97	**J Offset:**	N/A
Design Code:	AISC 15th (360-16): ASD	**T/C Only:**	Both Way

Material Properties

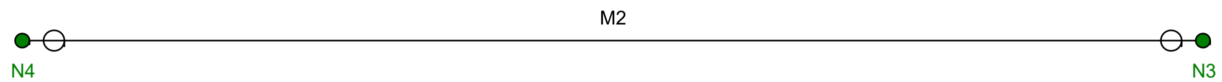
Material:	A500 Gr.B RECT	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.527	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

Shape Properties

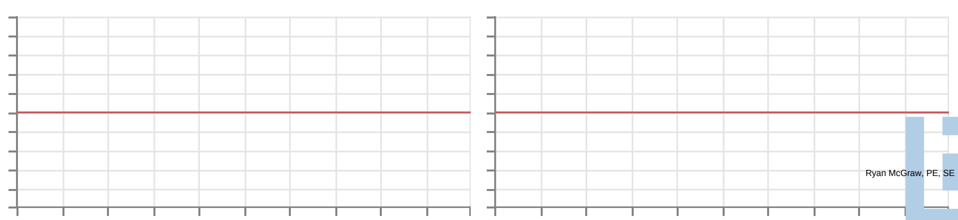
d (in):	3.5	**I_{yy} (in^4):**	4.05	**J (in^4):**	6.56
b_f (in):	3.5	**I_{zz} (in^4):**	4.05		
t (in):	0.174	**Area (in^2):**	2.24		

Design Properties

$L_{b\ y\text{-}y}$ (ft):	8.5	**$K_{y\text{-}y}$:**	1	**Seismic DR:**	None
$L_{b\ z\text{-}z}$ (ft):	8.5	**$K_{z\text{-}z}$:**	1	**Max Defl Ratio:**	L/10000
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Max Defl Location:**	0
$L_{comp\ bot}$ (ft):	8.5	**z sway:**	No	**Span:**	N/A
L_{torque} (ft):	8.5	**Function:**	Lateral	**τ_b:**	1



Diagrams:



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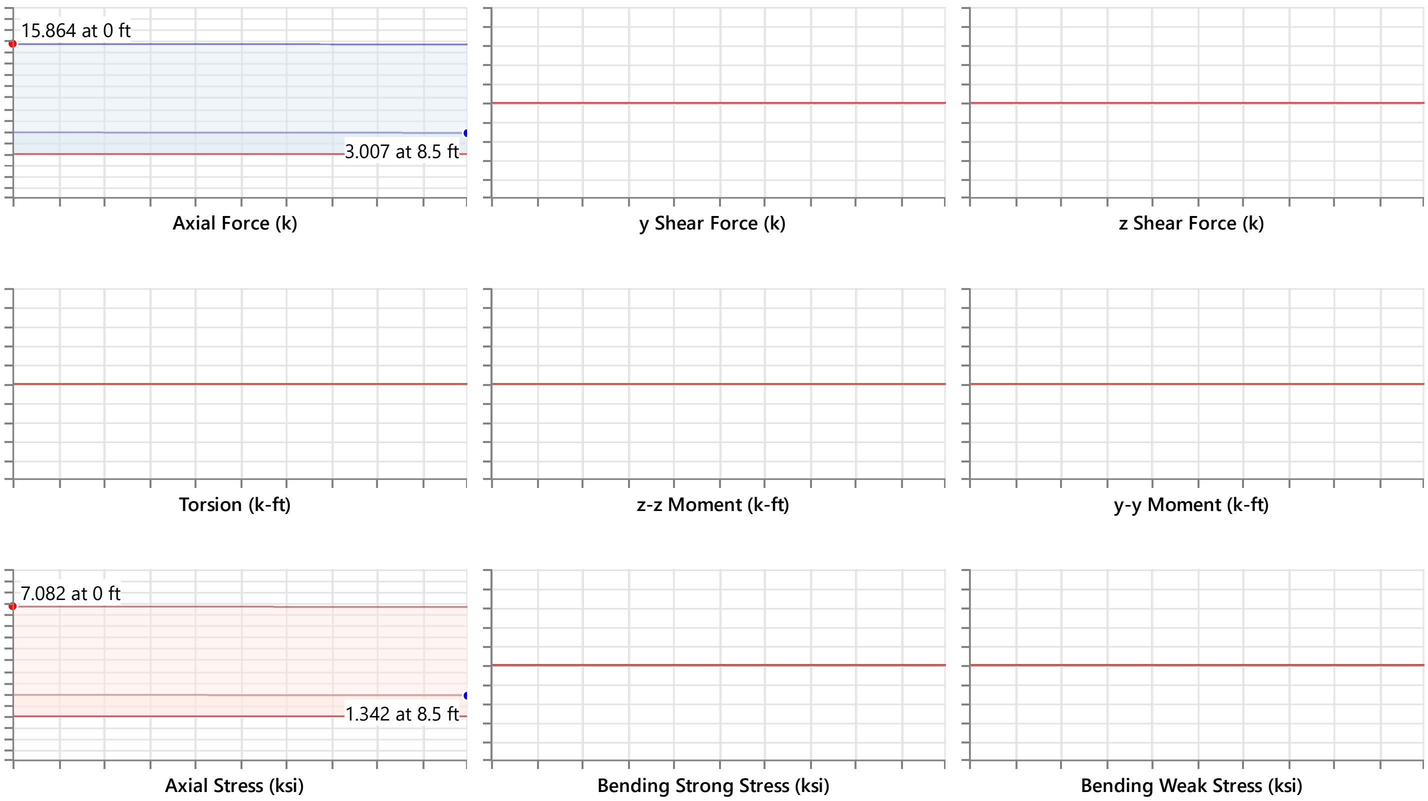

AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	7	-	-	-	-
Applied Loading - Shear + Torsion	7	-	-	-	-
Axial Tension Analysis	7	0 k	61.701 k	-	-
Axial Compression Analysis	7	15.864 k	41.897 k	-	-
Flexural Analysis (Strong Axis)	7	0 k-ft	6.335 k-ft	-	-
Flexural Analysis (Weak Axis)	-	0 k-ft	6.335 k-ft	-	-
Shear Analysis (Major Axis y)	7	0 k	17.128 k	**0**	**PASS**
Shear Analysis (Minor Axis z)	7	0 k	17.128 k	**0**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	7	-	-	**0.379**	**PASS**
Torsional Analysis	7	0 k-ft	5.274 k-ft	**0**	**PASS**

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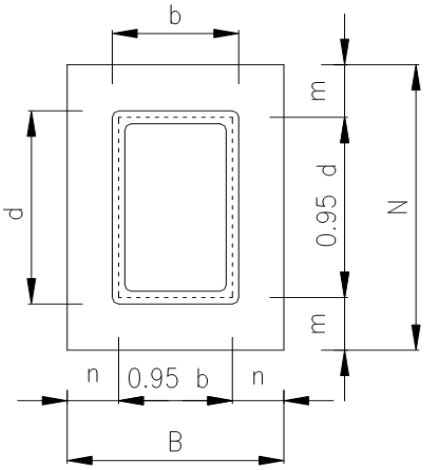
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:40-07'00'

MCGRAW STRUCTURAL ENGINEERING, LLC	Project: Alev Garage				Job Ref.: 22-108	
	Section: Steel Column Base Pl				Sheet no./rev.	
	Calc. by: RDM	Calc. Date: 5/4/2023	Chk'd by: RDM	Chk'd Date: 5/5/2023		App'd Date: 5/5/2023

INPUT DATA & ANALYSIS SUMMARY

COMPRESSIVE WORKING LOAD	P_a	=	16	kips
STEEL YIELD STRESS	F_y	=	46	ksi
COLUMN SHAPE	AISC HSS Shape	=	HSS3-1/2X3-1/2X3/16	
CONCRETE COMP STRENGTH	f'_c	=	2.5	ksi β = 0.85
BASE PLATE SIZE	N	=	7	in
	B	=	7	in
AREA OF BASEPLATE	A_1	=	49	in^2
AREA OF CONCRETE SUPPORT	A_2	=	1296	in^2



ANALYSIS | CONCRETE BEARING

CHECKING BEARING PRESSURE AISC 360 Section J8

$$P_p/\Omega_c = \frac{f'_c A_1}{\Omega_c} MIN\left[0.85 MAX\left(\sqrt{\frac{A_2}{A_1}}\ ,\ 1\right)\ ,\ 1.7\right] =$$ **83.3 kips** > P_a **[OK - acceptable]**

