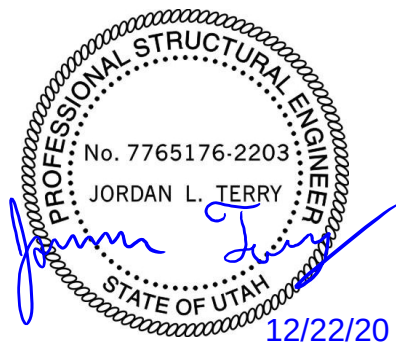




Bank of America - Centerville

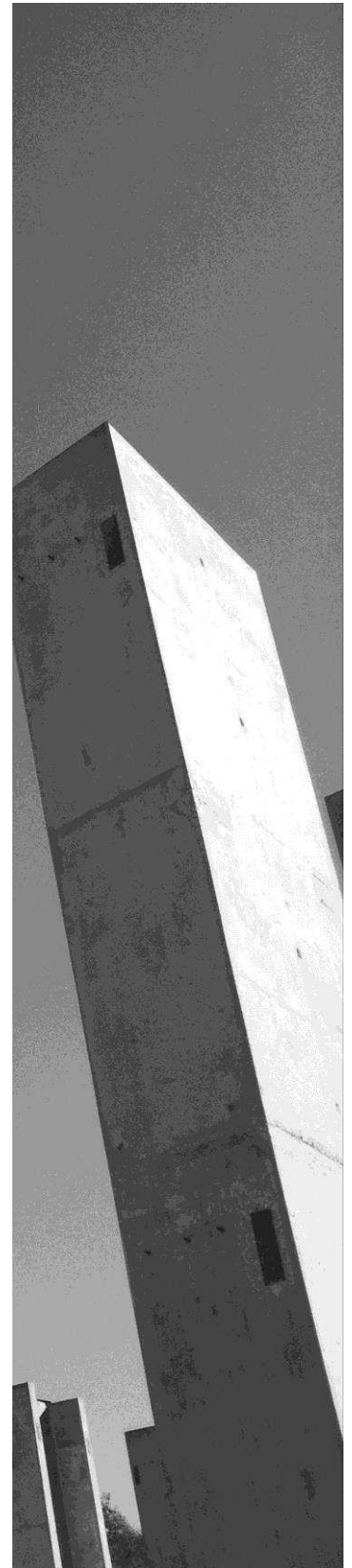
City Submittal

Calculation Package



KPFF JOB #10211900134

December 22, 2020



**Bank of America - Centerville, UT
City Submittal Structural Calculations**

KPFF JOB #10211900134

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DESIGN LOAD CRITERIA



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project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no. 10211900134		
1.0 Design Criteria				

CODE: **IBC 2018**

Risk Category: **II**

Project Location

Latitude: **40.9219** Longitude: **-111.8887** Address: **782 W Parrish Ln**

Snow Loading

$p_g =$	36.0 PSF	ASCE 7 Fig 7-1	$p_m =$	20.0 PSF	ASCE 7 Sec. 7.3.4
$C_s =$	1.0	ASCE 7 Fig 7-2	$p_r =$	25.2 PSF	ASCE 7 Eqn 7.3-1
$C_e =$	1.0	ASCE 7 Table 7-2	$p_s =$	25.2 PSF	ASCE 7 Eqn 7.4-1
$C_t =$	1.0	ASCE 7 Table 7-3	$\gamma =$	18.7 PCF	ASCE 7 Eqn 7.7-1
$I_s =$	1.0	ASCE 7 Table 1.5-2	$h_b =$	1.3 FT	ASCE 7 Sec. 7.1

Jurisdiction min : **30.0 PSF**

Rain Loading

Design Rainfall: **1.5 IN** (100-year, 1-hour per IBC Fig. 1611.1)

Seismic Criteria

Lateral System: **Light-frame (Cold-formed Steel) walls sheathed with Wood Structural Panels**

Risk Category:	II	ASCE 7 Table 1.5-2	R:	6.5	ASCE 7 Table 12.2-1
Site Class:	D - Default	ASCE 7 Sec. 20.1	Ω^* :	2.5	ASCE 7 Table 12.2-1
I_e :	1.00	ASCE 7 Table 1.5-2	C_d :	4	ASCE 7 Table 12.2-1
S_s :	1.346	ATC Seismic Maps	S_{DS} :	1.077	ATC Seismic Maps
1 S_1 :	0.499	ATC Seismic Maps	S_{D1} :	0.599	ATC Seismic Maps

NOTE: Ω_0 reduced by 0.5 for flexible diaphragm per ASCE 7 Table 12.2-1 Note b.

Wind Criteria

Wind Speed, Vult :	140 MPH	ASCE 7 Sec 26.5	K_{zt} :	1.0	ASCE 7 Sec. 26.8
Exposure Category:	C	ASCE 7 Sec 26.7.3	K_d :	0.85	ASCE 7 Table 26.6-1
Risk Category:	II	ASCE 7 Table 1.5-1	K_z :	0.849	ASCE 7 Table 26.10-1
Mean Roof Height:	15.0 FT		K_e :	0.857	ASCE 7 Table 26.9-1
Roof Slope:	0.25:12	Enter as Rise to Run	G :	0.85	ASCE 7 Sec. 26.11
Elevation, Z_g :	4248 FT		θ (deg) :	1.19	

Foundation Criteria

Report : **Yes**

Bearing : **2000 PSF** (Seismic +50%)

Friction ϕ : **0.4**

Passive : **300 PSF/FT**

Frost Depth : **30 IN**



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project	BOA Centerville	by	NRP	sheet no.
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	1.1 Roof - Design Load			

Dead Loads

Deck Self Weight: Weight of MTL Deck	3 PSF
Roof Insulation: Built up Insulation	6 PSF
Roof Covering: Single Ply Waterproofing	1 PSF
MEP: Mechanical Equipment, Plumbing, etc	3 PSF
Fire Sprinkler:	1 PSF
Ceiling Finish: Gyp or Suspended Accoustical Tile	3 PSF
Misc: Ceiling Insulation, etc	4 PSF
	Superimposed Dead Load = 18 PSF
	Load for Joists, Beams and Girders = 21 PSF
Joist/Beam:	2 PSF
	Load for Columns = 23 PSF
Column Weight:	1 PSF
	Load for Foundations = 24 PSF

Seismic Mass

Partitions: (If Required) Per ASCE 7 Sec 12.7.2	10 PSF
Storage Loads: (If Required) Per ASCE 7 Sec 12.7.2	0 PSF
Snow Loads: (If Required) Per ASCE 7 Sec 12.7.2	0 PSF
	Load used for Seismic Mass = 34 PSF
Equipment: Where occurs, if not included in MEP above	40 PSF

<u>Load Type Per ASCE 7 Table 4-1</u>	<u>Reducible?</u>	<u>Load</u>	<u>Point Load</u>
Roof Live	Yes	20 PSF	0.0 KIPS



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1.2 Canopy - Design Load				

Dead Loads

Deck Self Weight: **Weight of MTL Deck, Infill Framing** 5 PSF

Roof Insulation: **Built up Insulation** 6 PSF

Roof Covering: **Single Ply Waterproofing** 1 PSF

MEP: **Electrical** 1 PSF

Fire Sprinkler: **N/A** 0 PSF

Ceiling Finish: **Gyp Ceiling** 2 PSF

Misc: **Insulation, etc** 3 PSF

Superimposed Dead Load = 13 PSF

Load for Joists, Beams and Girders = 18 PSF

Beam/Girder: 5 PSF

Load for Columns = 23 PSF

Column Weight: 1 PSF

Load for Foundations = 24 PSF

Seismic Mass

Partitions: (If Required) Per ASCE 7 Sec 12.7.2 0 PSF

Storage Loads: (If Required) Per ASCE 7 Sec 12.7.2 0 PSF

Snow Loads: (If Required) Per ASCE 7 Sec 12.7.2 0 PSF

Load used for Seismic Mass = 24 PSF

Equipment: Where occurs, if not included in MEP above 40 PSF

<u>Load Type Per ASCE 7 Table 4-1</u>	<u>Reducible?</u>	<u>Load</u>	<u>Point Load</u>
Roof Live	Yes	20 PSF	0.0 KIPS



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project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no.	10211900134	
1.3 Exterior Wall - Design Load				

