Harriott Valentine Engineers Inc.

STRUCTURAL CALCULATIONS

Project:
Paige Garage
3431 74th Avenue Southeast
Mercer Island, WA 98040

Architect:
Patricia Brennan Architects
4515C Bagley Avenue North
Seattle, WA 98103

Structural Engineer:
Harriott Valentine Engineers, Inc.
1932 First Avenue, Suite 720
Seattle, WA 98101
tel. 206-624-4760

8/10/22

SECTION 1: FRAMING

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CRITERIA

FRAMING

roof	dead	asphalt shingles	2.5	live	snow	25.0 psf
garage		1/2" plywood	1.5			
		pre-fabricated trusses	5.0			
		R38 insulation	1.4			
		5/8" gyp. wallboard	2.8			
		slope factor	1.1			
		miscellaneous	1.7 11%			
			16.0 psf			
	total	dead + live	41.0 psf			

walls	hardie plank siding (7" exposure)	2.8	
	1/2" plywood	1.5	
	2x6 @ 16"oc	1.7	
	R21 insulation	0.8	
	1/2" gyp. wallboard	2.2	
	miscellaneous	1.0	10%
		10.0	psf

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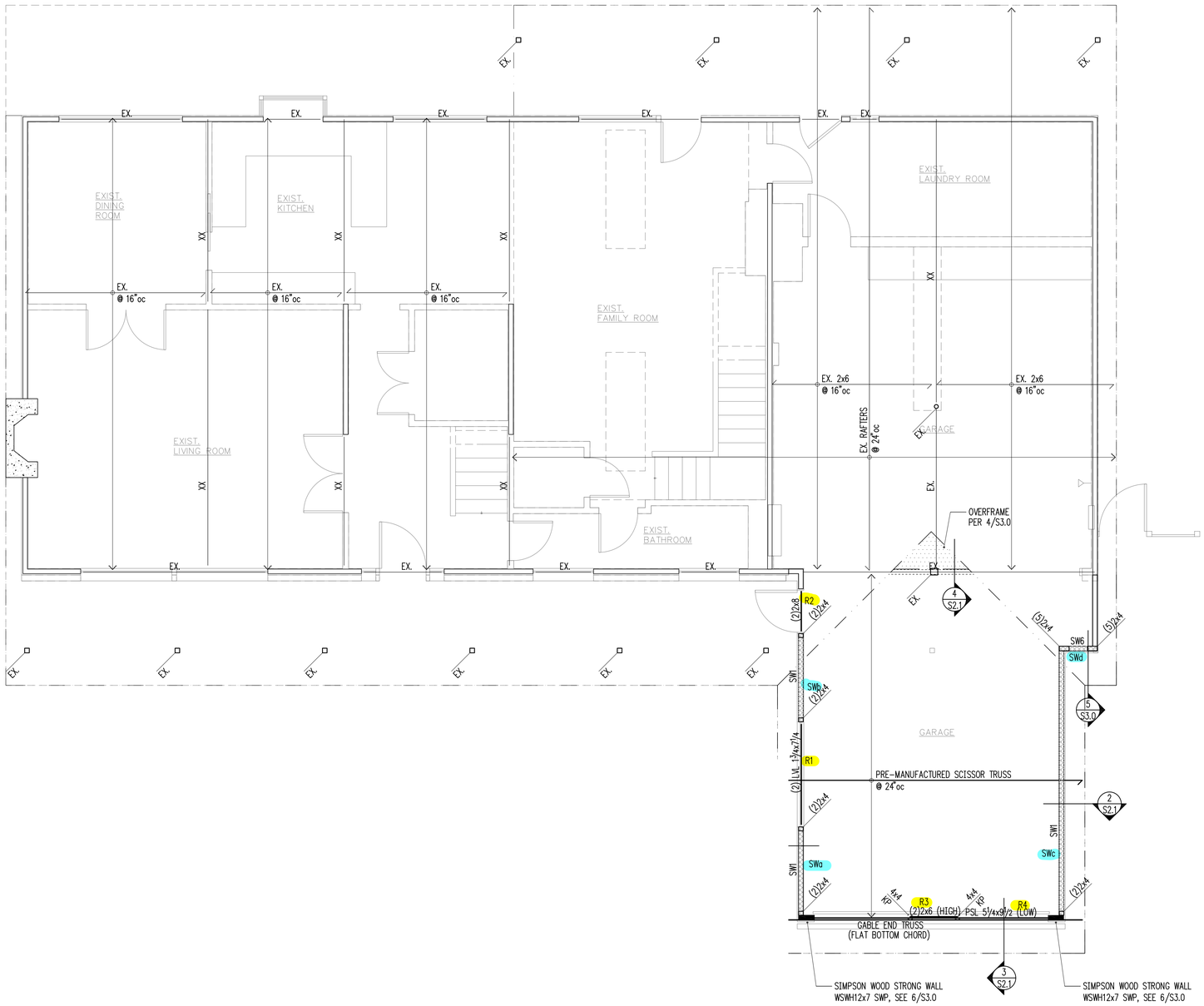


MEMBER MAPPING

Garage Roof			
Member Name	Results	Current Solution	Comments
R1	Passed	2 piece(s) 1 3/4" x 7 1/4" 2.0E Microllam® LVL	
R2	Passed	2 piece(s) 2 x 8 HF No.2	
R3	Passed	2 piece(s) 2 x 6 HF No.2	
R4	Passed	1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL	

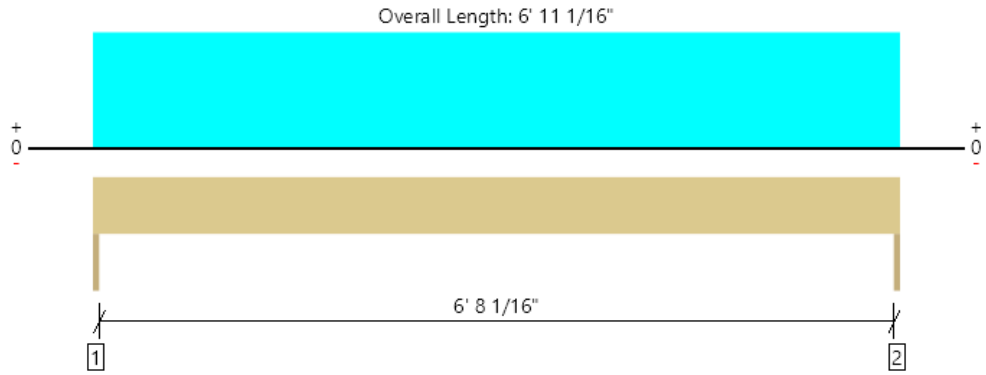
ForteWEB Software Operator	Job Notes
Elizabeth Ihrig Harriott Valentine Engineers (608) 228-0708 eihrig@harriottvalentine.com	

Weyerhaeuser

8/9/2022 6:17:00 PM UTC
ForteWEB v3.4
File Name: Paige Garage

Garage Roof, R1

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



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)	2828 @ 0	3806 (1.50")	Passed (74%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2232 @ 8 3/4"	5544	Passed (40%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4894 @ 3' 5 1/2"	8182	Passed (60%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.128 @ 3' 5 1/2"	0.231	Passed (L/648)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.212 @ 3' 5 1/2"	0.313	Passed (L/392)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (5/16").
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	1119	1709	2828	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1119	1709	2828	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 11" o/c	
Bottom Edge (Lu)	6' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 11 1/16"	N/A	7.4	--	
1 - Uniform (PSF)	0 to 6' 11 1/16"	19' 9"	16.0	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