Ω_c = 2.5

ANALYSIS | AISC PARAMETERS

d	=	3.5	in, column depth
b	=	3.5	in, column width
m = 0.5(N-0.95d)	=	1.8375	in
n = 0.5(B-0.95b)	=	1.8375	in
$n' = 0.25(db)^{0.5}$	=	0.875	in

$$X = MIN\left[\left(\frac{4db}{(d+b)^2}\right)\frac{\Omega_c P_a}{P_p}\ ,\ 1\right] =$$ **0.19**

$$\lambda = MIN\left(\frac{2\sqrt{X}}{1+\sqrt{1-X}}\ ,\ 1\right) =$$ **0.46**

ANALYSIS | AISC REQUIRED THICKNESS

$l = MAX(m,\ n,\ \lambda n') =$ **1.84** in

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y BN}} =$$ **0.28** in

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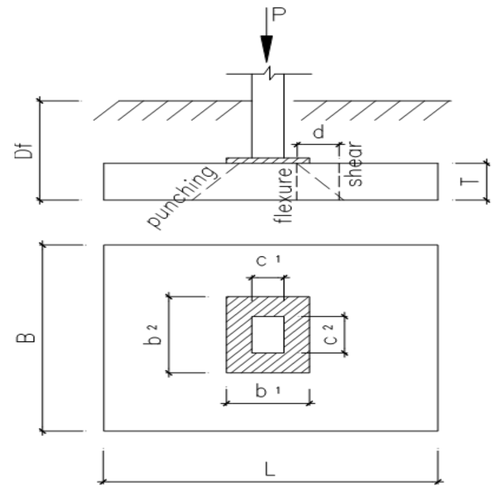
MCGRAW STRUCTURAL ENGINEERING, LLC

Project Name: Alev Deck Project
Client Name: Method Hardscapes
Job NO: 22-108
Design: Interior Footing
Date: 5/5/2023
Designer: Ryan M.
Checker: Ryan M.

(IBC 2021/2018 & ACI 318-14/19) Ordinary Rectangular Reinforced Concrete Footing Design Supporting Column

COLUMN WIDTH	c_1	=	3.5	in
COLUMN DEPTH	c_2	=	3.5	in
BASE PLATE WIDTH (TO BOLT C.L.)	b_1	=	7	in
BASE PLATE DEPTH	b_2	=	7	in
FOOTING CONCRETE STRENGTH	f_c'	=	2.5	ksi β_1 = 0.85
REBAR YIELD STRESS	f_y	=	60	ksi
SERVICE DEAD LOAD	P_D	=	2.25	kips
SERVICE LIVE LOAD	P_L	=	13.75	kips
SERVICE ROOF LIVE/SNOW LOAD	P_S	=	5	kips
SERVICE WIND LOAD	P_W	=	0	kips
SEISMIC AXIAL LOAD	P_E	=	0.5	kips (without 0.7 reduction)
LIVE LOAD REDUCTION FACTOR	f_1	=	0.5	
SHOULD OVERBURDEN BE USED TO REDUCE BEARING CAPACITY *(yes/no)*			no	
SLAB THICKNESS	t_s	=	0	in (measured above D_f)
SLAB UNIT WEIGHT	$\omega_{conc.}$	=	.150	ksf
SOIL UNIT WEIGHT	ω_{soil}	=	.120	ksf
FOOTING EMBEDMENT DEPTH	D_f	=	1	ft
FOOTING THICKNESS	T	=	12	in
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf
FOOTING WIDTH	B	=	3.25	ft
FOOTING LENGTH	L	=	3.25	ft
LONGITUDINAL BOTTOM REINF.		#	4	SATISFACTORY
TRANSVERSE BOTTOM REINF.		#	4	SATISFACTORY
BOTTOM COVER REQUIREMENT	clr.	=	3	in
SPACING OF FLEXURAL REINF.	S(long)	=	10.00	(Spacing is within limits of ACI section 7.12)
SPACING OF FLEXURAL REINF.	S(short)	=	10.00	(Spacing is within limits of ACI section 7.12)



DESIGN SUMMARY ALL LISTED CRITERIA SATISFIED

Long. Reinf.	=	5 no. 4's	
Transverse Reinf.	=	5 no. 4's	within square
Transverse Reinf.	=	0 no. 4's	outside square
Volume of Conc.	=	10.5625 cub. ft.	

