BSE

Brienen **S**tructural **E**ngineers, P.S.

PRCTI20241387

Centris Data Center
South Utility Yard Switchgear Building
1023 39th Avenue South East
Puyallup, WA 98374

City of Puyallup Building
REVIEWED FOR COMPLIANCE
SKinnear
12/24/2024
1:30:07 PM

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

Structural Calculations

Calculations required to be provided by the Permittee on site for all Inspections

PAUL J. BRIENEN
STATE OF WASHINGTON
32114
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Project Number 24201.5
11/08/2024

Phone: (206) 397-0000 1316 Central Avenue South, Suite 200, Kent, WA 98032 www.bse-ps.com

Vertical Design

Building Weights

-Roof

Metal Roofing	1.5 psf
Wood Sheathing	2.2 psf
Metal Stud Framing @ 24"oc	3.8 psf
Insulation	2.5 psf
Conduit	12.7 psf
Misc.	1.3 psf
Total Dead Load	24 psf
Total Live Load	20 psf
Total Snow Load	25 psf

-Exterior Walls

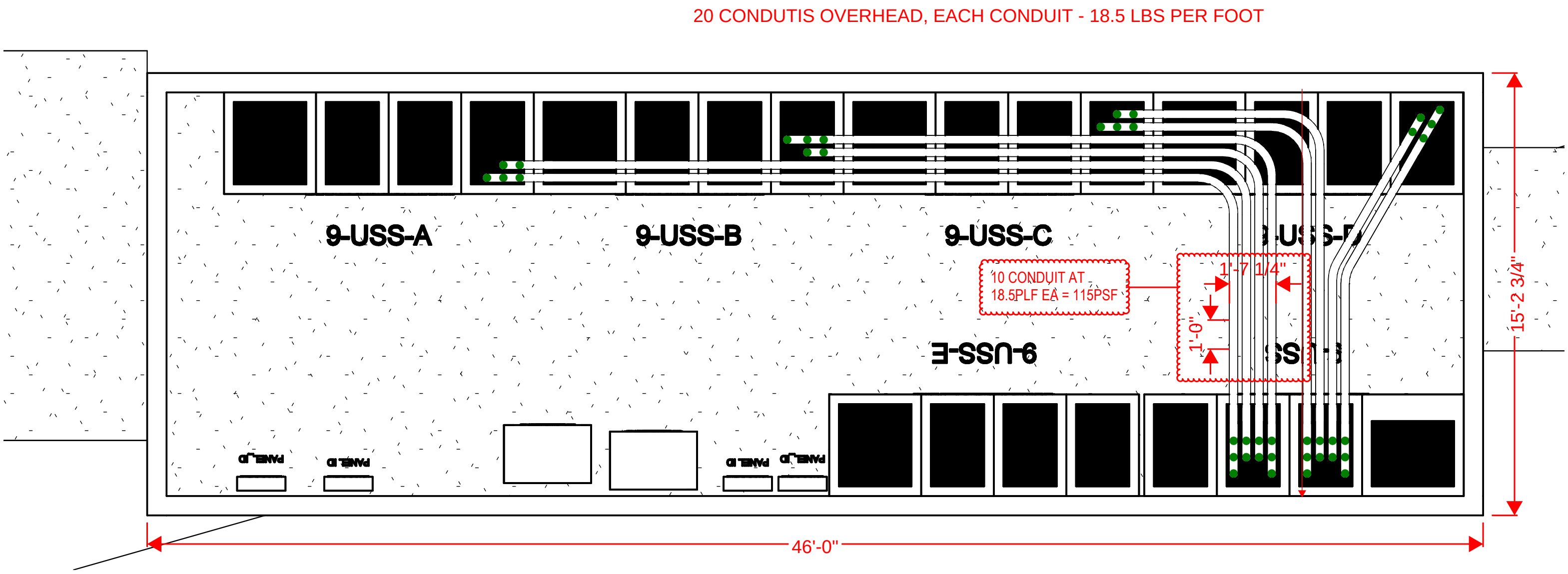
Panel Siding	2.4 psf
Wood Sheathing	1.5 psf
Metal Stud Framing	2.0 psf
Insulation	2.2 psf
Gyp Board	2.8 psf
Total Dead Load	10.9 psf ... USE 11psf
Lateral Live Load	5 psf
Wind Load	16 psf

BSE

B rienen **S** tructural **E** ngineers, P.S.

Conduit Weight Calculation

Conduit Plan



Total Conduit Length

- grouped into four groups of 5

L =

(5) * (25'-8" + 10'-2")

+ (5) * (16'-8" + 11'-6")

+ (5) * (7'-6" + 11'-10")

+ (5) * (6'-10" + 6'-5")

= 483ft

Conduit Weight

w_conduit = 18.5plf

w_total = (w_conduit) * (L) = (18.5plf) * (483ft) = 8936 lbs

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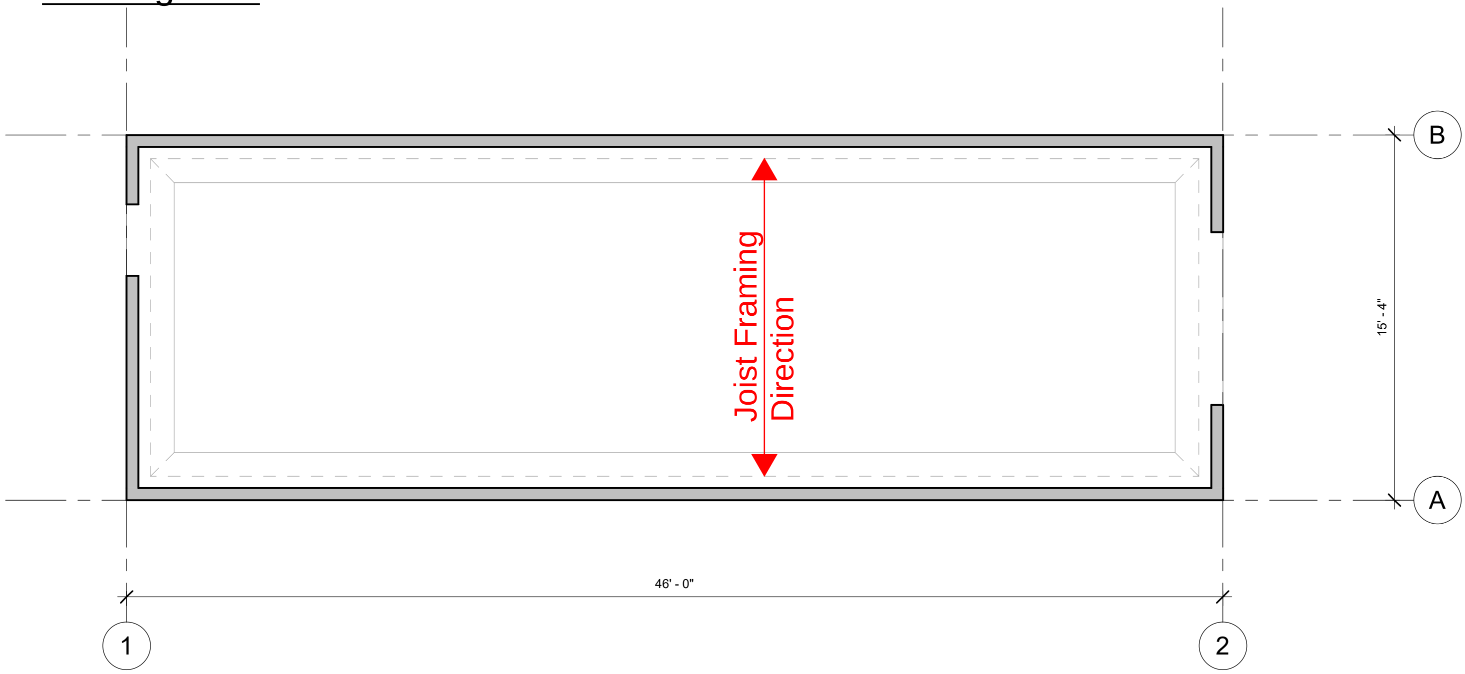
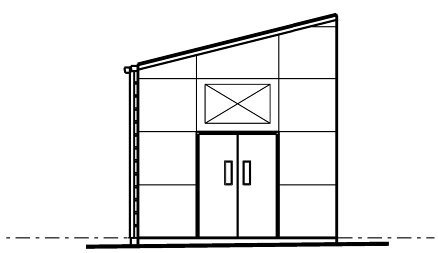
Brienen Structural Engineers, P.S.

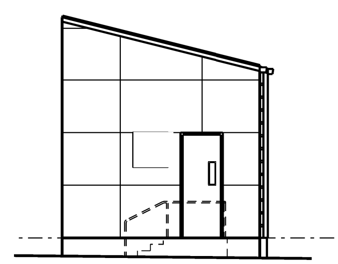
Building Plan



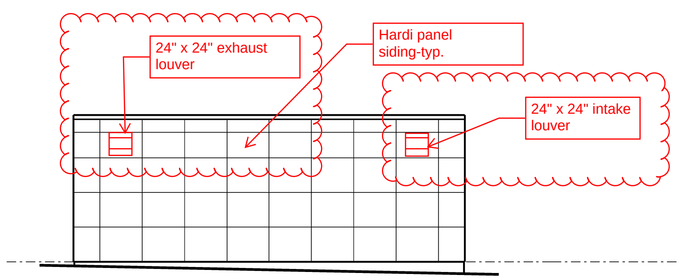
Building Elevations



ELEVATION 8
EAST



ELEVATION 10
WEST



NORTH ELEVATION

Phone: (206) 397-0000

1316 Central Avenue S, Suite 200, Kent, WA 98032

www.bse-ps.com

WIND LOAD - ASCE 7-16

98 mph, Exposure B, Mean Roof Height = 13.9 ft

K_{zt} at Base = 1

K_d = 0.85 , Roof Slope 14.04 degrees (3:12)

Enclosed Building, GC_{pi} = 0.18

(Wind Loads Shown are for Alternate Basic Load Combinations Using Allowable Stress Design and are Multiplied by a Factor of 0.6 to convert to ASD)

WALL COMPONENTS AND CLADDING per ASCE7-16 Figure 30.3-1

Tributary Area (ft2)	GCp by Zone Zone 4 (+/-)	Zone 5 (+/-)
10 ft^2	1.00/-1.10	1.00/-1.40
50 ft^2	0.88/-0.98	0.88/-1.15
500 ft^2	0.70/-0.80	0.70/-0.80

Height z (ft)	K_z	K_{zt}	K_e	q_z (psf)	Tributary Area (ft2)	Wind Pressures (psf) by Zone () Windward (4,5)	Leeward (4)	Leeward (5)
0 - 13.9	0.70	1.00	1.00	14.64	10	10.4	-11.2	-13.9
					50	9.6	-10.2	-11.7
					500	9.6	-9.6	-9.6

ROOF COMPONENTS AND CLADDING - MONOSLOPE ROOF ASCE7-16 Figure 30.3-5B

K_h = 0.70; K_{zt} at roof = 1.00; K_e = 1.00; q_h = 14.64 psf

	Positive Pressure, p (psf)				Negative Pressure, p (psf)			
	A=10		A=100		A=10		A=100	
Zone	GC_p	p	GC_p	p	GC_p	p	GC_p	p
1	0.40	9.60	0.30	9.60	-1.30	-13.00	-1.10	-11.24
2	0.40	9.60	0.30	9.60	-1.60	-15.64	-1.20	-12.12
3	0.40	9.60	0.30	9.60	-2.90	-27.06	-2.00	-19.15
2'	0.40	9.60	0.30	9.60	0.00	0.00	0.00	0.00
3'	0.40	9.60	0.30	9.60	0.00	0.00	0.00	0.00

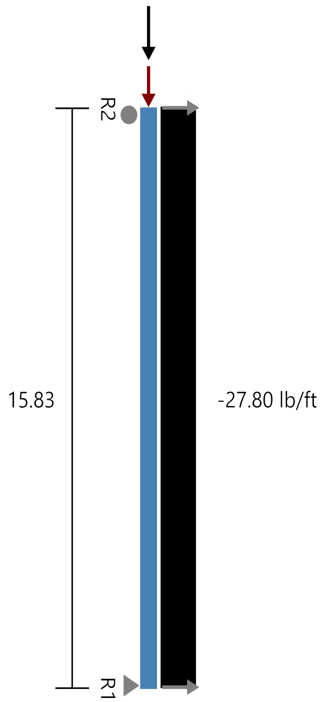
Project Name: Centeris

Model: 15'-10" Bearing Wall

Code: 2012 NASPEC [AISI S100-2012]

Date: 10/16/2024

Simpson Strong-Tie® CFS Designer™ 5.0.1.0



Section :	600S200-54 (50 ksi) @ 24" o.c. Single C Stud (punched)				
Maxo =	2532.9 ft-lb	**Va =**	2822.9 lb	**I =**	3.32 in^4

Loads have not been modified for strength checks
Loads have been multiplied by 0.70 for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	60.0", 60.0"	60.0", 190.0"	LSUBH3.25 (Min)	0.80

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	-220.0	--Slip Track Design, Ref Connectors--				NO
R1	-220.0	--Stud/Track Design, Ref Connectors--				NO