ForteWEB Software Operator	Job Notes
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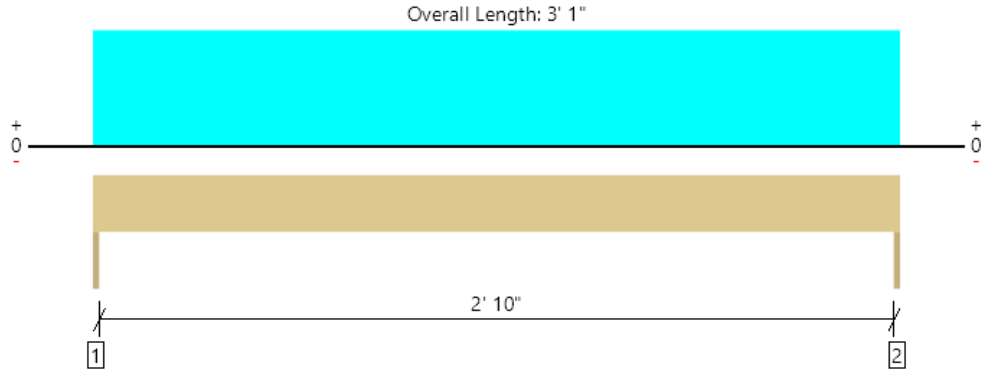
Garage Roof, R2
2 piece(s) 2 x 8 HF No.2



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)	1257 @ 0	1823 (1.50")	Passed (69%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	662 @ 8 3/4"	2501	Passed (26%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	969 @ 1' 6 1/2"	2569	Passed (38%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.008 @ 1' 6 1/2"	0.103	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.013 @ 1' 6 1/2"	0.154	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
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.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	496	761	1257	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	496	761	1257	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 1" o/c	
Bottom Edge (Lu)	3' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 1"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 3' 1"	19' 9"	16.0	25.0	Default Load

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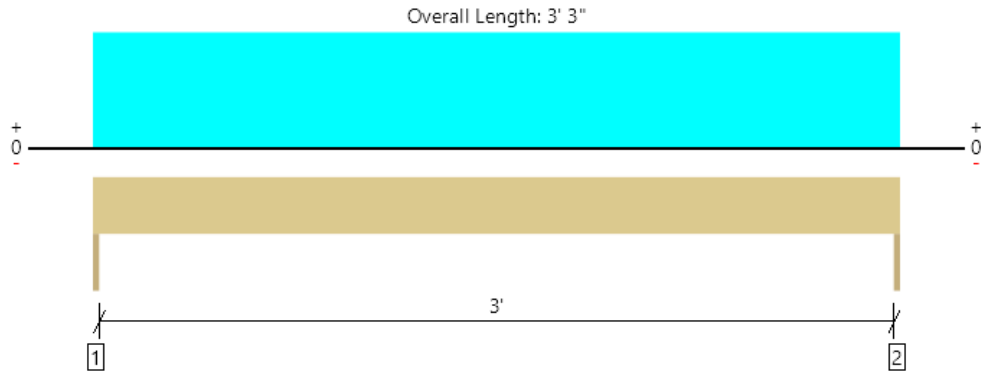
Elizabeth Ihrig Harriott Valentine Engineers (608) 228-0708 eihrig@harriottvalentine.com	

Weyerhaeuser

Garage Roof, R3
2 piece(s) 2 x 6 HF No.2



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)	273 @ 0	1823 (1.50")	Passed (15%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	175 @ 7"	1898	Passed (9%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	222 @ 1' 7 1/2"	1602	Passed (14%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.005 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.008 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
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.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	111	163	273	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	111	163	273	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	4.2	--	
1 - Uniform (PSF)	0 to 3' 3"	4'	16.0	25.0	Default Load

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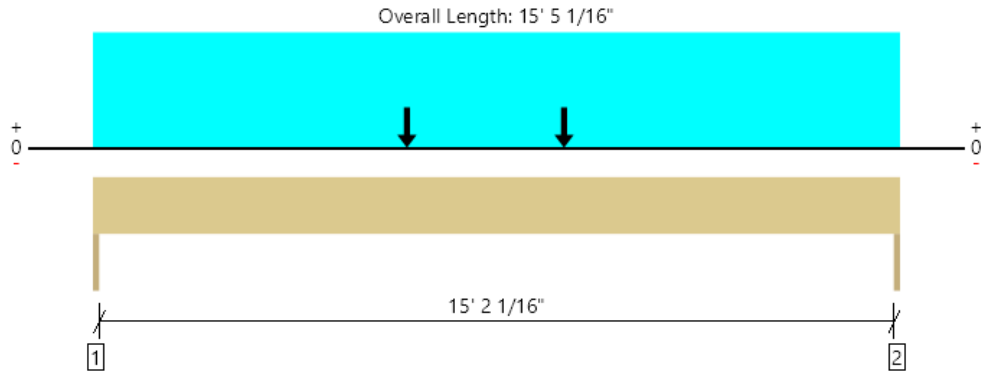
Weyerhaeuser

Garage Roof, R4

1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL



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)	1666 @ 0	4922 (1.50")	Passed (34%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1501 @ 11"	11089	Passed (14%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	7039 @ 7' 9"	22523	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.212 @ 7' 8 7/16"	0.514	Passed (L/873)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.375 @ 7' 8 7/16"	0.771	Passed (L/494)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	728	938	1666	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	722	930	1651	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 5" o/c	
Bottom Edge (Lu)	15' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 15' 5 1/16"	N/A	15.6	--	
1 - Uniform (PSF)	0 to 15' 5 1/16"	4'	16.0	25.0	Default Load
2 - Point (lb)	6'	N/A	111	163	Linked from: R3, Support 1
3 - Point (lb)	9'	N/A	111	163	Linked from: R3, Support 2

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Weyerhaeuser

SECTION 2: LATERAL

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CRITERIA

LATERAL

wind	wind importance factor	2		
	basic wind speed	110	mph	
	wind exposure	B		
	topographical factor (Kzt)	1.40		
seismic	seismic importance factor	1.0		
	latitude	47.579	°	
	longitude	-122.241	°	
	accel. at short periods (Ss)	1.412	g	(from ATC Hazard by Location
	accel. at 1-sec period (S1)	0.491	g	Tool)
	seismic design category	D		
	response modification factor (R)	6.5	Shear Walls	

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

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

Hazards by Location

Search Information

Address: 3431 74th Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5791236, -122.2408244

Elevation: 297 ft

Timestamp: 2022-08-02T18:58:46.925Z

Hazard Type: Wind

ASCE 7-16		ASCE 7-10		ASCE 7-05	
MRI 10-Year	67 mph	MRI 10-Year	72 mph	ASCE 7-05 Wind Speed	85 mph
MRI 25-Year	73 mph	MRI 25-Year	79 mph		
MRI 50-Year	78 mph	MRI 50-Year	85 mph		
MRI 100-Year	83 mph	MRI 100-Year	91 mph		
Risk Category I	92 mph	Risk Category I	100 mph		
Risk Category II	97 mph	Risk Category II	110 mph		
Risk Category III	104 mph	Risk Category III-IV	115 mph		
Risk Category IV	108 mph				