Dead Loads

Wall Type: Metal Stud Exterior Wall w/ Stucco

Self Weight: 8" Metal Studs 3 PSF

Wall Covering: 5/8" Gyp, 1/2" Ply 5 PSF

Wall Insulation: 2" Rigid 3 PSF

Veneer 7/8" Stucco 7 PSF

Misc: 2 PSF

Load used for Seismic Mass = 20 PSF

Wall Type: Metal Stud Exterior Wall w/ Metal Panel Popout

Self Weight: 8" Metal Studs, 4" exterior studs 4 PSF

Wall Covering: 5/8" Gyp, 1/2" Ply 5 PSF

Wall Insulation: 2" Rigid 3 PSF

Veneer Metal Panel 3 PSF

Misc: 3 PSF

Load used for Seismic Mass = 18 PSF

Wall Type: Stone Veneer Roof Wall

Self Weight: 2 layer 6" Metal Studs 4 PSF

Wall Covering: 2 layer 1/2" Ply 3 PSF

Wall Insulation: 1 PSF

Veneer 2 layer 2" max stone veneer 20 PSF

Misc: HSS Framing + Misc 7 PSF

Load used for Seismic Mass = 35 PSF

Wall Type: Exterior Glass Storefront

Self Weight: 15 PSF

Load used for Seismic Mass = 15 PSF

2018 Utah Ground Snow Load Map

BOA Centerville Ground Snow



Latitude: 40.922

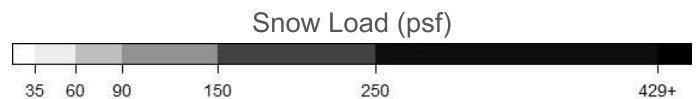
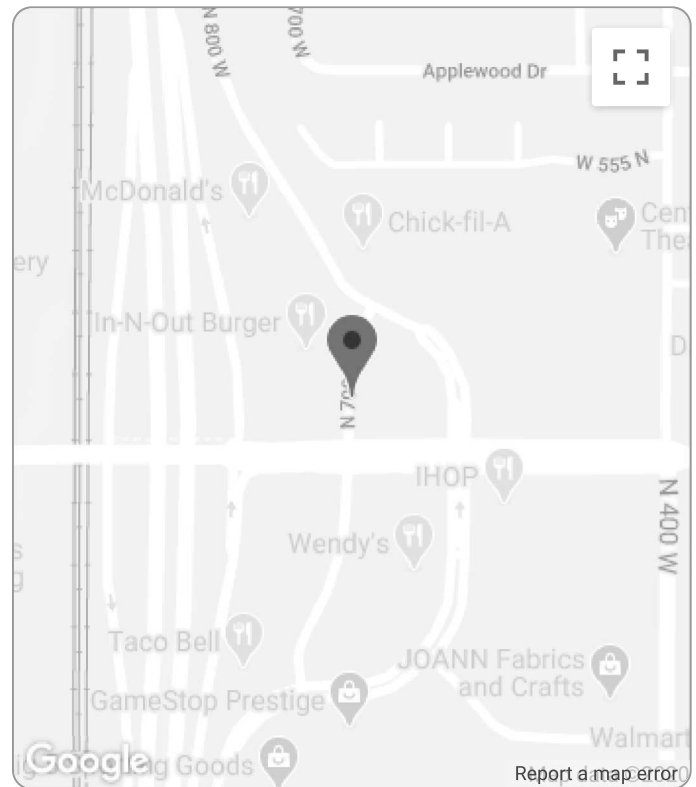
Longitude: -111.889

Elevation: 4,243 ft

Ground Snow Load:

33 psf / 1.57 kPa

36 psf per city



*This document is not legally binding. The user is urged to verify ground snow load values with the local authority having jurisdiction.

These ground snow load values represent 50-year ground snow load estimated value at a 2% probability of exceedance for the location given. The grid used in the map is 3350ft by 3350ft. Elevations for these grid cells were estimated by aggregating data from 100ft by 100ft USGS digital elevation models and may not coincide with the actual site elevation. These predictions are calculated using the process outlined in The Utah Snow Load Study.¹

Final predictions given are bounded at a lower limit for a minimum ground snow load of 21 psf to meet ASCE 7. Estimated values for snow loads at elevations significantly higher than all nearby stations lead to unreasonably high snow load estimates, therefore, the predictions in the map are not allowed to extend beyond the highest 50-year station ground snow load of 429 psf. Elevations over 9,000 ft are also considered less accurate due to the limited number of stations at these elevations. The results shown in this report have included a warning if the results have reached or exceeded the upper limit.

While great efforts have been made to ensure these predictions are as accurate as possible, designers must use expert judgement to ensure that such predictions are appropriate for their particular project. The SEAU and the authors cannot accept responsibility for prediction errors or any consequences resulting therefrom.

¹ Bean, Brennan; Maguire, Marc; and Sun, Yan, "The Utah Snow Load Study" (2018). Civil and Environmental Engineering Faculty Publications. Paper 3589.

project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no. 10211900134		
	1.4 Snow Drift			

Snow Loading - Roof

$p_g =$	36.0 PSF	ASCE 7 Fig 7-1	$p_m =$	20 PSF	ASCE 7 Sec. 7.3.4
$C_s =$	1.00	ASCE 7 Fig 7-2	$p_f =$	25 PSF	ASCE 7 Eqn 7.3-1
$C_e =$	1.00	ASCE 7 Table 7-2	$p_s =$	25 PSF	ASCE 7 Eqn 7.4-1 (balanced)
$C_t =$	1.00	ASCE 7 Table 7-3	$\gamma =$	19 PSF	ASCE 7 Eqn 7.7-1
$I_s =$	1.00	ASCE 7 Table 1.5-2	$h_b =$	1.3 FT	ASCE 7 Sec. 7.1
Jurisdiction min : 30.0 PSF					

Snow Drifts ASCE 7 Sec. 7.7 and 7.8

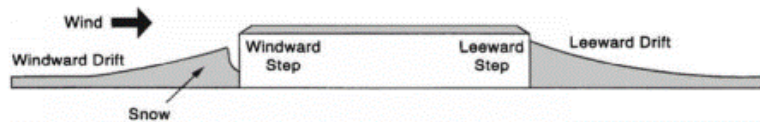


FIGURE 7.7-1 Drifts Formed at Windward and Leeward Steps

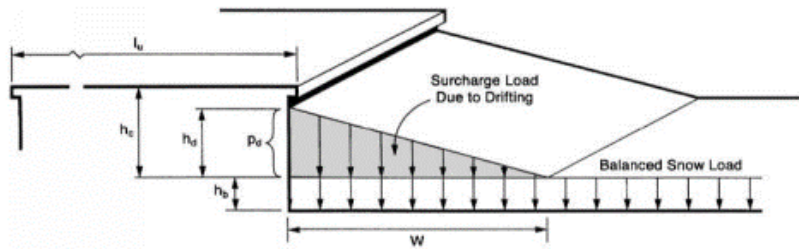
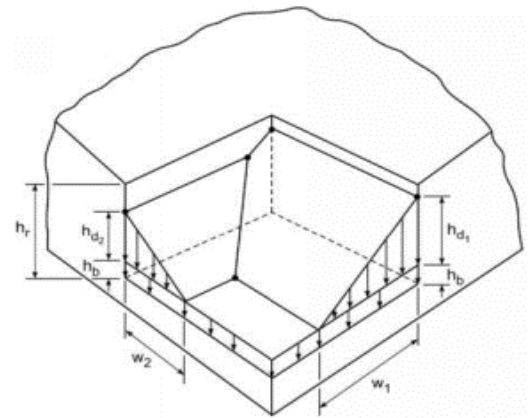


FIGURE 7.7-2 Configuration of Snowdrifts on Lower Roofs



Uniform Snow Loads on Upper Roof not Shown

FIGURE 7.7-3 Configuration of Intersecting Snowdrifts at Lower Roof

	Total Step Height							
		Upper Roof	Lower Roof	Upper Roof	Lower Roof			
Location	$h_c + h_b$	$l_{u_leeward}$	$l_{u_windward}$	$h_{d_leeward}$	$h_{d_windward}$	h_{d_max}	w	P_{d_max}
Grid 1/2	4.0 ft	0.0 FT	66.0 ft	0.0 FT	2.3 FT	2.3 FT	9 FT	42 PSF
3 step	2.0 ft	37.0 FT	29.0 ft	2.2 FT	1.5 FT	2.2 FT	5 FT	12 PSF
4.5 N	6.0 ft	0.0 FT	37.0 ft	0.0 FT	1.7 FT	1.7 FT	7 FT	40 PSF
4.5 S	6.0 ft	0.0 FT	6.0 ft	0.0 FT	0.4 FT	0.4 FT	2 FT	4 PSF
5 Roof	2.0 ft	0.0 FT	6.0 ft	0.0 FT	0.4 FT	0.4 FT	2 FT	6 PSF
A/D Hi	2.0 ft	0.0 FT	49.5 ft	0.0 FT	2.0 FT	2.0 FT	5 FT	33 PSF
A/D Lo	4.0 ft	0.0 FT	49.5 ft	0.0 FT	2.0 FT	2.0 FT	8 FT	27 PSF



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project	BOA Centerville	by	NRP	sheet no.
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client	Gensler	job no. 10211900134		
	1.5 Snow Drift			

Snow Loading - Canopy

$p_g =$	36.0 PSF	ASCE 7 Fig 7-1	$p_m =$	20 PSF	ASCE 7 Sec. 7.3.4
$C_s =$	1.0	ASCE 7 Fig 7-2	$p_f =$	30 PSF	ASCE 7 Eqn 7.3-1
$C_e =$	1.0	ASCE 7 Table 7-2	$p_s =$	30 PSF	ASCE 7 Eqn 7.4-1
$C_t =$	1.2	ASCE 7 Table 7-3	$\gamma =$	19 PSF	ASCE 7 Eqn 7.7-1
$I_s =$	1.0	ASCE 7 Table 1.5-2	$h_b =$	1.6 FT	ASCE 7 Sec. 7.1