Analysis

APPLICABLE DESIGN LOADS FOR CONCRETE

(BASED UPON IBC SEC.1605.2.1 & ACI 318 SEC.9.2.1)

(Eq. 16-1, 9-1)	$1.4\,D$	U	=	3.2 kips
(Eq. 16-2, 9-2)	$1.2\,D + 1.6\,L + 0.5\,(L_r \text{ or } S)$	U	=	27.2 kips
(Eq. 16-3, 9-3)	$1.2\,D + 1.6\,(L_r \text{ or } S) + (f_1 L \text{ or } 0.8W)$	U	=	17.6 kips
(Eq. 16-4, 9-4)	$1.2\,D + 1.6\,W + f_1 L + 0.5\,(L_r \text{ or } S)$	U	=	12.1 kips
(Eq. 16-5, 9-5)	$1.2\,D + 1.0\,E + f_1 L + 0.2\,S$	U	=	11.1 kips
(Eq. 16-6, 9-6)	$0.9\,D + 1.6\,W$	U	=	2.0 kips
(Eq. 16-7, 9-7)	$0.9\,D + 1.0\,E$	U	=	2.5 kips
		U_{max}	=	**27.2 kips**

APPLICABLE DESIGN LOADS FOR SOIL

(BASED UPON IBC SEC.1605.3.1)

(Eq. 16-8)	D	P	=	2.3 kips
(Eq. 16-9)	$D + L$	P	=	16.0 kips
(Eq. 16-10)	$D + (L_r \text{ or } S)$	P	=	7.3 kips
(Eq. 16-11)	$D + 0.75\,L + 0.75\,L_r$	P	=	16.3 kips
(Eq. 16-12)	$D + (W \text{ or } 0.7E)$	P	=	2.6 kips
(Eq. 16-13)	$D + 0.75(W \text{ or } 0.7E) + 0.75\,(L + L_r)$	P	=	12.8 kips
(Eq. 16-14)	$0.6\,D + W$	P	=	1.4 kips
(Eq. 16-15)	$0.6\,D + 0.7\,E$	P	=	1.7 kips
		$P_{req'd}$		**16.3 kips**

CHECKING SOIL BEARING CAPACITY

(BASED UPON ACI 318 SEC.15.2.2)

q_{net} = **1.5 ksf if surcharge reduction is not preferred.** = 1.50 ksf

$q_{net} = Qa - \left[(T + ts)\,x\,\omega_{conc.}\right]/12 - \left[(D_f - T)\,x\,\omega_{soil}\right]/12$ = 1.35 ksf

$A_{req'd}$ = 10.88 ft^2 **NO OVERBURDEN REDUCTION USED**

A_{given} = 10.56 ft^2 **BEARING AREA IS O.K.**

Factored Net Pressure = $q_{un} = U_{max}/A_{given} =$ 2.58 ksf

CHECKING 1-WAY (FLEXURAL) SHEAR

(BASED UPON ACI 318 SEC. 7.12, 9.3.2.3, 10.2, 11.1.3.1, 11.3, & 15.5.2)

$d = T - (3\text{ in.} + (\#4\text{ bar}) + d_b/2) =$ *8.0 in*

Controlling footing cross-section = 3.25 ft

$V_u = q_{un}\,(Bd)/12$ = **5.6 kips**

$\phi V_c = \phi 2(f'_c)^{0.5}\,b_w\,d$ = **23.4 kips** OK in one-way shear

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Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:41-07'00'

Project Name:	Alev Deck Project	Date:	5/5/2023
Client Name:	Method Hardscapes	Designer:	Ryan M.
Job NO:	22-108	Checker:	Ryan M.
Design:	Interior Footing		

Analysis (cont.)

CHECKING 2-WAY (PUNCHING) SHEAR

(BASED UPON ACI 318 10.2, 11.12.1.2, 11.12.2.1, 15.4.2, & 15.5.2)

$b_{o(steel)} = 2(c_1/2 + b_1/2 + d) + 2(c_2/2 + b_2/2 + d) = 53.0$ in

$\beta_c = 2.86$ $\quad \alpha_s = 40$ (interior column always assumed)

$V_u = q_{un}[BL - \{(b_o/4)/12\}^2] = 24.1$ kips

$\phi V_c = \min$ of:

ACI eq. 11-33	=	54.1 kips
ACI eq. 11-34	=	127.8 kips
ACI eq. 11-35	=	63.6 kips

= 54.1 kips **OK in two-way shear**

CHECKING FLEXURE IN LONG DIRECTION

(BASED UPON ACI 318 7.12.2, 10.5.3, 15.4.2, & 15.5.2)

$M_{u(long)} = q_{un}[B\{[L-(1/2)(b_1+c_1)/12]/2\}^2/2] = 8.3$ ft-kips

• Assuming $j > 0.9$, $\phi = 0.9$ for a tension-controlled section, $A_{s(long)}$ is the following

$A_{s(long)} = \max$ of:

$M_u/[\phi F_y(jd)]$	=	0.24	in^2	0.07	in^2/ft
$0.0018bh$	=		in^2		in^2/ft

S_{max} = 18.00 in.