Gravity Load

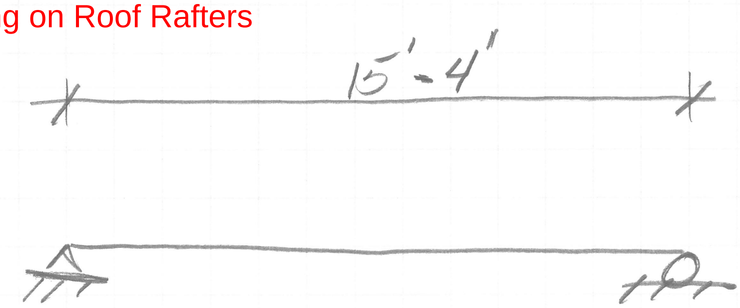
Type	Load (lb)
Uniform	22.00plf (Span)
P1y	2427.00lb @ 15.83ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	2775.3(c)	5499.4(c)	50%	KΦ=0.00 lb-in/in Max KL/r = 82
	Max. Shear, lbs	220.0	1947.4	11%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	870.8	2281.9	38%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	870.8	2332.3	37%	
	Shear/Moment	0.34	1.00	34%	Shear 0.0, Moment 870.8
	Axial/Moment	0.94	1.00	94%	Axial 2608.4(c), Moment 869.3
	Deflection Span, in	0.281	--meets L/677--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	-220.0	0.0	600T250-54 (50) & Anchorage Designed by Engineer	91.77 %	NA
R1	-220.0	2775.3	600T125-54 (50) & Anchorage Designed by Engineer	23.65 %	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Conduit Loading on Roof Rafters



$DL = 24\,psf - 13\,psf = 11\,psf$

$Conduit = 18.5\,plf$

$Length = 15.33\,ft$

$Conduit\ Length = 15.33\,ft - (1.5' + 1.5') = 12.33\,ft$

$Conduit\ Wt = (10)\cdot(18.5\,plf)\cdot(12.33\,ft) = 2281\#$

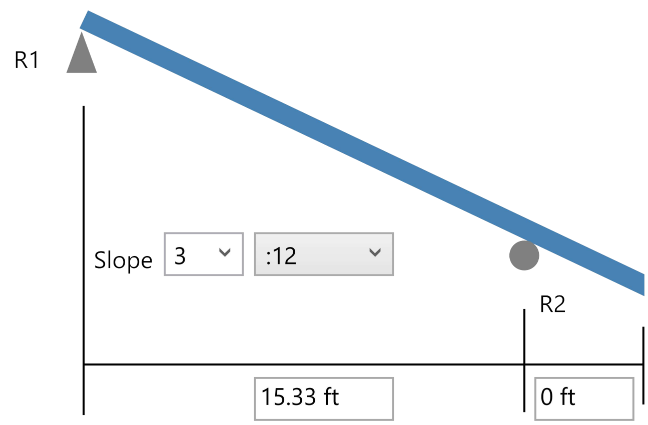
$= (2281\#)\,/\,(15.33\,ft)\,/\,(2ft) = 74.4\,plf$

$\rightarrow$ USE 75psf

$Total\ DL = 11\,psf + 75\,psf = 86\,psf$

Check DL + SL:
RXN = (11psf + 25psf) * (2ft * 15.333/2) = 552#

...adding conduit
RXN = 552# + (2281# / 2) = 1692#



Section :	(2) 1000S250-68 (50 ksi) @ 24" o.c. Back-To-Back C Stud (punched)		
Maxo =	13814.3(ft-lb)	**Va =** 6690.8	**I =** 31.481

Bracing, Interconnection and Distortional Buckling Parameters

	Span	Overhang
Flexural Bracing	96 in	None
Axial Bracing	NA	None
Distortional Buckling, K-phi	0 lb-in/in	0 lb-in/in
Distortional Buckling Bracing, Lm	None	None
Interconnection Req'mt	L/6 = 31.6	

See AISI S100 D1.1 for Add'nl Requirements

Load Cases

	Span (psf)	Overhang (psf)
Dead Load	86	NA
Live Load	20	NA
Snow Load	25	NA
Inward Wind Load	9.6	NA
Outward Wind Load	-27.1	NA

Load Combinations

LC Number	Dead	Live	Snow	Inward Wind	Outward Wind
1	1	1	0	0	0
2	1	0	1	0	0

3	1	0	0	0.6	0
4	1	0.75	0	0.45	0
5	1	0	0.75	0.45	0
6	0.6	0	0	0	0.6

Reactions

	Vertical				Horizontal			
	Max Rxn (lb)	**Load Comb.**	**Min Rxn (lb)**	**Load Comb.**	**Max Rxn (lb)**	**Load Comb.**	**Min Rxn (lb)**	**Load Comb.**
R1	1742.20	2	581.69	6	44.15	3	-124.63	6
R2	1742.20	2	550.53	6	0.00	1	0.00	1

Rafter Flexural and Deflection

Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
6677	11355	0.59	2	L/587	2	L/587	2

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	1690.2	2	6.00	7126.8	14253.6	0.10	2	NO
R2	1690.2	2	6.00	7126.8	14253.6	0.10	2	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	1690	2	1.000	0.25	0.00	0.25	2	N/A	N/A
R2	1690	2	1.000	0.25	0.00	0.25	2	N/A	N/A

USE SIMPSON S/DHUTF DRYWALL HANGER TO SUPPORT EA RAFTER @ 1-HR RATED WALL

S/DHUTF Drywall Hangers

SIMPSON Strong-Tie

The S/DHUTF top-mount hanger is designed to carry joist loads to a CFS stud wall through two layers of ⅝" gypsum board (drywall). This hanger installs after the drywall is in place and comes in sizes that accommodate most typical joists used in multi-family and commercial construction.

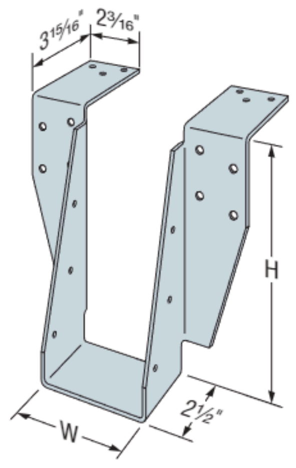
Material: 97 mil (12 ga.)

Finish: Galvanized (G90)

Installation:

- Use all specified fasteners; see General Notes
- Hanger to be framed in-line with vertical wall stud
- Drywall is installed first
- Wall top track must be restrained to counteract load eccentricity from hanger

Codes: See p. 13 for Code Reference Key Chart



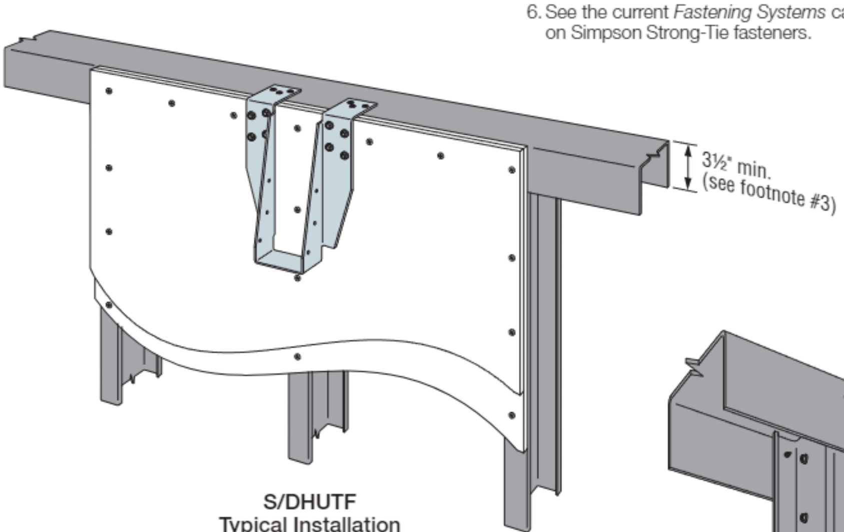
S/DHUTF
US Patent: 9,394,680

Model No.	Dimensions (in.)	
	W	H
S/DHU1.68/8TF	1 11⁄16	8
S/DHU1.68/10TF		10
S/DHU1.68/12TF		12
S/DHU2.1/8TF	2⅛	8
S/DHU2.1/10TF		10
S/DHU2.1/12TF		12
S/DHU2.56/8TF	2 9⁄16	8
S/DHU2.56/10TF		10
S/DHU2.56/12TF		12

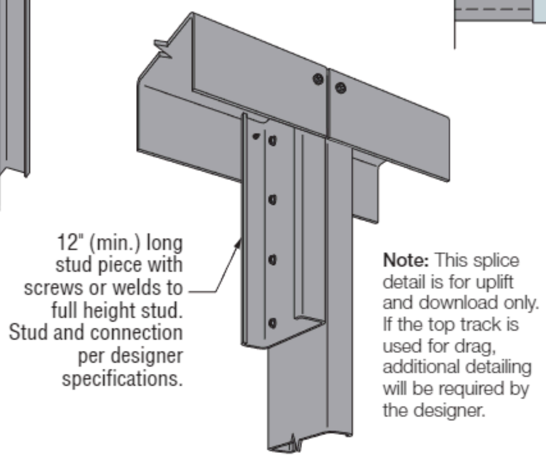
S/DHUTF Allowable Loads (lb.)

Model	Fasteners[6]			Allowable Load (lb.)		Code Ref.
	Top	Face	Joist	Uplift	Down	
S/DHUTF	(6) #10	(8) #14 x 2"	(3) #10	1,230	1,700	—

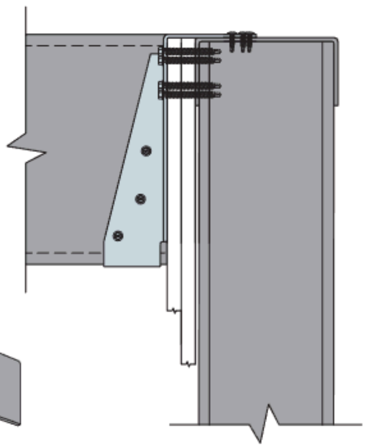
1. Designer shall ensure that the joist member adequately transfers load to the hanger.
2. Tabulated loads assume (2x) ⅝" Type X drywall attached per IBC.
3. Wall studs designed per designer specifications. At a minimum, the assembly must consist of 600T350-68, Gr. 50 ksi top track and 600S162-43, Gr. 33 ksi wall studs spaced at a maximum of 24" o.c.
4. Tabulated loads are based on testing with full bearing of 3 15⁄16" hanger top flange. The minimum joist gauge is 54 mil (16 ga.).
5. S/DHUTF hanger can be installed ¾" max. from the center of the vertical stud per the in-line framing specifications of the AISI General Provisions without load adjustment.
6. See the current *Fastening Systems* catalog at **strongtie.com** for more information on Simpson Strong-Tie fasteners.



S/DHUTF
Typical Installation

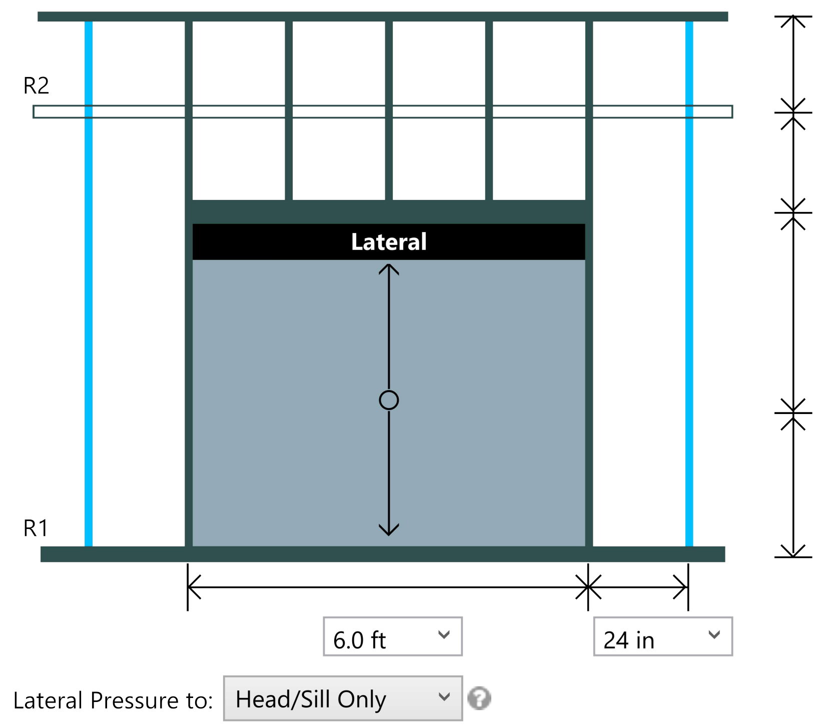


Typical Top Track Splice



Typical Installation Over (2) Layers of ⅝" Drywall

Joist Framing Connectors



Design Loads

Wind Selection :	**C&C Wind, Leeward (5)**
Trib. Area : Span :	**Length^2/3**
Wall Lateral Pressure :	**-11.3 psf**
Parapet Lateral Pressure :	
RO Lateral Pressure :	**Head/Sill Only**
Lateral element force multiplier	
Strength :	**1.0**
Deflection :	**0.7**
Header:	**Strongback, Lateral Track**
Gravity Load at Header:	**11 psf**
Additional Pt. Load ea. Stud :	**256 lbs**