Jurisdiction min : 30.0 PSF

Snow Drifts ASCE 7 Sec. 7.7 and 7.8

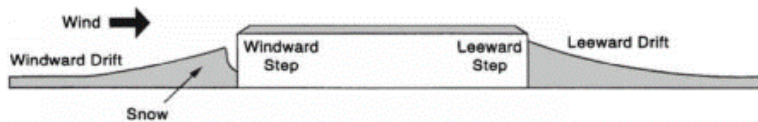


FIGURE 7.7-1 Drifts Formed at Windward and Leeward Steps

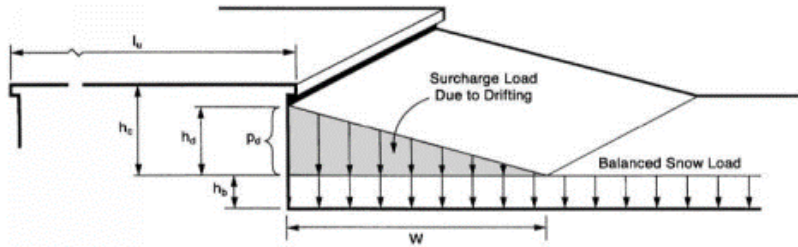
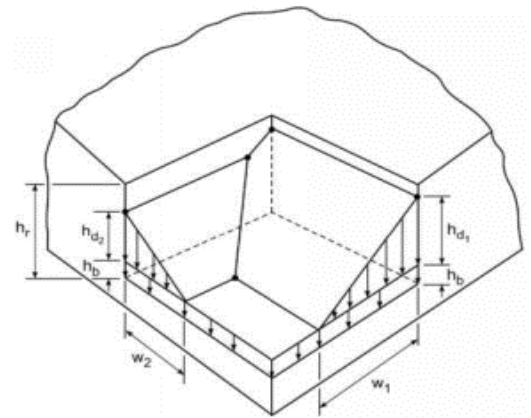


FIGURE 7.7-2 Configuration of Snowdrifts on Lower Roofs



Uniform Snow Loads on Upper Roof not Shown

FIGURE 7.7-3 Configuration of Intersecting Snowdrifts at Lower Roof

Location	Total Step Height $h_c + h_b$	Upper Roof	Lower Roof	Upper Roof	Lower Roof	h_{d_max}	w	P_{d_max}
		$l_{u_leeward}$	$l_{u_windward}$	$h_{d_leeward}$	$h_{d_windward}$			
Grid A	5.5 ft	0.0 FT	2.0 ft	0.0 FT	0 FT	0.0 FT	0 FT	0 PSF
Grid D	5.5 ft	0.0 FT	24.0 ft	0.0 FT	1.3 FT	1.3 FT	5 FT	24 PSF
4.5 Wall N	10.0 ft	0.0 FT	42.0 ft	0.0 FT	1.8 FT	1.8 FT	7 FT	40 PSF
Grid 5	5.5 ft	0.0 FT	6.0 ft	0.0 FT	0.4 FT	0.4 FT	2 FT	4 PSF

Search Information

Address: 782 W Parrish Ln, Centerville, UT 84014, USA

Coordinates: 40.9218852, -111.8886712

Elevation: 4248 ft

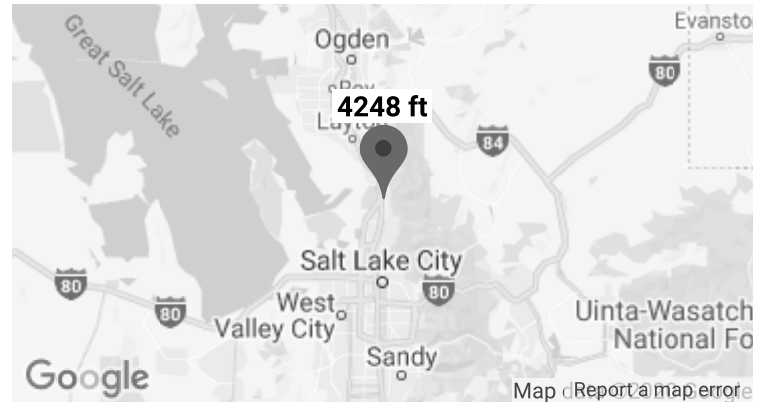
Timestamp: 2020-03-16T17:04:11.824Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default



Basic Parameters

Name	Value	Description
S_S	1.346	MCE_R ground motion (period=0.2s)
S_1	0.499	MCE_R ground motion (period=1.0s)
S_{MS}	1.615	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.077	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	0.864	Coefficient of risk (0.2s)
CR_1	0.877	Coefficient of risk (1.0s)
PGA	0.61	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.732	Site modified peak ground acceleration

T _L	8	Long-period transition period (s)
SsRT	1.346	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.559	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.843	Factored deterministic acceleration value (0.2s)
S1RT	0.499	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.57	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.225	Factored deterministic acceleration value (1.0s)
PGAd	1.114	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.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no. 10211900134		
2.0 Seismic Design				

Seismic Criteria

Lateral System: Light-frame (Cold-formed Steel) walls sheathed with Wood Structural Panels

Risk Category:	II	ASCE 7 Table 1.5-2	R:	6.5	ASCE 7 Table 12.2-1
Site Class:	D - Default	ASCE 7 Sec. 20.1	Ω :	2.50	ASCE 7 Table 12.2-1
I_e :	1.00	ASCE 7 Table 1.5-2	C_d :	4.00	ASCE 7 Table 12.2-1
S_s :	1.346	ATC Seismic Maps	S_{DS} :	1.077	ATC Seismic Maps
S_1 :	0.499	ATC Seismic Maps	S_{D1} :	0.599	ATC Seismic Maps
			p:	1.0	ASCE 7 Sec 12.3.4.1

Seismic Design Category Determination

ASCE 7 TABLE 11.6-1			ASCE 7 TABLE 11.6-2		
SEISMIC DESIGN CATEGORY BASED ON S_{DS}			SEISMIC DESIGN CATEGORY BASED ON S_{D1}		
	RISK CATEGORY:			RISK CATEGORY:	
Value of S_{DS}	I or II or III	IV	Value of S_{D1}	I or II or III	IV
< 0.167 g	A	A	< 0.067 g	A	A
< 0.330 g	B	C	< 0.133 g	B	C
< 0.500 g	C	D	< 0.200 g	C	D
≥ 0.500 g	D	D	≥ 0.200 g	D	D
Controlling:	D		Controlling:	D	
NOTE: Use controlling Seismic Design Category:				D	

Seismic Period Determination

C_t :	0.02	ASCE 7 Table 12.8-2	T_L :	8.0 Sec	ASCE 7 Sec 11.4.5
h_n :	15.0 FT	ASCE 7 Sec. 11.2	T_a :	0.152 Sec	ASCE 7 Eqn 12.8-7
x:	0.75	ASCE 7 Table 12.8-2	T_s :	0.556 Sec	ASCE 7 Sec 11.4.6
			T_{upper} :	0.213 Sec	See ASCE 7 Sec 12.8.2 for application

Seismic Response Coefficient Determination - ELF Method

C_s :	0.166	ASCE 7 Eqn 12.8-2	*Exception 2 per ASCE 7 Sec 11.4.8 applied	
C_{smax} :	0.604	ASCE 7 Eqn 12.8-3,4		
C_{smin} :	0.047	ASCE 7 Eqn 12.8-5,6	C_{s_design} :	0.166

Seismic Base Shear Determination - ELF Method

$V_{seismic}$:	0.166 *Seismic Mass	ASCE 7 Eq 12.8-1
-----------------	---------------------	------------------



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client	Gensler	job no. 10211900134		
	2.1 Seismic Mass			

Roof Diaphragm :

	<u>Area</u>	<u>Weight</u>	<u>Roof Mass</u>
Roof Weight : Roof	3530 SF	34 PSF	120.0 KIPS
Canopy/Awnings	1345 SF	22 PSF	29.6 KIPS
(4) RTU Units			3.6 KIPS

	<u>Length</u>	<u>Height</u>	<u>Weight</u>	<u>Wall Mass</u>
Wall Weight : Exterior Wall	250.0 FT	7.5 FT	20 PSF	37.5 KIPS
Roof Wall	55.0 FT	6.0 FT	35 PSF	11.6 KIPS
Canopy Wall	20.0 FT	16.0 FT	35 PSF	11.2 KIPS
Parapet	250.0 FT	2.0 FT	20 PSF	10.0 KIPS

Total : 223 KIPS

Total Area : 4875 SF

Normalized Mass : 46 PSF

Totals :

Total Area : 4875 SF

Total Mass : 223 KIPS

Normalized Mass : 46 PSF

Seismic Cs : 0.166

ELF Base Shear : 37.0 KIPS



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project	BOA Centerville	by	NRP	sheet no.
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client	Gensler	job no. 10211900134		
2.2 Seismic Vertical Distribution				

Seismic Story Force

T : 0.152 Sec
k : 1.000 ASCE 7 Sec. 12.8.3
V : 37.0 KIPS

Story	Height	Seismic Wt	$w_x h_x^k$	C_{vx}	F_x
Roof	15.0 FT	223.5 KIPS	3351.9	1.0000	37.0 KIPS
Total :		223.5 KIPS	3351.9		37.0 KIPS

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \quad (12.8-11)$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (12.8-12)$$

where

C_{vx} = vertical distribution factor;

V = total design lateral force or shear at the base of the structure [kip (kN)];

w_i and w_x = portion of the total effective seismic weight of the structure (W) located or assigned to level i or x ;

h_i and h_x = height [ft (m)] from the base to level i or x ; and

k = an exponent related to the structure period as follows:

- for structures that have a period of 0.5 s or less, $k = 1$;
- for structures that have a period of 2.5 s or more, $k = 2$; and
- for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.