$d_{b(min.)}$ = 0.375 in. *Use a no. 3 bar for long direction*

a = 0.7131 in. $\quad j = 0.955$

$A_{t(limit)} = \rho_{t(limit)}(bd)$ = 3.5 in^2 *Tension-controlled section*

$\phi M_{n(given)}$ = 33.9 ft-kips

CHECKING FLEXURE IN SHORT DIRECTION

(BASED UPON ACI 318 7.12.2, 10.5.3, 15.4.2, 15.4.4.2, & 15.5.2)

$M_{u(short)} = q_{un}[L\{[B-(1/2)(b_2+c_2)/12]/2\}^2/2] = 8.3$ ft-kips

• Assuming $j > 0.9$, $\phi = 0.9$ for a tension-controlled section, $A_{s(long)}$ is the following

$A_{s(short)} = \max$ of:

$M_u/[\phi F_y(jd)]$	=	0.24	in^2	0.07	in^2/ft
$0.0018bh$	=		in^2		in^2/ft

$R_{(within\ sq.)}A_s = \dfrac{2A_s}{\beta + 1}$, $\quad \beta = \dfrac{\text{long side}}{\text{short side}} = 1.00$, $\quad A_{s(short)}$ *(within square)* = 0.84 in^2 , 0.26 in^2/ft

$R_{(within\ sq.)}$ = 1.000 $\quad A_{s(short)}$ *(outside square)* = 0.00 in^2 , 0.00 in^2/ft

S_{max} = 18.00 in.

$d_{b(min.)}$ = 0.375 in. *Use a no. 3 bar for short direction*

a = 0.7131 in. $\quad j = 0.955$

$A_{t(limit)} = \rho_{t(limit)}(bd)$ = 3.5 in^2 *Tension-controlled section*

$\phi M_{n(given)}$ = 33.9 ft-kips

Ryan McGraw, PE, SE

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DN: C=US, E=ryanmcgrawse@gmail.com, O="MSE, LLC", CN="Ryan McGraw, PE, SE"
Reason: I am the author of this document.
Contact Info: Ryan McGraw
Date: 2023.05.08 08:07:41-07'00'

Notes and limitations

1. Reinforcement does not consider development length.
2. Lateral loads are not considered.

MCGRAW STRUCTURAL ENGINEERING, LLC	Project: Alev Residence Garage and Deck Replacement			Job Ref.: 22-108	
	Section: Structural Calculations - Wood Column (built-up stud)			Sheet no./rev.	
	Calc. by: RDM	Date: 5/7/2023	Chk'd by: RDM	Chk'd Date: 5/8/2023	App'd Date: 5/8/2023

Location: **2nd Story Multi-Story Column (beam#2 support)**

Species/Grade:	**DF#2**		
Nominal Size:	**2 x 6**		
Number of studs =	**3**	(if built-up)	
Stud spacing	**0**	in (used for wind tributary, as reqd)	

Member Design Values

Base Values

F_b =	875	
$F_{c||}$ =	1300	psi
E =	1600	ksi

Adjustment Factors

C_D =	1.00
$C_{F(b)}$ =	1.30
$C_{F(c)}$ =	1.10
C_{fu} =	1.15
$C_{i(b)}$ =	1.00
$C_{i(E)}$ =	1.00

Design Values (unadjusted for stability)

E' =	1600	ksi
F_b^* =	1138	psi
F_c^* =	1430	psi

Dimensional Information

h =	**7' - 6.00"**	=	90 in
d_1 =	5.5 in	d_2 =	4.5 in
S_x =	22.7 in^3	S_y =	18.6 in^3
A_c =	24.75 in^2		
l_{u1} =	90 in	l_{u2} =	90.0 in
K_1 =	1.00	K_2 =	0.50
e_1 =	0.00	e_2 =	0.75
K_{f1} =	1.00 (solid)	K_{f2} =	0.60 (nailed)

Loads

P_{down} =	6200 #	M_x =	0.00 k-in (about d_1 axis)
P_{up} =	0 #	M_y =	0.00 k-in (about d_2 axis)
Wind	25 psf	p_{WIND} =	0.0 plf

Allowable Bending Stresses (NDS 3.3)