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S200-54(50), Single	Full	48 in	48 in	0	None	N/A
Vertical Header	600S200-54(50), Single	Full	N/A	N/A	0	None	N/A
Lateral Header	600T125-33(33), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S200-54(50), Single	769.0	75	779.7	276.9	-327.7	-276.9
Vertical Header	600S200-54(50), Single	N/A	N/A	724.5	483.0	N/A	483.0
Lateral Header	600T125-33(33), Single	N/A	N/A	368.7	245.8	N/A	245.8

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S200-54(50), Single	L/895	L/0	0.46	0.33	NA	Yes
Vertical Header	600S200-54(50), Single	L/1502	NA	0.32	0.29	No	Yes
Lateral Header	600T125-33(33), Single	L/1597	NA	0.75	0.75	R1, R2	Yes

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	-276.85	0.00	600T250-68 (50) & Anchorage Designed by Engineer	87.10 %	NA
R1	-327.70	769.00	600T125-54 (50) & (2) .157", 1" embed SST PDPA/PDPAT to 4000 nw concrete	70.44 %	52.85 %

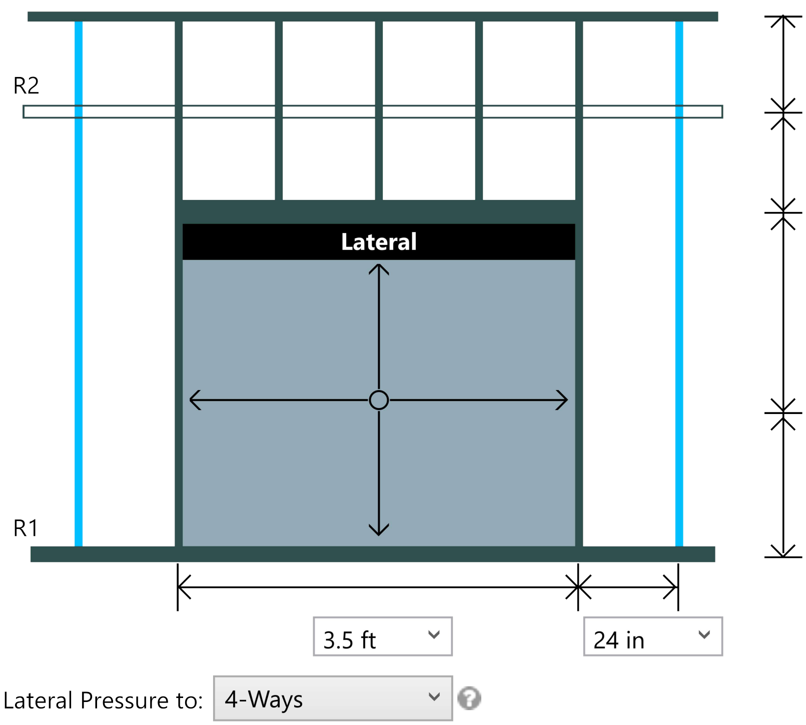
* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wind Selection :	**C&C Wind, Leeward (5)**
Trib. Area : Span :	**Length^2/3**
Wall Lateral Pressure :	**-11.6 psf**
Parapet Lateral Pressure :	
RO Lateral Pressure :	**4-Ways**
Lateral element force multiplier	
Strength :	**1.0**
Deflection :	**0.7**
Header:	**Single Member**
Gravity Load at Header:	**11 psf**
Additional Pt. Load ea. Stud :	**256 lbs**

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S200-54(50), Single	Full	48 in	48 in	0	None	N/A
Vertical Header	600S200-54(50), Y-Y Axis	Full	N/A	N/A	0	None	N/A
Lateral Header	600S200-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S200-54(50), Single	320.3	67	663.6	189.6	-207.4	-156.6
Vertical Header	600S200-54(50), Y-Y Axis	N/A	N/A	280.2	320.3	N/A	320.3
Lateral Header	600S200-54(50), Single	N/A	N/A	65.1	68.5	N/A	68.5

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S200-54(50), Single	L/1102	L/0	0.338	0.26	NA	Yes
Vertical Header	600S200-54(50), Y-Y Axis	L/488	NA	0.56	0.56	No	Yes
Lateral Header	600S200-54(50), Single	L/41439	NA	0.03	0.04	No	Yes
Combined Header				0.59	0		

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	-156.60	0.00	600T250-54 (50) & Anchorage Designed by Engineer	65.31 %	NA
R1	-207.35	551.25	600T125-54 (50) & (1) .157", 1" embed SST PDPA/PDPAT to 2500 nw concrete	44.57 %	72.75 %

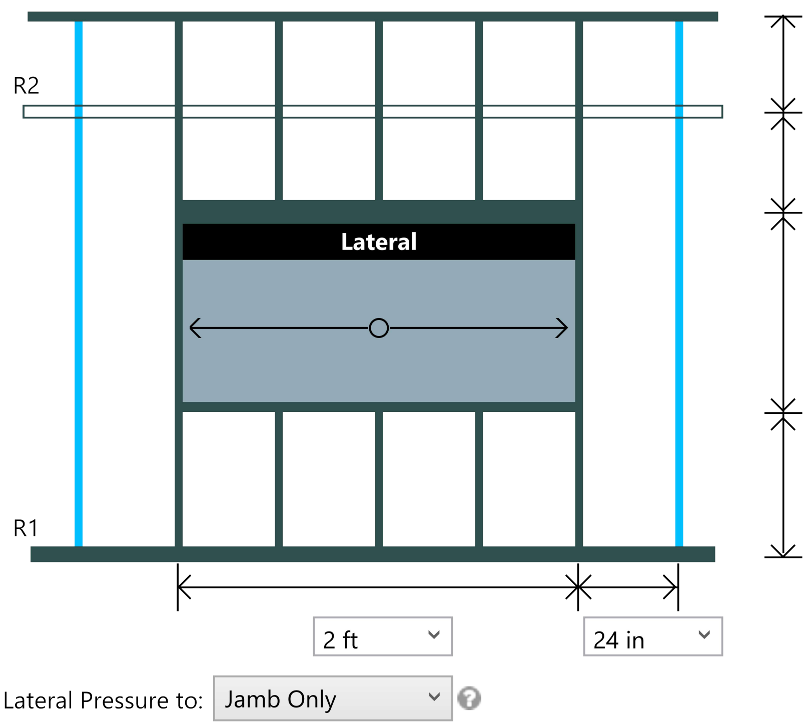
* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	Bracing Length(in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wind Selection :	C&C Wind, Leeward (5)
Trib. Area : Span :	Length^2/3
Wall Lateral Pressure :	-11.4 psf
Parapet Lateral Pressure :	
RO Lateral Pressure :	Jamb Only
Lateral element force multiplier	
Strength :	1.0
Deflection :	0.7
Header:	Single Member
Gravity Load at Header:	11 psf
Additional Jamb Axial Load :	2424 lbs

Brace Settings

Component(s)	Members(s)	Flexural Bracing	Axial KyLy	Axial KtLt	Distortional K-Phi(lb-in/in)	Distortional Lm	Interconnection Spacing
Jamb Studs	600S200-54(50), Single	Full	60 in	60 in	0	None	N/A
Vertical Header	600S200-54(50), Y-Y Axis	Full	N/A	N/A	0	None	N/A
Lateral Header	600S200-54(50), Single	Full	N/A	N/A	0	None	N/A
Sill	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max KL/r	Max. Moment (ft-lb)	Max. Shear (lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Jamb Studs	600S200-54(50), Single	2730.2	82	545.1	135.9	-114.5	-135.9
Vertical Header	600S200-54(50), Y-Y Axis	N/A	N/A	22.0	44.0	N/A	44.0
Lateral Header	600S200-54(50), Single	N/A	N/A	11.4	22.8	N/A	22.8
Sill	600T125-54(50), Single	N/A	N/A	22.1	44.2	N/A	44.2

Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	A + M Interaction	V + M Interaction	Web Stiffners	Design OK
Jamb Studs	600S200-54(50), Single	L/1320	L/0	0.74	0.22	NA	Yes
Vertical Header	600S200-54(50), Y-Y Axis	L/10877	NA	0.04	0.04	No	Yes
Lateral Header	600S200-54(50), Single	L/409027	NA	0.00	0.01	No	Yes
Combined Header				0.05	0		
Sill	600T125-54(50), Single	L/142519	NA	0.01	0.02	No	Yes

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	-135.89	0.00	600T250-54 (50) & Anchorage Designed by Engineer	56.67 %	NA

R1	-114.51	2730.24	600T125-54 (50) & Anchorage Designed by Engineer	12.31 %	NA

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/Parapet	**Bracing Length(in.)**	**Design Number of Braces**	**Pn(lb.)**	**LSUBH (Min)[1]**	**LSUBH (Max)[1]**	**SUBH (Min)[1]**	**SUBH (Max)[1]**	**MSUBH (Min)[1]**	**MSUBH (Max)[1]**
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

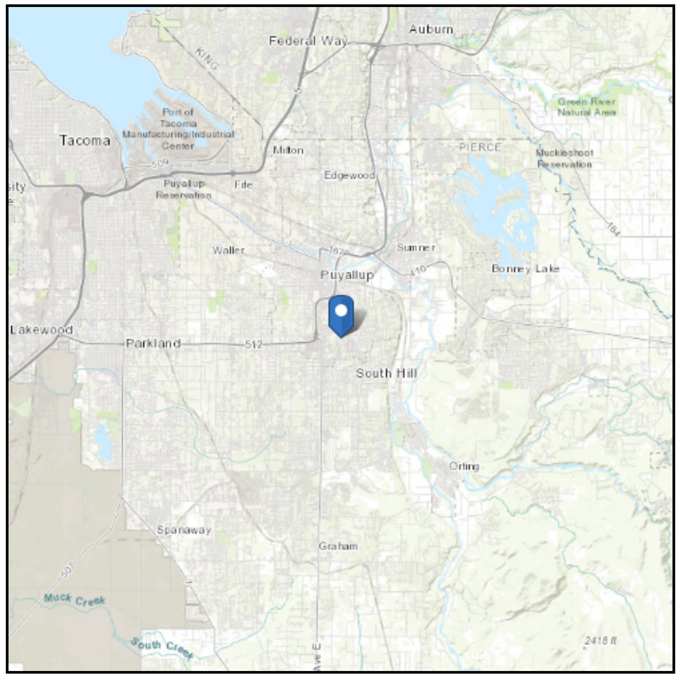
1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back, box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes.
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.

Lateral Design

ASCE Hazards Report

Address:
1023 39th Ave SE
Puyallup, Washington
98374

Standard:	ASCE/SEI 7-16	**Latitude:**	47.160853
Risk Category:	II	**Longitude:**	-122.279318
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	482.88472036372787 ft (NAVD 88)



Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Mon Feb 05 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.257	S_{D1} :	N/A
S_1 :	0.434	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.509	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.006	C_v :	1.351

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Mon Feb 05 2024

Date Source: USGS Seismic Design Maps

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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MecaWind v2481

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File Location: G:\2024\24201.5 Centeris Switchgear Bldg 2\Calcs\Centris Wind.wnd

General:

Wind Load Standard	=	ASCE 7-16	Basic Wind Speed	=	98.0 mph
Exposure Classification	=	B	Risk Category	=	II
Structure Type	=	Building	Design Basis for Wind Pressures	=	LRFD
MWFRS Analysis Method	=	Ch 27 Pt 1	C&C Analysis Method	=	Ch 30 Pt 1
Dynamic Type of Structure	=	Rigid	Show Advanced Options	=	True
Reset Advanced Options to Default Values	=	Defaults	Simple Diaphragm Building	=	False
Show Base Reactions in Output	=	Summary	Altitude above Sea Level	=	482.900 ft
Base Elevation Of Structure	=	0.000 ft	MWFRS Pressure Elevations	=	Mean Ht
Topographic Effects	=	None	Override Directionality Factor K_d	=	False
Override the Gust Factor G	=	False	Override Minimum Pressure	=	False

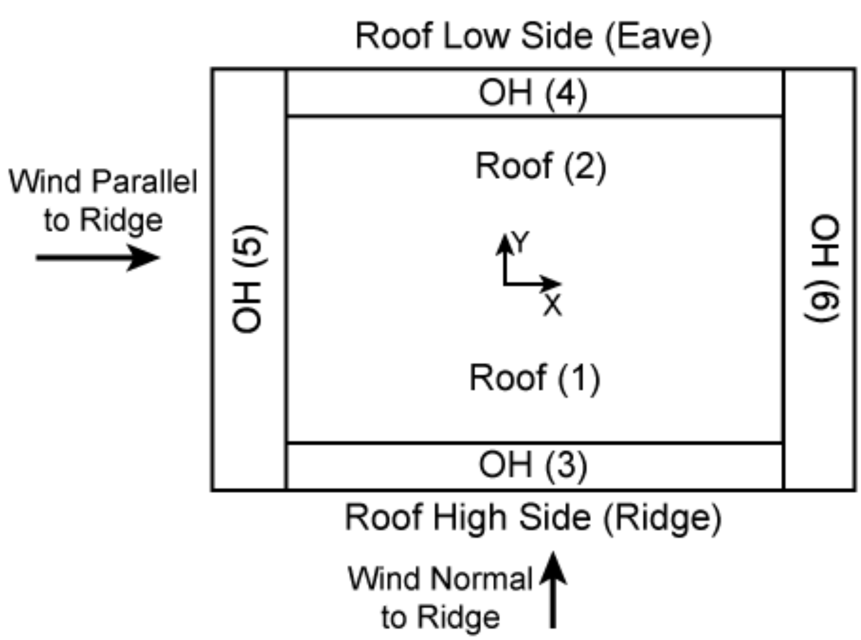
Building:

Roof	=	Roof Type	=	Monoslope	Encl	=	Enclosure Classification	=	Enclosed
Help	=	Help on Building Roof Type	=	Help	Pitch	=	Pitch of Roof	=	2.0 :12
θ	=	Slope of Roof	=	9.46 Deg	R_{Ht}	=	Ridge Height	=	14.555 ft
E_{Ht}	=	Eave Height	=	12.000 ft	W	=	Building Width	=	15.330 ft
L	=	Building Length	=	46.000 ft	OH	=	Type of Overhang	=	None
Par	=	Parapet	=	None	HT_{over}	=	Override Mean Roof Height	=	False
Ht_{man}	=	Mean Roof Height	=	12.000 ft	RA_{over}	=	Override Roof Area	=	False
GC_{pi_o}	=	Override GC_{pi} value	=	False		=			

Exposure Constants [Tbl 26.11-1]:

α = 3-s Gust-speed exponent	=	7.000	Z_g = Nominal Ht of Boundary Layer	=	1200.000 ft
â = Reciprocal of α	=	0.143	b = 3 sec gust speed factor	=	0.840
α_m = Mean hourly Wind-Speed Exponent	=	0.250	b_m = Mean hourly Windspeed Exponent	=	0.450
c = Turbulence Intensity Factor	=	0.300	ε = Integral Length Scale Exponent	=	0.3333

Main Wind Force Resisting System (MWFRS) Wind Calculations per Ch 27 Pt1



h	=	Mean structure height	=	12.000 ft
h_{grade}	=	Elevation from Grade to Top of Structure	=	12.000 ft
K_h	=	$2.01 \cdot (15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]	=	0.575
K_{zt}	=	No Topographic feature specified	=	1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	=	0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	=	+0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	=	-0.18
LF	=	Load Factor based upon STRENGTH Design	=	1.00

K_e	=	Ground Elev Factor [Tbl 26.9-1]	=	0.983
q_h	=	0.00256•K_h•K_{zt}•K_d•K_e•V^2*LF [Eq 26.10-1]	=	11.80 psf
RA	=	Roof Area	=	714.91 ft^2
q_h	=	0.00256•K_h•K_{zt}•K_d•K_e•V^2*LF [Eq 26.10-1]	=	11.80 psf
q_{in}	=	Negative Internal Pressure: q_h•LF	=	11.80 psf
q_{ip}	=	Positive Internal Pressure: q_h•LF	=	11.80 psf

MWFRS Wind Loads [Normal to Ridge]

h	=	Mean Roof Height Of Building	=	12.000 ft
RHt	=	Ridge Height Of Roof	=	14.555 ft
B	=	Horizontal Dimension Of Building Normal To Wind Direction	=	46.000 ft
L	=	Horizontal Dimension Of building Parallel To Wind Direction	=	15.330 ft
L/B	=	Ratio Of L/B used For Cp determination	=	0.333
h/L	=	Ratio Of h/L used For Cp determination	=	0.783
Slope	=	Slope Of Roof	=	9.46 Deg

Gust Factor Calculation for Wind: [Normal to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	=	Simplified: For Rigid Structures can use 0.85	=	0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	=	Equiv Struc Height: Max(0.6•h, Z_{min})	=	30.000 ft
I_{zm}	=	Turbulence Intensity: c•$(33/Z_m)^{1/6}$ [Eq 26.11-1]	=	0.305
L_{zm}	=	Turbulence Integral Length Scale: ℓ•$(Z_m/33)^{\varepsilon}$ [Eq 26.11-9]	=	309.993 ft
B	=	Building Width Width Normal to Wind Direction	=	46.000 ft
Q	=	$[1/(1+0.63•[(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	=	0.906
G_2	=	Detailed: 0.925•[(1+1.7•g_q•I_{zm}•Q)/(1+1.7•g_v•I_{zm})] [Eq 26.11-6]	=	0.869

Gust Factor Used in Analysis

G	=	Gust Factor: Min(G_1, G_2)	=	0.850
Cp_{WW}	=	Windward Wall Coefficient (All L/B Values)	=	0.800
Cp_{LW}	=	Leeward Wall Coefficient using L/B	=	-0.500
Cp_{SW}	=	Side Wall Coefficient (All L/B values)	=	-0.700

Wind Pressures [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
14.555	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-7.14	-9.15	13.04	16.00
12.000	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-7.14	-9.15	13.04	16.00
14.555	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-2.89	-4.90	13.04	16.00
12.000	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-2.89	-4.90	13.04	16.00

K_z = 2.01•$(15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: q_h•LF
Side = q_h•G•Cp_{SW}-q_{ip}•(+GC_{pi}) [Eq 27.3-1]
Windward = q_z•G•Cp_{WW}-q_{ip}•(+GC_{pi}) [Eq 27.3-1]
K_{zt} = No Topographic feature specified
*q_z = 0.00256•K_z•K_{zt}•K_d•K_e•V^2*LF [Eq 26.10-1]*
q_{in} = Negative Internal Pressure: q_h•LF
Leeward = q_h•G•Cp_{LW}-q_{ip}•(+GC_{pi}) [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures [Normal to Ridge]
All wind pressures include a Load Factor (LF) of 1.0

Component	Description	Location	Start ft	End ft	GC_{pi}	C_{pMin}	C_{pMax}	P_{CpMin} psf	P_{CpMax} psf	P_{min} psf
Roof	Roof (0 to h/2)	All	0.000	6.000	+0.18	-1.007	-0.180	-12.23	-3.93	8.00
Roof	Roof (h/2 to h)	All	6.000	12.000	+0.18	-0.787	-0.180	-10.02	-3.93	8.00
Roof	Roof (h to 2*h)	All	12.000	15.330	+0.18	-0.613	-0.180	-8.28	-3.93	8.00
Roof	Roof (0 to h/2)	All	0.000	6.000	-0.18	-1.007	-0.180	-7.98	0.32	8.00
Roof	Roof (h/2 to h)	All	6.000	12.000	-0.18	-0.787	-0.180	-5.77	0.32	8.00
Roof	Roof (h to 2*h)	All	12.000	15.330	-0.18	-0.613	-0.180	-4.03	0.32	8.00

Roof Pressures based upon Ch 27 Pt1:
Component = The building component for pressures
Start = Start Dist from Windward Edge
C_{pMin} = Smallest Coefficient Magnitude
P_{CpMin} = q_h•G•C_{pMin}-q_{ip}•GC_{pi} [Eq 27.3-1]
Location = Reference Graphic in Output for Values
End = End Dist from Windward Edge
C_{pMax} = Largest Coefficient Magnitude
P_{CpMax} = q_h•G•C_{pMax}-q_{in}•GC_{pi} [Eq 27.3-1]
P_{min} = Min Press projected on vertical plane [§ 27.1.5]

- *0.838 Reduction Factor applied for h/L>=1 & Slope<10 Deg*

- *The smaller uplift pressures due to C_{pMin} can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

MWFRS Wind Loads [Normal to Eave]

h	=	Mean Roof Height Of Building	=	12.000 ft
RHt	=	Ridge Height Of Roof	=	14.555 ft
B	=	Horizontal Dimension Of Building Normal To Wind Direction	=	46.000 ft
L	=	Horizontal Dimension Of building Parallel To Wind Direction	=	15.330 ft
L/B	=	Ratio Of L/B used For Cp determination	=	0.333
h/L	=	Ratio Of h/L used For Cp determination	=	0.783
Slope	=	Slope Of Roof	=	9.46 Deg

Gust Factor Calculation for Wind: [Normal to Eave]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	=	Simplified: For Rigid Structures can use 0.85	=	0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	=	Equiv Struc Height: Max(0.6•h, Z_{min})	=	30.000 ft
I_{zm}	=	Turbulence Intensity: c•$(33/Z_m)^{1/6}$ [Eq 26.11-1]	=	0.305
L_{zm}	=	Turbulence Integral Length Scale: ℓ•$(Z_m/33)^{\varepsilon}$ [Eq 26.11-9]	=	309.993 ft
B	=	Building Width Width Normal to Wind Direction	=	46.000 ft
Q	=	$[1/(1+0.63•[(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	=	0.906
G_2	=	Detailed: $0.925•[(1+1.7•g_q•I_{zm}•Q)/(1+1.7•g_v•I_{zm})]$ [Eq 26.11-6]	=	0.869

Gust Factor Used in Analysis

G	=	Gust Factor: Min(G_1, G_2)	=	0.850
Cp_{WW}	=	Windward Wall Coefficient (All L/B Values)	=	0.800
Cp_{LW}	=	Leeward Wall Coefficient using L/B	=	-0.500
Cp_{SW}	=	Side Wall Coefficient (All L/B values)	=	-0.700

Wind Pressures [Normal to Eave]
All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
12.000	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-7.14	-9.15	13.04	16.00
12.000	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-2.89	-4.90	13.04	16.00

K_z = $2.01•(15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: q_h•LF
Side = $q_h•G•Cp_{SW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
Windward = $q_z•G•Cp_{WW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
K_{zt} = No Topographic feature specified
*q_z = $0.00256•K_z•K_{zt}•K_d•K_e•V^2$*LF [Eq 26.10-1]*
q_{in} = Negative Internal Pressure: q_h•LF
Leeward = $q_h•G•Cp_{LW}-q_{ip}•(+GC_{pi})$ [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures [Normal to Eave]
All wind pressures include a Load Factor (LF) of 1.0

Component	Description	Location	Start ft	End ft	GC_{pi}	C_{pMin}	C_{pMax}	$P_{C_{pMin}}$ psf	$P_{C_{pMax}}$ psf	P_{min} psf
Roof	Roof (0 to h/2)	All	0.000	6.000	+0.18	-1.007	-0.180	-12.23	-3.93	8.00
Roof	Roof (h/2 to h)	All	6.000	12.000	+0.18	-0.787	-0.180	-10.02	-3.93	8.00
Roof	Roof (h to 2*h)	All	12.000	15.330	+0.18	-0.613	-0.180	-8.28	-3.93	8.00
Roof	Roof (0 to h/2)	All	0.000	6.000	-0.18	-1.007	-0.180	-7.98	0.32	8.00
Roof	Roof (h/2 to h)	All	6.000	12.000	-0.18	-0.787	-0.180	-5.77	0.32	8.00
Roof	Roof (h to 2*h)	All	12.000	15.330	-0.18	-0.613	-0.180	-4.03	0.32	8.00

Roof Pressures based upon Ch 27 Pt1:
Component = The building component for pressures
Start = Start Dist from Windward Edge
C_{pMin} = Smallest Coefficient Magnitude
$P_{C_{pMin}}$ = $q_h•G•C_{pMin}-q_{ip}•GC_{pi}$ [Eq 27.3-1]
Location = Reference Graphic in Output for Values
End = End Dist from Windward Edge
C_{pMax} = Largest Coefficient Magnitude
$P_{C_{pMax}}$ = $q_h•G•C_{pMax}-q_{in}•GC_{pi}$ [Eq 27.3-1]
P_{min} = Min Press projected on vertical plane [§ 27.1.5]

- *0.838 Reduction Factor applied for h/L>=1 & (0 to h/2)*
- *The smaller uplift pressures due to C_{pMin} can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

MWFRS Wind Loads [Parallel to Ridge]

h	=	Mean Roof Height Of Building	=	12.000 ft
RHt	=	Ridge Height Of Roof	=	14.555 ft
B	=	Horizontal Dimension Of Building Normal To Wind Direction	=	15.330 ft
L	=	Horizontal Dimension Of building Parallel To Wind Direction	=	46.000 ft
L/B	=	Ratio Of L/B used For Cp determination	=	3.001
h/L	=	Ratio Of h/L used For Cp determination	=	0.261
Slope	=	Slope Of Roof	=	9.46 Deg

Gust Factor Calculation for Wind: [Parallel to Ridge]

Gust Factor Category I Rigid Structures - Simplified Method

G_1	=	Simplified: For Rigid Structures can use 0.85	=	0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Z_m	=	Equiv Struc Height: $Max(0.6 \cdot h, Z_{min})$	=	30.000 ft
I_{zm}	=	Turbulence Intensity: $c \cdot (33/Z_m)^{1/6}$ [Eq 26.11-1]	=	0.305
L_{zm}	=	Turbulence Integral Length Scale: $\ell \cdot (Z_m/33)^{\varepsilon}$ [Eq 26.11-9]	=	309.993 ft
B	=	Building Width Width Normal to Wind Direction	=	15.330 ft
Q	=	$[1/(1+0.63 \cdot [(B+h)/L_{zm}]^{0.63})]^{0.5}$ [Eq 26.11-8]	=	0.938
G_2	=	Detailed: $0.925 \cdot [(1+1.7 \cdot g_q \cdot I_{zm} \cdot Q)/(1+1.7 \cdot g_v \cdot I_{zm})]$ [Eq 26.11-6]	=	0.888