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client	Gensler	job no. 10211900134		
2.2 Seismic Vertical Distribution				

S_{DS} : 1.300 g I_e : 1.00

Seismic Diaphragm Forces ASCE 7 Sec. 12.10.1

Diaphragm	W_{px}	ΣW_i	ELF F_i	ΣF_i	
Roof	223 KIPS	223 KIPS	37 KIPS	37 KIPS	
	ASCE 7 Eqn 12.10-1	ASCE 7 Eqn 12.10-2	ASCE 7 Eqn 12.10-3		
Diaphragm	F_{px}	F_{px_min}	F_{px_max}	F_{px_design}	Scale
Roof	37 KIPS	58 KIPS	116 KIPS	58 KIPS	1.57

Seismic Collector Forces ASCE 7 Sec. 12.10.1

Ω : 2.50 * Multiply Model ELF Forces by Scale value
p : 1

Maximum of:

Diaphragm	$F_{px_diaphragm}$	ELF F_i	ΩF_{px} ELF	ΩF_{px} 12.10-1	p F_{px_min}	F_{px_design}	Scale*
Roof	37 KIPS	37 KIPS	93 KIPS	93 KIPS	37 KIPS	93 KIPS	2.50

Search Information

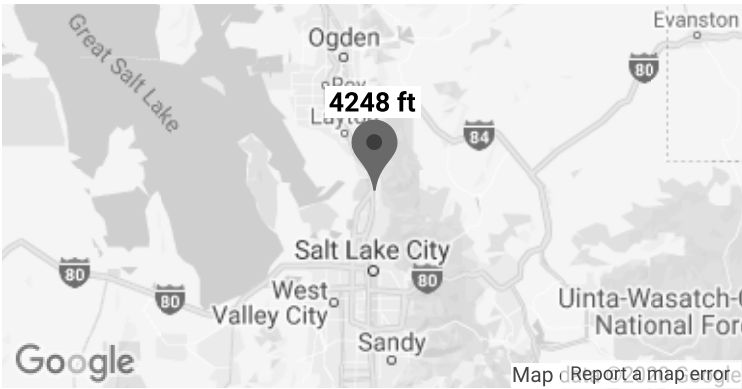
Address: 782 W Parrish Ln, Centerville, UT 84014, USA

Coordinates: 40.9218852, -111.8886712

Elevation: 4248 ft

Timestamp: 2020-03-13T22:43:26.349Z

Hazard Type: Wind



ASCE 7-16		ASCE 7-10		ASCE 7-05	
MRI 10-Year	74 mph	MRI 10-Year	76 mph	ASCE 7-05 Wind Speed	90 mph
MRI 25-Year	79 mph	MRI 25-Year	84 mph		
MRI 50-Year	84 mph	MRI 50-Year	90 mph		
MRI 100-Year	89 mph	MRI 100-Year	96 mph		
Risk Category I	97 mph	Risk Category I	105 mph		
Risk Category II	103 mph	Risk Category II	115 mph		
	140 mph per city				
Risk Category III	109 mph	Risk Category III-IV	120 mph		
Risk Category IV	113 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.

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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699 E. South Temple, Suite 200
Salt Lake City, UT 84102
(801) 441-2204

project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no. 10211900134		
	3.0 Wind Design			

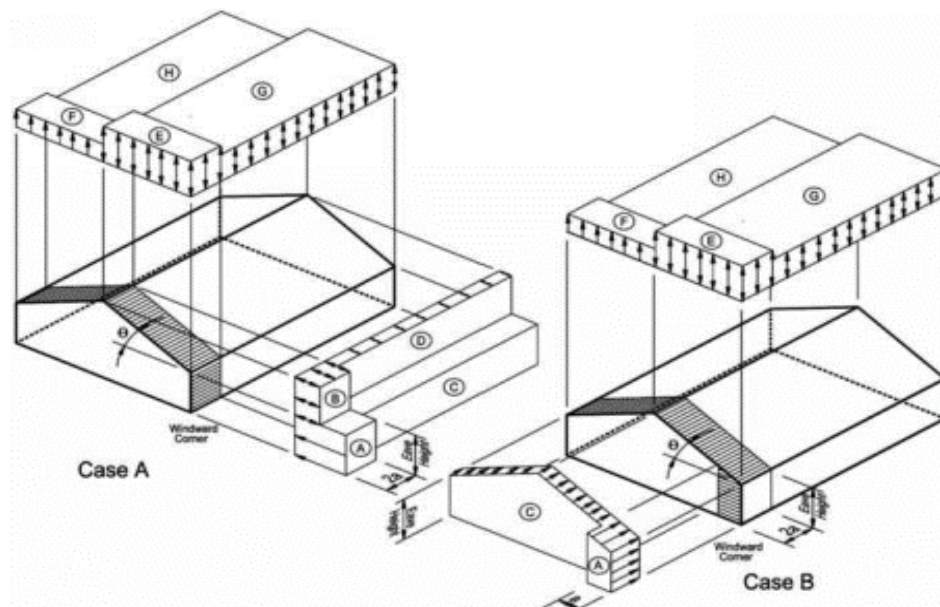
Wind Criteria

Wind Speed, Vult :	140	ASCE 7 Sec 26.5	K_{zt} :	1.00	ASCE 7 Sec. 26.8
Exposure Category:	C	ASCE 7 Sec 26.7.3	K_d :	0.85	ASCE 7 Table 26.6-1
Risk Category:	II	ASCE 7 Table 1.5-1	K_z :	0.85	ASCE 7 Table 26.10-1
Mean Roof Height:	15.0 FT		K_e :	0.86	ASCE 7 Table 26.9-1
Roof Slope:	0.3:12		G :	0.85	ASCE 7 Sec. 26.11
θ (deg) :	1.19		Least Horiz Dim:	51 FT	
Elevation, Z_g :	4248.0 FT		a :	5.1 FT	

NOTE: Applicable to enclosed, simple diaphragm, low-rise building (<60') per ASCE 7 Chapt. 28 Part 2

	Horizontal Pressures:				Vertical Pressures				Overhangs	
Zone:	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
Case 1:	37.6	-19.5	24.9	-11.6	-45.1	-25.7	-31.5	-19.8	-63.3	-49.5
Case 2:	-	-	-	-	-	-	-	-	-	-

Diagrams



Notation

- a 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
EXCEPTION: For buildings with $\theta = 0$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.
 h Mean roof height, in ft (m), except that eave height shall be used for roof angles $< 10^\circ$.
 θ Angle of plane of roof from horizontal, in degrees.



699 E. South Temple, Suite 200
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(801) 441-2204

project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	11/24/20	
client	Gensler	job no.	10211900134	
	3.1 Wind C&C Design			

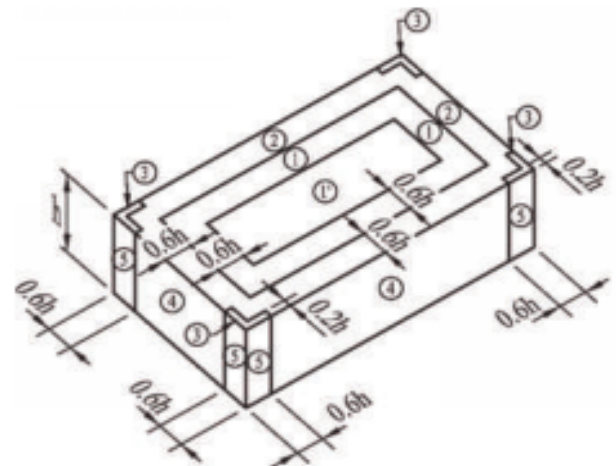
Wind Components & Cladding Criteria

Wind Speed, Vult :	140	ASCE 7 Sec 26.5	K_{zt} :	1.00	ASCE 7 Sec. 26.8
Exposure Category:	C	ASCE 7 Sec 26.7.3	K_d :	0.85	ASCE 7 Table 26.6-1
Risk Category:	II	ASCE 7 Table 1.5-1	K_z :	0.85	ASCE 7 Table 26.10-1
Mean Roof Height:	15.0 FT		K_e :	0.86	ASCE 7 Table 26.9-1
Roof Slope:	0.25:12		G :	0.85	ASCE 7 Sec. 26.11
θ (deg) :	1.19		Least Horiz Dim:	51 FT	
Elevation, Z_g :	4248.0 FT		a :	5.1 FT	0.2h: 3.0 FT
					0.6h: 9.0 FT

NOTE: Applicable to enclosed buildings that are regular in shape, $h \leq 60'$, Flat/Hip/Gable roof $\theta < 7^\circ$

Effect Wind Area (ft ²)	Roof Pressures								Wall Pressures			
	1+	1-	1'+	1'-	2+	2-	3+	3-	4+	4-	5+	5-
10	17.3	-68.0	17.3	-39.1	17.3	-89.7	17.3	-122.2	42.7	-46.2	42.7	-57.1
20	16.2	-63.5	16.2	-39.1	16.2	-83.9	16.2	-110.7	40.8	-44.4	40.8	-53.2
50	14.9	-57.6	14.9	-39.1	14.9	-76.2	14.9	-95.5	38.2	-41.9	38.2	-48.2
100	13.8	-53.1	13.8	-39.1	13.8	-70.5	13.8	-83.9	36.3	-39.9	36.3	-44.4

Effect Wind Area (ft ²)	Overhangs			Parapets per Fig 30.9-1			
	1/1'o	2o	3o	4+	4-	5+	5-
10	-61.5	-83.1	-115.7	132.4	-88.9	164.9	-99.8
20	-60.4	-75.5	-102.2	124.6	-85.2	151.5	-94.0
50	-58.9	-65.3	-84.5	114.5	-80.1	133.7	-86.4
100	-57.8	-57.6	-71.0	106.8	-76.2	120.2	-80.7



Flat/Hip/Gable ($0^\circ \leq \theta \leq 7^\circ$)



699 E. South Temple, Suite 200
Salt Lake City, UT 84102
(801) 441-2204

project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	12/08/20	
client	Gensler	job no. 10211900134		
3.2 Add'l Wind C&C				