K_{bE} =	0.439		
l_{eb1} =	165.6 in	l_{eb2} =	165.6 in
R_{B1} =	6.7	R_{B2} =	5.0
F_{bE1} =	15617	F_{bE2} =	28513
C_{L1} =	0.996	C_{L2} =	0.998
F_{b1}' =	1303 psi	F_{b2}' =	1305 psi

Allowable Axial Stresses (NDS 3.7)

c =	0.8		
K_{cE} =	0.3		
l_{ec1} =	90 in	l_{ec2} =	45 in
l_{e1}/d_1 =	16.36	l_{e2}/d_2 =	10.00
F_{cE1} =	1793	F_{cE2} =	4800
C_{p1} =	0.763	C_{p2} =	0.557
F_c' =	797 psi <d2 governs>		

Column Analysis (NDS 15.3 and 15.4)

f_c =	251 psi
f_{bx} =	0 psi
f_{by} =	0 psi

Combined axial/bending check

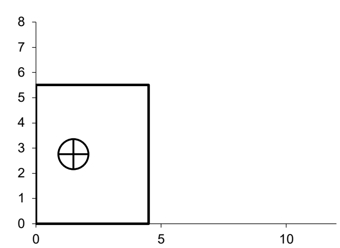
basic axial ratio =	0.099	Eq. (15.4.2)
e1 effects =	0.000	
e2 effects =	0.205	
Total ratio =	**0.304** < 1.0, OK	

Column Notes:
- Fasten studs of column together with two rows of 10d box nails @ 8" oc.
- Install tight-fit blocking to next stud at column 3rd points or window sill/hdr.
- Install tight-fit blocking in floor for full bearing under column

Bearing at Top of Column

Bearing width =	5.5 in		
Bearing length =	4.5 in	A_b =	24.75 in^2
Beam species:	DF Std	$F_{c\perp}$ =	625 psi

$f_{c\perp}$ = 251 < 625, OK



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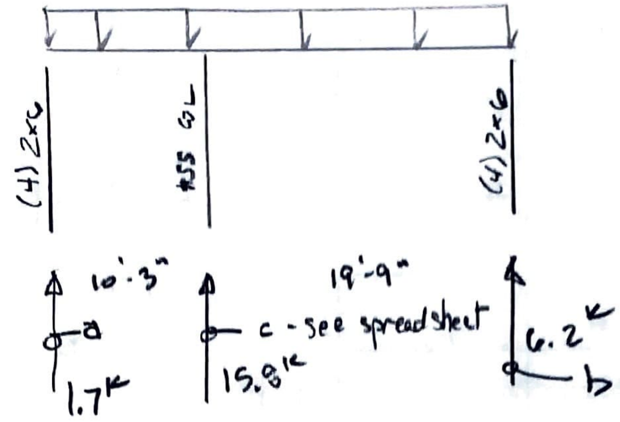
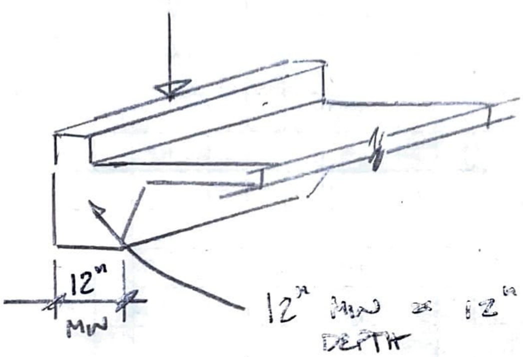
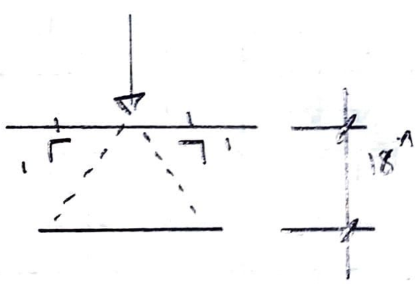


a. 1.7^K ON 12" WIDE FOOTING



12" MIN & 12" WIDTH DEPTH SHALL BE VERIFIED BY CONTRACTOR VIA ROTOHAMMER CORE DRILLING OR SIMILAR MEANS



$P_{ALL} = 1.5' \times 1' \times 1500 \times 2 = 4500^{\#}$

$P_{REQ} = 1700^{\#} < P_{ALL}$, ∴ OK

b. $6.2^k / 1500$ PSF = 4.13 SQ FT

* ADD 1'-0" x 3'-0" BURIED FOOTING & (3) # 4 DOWELED REBAR

3 SQ FT + 1 FT x 3 = 7 SQ FT ∴ > AREA REQ'D

∴ OK