Gust Factor Used in Analysis

G	=	Gust Factor: $Min(G_1, G_2)$	=	0.850
Cp_{WW}	=	Windward Wall Coefficient (All L/B Values)	=	0.800
Cp_{LW}	=	Leeward Wall Coefficient using L/B	=	-0.250
Cp_{SW}	=	Side Wall Coefficient (All L/B values)	=	-0.700

Wind Pressures [Parallel to Ridge]

All wind pressures include a Load Factor (LF) of 1.0

Elev ft	GC_{pi} Windward	GC_{pi} Leeward	q_i psf	K_z	K_{zt}	q_z psf	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
14.555	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-4.63	-9.15	10.53	16.00
12.000	+0.18	+0.18	11.80	0.575	1.000	11.80	5.90	-4.63	-9.15	10.53	16.00
14.555	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-0.38	-4.90	10.53	16.00
12.000	-0.18	-0.18	11.80	0.575	1.000	11.80	10.15	-0.38	-4.90	10.53	16.00

K_z = $2.01 \cdot (15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]
GC_{pi} = Enclosed Internal Pressure Tbl 26.13-1
q_{ip} = Positive Internal Pressure: $q_h \cdot LF$
Side = $q_h \cdot G \cdot Cp_{SW} - q_{ip} \cdot (+GC_{pi})$ [Eq 27.3-1]
Windward = $q_z \cdot G \cdot Cp_{WW} - q_{ip} \cdot (+GC_{pi})$ [Eq 27.3-1]
K_{zt} = No Topographic feature specified
q_z = $0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]
q_{in} = Negative Internal Pressure: $q_h \cdot LF$
Leeward = $q_h \cdot G \cdot Cp_{LW} - q_{ip} \cdot (+GC_{pi})$ [Eq 27.3-1]
Total = Windward - Leeward

- *Minimum Pressure: § 27.1.5 no less than 16.00 psf (Incl LF) applied to Walls*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Roof Wind Pressures [Parallel to Ridge]

All wind pressures include a Load Factor (LF) of 1.0

Component	Description	Location	Start ft	End ft	GC_{pi}	C_{pMin}	C_{pMax}	P_{CpMin} psf	P_{CpMax} psf	P_{min} psf
Roof	Roof (0 to h)	All	0.000	12.000	+0.18	-0.900	-0.180	-11.15	-3.93	8.00
Roof	Roof (h to 2*h)	All	12.000	24.000	+0.18	-0.500	-0.180	-7.14	-3.93	8.00
Roof	Roof (>= 2*h)	All	24.000	46.000	+0.18	-0.300	-0.180	-5.13	-3.93	8.00
Roof	Roof (0 to h)	All	0.000	12.000	-0.18	-0.900	-0.180	-6.90	0.32	8.00
Roof	Roof (h to 2*h)	All	12.000	24.000	-0.18	-0.500	-0.180	-2.89	0.32	8.00
Roof	Roof (>= 2*h)	All	24.000	46.000	-0.18	-0.300	-0.180	-0.89	0.32	8.00

Roof Pressures based upon Ch 27 Pt1:
Component = The building component for pressures
Start = Start Dist from Windward Edge
C_{pMin} = Smallest Coefficient Magnitude
P_{CpMin} = $q_h \cdot G \cdot C_{pMin} - q_{ip} \cdot GC_{pi}$ [Eq 27.3-1]
Location = Reference Graphic in Output for Values
End = End Dist from Windward Edge
C_{pMax} = Largest Coefficient Magnitude
P_{CpMax} = $q_h \cdot G \cdot C_{pMax} - q_{in} \cdot GC_{pi}$ [Eq 27.3-1]
P_{min} = Min Press projected on vertical plane [§ 27.1.5]

- *No reduction factor was applicable*
- *The smaller uplift pressures due to C_{pMin} can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7*
- *Positive Pressures Act TOWARD Surface and Negative Pressures Act AWAY from Surface*

Reaction Summary Wind (MWFRS)

Description	F_x Kip	F_y Kip	F_z Kip	M_x k•ft	M_y k•ft	M_z k•ft

Normal to Ridge: Walls+Roof|+GCPi	0.00	-9.13	-7.41	73.37	0.00	0.00
Normal to Ridge: Walls Only|+GCPi	0.00	-7.89	0.00	52.40	0.00	0.00
Normal to Ridge: Walls+Roof|-GCPi	0.00	-8.35	0.22	58.54	0.00	0.00
Normal to Ridge: Walls Only|-GCPi	0.00	-8.39	0.00	59.04	0.00	0.00
Normal to Ridge: Walls+Roof|Min Pressure	0.00	-10.71	0.00	77.96	0.00	0.00
Normal to Eave: Walls+Roof|+GCPi	0.00	6.80	-7.41	-42.51	0.00	0.00
Normal to Eave: Walls Only|+GCPi	0.00	8.04	0.00	-54.34	0.00	0.00
Normal to Eave: Walls+Roof|-GCPi	0.00	7.58	0.22	-48.20	0.00	0.00
Normal to Eave: Walls Only|-GCPi	0.00	7.54	0.00	-47.71	0.00	0.00
Normal to Eave: Walls+Roof|Min Pressure	0.00	9.77	0.00	-65.48	0.00	0.00
Parallel to Ridge: Walls+Roof|+GCPi	-2.14	0.23	-5.10	-3.00	-34.95	2.92
Parallel to Ridge: Walls Only|+GCPi	-2.14	1.08	0.00	-14.28	-14.28	-0.53
Parallel to Ridge: Walls+Roof|-GCPi	-2.14	0.61	0.22	-8.14	-14.28	-0.53
Parallel to Ridge: Walls Only|-GCPi	-2.14	0.58	0.00	-7.64	-14.28	-0.53
Parallel to Ridge: Walls+Roof|Min Pressure	-3.26	0.00	0.00	0.00	-21.69	-0.80

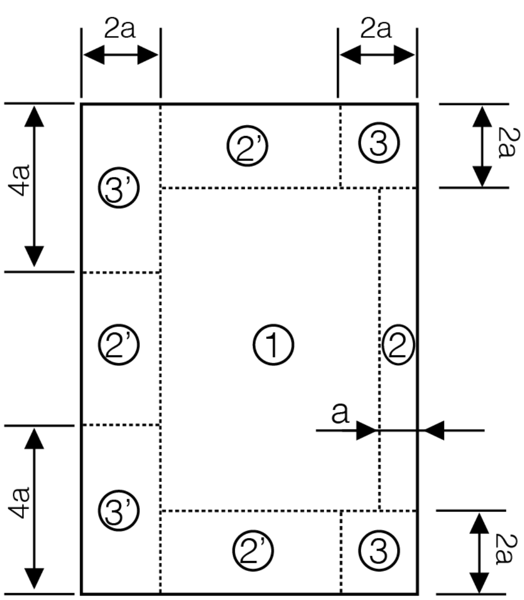
** Per Fig 27.3-1, Use greater of Shear calculated with or without roof.*
** X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical*
** Minimum Pressures applied to a vertical plane normal to wind.*
** Reactions calculated about the geometric center of the footprint*

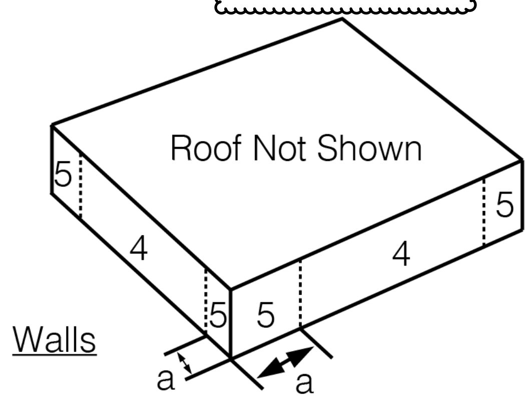
Components and Cladding (C&C) Zone Summary per Ch 30 Pt 1:

h/W	=	Ratio of mean roof height to building width	= 0.783
h/L	=	Ratio of mean roof height to building length	= 0.261
h	=	Mean structure height	= 12.000 ft
h_{grade}	=	Elevation from Grade to Top of Structure	= 12.000 ft
K_h	=	$2.01 \cdot (15/Z_g)^{2/\alpha}$ [Tbl 26.10-1]	= 0.575
K_{zt}	=	No Topographic feature specified	= 1.000
K_d	=	Wind Directionality Factor per Tbl 26.6-1	= 0.85
$+GC_{pi}$	=	Enclosed Positive Internal Pressure Tbl 26.13-1	= +0.18
$-GC_{pi}$	=	Enclosed Negative Internal Pressure Tbl 26.13-1	= -0.18
LF	=	Load Factor based upon STRENGTH Design	= 1.00
K_e	=	Ground Elev Factor [Tbl 26.9-1]	= 0.983
q_h	=	$0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot K_e \cdot V^2 * LF$ [Eq 26.10-1]	= 11.80 psf
LHD	=	Least Horizontal Dimension: Min(B, L)	= 15.330 ft
a_1	=	$Min(0.1 \cdot LHD, 0.4 \cdot h)$	= 1.533 ft
a	=	$Max(a_1, 0.04 \cdot LHD, 3\text{ ft }[0.9\text{ m}])$	= 3.000 ft
h/B	=	Ratio of mean roof height to least horizontal dim: h/B	= 0.783

Controlling lateral wind design load (longitudinal direction) - Compare with Seismic forces

Controlling lateral wind design load (transverse design)- Compare with Seismic forces





Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 1 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	Pos A ≤ 10 ft² psf	Neg A ≤ 10 ft² psf	Pos A = 20 ft² psf	Neg A = 20 ft² psf	Pos A = 50 ft² psf	Neg A = 50 ft² psf
1	30.3-5A	16.00	-16.00	16.00	-16.00	16.00	-16.00
2	30.3-5A	16.00	-17.47	16.00	-17.11	16.00	-16.64
2'	30.3-5A	16.00	-21.01	16.00	-20.65	16.00	-20.18
3	30.3-5A	16.00	-23.37	16.00	-21.24	16.00	-18.42
3'	30.3-5A	16.00	-32.81	16.00	-29.26	16.00	-24.56

4	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00
5	30.3-1	16.00	-17.00	16.00	-16.00	16.00	-16.00

Wind Pressure Summary for C&C Zones based Upon Areas Ch 30 Pt 1 (Table 2 of 2)
All wind pressures include a Load Factor (LF) of 1.0

Zone	Figure	Pos A = 100 ft^2 psf	Neg A = 100 ft^2 psf	Pos A = 200 ft^2 psf	Neg A = 200 ft^2 psf	Pos A > 500 ft^2 psf	Neg A > 500 ft^2 psf
1	30.3-5A	16.00	-16.00	16.00	-16.00	16.00	-16.00
2	30.3-5A	16.00	-16.29	16.00	-16.29	16.00	-16.29
2'	30.3-5A	16.00	-19.83	16.00	-19.83	16.00	-19.83
3	30.3-5A	16.00	-16.29	16.00	-16.29	16.00	-16.29
3'	30.3-5A	16.00	-21.01	16.00	-21.01	16.00	-21.01
4	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00
5	30.3-1	16.00	-16.00	16.00	-16.00	16.00	-16.00

** A is effective wind area for C&C: Span Length * Effective Width*
** Effective width need not be less than 1/3 of the span length*
** Maximum and minimum values of pressure shown.*
** + Pressures acting toward surface, - Pressures acting away from surface*
** Per § 30.2.2 the Minimum Pressure for C&C is 16.00 psf [0.766 kPa] {Includes LF}*
** Interpolation can be used for values of A that are between those values shown.*

Calculation of Seismic Response Coefficient, Cs

(ASCE 7-16, Chapter 11 and 12, Equivalent Lateral Force "ELF" Procedure)

BUILDING INFORMATION :

Risk Category :	II	(ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)
Importance Factor, Ie :	1.00	(ASCE 7-16 Table 1.5-2)
Response Modification Factor, R :	6.5	(ASCE 7-16 Table 12.2-1)
Overstrength Factor, Ω :	3	(ASCE 7-16 Table 12.2-1)
Deflection Amplification Factor, Cd :	4	(ASCE 7-16 Table 12.2-1)

SITE INFO & SEISMIC ACCELERATIONS :

Site Class :	D (default)	(IBC Section 1613.2.2, "D" Assumed or per Geotech.)
Ss :	1.257	
S1 :	0.434	
Fv:	1.866	(ASCE 7-16 TABLE 11.4-2)
Sds :	1.006	
Sd1 :	0.540	(Eqn 11.4-2 & 11.4-4)
Seismic Design Category :	D	(ASCE 7-16 TABLE 11.6-1 & 11.6-2)

PERIOD DETERMINATION :

Ct :	0.02	(ASCE 7-16 Table 12.8-2)
x :	0.75	(ASCE 7-16 Table 12.8-2)
hn (ft) :	14.67	
Ta = Ct*hn^x :	0.150	(Eqn 12.8-7)
Ts = (Sd1/Sds) :	0.537	(ASCE 7-16 11.4.6)
1.5*Ts :	0.805	

CALCULATE Cs :