Per City of Mercer Island Design Criteria, Basic Wind Speed, V, is 110 mph

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. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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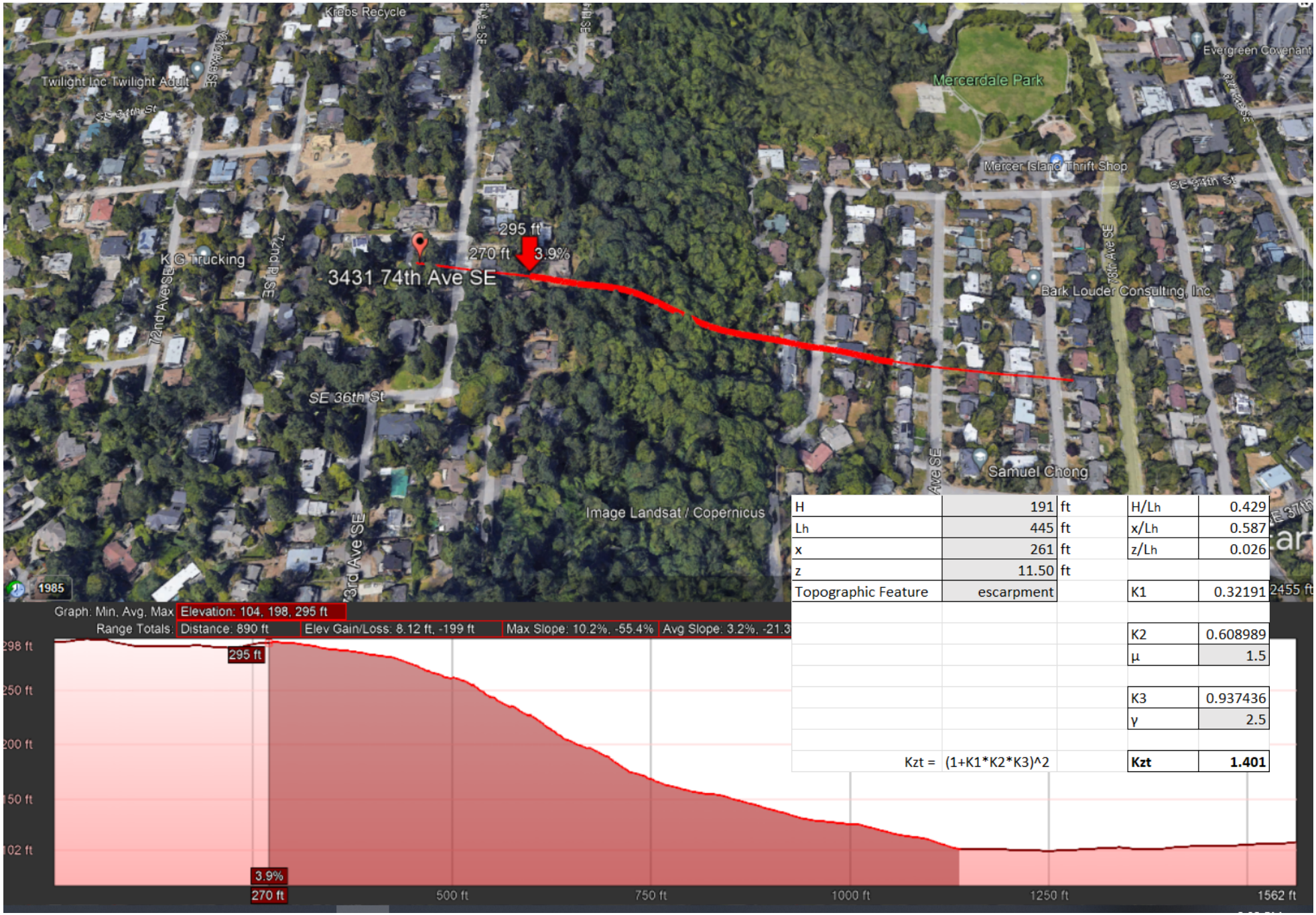
Kzt Calculation



H	191	ft	H/Lh	0.429
Lh	445	ft	x/Lh	0.587
x	261	ft	z/Lh	0.026
z	11.50	ft		
Topographic Feature	escarpment		K1	0.32191
			K2	0.608989
			μ	1.5
			K3	0.937436
			γ	2.5
Kzt =	(1+K1*K2*K3)^2		**Kzt**	**1.401**

Ch. 27 -- MAIN WIND FORCE RESISTING SYSTEM (DIRECTIONAL PROCEDURE)

	BUILDING GEOMETRY		
avg grade to avg height	mean roof height, h (N/S)	11.5	ft
avg grade to avg height	mean roof height, h (E/W)	11.5	ft
dimension perp. to wind	B (N/S)	22.25	ft
dimension Parallel to wind	L (N/S)	16.83	ft
	B (E/W)	16.83	ft
	L (E/W)	22.25	ft
	L/B (N/S)	0.757	
	L/B (E/W)	1.322	
	h/L (N/S)	0.683	
	h/L (E/W)	0.517	
	Parallel to Ridge	E/W	
	Perpendicular to Ridge	N/S	
	Windward Roof Angle, θ	25.60	degrees
	Leeward Roof Angle, θ	25.60	degrees

	CRITERIA		
	Structure Location	Seattle, WA	
	ASCE Edition	7 - 16	
ATC Online Hazards Tool	Elevation above sea level	297	ft
	Risk Cat.	II	
Hazard Tool	Basic Wind Speed, V	110	mph

	WIND LOAD PARAMETERS	
Wind directionality factor	K_d	0.85
	Exposure Cat.	B
Topographic Factor	K_{zt}	1.401
ground elevation factor	K_e	1.00
gust-effect factor	G or G_f	0.85
	Enclosure Classification	Enclosed
internal pressure coefficient	GC_{pi} (+)	0.18
	GC_{pi} (-)	-0.18

Table 26.11-1

TERRAIN EXPOSURE CONSTANTS	
α	7
Z_g	1200

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \ (\text{lb/ft}^2);\ V \text{ in mi/h} \qquad (26.10\text{-}1)$$

VELOCITY PRESSURE				
z (ft)	K_z	q_z (psf)	K_h	q_h (psf)
0-15	0.85	31.4	0.85	31.4

WALL EXTERNAL PRESSURE COEFFICIENTS - N/S		
Wall	C_p	Use with
Windward	0.8	q_z
Leeward	-0.5	q_h
Side	-0.7	q_h

WALL EXTERNAL PRESSURE COEFFICIENTS - E/W		
Wall	C_p	Use with
Windward	0.8	q_z
Leeward	-0.4356	q_h
Side	-0.7	q_h

ROOF EXT. PRESSURE COEFFICIENTS (⊥)	
Side	Cp
Windward	-0.263
	0.2
Leeward	-0.6

ROOF EXT. PRESSURE COEFFICIENTS (II)		
Dist. From Windward Edge	Cp	
0 to h/2	-0.9	-0.18
h/2 to h	-0.9	-0.18
h to 2h	-0.5	-0.18
> 2h	-0.3	-0.18

WIND PRESSURE

$$p = qGC_p - q_i(GC_{pi}) \qquad (27.3\text{-}1)$$

WINDWARD WALL WIND PRESSURE, P - (N/S)				
z (ft)	P_{ext}	P_{int} (+)	P_{int} (-)	p
0-15	21.3	5.6	-5.6	27.0

LEEWARD WALL WIND PRESSURE, P (N/S)

p	-18.97
P_{ext}	-13.33
P_{int} (+)	5.64
P_{int} (-)	-5.64

SIDEWALL WIND PRESSURE, P (N/S)

P	-24.30
P_{ext}	-18.66
P_{int} (+)	5.64
P_{int} (-)	-5.64

WINDWARD ROOF PRESSURE - (N/S)

p (+)	10.97
p (-)	-12.65
P_{ext} (+)	5.33
P_{ext} (-)	-7.01
P_{int} (+)	5.64
P_{int} (-)	-5.64