Wind Components & Cladding Criteria

Wind Speed, Vult :	140	ASCE 7 Sec 26.5	K_{zt} :	1.00	ASCE 7 Sec. 26.8
Exposure Category:	C	ASCE 7 Sec 26.7.3	K_d :	0.85	ASCE 7 Table 26.6-1
Risk Category:	II	ASCE 7 Table 1.5-1	K_z :	0.85	ASCE 7 Table 26.10-1
Mean Roof Height:	15.0 FT		K_e :	0.86	ASCE 7 Table 26.9-1
Elevation, Z_g :	4248.0 FT		G :	0.85	ASCE 7 Sec. 26.11

Freestanding Rock Wall

$$F = q_z * G * C_f * A_r \quad \text{ASCE 7 Sec 29.4}$$

$$q_z = 31.0 \text{ PSF} \quad \text{ASCE 7 Sec 26.10}$$

1) 10'-0" Wall Section	s/h = 0.5	B/s = 2.0
	$C_f = 1.7$	$F = 44.9 \text{ PSF}$
2) 6'-0" Wall Section	s/h = 0.3	B/s = 8.0
	$C_f = 1.8$	$F = 47.5 \text{ PSF}$
3) 22'-0" Wall Section	s/h = 1.0	B/s = 0.2
	$C_f = 1.8$	$F = 47.5 \text{ PSF}$

Attached Canopy

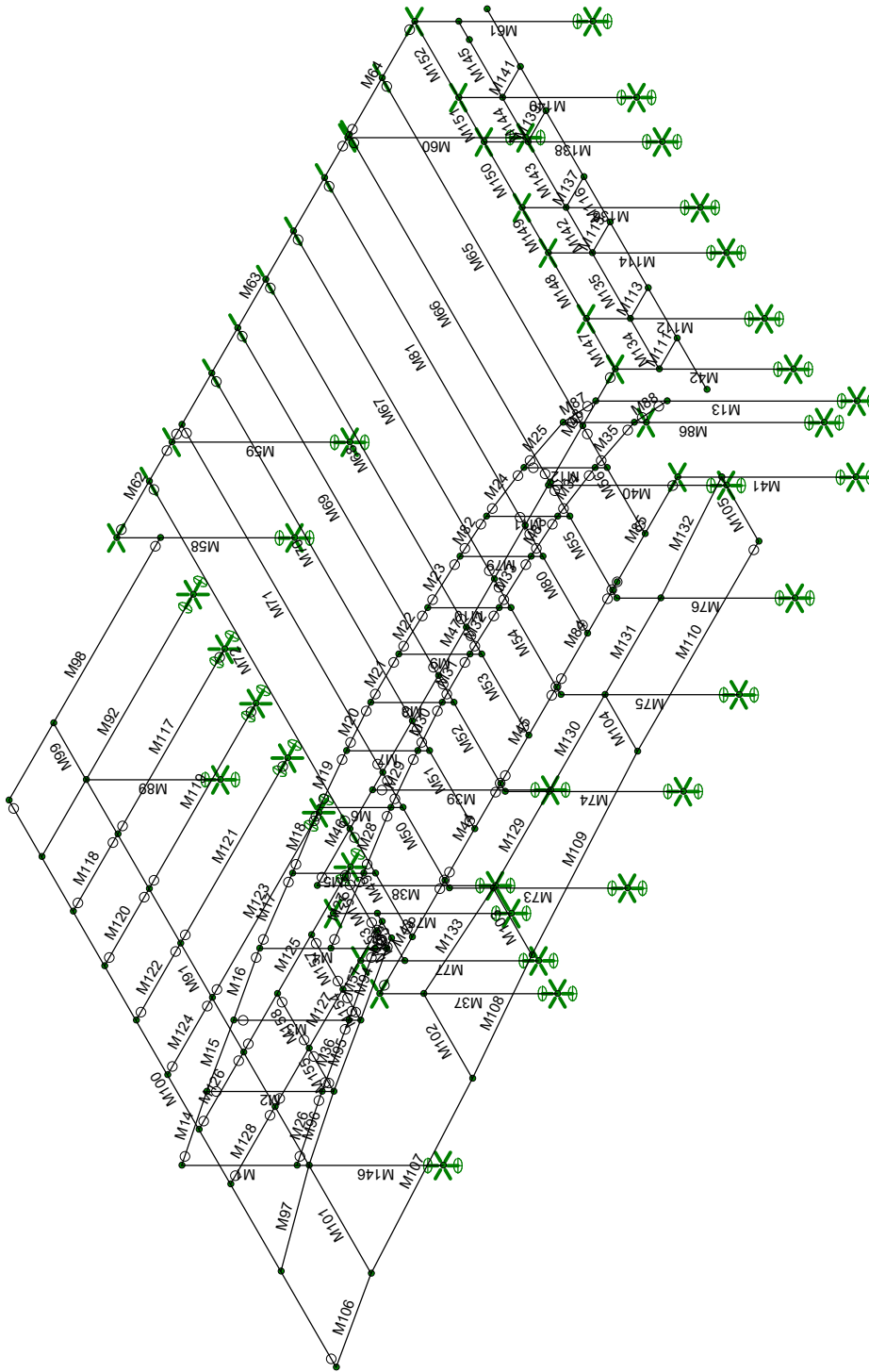
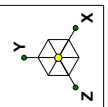
$$p = q_h * G C_p \quad \text{ASCE 7 Sec 30.11}$$

$$q_h = 31.0 \text{ PSF} \quad \text{ASCE 7 Sec 26.10} \quad h_c/h_e = 0.7 \quad \text{ASCE 7 Sec 30.11}$$

	10 sf	100 sf
Upper Surface	-37.3 PSF	-24.8 PSF
Lower Surface	-24.8 PSF	-21.7 PSF
Both Surface	-27.9 PSF	-18.6 PSF