Cs = Sds/(R/I) :	0.155	(Eqn 12.8-2)
Max Cs = Sd1/(Ta*(R/I)) :	0.554	(Eqn 12.8-3)
Min Cs = 0.044*Sds*I > 0.01 :	0.044	(Eqn 12.8-5)
Min Cs = 0.5*S1/(R/I) :	0.000	(Eqn 12.8-6, for S1 > 0.6g)
Minimum Cs :	0.044	

Cs : **0.155**

Base Shear, V = Cs * W : 0.155 * W

SITE CLASS CHECKS :

Check ASCE-16, 11.4.8, Site Class F :	Site Response Analysis Not Required	Ground Motion Hazard Analysis <u>is</u> Required for seismically isolated structures or structures with damping systems on sites with S1 >/= 0.6
Check ASCE-16, 11.4.8, Site Class E :	Ground Motion Hazard Analysis Not Required	
Check ASCE-16, 11.4.8, Site Class D, Exception 2:	Ground Motion Hazard Analysis Not Required	

Seismic Weight

-Roof

Metal Roofing	1.5 psf
Wood Sheathing	2.2 psf
Metal Stud Framing @ 24"oc	3.8 psf
Insulation	2.5 psf
Conduit	12.7 psf
Misc.	1.3 psf
Total	24 psf

-Exterior Walls

Panel Siding	2.4 psf
Wood Sheathing	1.5 psf
Metal Stud Framing	2.0 psf
Insulation	2.2 psf
Gypsum Board	2.8 psf
Total	10.9 psf...USE 11psf

Seismic Base Shear

Roof:
(46ft x 15.33ft) x (24psf) = 16924 lbs

Exterior Walls:
perimeter = (2 x 46ft) + (2 x 15.33ft) = 123 ft
wall height = 13.9 ft (average)
(123ft x 13.9ft / 2) x (11 psf) = 9403 lbs

Seismic Weight = 16924lbs + 9403lbs = 26327lbs

Base Shear, V = Cs * W = 0.155 * 26327lbs = 4081lbs = 4.1kips

Seismic: (ASD) V_seismic = 0.7*(4.1kips) = 2.9kips
-
Compare with Wind Base Shear
(Transverse Direction)
(ASD) V_wind = 0.6*10.71kips = 6.4kips > V_seismic ∴ [Wind Controls, Transverse]

Longitudinal Direction
(ASD) V_wind = 0.6*3.26kips = 2.0kips < V_seismic ∴ [Seismic Controls, Longitudinal Direction]

BSE

B rienen **S** tructural **E** ngineers, P.S.

Diaphragm Design

-Diphragm Forces

Transverse Direction:
V = 6.4kips (ASD)

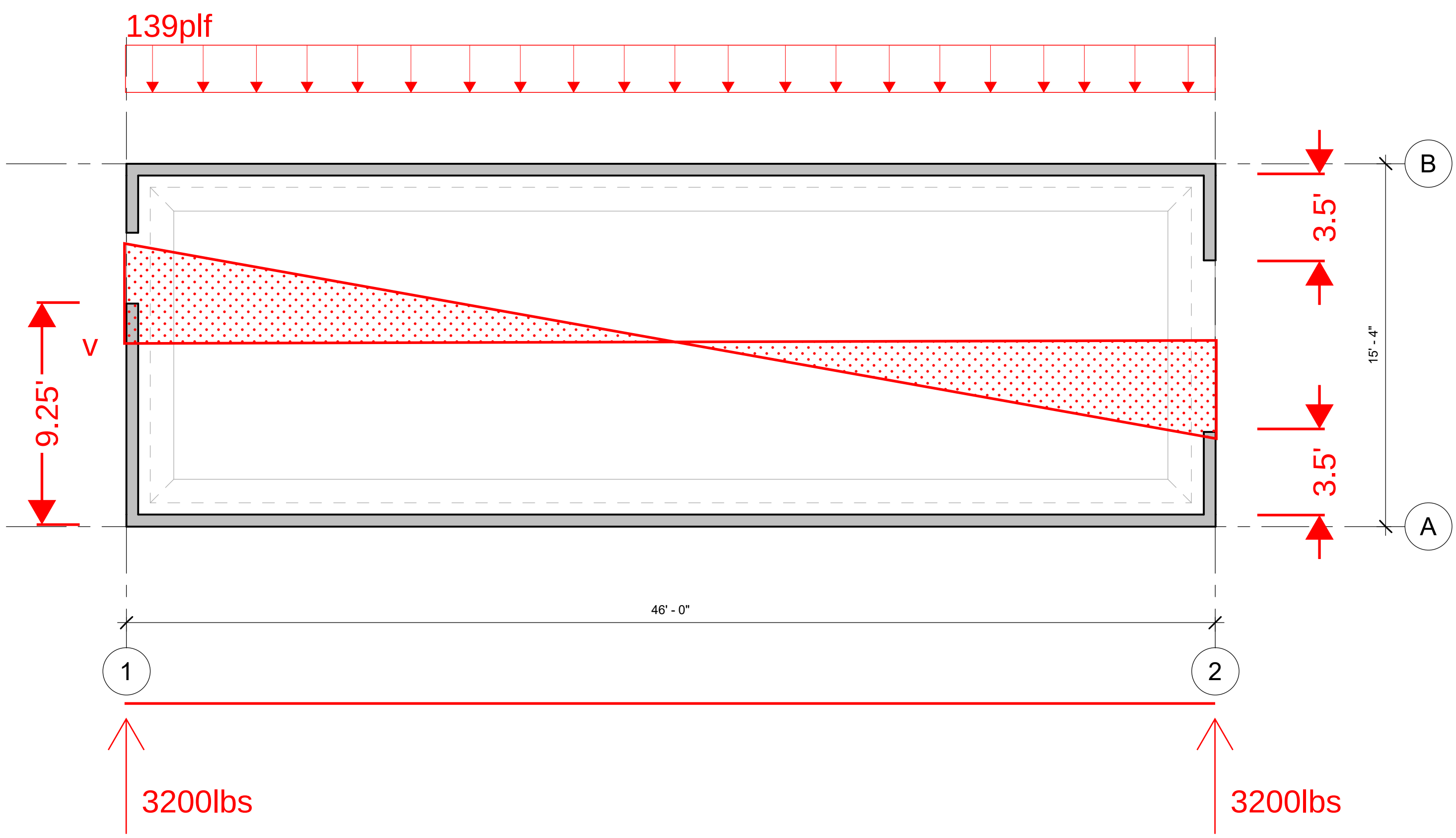
Distributed Wind Load w = 6.4kips / 46ft = 0.139k/ft = 139plf

- 2 walls ea end of diaphragm

Force to each wall = (6.4kips) / 2 = 3.2kips = 3200lbs

-Max Diaphragm shear @ gridline 1 & 2
v = (3200lbs) / 15.33ft = 209plf

-Max chord Forces @ gridlines A & B
M_max = (139lbs/ft) * (46ft)^2 / 8 = 36766 lb-ft
Total Chord Force, T/C = (36766lb-ft) / 15.33ft = 2398lbs
Linear chord force = 4056lbs / 52ft = 78lbs/ft



Diaphragm Design

-Diphragm Forces

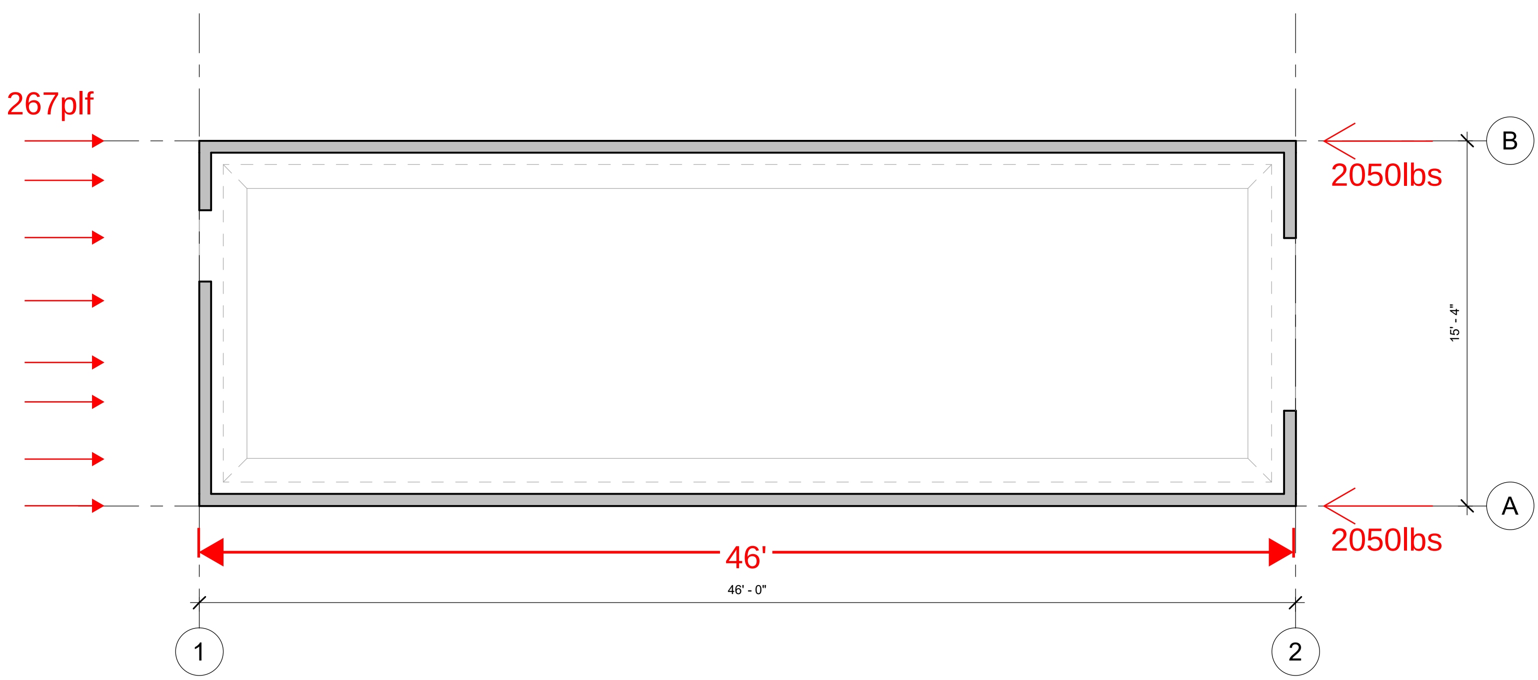
Longitudinal Direction
V = 4.1kips (ASD)

Distributed Seismic Load w = 4.1kips / 15.33ft = 0.267k/ft = 267lbs/ft

- 2 walls ea end of diaphragm
Force to each wall = (4.1kips) / 2 = 2.1kips = 2050lbs

- Max Diaphragm shear @ gridline A & B
v = (2050lbs) / 46ft = 45 lbs/ft

- Max chord Forces @ gridlines 1&2
Mmax = (61lbs/ft) * (22ft)^2 / 8 = 3691 lb-ft
Total Chord Force, T/C = (3691lb-ft) /52ft = 71lbs
Linear chord force = 71lbs / (9ft+24ft) = 2lbs/ft



BSE

B rienen **S** tructural **E** ngineers, P.S.

Diaphragm Design

-Diphragm Forces

Table F2.4-1
Nominal Shear Strength (v_n) per Unit Length for Diaphragms Sheathed With Wood Structural Panel Sheathing [1, 2]
***United States and Mexico* (lb/ft)**

Sheathing	Thickness (in.)	Blocked				Unblocked	
		Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.)				Screws spaced maximum of 6 in. on all supported edges	
		6	4	2.5	2	Load perpendicular to unblocked edges and continuous panel joints	All other configurations
		Screw spacing at all other panel edges (in.)					
		6	6	4	3		
Structural I	3/8	768	1022	1660	2045	685	510
	7/16	768	1127	1800	2255	755	565
	15/32	925	1232	1970	2465	825	615
C-D, C-C and other graded wood structural panels	3/8	690	920	1470	1840	615	460
	7/16	760	1015	1620	2030	680	505
	15/32	832	1110	1770	2215	740	555

1. For SI: 1" = 25.4 mm, 1 ft = 0.305 m, 1 lb = 4.45 N
2. For *diaphragms* sheathed with *wood structural panels*, tabulated R_n values are applicable for short-term *load* duration (seismic *loads*).