LEEWARD ROOF PRESSURE - (N/S)

p	-21.63
P_{ext}	-15.99
P_{int} (+)	5.64
P_{int} (-)	-5.64

WINDWARD WALL WIND PRESSURE, P - (E/W)				
z (ft)	P_{ext} (+)	P_{int} (+)	P_{int} (-)	p
0-15	21.3	5.6	-5.6	27.0

LEEWARD WALL WIND PRESSURE, P (E/W)

p	-17.25
P_{ext}	-11.61
P_{int} (+)	5.64
P_{int} (-)	-5.64

SIDEWALL WIND PRESSURE, P (E/W)

P	-24.30
P_{ext}	-18.66
P_{int} (+)	5.64
P_{int} (-)	-5.64

ROOF PRESSURE - (E/W)				
Distance	P_{ext}	P_{int} (+)	P_{int} (-)	p
0 to h/2	-23.99	5.64	-5.64	-29.63
h/2 to h	-23.99	5.64	-5.64	-29.63
h to 2h	-13.33	5.64	-5.64	-18.97
> 2h	-8.00	5.64	-5.64	-13.64

DIAPHRAGM LOADING

N/S Direction

wall trib = 4.165 ft

w = (27 psf + 18.97 psf)(wall trib)*0.6 = 114.8 plf

roof trib = 10.75 ft

w = (10.97 psf + 21.63 psf)*(roof trib)*(5.75/13.31)*0.6 = 90.8 plf

w = 205.6 plf

E/W Direction

wall trib = 4.165 ft

w = (27 psf psf)(wall trib)*0.6 = **67.4 plf**

roof area = 45 ft^2

w = 27 psf)*(roof area)*2/16.83'*0.6 = **86.5 plf**

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

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

Hazards by Location

Search Information

Address:	3431 74th Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.5791236, -122.2408244
Elevation:	297 ft
Timestamp:	2022-08-02T20:56:55.606Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.412	MCE_R ground motion (period=0.2s)
S_1	0.491	MCE_R ground motion (period=1.0s)
S_{MS}	1.694	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.129	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	0.897	Coefficient of risk (1.0s)
PGA	0.604	MCE$_G$ peak ground acceleration

PGA	0.604	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.725	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.412	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.564	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.435	Factored deterministic acceleration value (0.2s)
S1RT	0.491	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.548	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.385	Factored deterministic acceleration value (1.0s)
PGAd	1.177	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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SEISMIC DESIGN

ASCE 7-16
Equivalent Lateral Force Procedure

Occupancy Category	II		Table 1-1
Seismic Design Category	D		Table 11.6-1
Importance Factor	1.00		Table 11.5-1
Site Class	D		Table 20.3-1 **DEFAULT**
S_S	141.20	%g	(from USGS Seismic Hazard Curves, 2002 data)
S_1	49.10	%g	(from USGS Seismic Hazard Curves, 2002 data)
F_a	1.20		Table 11.4-1
F_v	1.81		Table 11.4-2
C_t	0.02		Table 12.8-2
x	0.75		Table 12.8-2
h_n	11.50	feet	(height to highest level)

$S_{MS} = Fa*Ss$	1.6944		Eq. 11.4-1
$S_{M1} = Fv*S1$	0.8882		Eq. 11.4-2
$S_{DS} = (2/3)*S_{MS}$	1.1296	g	Eq. 11.4-3
$S_{D1} = (2/3)*S_{M1}$	0.5921	g	Eq. 11.4-4
Period $T_a = C_t*h_n\text{^}x$	0.1249	s	Eq. 12.8-7
T_o	0.1048	s	per section 11.4.5
T_S	0.5242	s	per section 11.4.5
S_a	1.1296	g	per section 11.4.5

R	6.5	Table 12.2-1
Ωo	3	Table 12.2-1
C_d	4	Table 12.2-1
Section 9.5.5 ok?	Yes	Table 12.6-1

Equivalent Lateral Force Procedure (section 12.8)

Cs	0.1738		Eq. 12.8-2
W, weight	12,633	lb	per table below
Q_E	2,195	lb	Eq. 12.8-1

Vertical Force Distribution (section 12.8.3)
k = 1.00

Level	Hx (ft)	Floor Area (ft2)	Floor Wt. (psf)	Floor Wt. (k)	Wall Length (ft)	Wall Wt. (k)	Total Wt. (k)	WxHx (k-ft)	Cvx (%)	(LRFD) Q_E (k)	(ASD) $0.7Q_E$ (k)
garage roof	11.50	492	16	7.9	92	4.8	12.6	145.3	100.0	2.20	1.537
							12.6	145.3	100.0	2.20	1.54

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DIAPHRAGM LOADING

N/S Direction

0.7Qe =	1.537	kip
w = 0.7Qe / 22' *0.7 =	**48.9**	**plf**

N/S Direction

0.7Qe =	1.537	kip
w = 0.7Qe / 16.83' * 0.7 =	**63.9**	**plf**

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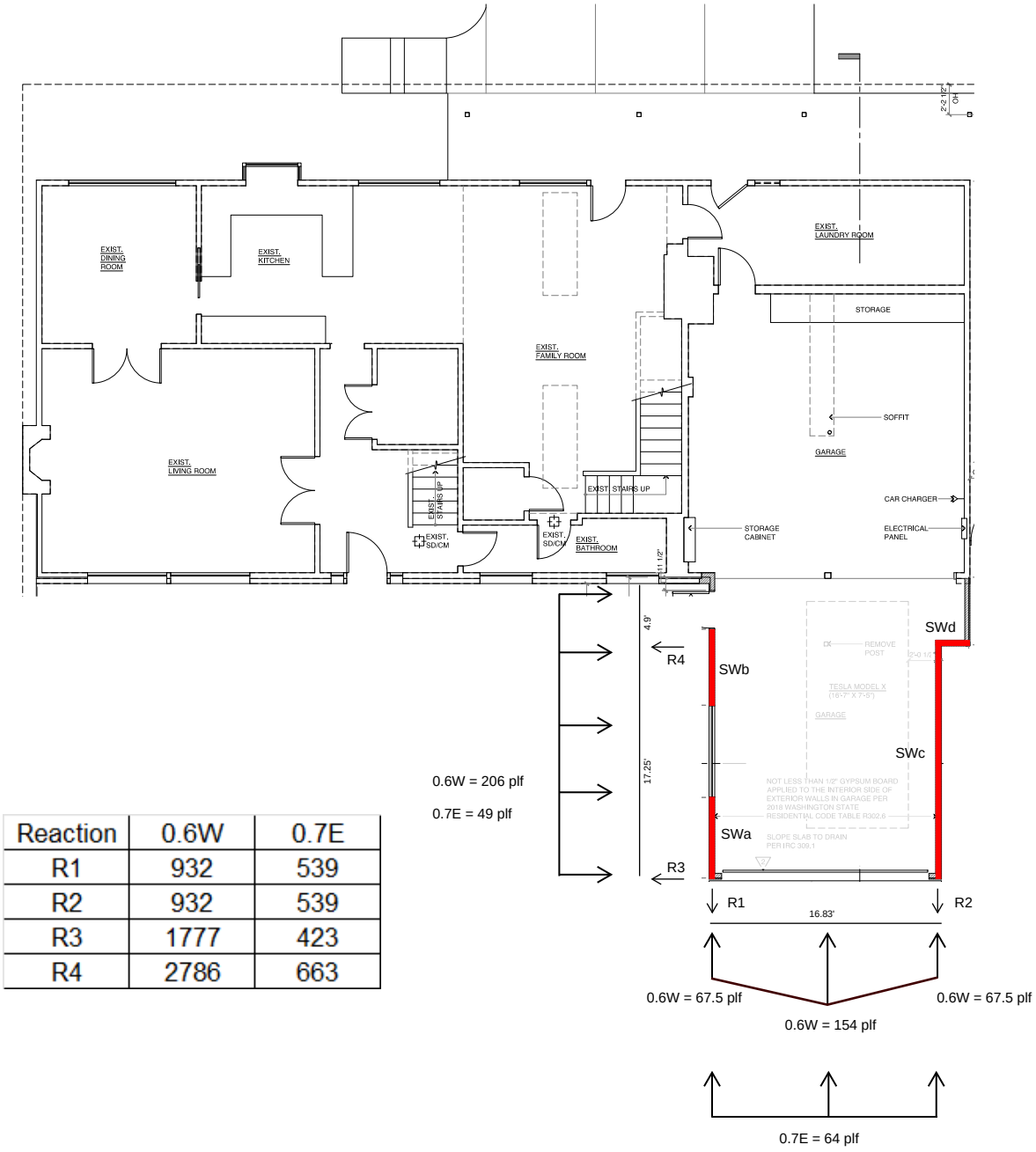
GARAGE ROOF DIAPHRAGM LOADING