GRAVITY SYSTEM DESIGN

RISA Model
Member Labels



Envelope Only Solution

KPFF

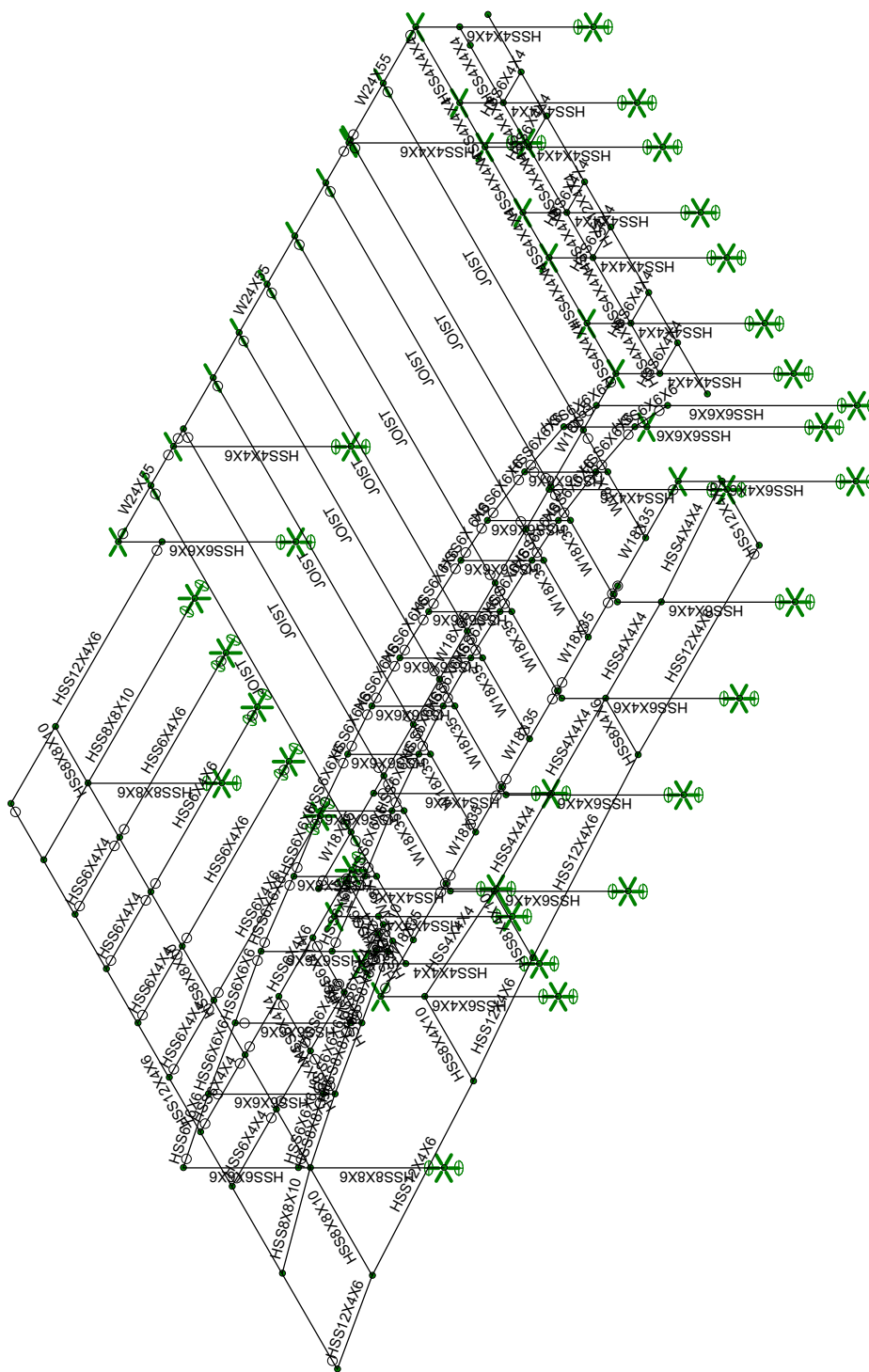
NRP

SK - 1

BOA Canopy Sail Wall

Dec 7, 2020 at 7:44 PM

BOA Canopy Sail (LRFD).r3d



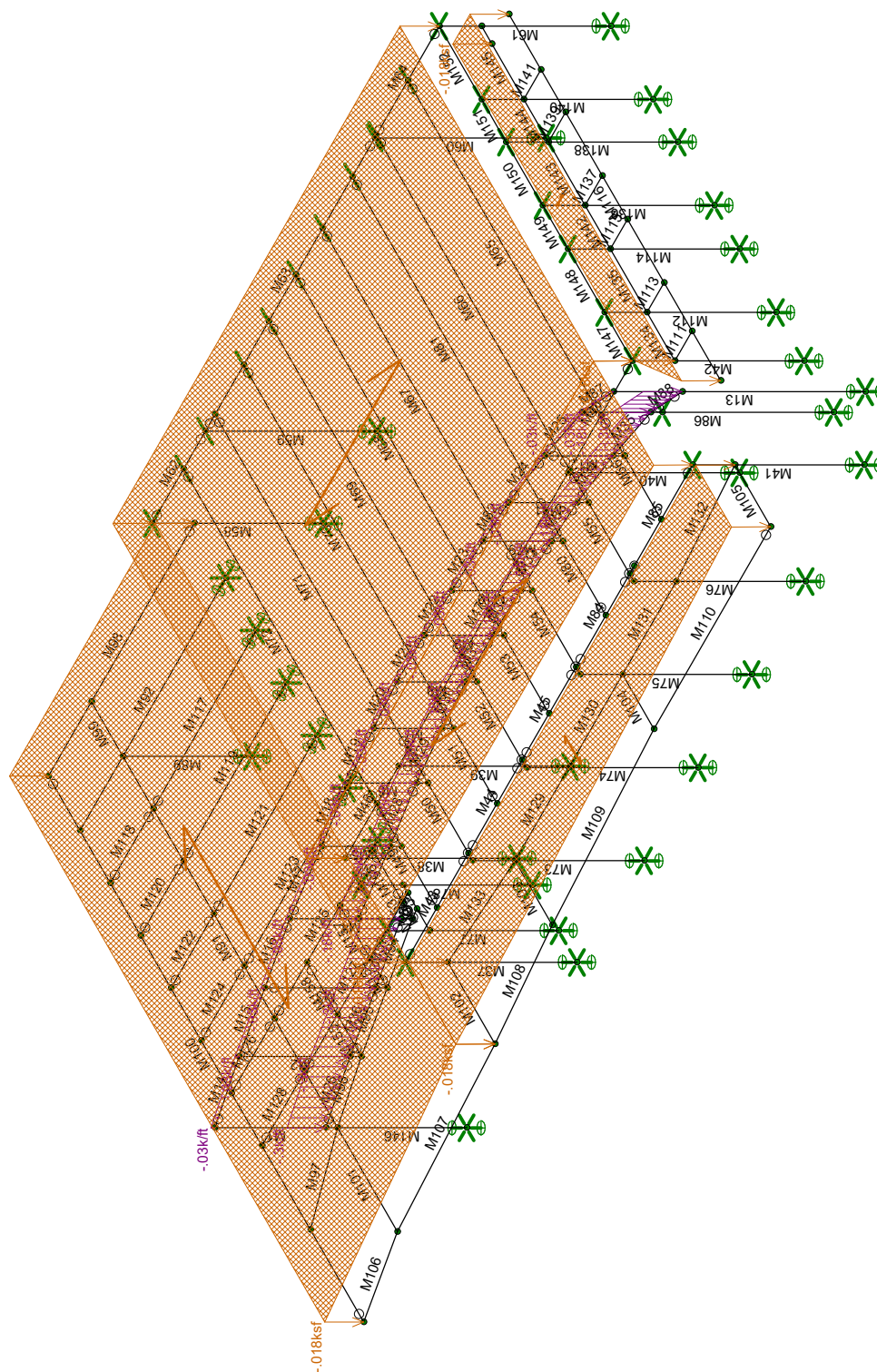
KPFF

NRP

BOA Canopy Sail Wall

Dec 7, 2020 at 7:43 PM

BOA Canopy Sail (LRFD).r3d



KPFF

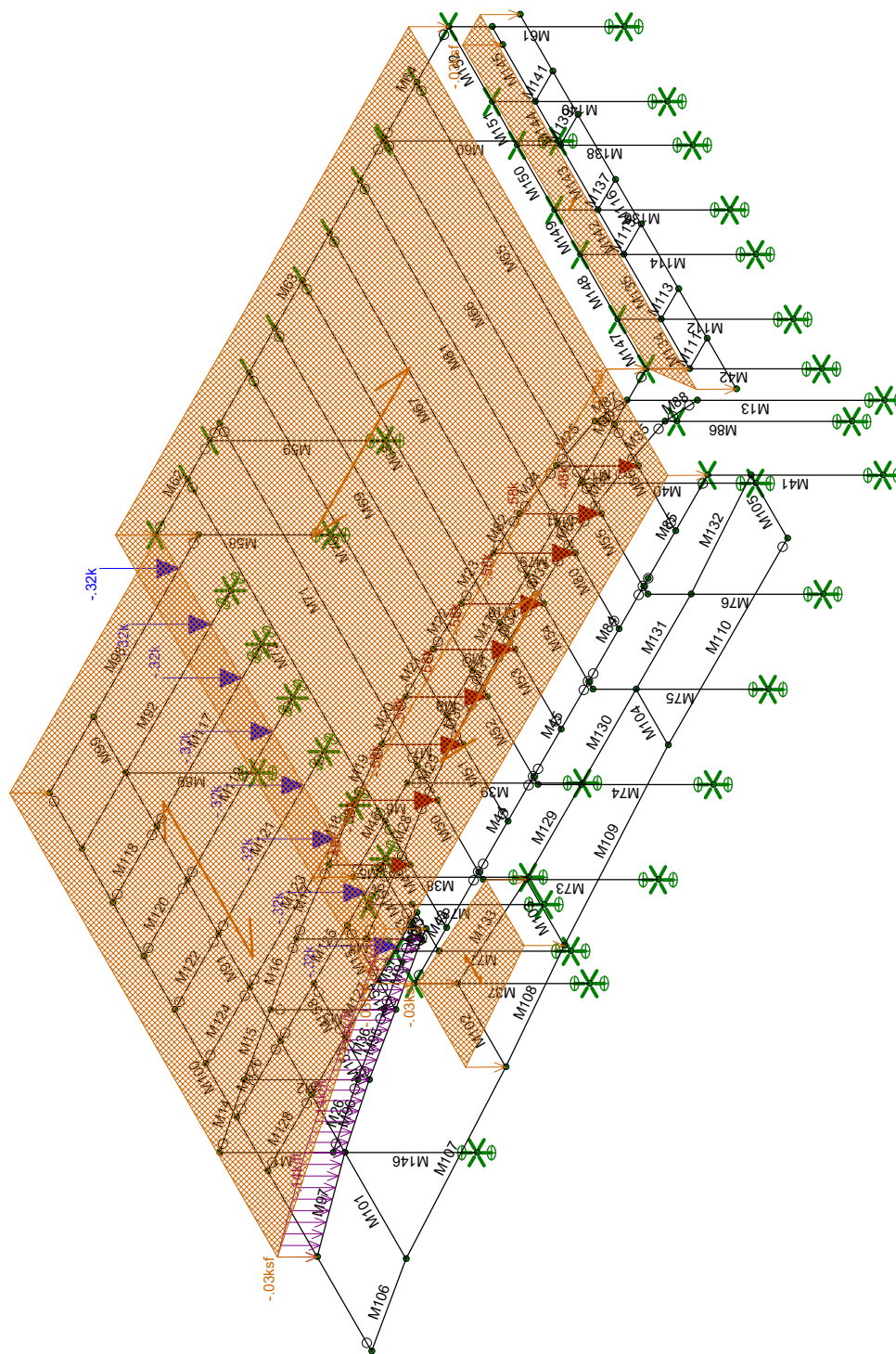
NRP

SK - 3

BOA Canopy Sail Wall

Dec 7, 2020 at 7:45 PM

BOA Canopy Sail (LRFD).r3d



KPFF

NRP

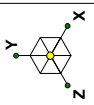
BOA Canopy Sail Wall

SK - 4

Dec 7, 2020 at 7:45 PM

BOA Canopy Sail (LRFD).r3d

Wind Loading (@ canopy, roof, & roof wall)



Loads: BLC 6, Wind Envelope Only Solution

KPFF

NRP

BOA Canopy Sail Wall

SK - 5

Dec 7, 2020 at 7:47 PM

BOA Canopy Sail (LRFD).r3d

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (/1E...	Density[k/ft...	Yield[ksi]	Ry	Fu[ksi]	Rt
1	A992	29000	11154	.3	.65	.49	50	1.1	65	1.1
2	A36 Gr.36	29000	11154	.3	.65	.49	36	1.5	58	1.2
3	A572 Gr.50	29000	11154	.3	.65	.49	50	1.1	65	1.1
4	A500 Gr.B RND	29000	11154	.3	.65	.527	42	1.4	58	1.3
5	A500 Gr.B Rect	29000	11154	.3	.65	.527	46	1.4	58	1.3
6	A53 Gr.B	29000	11154	.3	.65	.49	35	1.6	60	1.2
7	A1085	29000	11154	.3	.65	.49	50	1.4	65	1.3

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	HR1A	W10X33	Beam	None	A992	Typical	9.71	36.6	171	.583