Using 15/32" min thickness OSB @ roof w/ #8 SMS @ 6"oc at panel edges-
825lb/ft / Ω = 330plf > 209plf [OK]

Gridline 1 Wall

Project Name: Centeris

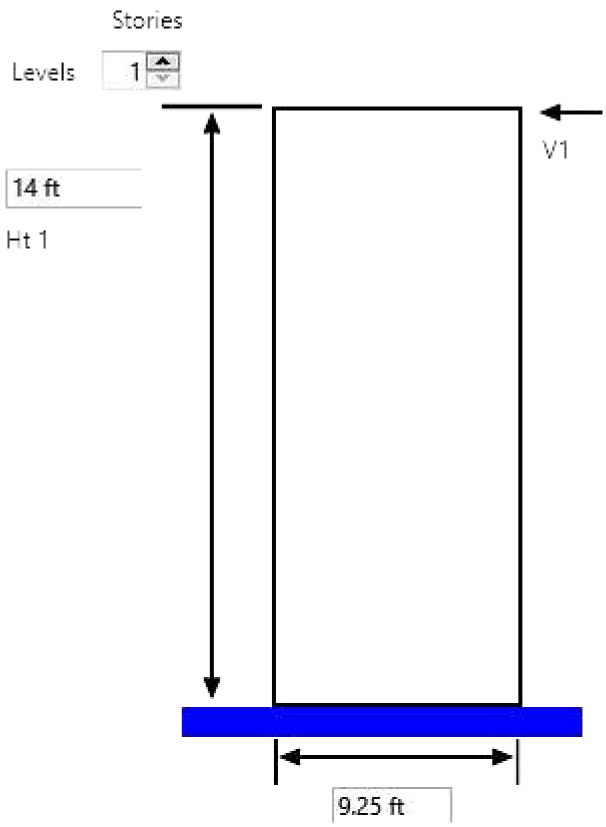
Model: LFRS Shearwall –1

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report



Load Inputs (All Loads are Unfactored Forces)

Top of Level Shear (lb)

Level	Wind	Seismic	Aspect Ratio
1	3200	2050	1.51

Seismic Design Parameters:

Seismic Design Category = D S_{DS} = 1 Ie = 1

Level	Overstrength Factor, Ω_0	Defl Amplification Factor, Cd
1	3	4

Allowable Strength Increase for Load Combinations w/Overstrength Included? Yes

Additional Applied Chord Axial Loads (lb) - Unfactored

Level	D	L	Lr	S	W
1	20	0	40	50	0

Additional Applied Chord Moments (ft-lb) - Unfactored

Level	D	L	Lr	S	W	E
1	0	0	0	0	0	0

Total and Unit Shear Forces (Factored)

	Wind Shear Forces		Seismic Shear Forces	
Level	V, Total (lb)	v, per ft (lb/ft)	V, Total (lb)	v, per ft (lb/ft)
1	1920	207.	1435	155.

USE OSB SHEATHING (1) SIDE W/ 6"OC E.N. & 12"OC F.N.

Shear Wall Sheathing and Fastener Selection

Level	Sheathing	Fastener Size	Edge/Field Fastener Spac (in)	Framing Thickness (mils)	Max Framing Spac (in)	One or Two Sides
1	7/16" OSB	No. 8	6/12	54	24	1

Shear Strength Modification Factors

Level	Wind Modifiers	Seismic Modifiers

Gridline 1 Wall

Project Name: Centeris

Model: LFRS Shearwall –1

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

1 None None

Available Shear Strength and Shear Ratios

	Wind			Seismic		
Level	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va
1	1	455	0.456	1	376	0.413

USE (1) 600S200-54
@ EA HOLD DOWN

Chords

				Bracing (in)					
Level	Section	Fy (ksi)	Configuration	Flexural	Axial KyLy	Axial KtLt	Flex Kϕ (lb-in/in)	Axial Kϕ (lb-in/in)	Bracing, Lm (in)
1	600S200-54	50	Single	60	60	60	0	0	None

Load Combinations ASCE7-16 ASD

LC1 = D
LC2 = D + L
LC3 = D + (Lr or S)
LC4 = D + 0.75L + 0.75(Lr or S)
LC5 = D + (0.6W or 0.7E)
LC6 = D + 0.75(0.6W or 0.7E) + 0.75L + 0.75(Lr or S)
LCO5 = $(1.0 + 0.14\text{Sds})\text{D} + 0.7\Omega_o\text{Qe}$
LCO6 = $(1.0 + 0.105\ \text{Sds})\text{D} + 0.525\Omega_o\text{Qe} + 0.75\text{L} + 0.75(\text{Lr or S})$

Note: LCO5 and LCO6 based on the lower of Overstrength or Expected Strength

Factored Chord Compression, P (lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	20	20	70	58	2906	2179	6538	4946

Factored Chord Strong-Axis Bending, Mx (ft-lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	0	0	0	0	0	0	0	0

			Interactions				
Level	Minimum Ma (ft-lb)	Minimum Pa (lb)	LC1	LC5	LC6	LC05	LC06
1	2282	5862	0.003	0.496	0.372	0.93	0.703

USE S/HDU9-54
HOLD DOWN

Ties and Holdowns

Level	Holdown	Quantity	Config	Exposed Rod Length (in)	Holdown Capacity Ta (lb/Each)	LRFD Capacity Φ Tn (lb)	Holdown Disp at Φ Tn (lb)	Holdown height (in)	Rod Dia. (in)
1	S/HDU9 - 54	1	Base	4	6750	10805	0.131	12.875	0.875

Gridline 1 Wall

Project Name: Centeris

Model: LFRS Shearwall –1

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

Level	Holdown Offset from End of Shear Wall (in)
1	0.0

Load Combinations (ASCE7-16 ASD)

LC7 = 0.6D + 0.6W

LC8 = 0.6D + 0.7E

LCO8 = (0.6-0.14Sds)D + 0.7Ω_0Qe Note: LCO8 based on the lower of Overstrength or Expected Strength

	Factored Net Uplift (lb) (Negative values represent uplift, Positive values indicate no net uplift)			Shear Forces (lb)		
Level	LC7	LC8	LC08	Wind	Seismic	Seismic w/Overstrength
1	-2894	-2160	-6506	1920	1435	4305

Ratio (Factored Net Uplift)/(Holdown Capacity)

Level	LC7	LC8	LC08
1	0.429	0.32	0.803

Displacement

	Floor-Floor Relative Displacement (in)			Drift %		
Level	Wind	Seismic	Seismic, Cd	Wind	Seismic	Seismic, Cd
1	0.32	0.17	0.68	0.19	0.1	0.4

Gridline 2 Wall

Project Name: Centeris

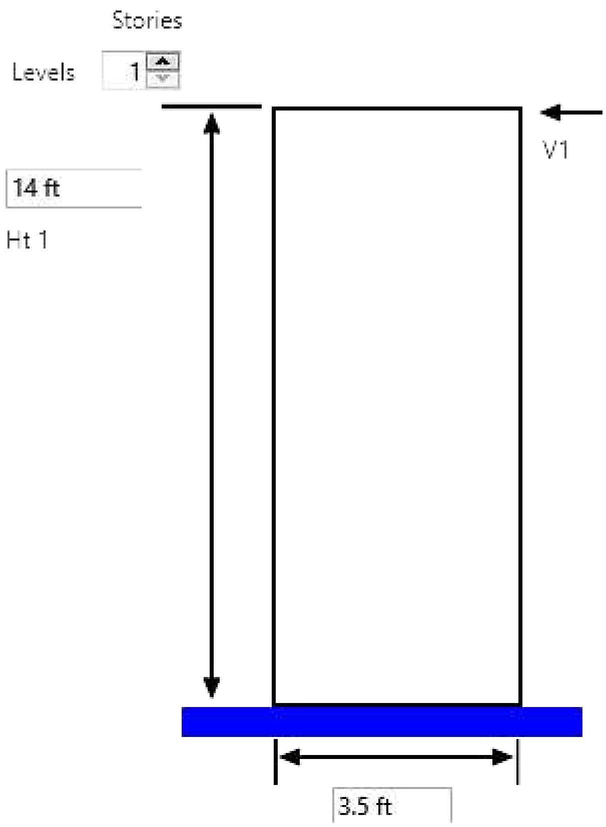
Model: LFRS Shearwall –2

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report



Load Inputs (All Loads are Unfactored Forces)

Top of Level Shear (lb)

Level	Wind	Seismic	Aspect Ratio
1	1600	1025	4

Seismic Design Parameters:

Seismic Design Category = D S_{DS} = 1.006 Ie = 1

Level	Overstrength Factor, Ω_0	Defl Amplification Factor, Cd
1	3	4

Allowable Strength Increase for Load Combinations w/Overstrength Included? Yes

Additional Applied Chord Axial Loads (lb) - Unfactored

Level	D	L	Lr	S	W
1	20	0	40	50	0

Additional Applied Chord Moments (ft-lb) - Unfactored

Level	D	L	Lr	S	W	E
1	0	0	0	0	0	0

Total and Unit Shear Forces (Factored)

	Wind Shear Forces		Seismic Shear Forces	
Level	V, Total (lb)	v, per ft (lb/ft)	V, Total (lb)	v, per ft (lb/ft)
1	960	274.	717.5	205

USE OSB SHEATHING (2) SIDES W/ 4"OC E.N. & 12"OC F.N.

Shear Wall Sheathing and Fastener Selection

Level	Sheathing	Fastener Size	Edge/Field Fastener Spac (in)	Framing Thickness (mils)	Max Framing Spac (in)	One or Two Sides
1	7/16" OSB	No. 8	4/12	54	24	2

Shear Strength Modification Factors

Level	Wind Modifiers	Seismic Modifiers

Gridline 2 Walls

Project Name: Centeris

Model: LFRS Shearwall –2

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

1 None None

Available Shear Strength and Shear Ratios

	Wind			Seismic		
Level	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va
1	0.5	512	0.535	0.5	494	0.415

Chords

USE 600S200-54 BACK-TO-BACK @ EA HOLD DOWN

				Bracing (in)					
Level	Section	Fy (ksi)	Configuration	Flexural	Axial KyLy	Axial KtLt	Flex Kϕ (lb-in/in)	Axial Kϕ (lb-in/in)	Bracing, Lm (in)
1	600S200-54	50	(2) Back-To-Back	60	60	60	0	0	None

Interconnection Spacing = 12 in

Load Combinations ASCE7-16 ASD

LC1 = D
LC2 = D + L
LC3 = D + (Lr or S)
LC4 = D + 0.75L + 0.75(Lr or S)
LC5 = D + (0.6W or 0.7E)
LC6 = D + 0.75(0.6W or 0.7E) + 0.75L + 0.75(Lr or S)
LCO5 = (1.0 + 0.14Sds)D + 0.7ΩₒQe
LCO6 = (1.0 + 0.105 Sds)D + 0.525ΩₒQe + 0.75L + 0.75(Lr or S)

Note: LCO5 and LCO6 based on the lower of Overstrength or Expected Strength

Factored Chord Compression, P (lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	20	20	70	58	3840	2880	8633	6517

Factored Chord Strong-Axis Bending, Mx (ft-lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	0	0	0	0	0	0	0	0

			Interactions							
Level	Minimum Ma (ft-lb)	Minimum Pa (lb)	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	4564	15040	0.001	0.001	0.005	0.004	0.255	0.191	0.478	0.361

Ties and Holdowns

Level	Holdown	Quantity	Config	Exposed Rod Length (in)	Holdown Capacity Ta (lb/Each)	LRFD Capacity Φ Tn (lb)	Holdown Disp at Φ Tn (lb)	Holdown height (in)	Rod Dia. (in)

Gridline 2 Walls

USE S/HDU11-54
HOLD DOWN

Project Name: Centeris

Model: LFRS Shearwall –2

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

1	S/HDU11 - 54	1	Base	4	7665	12265	0.109	16.625	0.875

Level	Holdown Offset from End of Shear Wall (in)
1	0.0

Load Combinations (ASCE7-16 ASD)

LC7 = 0.6D + 0.6W

LC8 = 0.6D + 0.7E

LCO8 = (0.6-0.14Sds)D + 0.7Ω_oQe Note: LCO8 based on the lower of Overstrength or Expected Strength

	Factored Net Uplift (lb) (Negative values represent uplift, Positive values indicate no net uplift)			Shear Forces (lb)		
Level	LC7	LC8	LC08	Wind	Seismic	Seismic w/Overstrength
1	-3828	-2858	-8601	960	718	2152

Ratio (Factored Net Uplift)/(Holdown Capacity)

Level	LC7	LC8	LC08
1	0.499	0.373	0.935

Displacement

	Floor-Floor Relative Displacement (in)			Drift %		
Level	Wind	Seismic	Seismic, Cd	Wind	Seismic	Seismic, Cd
1	0.43	0.25	1	0.25	0.15	0.59

Gridline A/B Walls

Project Name: Centeris

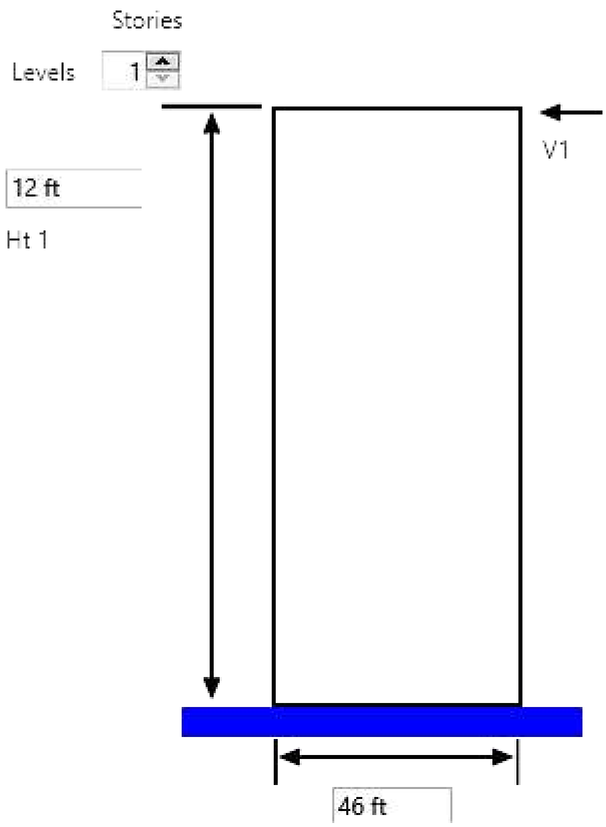
Model: LFRS Shearwall – A

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report



Load Inputs (All Loads are Unfactored Forces)

Top of Level Shear (lb)

Level	Wind	Seismic	Aspect Ratio
1	1000	2050	0.26

Seismic Design Parameters:

Seismic Design Category = D S_{DS} = 1.006 Ie = 1

Level	Overstrength Factor, Ω_0	Defl Amplification Factor, Cd
1	3	4

Allowable Strength Increase for Load Combinations w/Overstrength Included? Yes

Additional Applied Chord Axial Loads (lb) - Unfactored

Level	D	L	Lr	S	W
1	0	0	0	0	0

Additional Applied Chord Moments (ft-lb) - Unfactored

Level	D	L	Lr	S	W	E
1	0	0	0	0	0	0

Total and Unit Shear Forces (Factored)

	Wind Shear Forces		Seismic Shear Forces	
Level	V, Total (lb)	v, per ft (lb/ft)	V, Total (lb)	v, per ft (lb/ft)
1	600	13.	1435	31.