Reaction	0.6W	0.7E
R1	932	539
R2	932	539
R3	1777	423
R4	2786	663

LATERAL FORCE DISTRIBUTION (SEISMIC)

WALLS BELOW GARAGE ROOF

East-West

va' = allowable shear values multiplied by 1.25-0.125 h / L
for wall aspect ratios greater than 2:1

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)	SW	h (ft)	h/l	va'	M ot (lbft)	OT (lb)	OT (abv)	OT (total)	DL max (lb)	T (lb)	HD	TL (lb)	C (lb)	POST
SWa	269	0	269	5.60	63	--	8.00	1.43	N/A	2801	500	0	500	396	104	--	1343	1843	--
											500	0	500	396	104	--	1343	1843	--
SWb	269	0	269	5.60	62	--	8.00	1.43	975	2801	500	0	500	397	103	--	1344	1844	--
											500	0	500	397	103	--	1344	1844	--
SWc	539	0	539	17.10	41	--	8.00	0.47	N/A	5601	327	0	327	1211	-884	--	4103	4430	--
											327	0	327	1211	-884	--	4103	4430	--

rho = 1.30

North-South

va' = allowable shear values multiplied by 2w/h
for wall aspect ratios greater than 2:1

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)	SW	h (ft)	h/l	va'	M ot (lbft)	OT (lb)	OT (abv)	OT (total)	DL max (lb)	T (lb)	HD	TL (lb)	C (lb)	POST
SWd	663	0	663	2.42	357	--	8.00	3.31	N/A	6892	2852	0	2852	81	2771	--	196	3048	--
											2852	0	2852	81	2771	--	196	3048	--

rho = 1.30

LATERAL FORCE DISTRIBUTION (WIND)

WALLS BELOW GARAGE ROOF

East-West

va' = allowable shear values multiplied by 1.25-0.125 h / L
for wall aspect ratios greater than 2:1

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)	SW	h (ft)	h/l	va'	M ot (lbft)	OT (lb)	OT (abv)	OT (total)	DL max (lb)	T (lb)	HD	TL (lb)	C (lb)	POST
SWa	466	0	466	5.60	83	SW1	8.00	1.43	N/A	3728	666	0	666	396	269	HDU2	1343	2009	(2)2x4
											666	0	666	396	269	HDU2	1343	2009	(2)2x4
SWb	466	0	466	5.60	83	SW1	8.00	1.43	975	3728	665	0	665	397	269	HDU2	1344	2009	(2)2x4
											665	0	665	397	269	HDU2	1344	2009	(2)2x4
SWc	932	0	932	17.10	54	SW1	8.00	0.47	N/A	7456	436	0	436	1211	-775	HDU2	4103	4539	(3)2x4
											436	0	436	1211	-775	HDU2	4103	4539	(3)2x4

North-South

va' = allowable shear values multiplied by 2w/h
for wall aspect ratios greater than 2:1

WALL	F (lb)	V (abv)	V (total)	L (ft)	v (plf)	SW	h (ft)	h/l	va'	M ot (lbft)	OT (lb)	OT (abv)	OT (total)	DL max (lb)	T (lb)	HD	TL (lb)	C (lb)	POST
SWd	2786	0	2786	2.42	1153	SW6	8.00	3.31	1393	22289	9223	0	9223	81	9142	HDU14	196	9419	(5)2x4
											9223	0	9223	81	9142	HDU14	196	9419	(5)2x4

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Job Name: Paige Garage
Wall Name: Garage Front
Application: Garage Front

Design Criteria:

* 2018 International Bldg Code
* Wind
* 2500 psi concrete
* ASD Design Shear = 900 lbs
* Shearwall Height = 7' with header on top of Strong-Wall

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSWH12x7 SWP	Wood	12	78	3.5	N/A	2 - 1"	40	7345 lb

Actual Shear & Drift Distribution:

Model	Actual Shear (lbs)		Allowable Shear (lbs)		Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSWH12x7 SWP	900	≤	2285	OK	0.39	0.21	0.53

Notes:

1. Strong-Wall High-Strength Wood Shearwalls have been evaluated to the 2021 IBC/IRC. See www.strongtie.com for additional design and installation information.

2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.

3. Check that wall height "H" plus curb height (above slab) will attain overall rough header opening height (top of driveway slab to bottom of header).

4. WSWH Portal Connection Kit WSWH-PK is included with panels less than 100 inches in height and must be ordered separately for panels over 100 inches tall.

5. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam. For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.

6. Panels may be trimmed to a minimum height of 74½".

7. 2-ply headers may be used with Strong-Wall High-Strength Wood Shearwall panels. Minimum 11¼ inch deep nominal header is required with header design by others.

Disclaimer:

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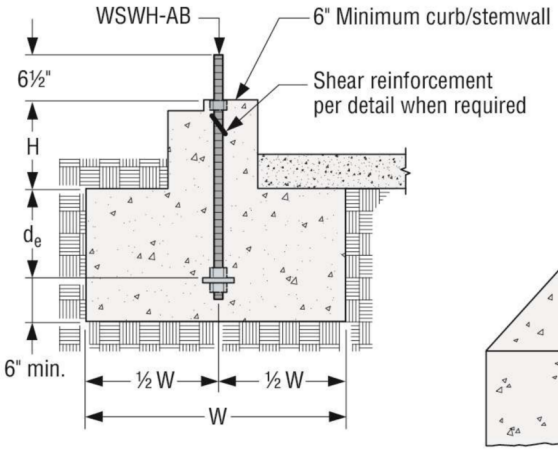
Job Name: Paige Garage
Wall Name: Garage Front
Application: Garage Front

Design Criteria:

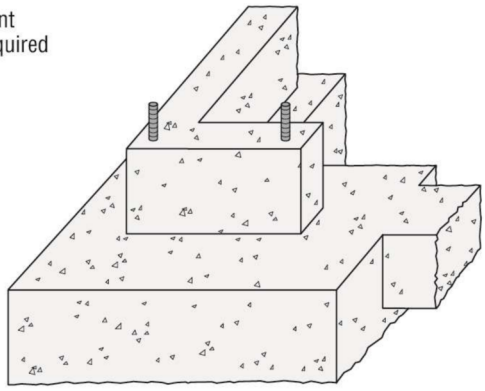
* Slab on grade - Garage curb
* 2018 International Bldg Code
* Wind
* 2500 psi concrete