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N1T	0	22	0	0	
3	N1B	0	12.3	0	0	
4	N2	5.7	11.3	-1.5	0	
5	N2T	5.7	22	-1.5	0	
6	N2B	5.7	12.3	-1.5	0	
7	N3	11.4	11.3	-2.7	0	
8	N3T	11.4	22	-2.7	0	
9	N3B	11.4	12.3	-2.7	0	
10	N4	17.1	11.3	-4	0	
11	N4T	17.1	22	-4	0	
12	N4B	17.1	12.3	-4	0	
13	N5	23.5	15	-4.9	0	
14	N5T	23.5	22	-4.9	0	
15	N5B	23.5	16	-4.9	0	
16	N6	29	15	-5.8	0	
17	N6T	29	22	-5.8	0	
18	N6B	29	16	-5.8	0	
19	N7	34	15	-6.3	0	
20	N7T	34	22	-6.3	0	
21	N7B	34	16	-6.3	0	
22	N8	38.4	15	-6.6	0	
23	N8T	38.4	22	-6.6	0	
24	N8B	38.4	16	-6.6	0	
25	N9	43.1	15	-6.6	0	
26	N9T	43.1	22	-6.6	0	
27	N9B	43.1	16	-6.6	0	
28	N10	47.8	15	-6.4	0	
29	N10T	47.8	22	-6.4	0	
30	N10B	47.8	16	-6.4	0	
31	N12	57.2	15	-5.9	0	
32	N12T	57.2	22	-5.9	0	
33	N12B	57.2	16	-5.9	0	
34	N13	62.7	15	-5.1	0	
35	N13T	62.7	22	-5.1	0	
36	N13B	62.7	16	-5.1	0	
37	N15	72	0	-2.3	0	
38	N15T	72	22	-2.3	0	
39	N15B	72	16	-2.3	0	

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
40	N40	18	0	1.3	0	
41	N43	68.2	0	1.3	0	
42	N44	18	0	-9.2	0	
43	N45	27.3	0	-9.2	0	
44	N46	56.9	0	-9.2	0	
45	N47	68.2	0	-9.2	0	
46	N48	18	15	1.3	0	
47	N49	29	15	1.3	0	
48	N50	38.4	15	1.3	0	
49	N51	68.2	15	1.3	0	
50	N52	18	15	-9.2	0	
51	N53	27.3	15	-9.2	0	
52	N54	56.9	15	-9.2	0	
53	N55	68.2	15	-9.2	0	
54	N56	23.5	15	1.3	0	
55	N57	23.5	15	-9.2	0	
56	N59	29	15	-9.2	0	
57	N60	34	15	1.3	0	
58	N61	34	15	-9.2	0	
59	N62	47.8	15	1.3	0	
60	N63	38.4	15	-9.2	0	
61	N64	43.1	15	1.3	0	
62	N65	43.1	15	-9.2	0	
63	N66	57.2	15	1.3	0	
64	N67	47.8	15	-9.2	0	
65	N68	58	15	1.3	0	
66	N69	57.2	15	-9.2	0	
67	N70	62.7	15	1.3	0	
68	N71	62.7	15	-9.2	0	
69	N4M	17.1	16	-4	0	
70	N74	18	0	-43	0	
71	N75	27.3	0	-43	0	
72	N76	56.9	0	-43	0	
73	N77	68.2	0	-43	0	
74	N78	18	15	-43	0	
75	N79	27.3	15	-43	0	
76	N80	56.9	15	-43	0	
77	N81	68.2	15	-43	0	
78	N82	23.5	15	-43	0	
79	N83	29	15	-43	0	
80	N84	34	15	-43	0	
81	N85	38.4	15	-43	0	
82	N86	43.1	15	-43	0	
83	N87	47.8	15	-43	0	
84	N88	57.2	15	-43	0	
85	N89	62.7	15	-43	0	
86	N91	29	0	2.05	0	
87	N92	38.4	0	2.05	0	
88	N95	29	15	2.05	0	
89	N96	38.4	15	2.05	0	
90	N96A	47.8	0	2.05	0	
91	N97A	47.8	15	2.05	0	
92	N98	57.2	0	2.05	0	
93	N99	57.2	15	2.05	0	
94	N100	18	0	-1.9	0	
95	N101	18	15	-1.9	0	
96	N102	18	0	-6.5	0	

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
97	N103	18	15	-6.5	0	
98	N11	53	15	-6.2	0	
99	N11T	53	22	-6.2	0	
100	N11B	53	16	-6.2	0	
101	N105	53	15	1.3	0	
102	N106	53	15	-9.2	0	
103	N107	53	15	-43	0	
104	N14	68.2	0	-4	0	
105	N14T	68.2	22	-4	0	
106	N14B	68.2	16	-4	0	
107	N111	0	0	-37.5	0	
108	N113	0	11.3	-37.5	0	
109	N113A	18	11.3	-1.9	0	
110	N114	18	11.3	-6.5	0	
111	N115	0	11.3	0	0	
112	N116	-7.5	11.3	-37.5	0	
113	N117	18	11.3	-4.1	0	
114	N118	18	11.3	-37.5	0	
115	N119	-7.5	11.3	2.8	0	
116	N120	-7.5	11.3	12.1	0	
117	N121	0	11.3	-43	0	
118	N122	-7.5	11.3	-43	0	
119	N123	18	11.3	-43	0	
120	N124	0	11.3	10.5	0	
121	N125	18	11.3	1.3	0	
122	N126	29	11.3	2.05	0	
123	N127	38.4	11.3	2.05	0	
124	N128	68.2	11.3	1.3	0	
125	N129	47.8	11.3	2.05	0	
126	N130	57.2	11.3	2.05	0	
127	N131	18	11.3	9.55	0	
128	N132	29	11.3	8.55	0	
129	N134	47.8	11.3	7.55	0	
130	N136	68.2	11.3	7.55	0	
131	N137	68.2	11.3	-9.2	0	
132	N138	71.2	11.3	-9.2	0	
133	N139	68.2	0	-14.1	0	
134	N140	68.2	15	-14.1	0	
135	N141	68.2	11.3	-14.1	0	
136	N142	71.2	11.3	-14.1	0	
137	N143	68.2	0	-20.5	0	
138	N144	68.2	15	-20.5	0	
139	N145	68.2	11.3	-20.5	0	
140	N146	71.2	11.3	-20.5	0	
141	N147	68.2	11.3	-41.2	0	
142	N148	71.2	11.3	-41.2	0	
143	N149	71.2	11.3	-4.2	0	
144	N150	0	11.3	-32.2	0	
145	N151	-7.5	11.3	-32.2	0	
146	N152	18	11.3	-32.2	0	
147	N153	0	11.3	-26.9	0	
148	N154	-7.5	11.3	-26.9	0	
149	N155	18	11.3	-26.9	0	
150	N156	0	11.3	-21.6	0	
151	N157	-7.5	11.3	-21.6	0	
152	N158	18	11.3	-21.6	0	
153	N159	0	11.3	-16.3	0	

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
154	N160	-7.5	11.3	-16.3	0	
155	N161	18	11.3	-16.3	0	
156	N162	0	11.3	-11	0	
157	N163	-7.5	11.3	-11	0	
158	N164	18	11.3	-11	0	
159	N165	0	11.3	-5.7	0	
160	N166	-7.5	11.3	-5.7	0	
161	N167	18	11.3	-5.7	0	
162	N168	68.2	15	-4	0	
163	N164A	68.2	0	-24.9	0	
164	N165A	68.2	15	-24.9	0	
165	N166A	68.2	11.3	-24.9	0	
166	N167A	71.2	11.3	-24.9	0	
167	N168A	68.2	0	-31.3	0	
168	N169	68.2	15	-31.3	0	
169	N170	68.2	11.3	-31.3	0	
170	N171	71.2	11.3	-31.3	0	
171	N172	68.2	0	-35.6	0	
172	N173	68.2	15	-35.6	0	
173	N174	68.2	11.3	-35.6	0	
174	N175	71.2	11.3	-35.6	0	
175	N176	68.2	11.3	-43	0	
176	N176A	5.7	11.3	-5.7	0	
177	N177	11.4	11.3	-5.7	0	
178	N178	17.1	11.3	-5.7	0	
179	N179	5.7	11.3	-11	0	
180	N180	11.4	11.3	-11	0	
181	N181	17.1	11.3	-11	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	N1	Reaction	Reaction	Reaction		Reaction	
2	N15	Reaction	Reaction	Reaction		Reaction	
3	N40	Reaction	Reaction	Reaction		Reaction	
4	N44	Reaction	Reaction	Reaction		Reaction	
5	N45	Reaction	Reaction	Reaction		Reaction	
6	N46	Reaction	Reaction	Reaction		Reaction	
7	N43	Reaction	Reaction	Reaction		Reaction	
8	N47	Reaction	Reaction	Reaction		Reaction	
9	N2						
10	N3						
11	N52						
12	N55	Reaction		Reaction			
13	N54						
14	N53						
15	N48	Reaction		Reaction			
16	N51	Reaction		Reaction			
17	N1B						
18	N57			Reaction			
19	N61						
20	N63						
21	N65						
22	N67						
23	N69						
24	N71						

Joint Boundary Conditions (Continued)