USE OSB SHEATHING (1) SIDE W/ 6"OC E.N. & 12"OC F.N.

Shear Wall Sheathing and Fastener Selection

Level	Sheathing	Fastener Size	Edge/Field Fastener Spac (in)	Framing Thickness (mils)	Max Framing Spac (in)	One or Two Sides
1	7/16" OSB	No. 8	6/12	54	24	2

Shear Strength Modification Factors

Level	Wind Modifiers	Seismic Modifiers

Gridline A/B Walls

Project Name: Centeris

Model: LFRS Shearwall – A

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

1 None None

Available Shear Strength and Shear Ratios

	Wind			Seismic		
Level	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va	Aspect Ratio Factor	Available Shear Strength, va (lb/ft)	Shear Ratio v/va
1	1	910	0.014	1	752	0.041

USE (1) 600S200-54
@ EA HOLD DOWN

Chords

				Bracing (in)					
Level	Section	Fy (ksi)	Configuration	Flexural	Axial KyLy	Axial KtLt	Flex Kϕ (lb-in/in)	Axial Kϕ (lb-in/in)	Bracing, Lm (in)
1	600S200-54	50	Single	60	60	60	0	0	None

Load Combinations ASCE7-16 ASD

LC1 = D
LC2 = D + L
LC3 = D + (Lr or S)
LC4 = D + 0.75L + 0.75(Lr or S)
LC5 = D + (0.6W or 0.7E)
LC6 = D + 0.75(0.6W or 0.7E) + 0.75L + 0.75(Lr or S)
LCO5 = (1.0 + 0.14Sds)D + 0.7Ω_oQe
LCO6 = (1.0 + 0.105 Sds)D + 0.525Ω_oQe + 0.75L + 0.75(Lr or S)

Note: LCO5 and LCO6 based on the lower of Overstrength or Expected Strength

Factored Chord Compression, P (lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	0	0	0	0	374	281	1123	842

Factored Chord Strong-Axis Bending, Mx (ft-lb)

Level	LC1	LC2	LC3	LC4	LC5	LC6	LC05	LC06
1	0	0	0	0	0	0	0	0

			Interactions				
Level	Minimum Ma (ft-lb)	Minimum Pa (lb)	LC1	LC5	LC6	LC05	LC06
1	2282	6175	0	0.061	0.045	0.152	0.114

USE S/HDU4-54
HOLD DOWN

Ties and Holdowns

Level	Holdown	Quantity	Config	Exposed Rod Length (in)	Holdown Capacity Ta (lb/Each)	LRFD Capacity Φ Tn (lb)	Holdown Disp at Φ Tn (lb)	Holdown height (in)	Rod Dia. (in)
1	S/HDU4 - 54	1	Base	4	2550	4080	0.053	7.875	0.625

Gridline A/B Walls

Project Name: Centeris

Model: LFRS Shearwall – A

Date: 10/16/2024

Code: 2012 NASPEC [AISI S100-2012]
AISI S400-15/S1-16 AISI S240-15

Simpson Strong-Tie® CFS Designer™ 5.0.1.0

LFRS Shearwall Summary Report

Level	Holdown Offset from End of Shear Wall (in)
1	0.0

Load Combinations (ASCE7-16 ASD)

LC7 = 0.6D + 0.6W

LC8 = 0.6D + 0.7E

LCO8 = (0.6-0.14Sds)D + 0.7Ω_oQe Note: LCO8 based on the lower of Overstrength or Expected Strength

	Factored Net Uplift (lb) (Negative values represent uplift, Positive values indicate no net uplift)			Shear Forces (lb)		
Level	**LC7**	**LC8**	**LC08**	**Wind**	**Seismic**	**Seismic w/Overstrength**
1	-157	-374	-1123	600	1435	4305

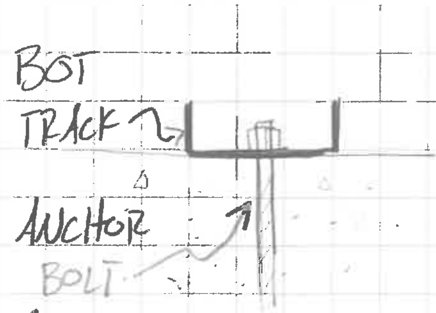
Ratio (Factored Net Uplift)/(Holdown Capacity)

Level	LC7	LC8	LC08
1	0.061	0.147	0.367

Displacement

	Floor-Floor Relative Displacement (in)			Drift %		
Level	**Wind**	**Seismic**	**Seismic, Cd**	**Wind**	**Seismic**	**Seismic, Cd**
1	0	0	0.02	0	0	0.01

Shear Transfer - Bot Track Anchors



AISI S100 - Section E3

Track thickness, $t = 54$ mil
$F_y = 50$ ksi ; $F_u = 65$ ksi

Anchor Bolt ϕ, $D = 0.625''$
$A_b = 0.31 in^2$

AISI S100 E3.3.1.1 $P_n = C \times m_f \times d \times t \times F_u$ $\Omega = 2.5$

$C = 4 - 0.1(d/t) = 4 - 0.1\left(0.625''/0.054''\right) = 2.84$

$m_f = 0.75$ (w/ washer)

$P_n = (2.84)(0.75)(0.625'')(.054'')(65 ksi)$
$= 4.67 kips$

$P_n/\Omega = 4.67 kips / 2.5 = 1.87 kips$

Try Bolt Hole Deformation

AISI S100 E3.3.2.1 $P_n = (4.64\alpha t + 1.53) d \times t \times F_u$; $\Omega = 2.22$

$\alpha = 1.0$

$P_n = (4.64 \cdot (1.0) \cdot (.054'') + 1.53) \cdot (0.625'')(.054'') \cdot (65 ksi)$
$= 3.91 kips$

$P_n/\Omega = 3.91 kips / 2.22 = \underline{1.76 kips}$ per-anchor

ALTERNATE BOTTOM TRACK ANCHOR

Hilti PROFIS Engineering 3.0.91

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

Specifier's comments:

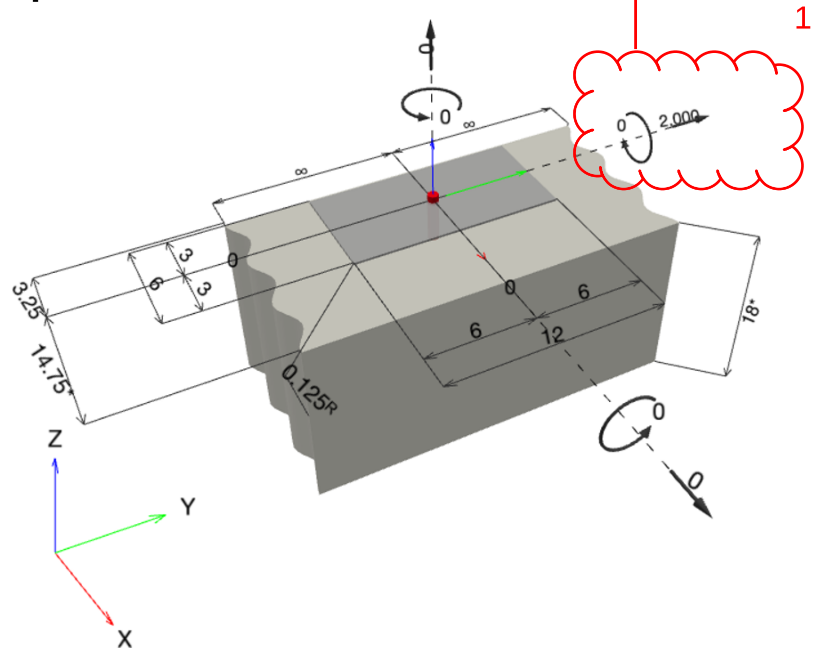
1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 5/8 (3 1/4)**
Item number:	418078 KH-EZ 5/8"x3 1/2"
Effective embedment depth:	$h_{ef,act}$ = 2.390 in., h_{nom} = 3.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	4/1/2022 | 12/1/2023
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.125 in.
Anchor plate^R :	l_x x l_y x t = 6.000 in. x 12.000 in. x 0.125 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 18.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

2000# PER ANCHOR
∴ BOLT HOLE
DEFORMATION
CONTROLS @
1.7KIPS/BOLT

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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Address:		Specifier:	
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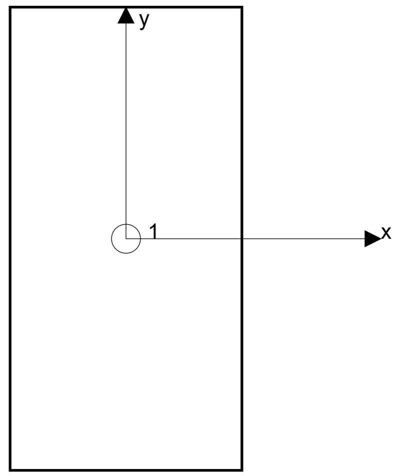
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 0; V_y = 2,000; M_x = 0; M_y = 0; M_z = 0;	no	99

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	2,000	0	2,000

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

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Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2,000	6,732	30	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	2,000	2,037	99	OK
Concrete edge failure in direction x-**	2,000	2,968	68	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.27	90,180

Calculations

V_{sa} [lb]
11,220

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
11,220	0.600	6,732	2,000

4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \, \psi_{c,N} \, \psi_{cp,N} \, N_b \right]$ ACI 318-19 Eq. (17.7.3.1a)

$\phi \, V_{cp} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \, h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \, \lambda_a \, \sqrt{f'_c} \, h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1	2.390	3.250	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
3.630	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
49.01	51.41	0.972	1.000	3,141

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi \, V_{cp}$ [lb]	V_{ua} [lb]
2,910	0.700	2,037	2,000

4.3 Concrete edge failure in direction x-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$	ACI 318-19 Eq. (17.7.2.1a)
$\phi\ V_{cb} \geq V_{ua}$	ACI 318-19 Table 17.5.2
A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)	
$A_{Vc0} = 4.5\ c_{a1}^2$	ACI 318-19 Eq. (17.7.2.1.3)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-19 Eq. (17.7.2.4.1b)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-19 Eq. (17.7.2.6.1)
$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$	ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
3.250	-	1.000	18.000	2.390

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.625	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
47.53	47.53	1.000	1.000	2,120

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
4,240	0.700	2,968	2,000

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

Fastening meets the design criteria!

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

6 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 5/8 (3 1/4)

Item number: 418078 KH-EZ 5/8"x3 1/2"

Maximum installation torque: 1,020 in.lb

Hole diameter in the base material: 0.625 in.

Hole depth in the base material: 3.625 in.

Minimum thickness of the base material: 5.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.750 in.

Plate thickness (input): 0.125 in.

Recommended plate thickness: not calculated

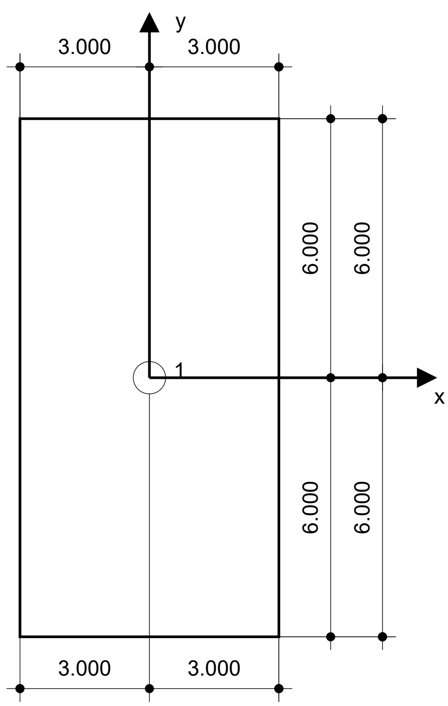
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KH-EZ screw anchor with 3.25 in embedment, 5/8 (3 1/4), Carbon steel, installation per ESR-3027

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque wrench • Hilti SIW 9-A22 Impact Wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	3.250	14.750	-	-

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Alt Bot Track Anchor	Date:	2/14/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

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