Anchor Solution Details:

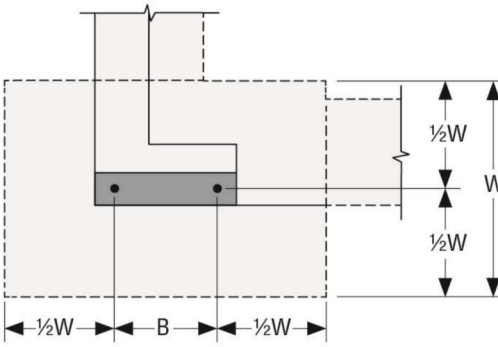
Curb Installation



Curb Section View



Perspective View
(Slab not shown for clarity)



Footing Plan

Anchor Solution Assuming Cracked Concrete Design:

Model	W	de	B	Anchor Bolt	Strength
WSWH12x7 SWP	24	8	8.125	WSWH-AB	Standard

Anchor Solution Assuming Uncracked Concrete Design:

Model	W	de	B	Anchor Bolt	Strength
WSWH12x7 SWP	22	8	8.125	WSWH-AB	Standard

Notes:

1. Anchorage designs conform to ACI 318-19, ACI 318-14 and 318-11 Appendix D with no supplementary reinforcement for cracked and uncracked concrete as noted.
2. Anchorage strength indicates required grade of anchor bolt. Standard (ASTM F1554 grade 36) or High Strength (HS)(ASTM A193 Grade B7).
3. Wind includes Seismic Design Category A and B and detached 1 and 2 family dwellings in SDC C.
4. Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) by others. The registered design professional may specify alternate embedment, footing size or anchor bolt.

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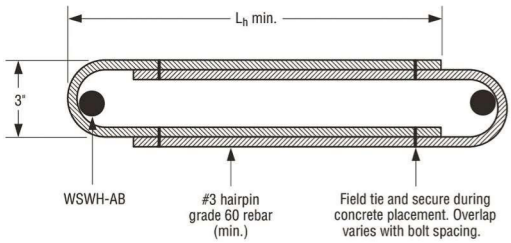
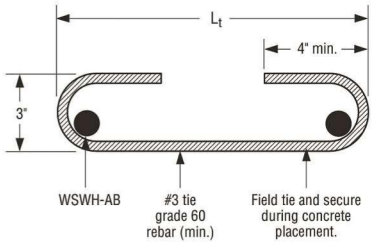
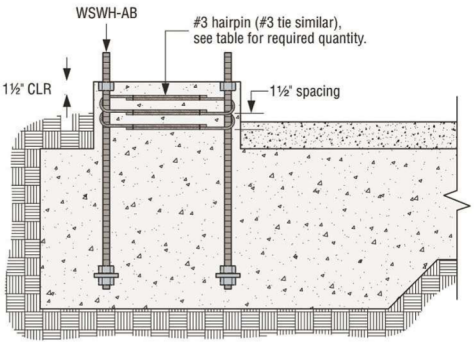


Hairpin Shear Reinforcement



Tie Shear Reinforcement



Hairpin Installation
(Garage curb shown, other footing types similar)

Shear Anchorage Solutions

Strong-Wall High-Strength Wood Shearwall Model No.	L_t or L_h (in.)	Seismic [3]		Wind [4]			
		Shear Reinforcement	Minimum Curb/ Stemwall Width (in.)	Shear Reinforcement	Minimum Curb/ Stemwall Width (in.)	ASD Allowable Shear Load, V (lb.) [7]	
						Uncracked	Cracked
WSWH12	10¼	(1) #3 Tie	6	See Note 7	6	1,080	770
WSWH18	15	(2) #3 hairpins[5,6]	6	(1) #3 hairpin	6	Hairpin reinforcement achieves maximum allowable shear load of the Strong-Wall® WSWH	
WSWH24	19	(2) #3 hairpins[5]	6	(2) #3 hairpins[5]	6		

1. Shear anchorage designs conform to ACI 318-14 Chapter 17 and ACI 318-11 and assume minimum 2,500 psi concrete.
2. Shear reinforcement is not required for interior foundation applications (panel installed away from edge of concrete), or braced wall panel applications.
3. Seismic indicates seismic design category C through F. Detached one- and two-family dwellings in SDC C may use wind anchorage solutions. Seismic shear reinforcement designs conform to ACI 318-14, section 17.2.3.5.3 and ACI 318-11 section D.3.3.5.
4. Wind includes seismic design category A and B and detached one- and two-family dwellings in SDC C.
5. Additional ties may be required at garage curb or stemwall installations below anchor reinforcement per designer.
6. Use (1) #3 hairpin for WSWH18 when standard strength anchor is used.
7. Use (1) #3 tie for WSWH12 when panel design shear force exceeds tabulated anchorage allowable shear load.
8. No. 4 grade 40 shear reinforcement may be substituted for WSWH shear anchorage solutions.
9. Concrete edge distance for anchors must comply with ACI 318-14 section 17.7.2 and ACI 318-11 section D.8.2.
10. The designer may specify alternate shear anchorage.

STRONG-WALL® WSWH SHEAR ANCHORAGE SCHEDULE AND DETAILS

SECTION 3: FOUNDATION

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FOOTING WITH COMBINED AXIAL AND FLEXURAL LOADS

Footing Under SWd

Sizes and Loads:

superstructure:			
frame	116	lb	
footing:			
length	12.00	ft	(along same axis as applied moment)
width	2.50	ft	(perpendicular to applied moment)
depth	1.17	ft	
weight	5,075	lb	
soil abv.	1,200	lb	
total R =	6,391	lb	
M =	22,289	lbft	
e =	3.49	ft	
B/6 =	2.00	ft	

Bearing Pressures:

Reaction is OUTSIDE kern.

(Use these results)			(Do not use these results)		
x =	2.51	ft	fa =	213	psf
			fb =	371	psf
fp =	678	psf	fp =	585	psf
Fa =	2,000	psf	Fa =	2,000	psf

Stability:

Mot =	22,289	lbft	(using 0.6W, per ASD Load Combinations)
Mr =	23,008	lbft	(using 0.6D, per ASD Load Combinations)

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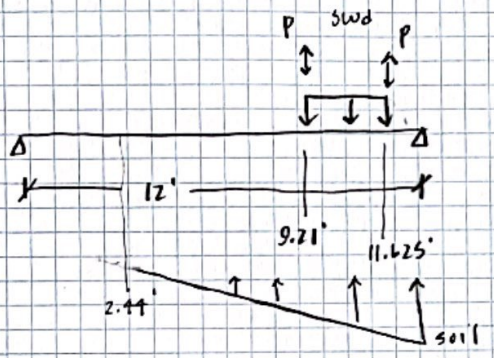
FOOTING REINFORCEMENT DESIGN

→ ftg. under SWd



- loading

SWd:
- $w = (110psf)(8') + (116psf)(2') \} (25psf)(2')$
- $w_D = 11.2 plf$
- $w_S = 50 plf$
- $P_W = 9.223 K / 0.6 = 15.37 K$
- $P_E = 2.852 K / 0.7 = 4.074 K$

$f_p = 684 psf \rightarrow$ ftg width = 2.5'

$w_{soil} = 1710 plf$

- demand

$M_u = 27.65 K\text{-ft}$

$V_u = 12.36 K$

* see Footing Reinf. design spreadsheet

use (5) #4 bars, top & bott

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PAIGE GARAGE Project

Date

Page ____ of ____

DESIGN FOOTING W/ MOMENT AND SHEAR DEMAND

Footing below SWd

Footing Criteria

W	30	in
D	14	in
f'c	2.5	ksi
cover,f	3.5	in
d	10.5	in
f_y	60	ksi
β	0.85	

Demands

$M_{u,a}$	27.62	k-ft
$V_{u,a}$	12.36	k
M_u	530.304	k-in
V_u	19.776	k

Design Criteria

Φ_b	0.9
Φ_v	0.75

Flexural Reinforcement Design

	M_n	589.2267	k-in
	μ_n	0.084	
	ω_d	0.088	
	ρ	0.003	
	$A_{s,req'd}$	0.978	in.2
	$A_{s,min}$	1.05	in.2
	ρmax	0.006779	
	Bar	#4	
	A_{bar}	0.2	in.2
	no. bar	5	
NOT GOOD	$A_{s,actual}$	1	in.2
	$A_{s,min}$	1.05	in.2
	ρactual	0.002381	
OK	ρmax	0.011298	
	a	0.94	in.2
	$\Phi*M_{n,actual}$	541.59	k-in

Shear Strength

V_c	31.5
$\Phi*V_c$	23.625

No Shear Reinf.Req'd

Min. Shear Reinf.
Beam Integral w/ Slab:
h <= 0.6*b
h <= 24 in

Min. Shear Reinf. Not Req'd

DESIGN SUMMARY

M_u	530.304	k-in
V_u	19.776	kip
$\Phi*M_n$	541.59	k-in
$\Phi*V_c$	23.625	kip

Use (5) #4 bars, Top & Bott.
no shear reinf. Req'd

Company:		Date:	8/4/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description:
Location:
Fastening description:

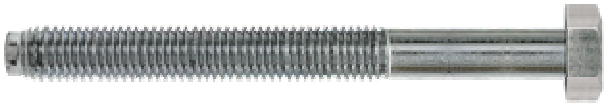
2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 55
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 4.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.38
C_{min} (inch): 1.11
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
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: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 5/8"Ø Heavy Hex Bolt, F1554 Gr. 55

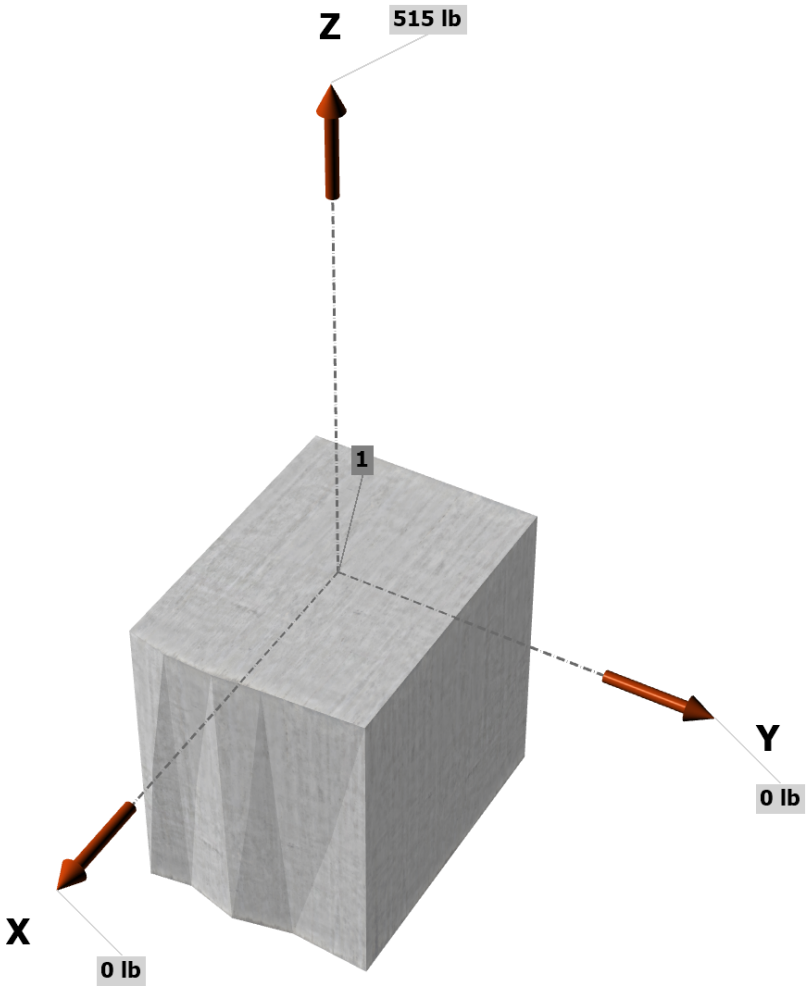


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

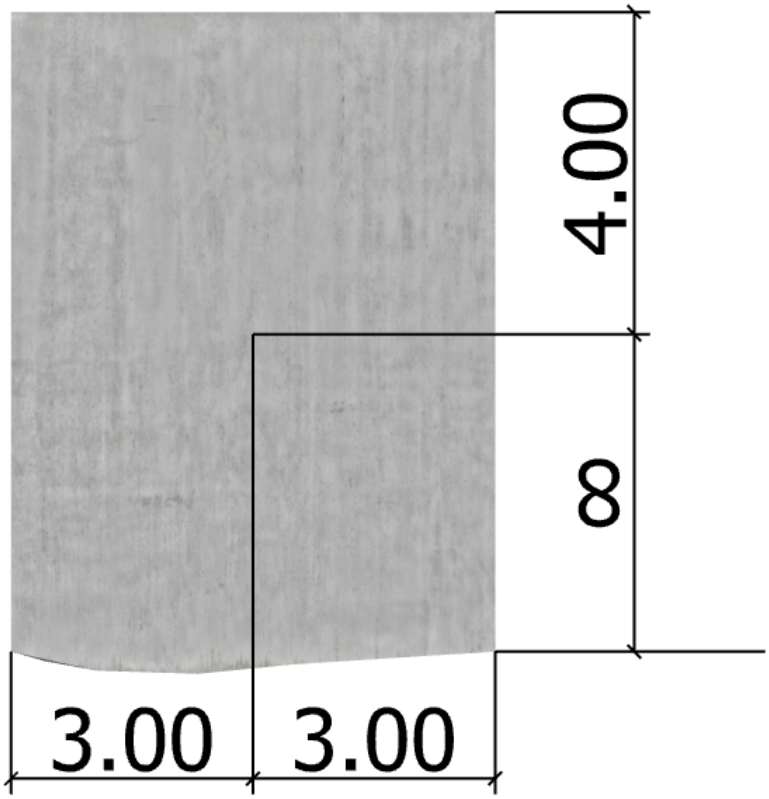
Strength level loads:

N_{ua} [lb]: 515
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	515.0	0.0	0.0	0.0
Sum	515.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 515
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)
16950	0.75	12713

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	2.667	5226

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
48.00	64.00	3.00	0.925	1.00	1.000	5226	0.70	2538

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

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.67	2500	0.70	9394

11. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	515	12713	0.04	Pass
Concrete breakout	**515**	**2538**	**0.20**	**Pass (Governs)**
Pullout	515	9394	0.05	Pass

5/8"Ø Heavy Hex Bolt, F1554 Gr. 55 with hef = 4.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

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

Company:		Date:	8/4/2022
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description:
Location:
Fastening description:

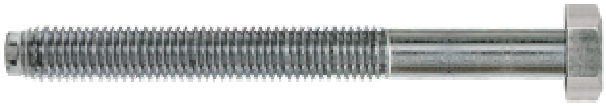
2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.75
C_{min} (inch): 1.44
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 14.00
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: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 1"Ø Heavy Hex Bolt, F1554 Gr. 36

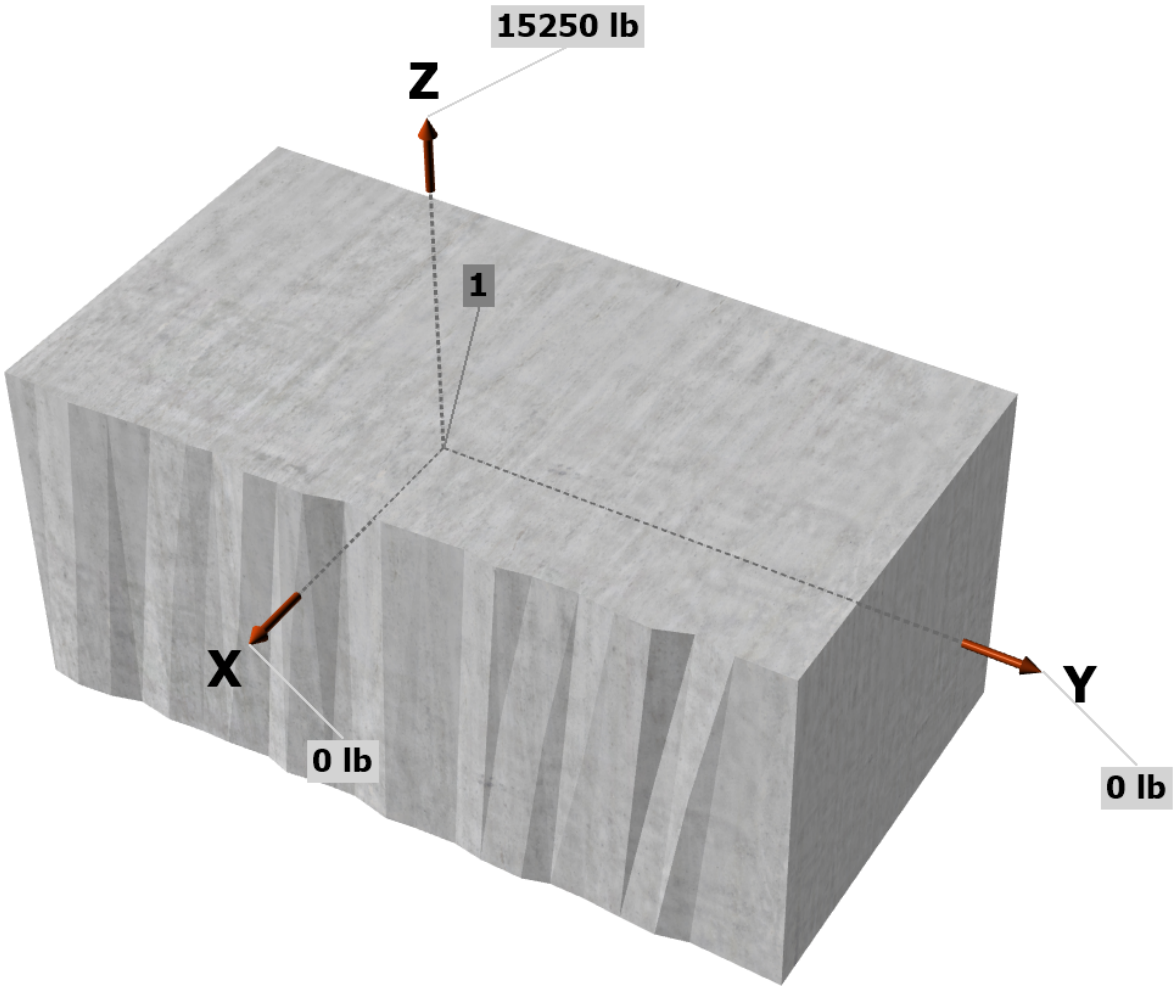


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

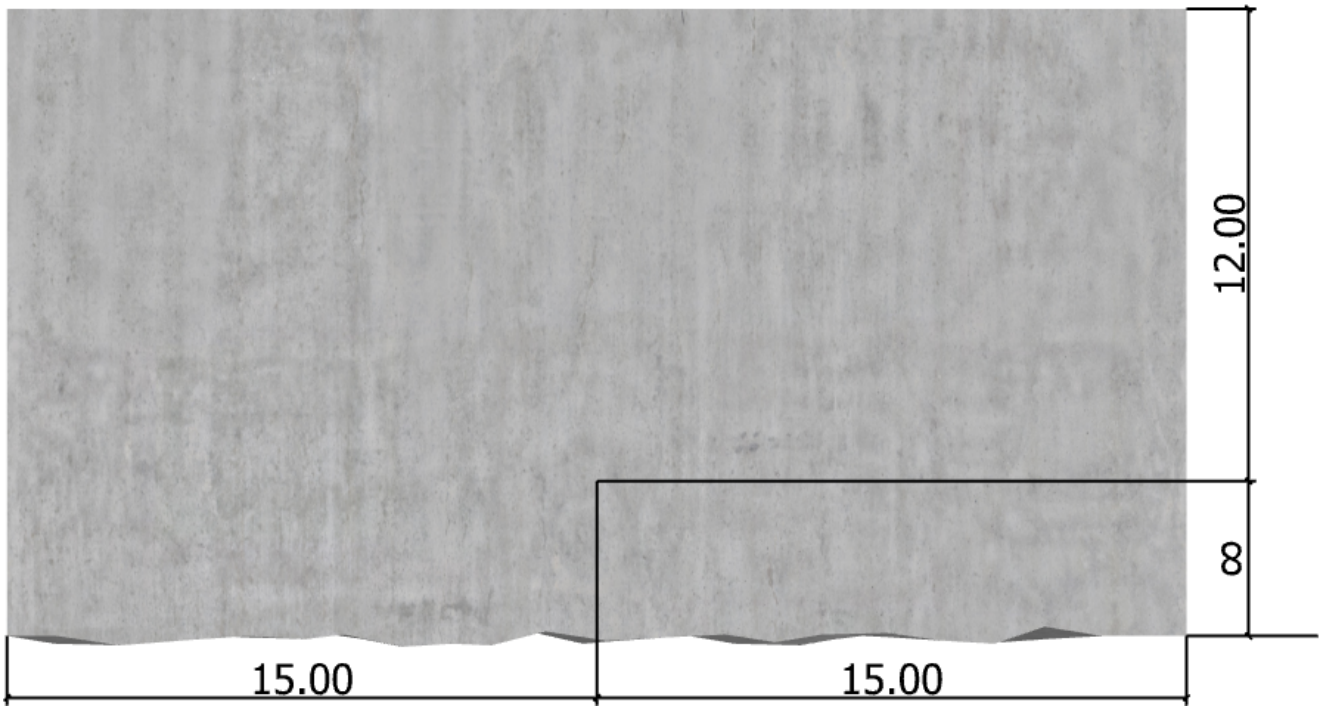
Strength level loads:

N_{ua} [lb]: 15250
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	15250.0	0.0	0.0	0.0
Sum	15250.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 15250
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)
35150	0.75	26363

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	8.000	27153

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

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
576.00	576.00	12.00	1.000	1.00	1.000	27153	0.70	19007

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

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	1.50	2500	0.70	21014

11. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	15250	26363	0.58	Pass
Concrete breakout	**15250**	**19007**	**0.80**	**Pass (Governs)**
Pullout	15250	21014	0.73	Pass

1"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

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