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
25	N74	Reaction	Reaction	Reaction		Reaction	
26	N75	Reaction	Reaction	Reaction		Reaction	
27	N76	Reaction	Reaction	Reaction		Reaction	
28	N77	Reaction	Reaction	Reaction		Reaction	
29	N78	Reaction		Reaction			
30	N79			Reaction			
31	N80			Reaction			
32	N81	Reaction		Reaction			
33	N82			Reaction			
34	N84			Reaction			
35	N85			Reaction			
36	N86			Reaction			
37	N87			Reaction			
38	N88			Reaction			
39	N89			Reaction			
40	N91	Reaction	Reaction	Reaction		Reaction	
41	N92	Reaction	Reaction	Reaction		Reaction	
42	N96A	Reaction	Reaction	Reaction		Reaction	
43	N98	Reaction	Reaction	Reaction		Reaction	
44	N100	Reaction	Reaction	Reaction		Reaction	
45	N101	Reaction		Reaction			
46	N102	Reaction	Reaction	Reaction		Reaction	
47	N103	Reaction		Reaction			
48	N106						
49	N107			Reaction			
50	N14	Reaction	Reaction	Reaction		Reaction	
51	N111	Reaction	Reaction	Reaction		Reaction	
52	N113						
53	N113A						
54	N114						
55	N115						
56	N116						
57	N117						
58	N118	Reaction	Reaction	Reaction	Reaction		
59	N119						
60	N120						
61	N121						
62	N122						
63	N123						
64	N124						
65	N125						
66	N128						
67	N131						
68	N136						
69	N137						
70	N138						
71	N139	Reaction	Reaction	Reaction		Reaction	
72	N140	Reaction		Reaction			
73	N141						
74	N142						
75	N143	Reaction	Reaction	Reaction		Reaction	
76	N144	Reaction		Reaction			
77	N145						
78	N146						
79	N147						
80	N148						
81	N149						

Joint Boundary Conditions (Continued)

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
82	N150						
83	N151						
84	N152	Reaction	Reaction	Reaction	Reaction		
85	N153						
86	N154						
87	N155	Reaction	Reaction	Reaction	Reaction		
88	N156						
89	N157						
90	N158	Reaction	Reaction	Reaction	Reaction		
91	N159						
92	N160						
93	N161	Reaction	Reaction	Reaction	Reaction		
94	N162						
95	N163						
96	N164	Reaction	Reaction	Reaction	Reaction		
97	N165						
98	N166						
99	N167						
100	N168	Reaction		Reaction			
101	N164A	Reaction	Reaction	Reaction		Reaction	
102	N165A	Reaction		Reaction			
103	N166A						
104	N167A						
105	N168A	Reaction	Reaction	Reaction		Reaction	
106	N169	Reaction		Reaction			
107	N170						
108	N171						
109	N172	Reaction	Reaction	Reaction		Reaction	
110	N173	Reaction		Reaction			
111	N174						
112	N175						
113	N176						
114	N176A						
115	N177						
116	N179						
117	N180						

Hot Rolled Steel Design Parameters

	Label	Shape	Length[...]	Lbyy[ft]	Lbzz[ft]	Lcomp top..	Lcomp bot..	L-torq...	Kyy	Kzz	Cb	Function
1	M1	HSS6X6X6	10.7									Lateral
2	M2	HSS6X6X6	10.7									Lateral
3	M3	HSS6X6X6	10.7									Lateral
4	M4	HSS6X6X6	10.7									Lateral
5	M5	HSS6X6X6	7									Lateral
6	M6	HSS6X6X6	7									Lateral
7	M7	HSS6X6X6	7									Lateral
8	M8	HSS6X6X6	7									Lateral
9	M9	HSS6X6X6	7									Lateral
10	M10	HSS6X6X6	7									Lateral
11	M11	HSS6X6X6	7									Lateral
12	M12	HSS6X6X6	7									Lateral
13	M13	HSS6X6X6	22									Lateral
14	M14	HSS6X6X6	5.894									Lateral
15	M15	HSS6X6X6	5.825									Lateral
16	M16	HSS6X6X6	5.846									Lateral

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Length[...]	Lbyy[ft]	Lbzz[ft]	Lcomp top..	Lcomp bot..	L-torq...	Kyy	Kzz	Cb	Function
17	M17	HSS6X6X6	6.463									Lateral
18	M18	HSS6X6X6	5.573									Lateral
19	M19	HSS6X6X6	5.025									Lateral
20	M20	HSS6X6X6	4.41									Lateral
21	M21	HSS6X6X6	4.7									Lateral
22	M22	HSS6X6X6	4.704									Lateral
23	M23	HSS6X6X6	5.204									Lateral
24	M24	HSS6X6X6	5.558									Lateral
25	M25	HSS6X6X6	5.609									Lateral
26	M26	HSS6X6X6	5.894									Lateral
27	M27	HSS6X6X6	6.463									Lateral
28	M28	HSS6X6X6	5.573									Lateral
29	M29	HSS6X6X6	5.025									Lateral
30	M30	HSS6X6X6	4.41									Lateral
31	M31	HSS6X6X6	4.7									Lateral
32	M32	HSS6X6X6	4.704									Lateral
33	M33	HSS6X6X6	5.204									Lateral
34	M34	HSS6X6X6	5.558									Lateral
35	M35	HSS6X6X6	5.609									Lateral
36	M36	HSS6X6X6	5.825									Lateral
37	M37	HSS6X4X6	15									Gravity
38	M38	HSS4X4X6	15									Gravity
39	M39	HSS4X4X6	15									Gravity
40	M40	HSS4X4X6	15									Gravity
41	M41	HSS6X4X6	15									Lateral
42	M42	HSS4X4X4	15									Gravity
43	M43	W18X35	11	1		Lbyy						Gravity
44	M44	W18X35	9.4	1		Lbyy						Gravity
45	M45	W18X35	9.4	1		Lbyy						Gravity
46	M46	W18X55	9.3	1		Lbyy						Gravity
47	M47	W18X55	29.6	1		Lbyy						Gravity
48	M48	W18X55	11.3	1		Lbyy						Gravity
49	M49	W18X35	10.5	1		Lbyy						Gravity
50	M50	W18X35	11.25	1		Lbyy						Gravity
51	M51	W18X35	10.5	1		Lbyy						Gravity
52	M52	W18X35	11.25	1		Lbyy						Gravity
53	M53	W18X35	10.5	1		Lbyy						Gravity
54	M54	W18X35	11.25	1		Lbyy						Gravity
55	M55	W18X35	11.25	1		Lbyy						Gravity
56	M56	W18X35	10.5	1		Lbyy						Gravity
57	M57	HSS6X6X4	5.846			Lbyy						Gravity
58	M58	HSS6X6X6	15									Gravity
59	M59	HSS4X4X6	15									Gravity
60	M60	HSS4X4X6	15									Gravity
61	M61	HSS4X4X6	15									Gravity
62	M62	W24X55	9.3	1		Lbyy						Gravity
63	M63	W24X55	29.6	1		Lbyy						Gravity
64	M64	W24X55	11.3	1		Lbyy						Gravity
65	M65	W18X40	33.8	1		Lbyy						Gravity
66	M66	W18X40	33.8	1		Lbyy						Gravity
67	M67	W18X40	33.8	1		Lbyy						Gravity
68	M68	W18X40	33.8	1		Lbyy						Gravity
69	M69	W18X40	33.8	1		Lbyy						Gravity
70	M70	W18X40	33.8	1		Lbyy						Gravity
71	M71	W18X40	33.8	1		Lbyy						Gravity
72	M72	W18X40	33.8	1		Lbyy						Lateral
73	M73	HSS6X4X6	15									Gravity

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Length[...]	Lbyv[ft]	Lbzz[ft]	Lcomp top..	Lcomp bot..	L-torg...	Kvy	Kzz	Cb	Function
74	M74	HSS6X4X6	15									Gravity
75	M75	HSS6X4X6	15									Gravity
76	M76	HSS6X4X6	15									Gravity
77	M77	HSS4X4X4	15									Gravity
78	M78	HSS4X4X4	15									Gravity
79	M79	HSS6X6X6	7									Gravity
80	M80	W18X35	10.5	1		Lbyy						Gravity
81	M81	W18X40	33.8	1		Lbyy						Gravity
82	M82	HSS6X6X6	4.211									Lateral
83	M83	HSS6X6X6	4.211									Lateral
84	M84	W18X35	9.4	1		Lbyy						Gravity
85	M85	W18X35	11	1		Lbyy						Gravity
86	M86	HSS6X6X6	22									Lateral
87	M87	HSS6X6X6	4.163									Lateral
88	M88	HSS6X6X6	4.163									Lateral
89	M89	HSS8X8X6	11.3	1		Lbyy						Gravity
90	M90	HSS8X4X4	4.6	1		Lbyy						Gravity
91	M91	HSS8X8X...	37.5	1		Lbyy						Gravity
92	M92	HSS8X8X...	25.5	1		Lbyy						Gravity
93	M93	HSS8X8X...	.906	1		Lbyy						Gravity
94	M94	HSS8X8X...	5.846	1		Lbyy						Gravity
95	M95	HSS8X8X...	5.825	1		Lbyy						Gravity
96	M96	HSS8X8X...	5.894	1		Lbyy						Gravity
97	M97	HSS8X8X...	8.006	1		Lbyy						Gravity
98	M98	HSS12X4...	25.5	1		Lbyy						Gravity
99	M99	HSS8X8X...	5.5	1		Lbyy						Gravity
100	M100	HSS12X4...	55.1	Segment		Lbyy						Gravity
101	M101	HSS8X8X...	10.5									Gravity
102	M102	HSS8X4X...	8.25	1		Lbyy						Gravity
103	M103	HSS8X4X...	6.5	1		Lbyy						Gravity
104	M104	HSS8X4X6	5.5									Gravity
105	M105	HSS12X4...	6.25									Gravity
106	M106	HSS12X4...	7.669									Gravity
107	M107	HSS12X4...	18.025									Gravity
108	M108	HSS12X4...	11.045									Gravity
109	M109	HSS12X4...	18.827									Lateral
110	M110	HSS12X4...	20.4									Lateral
111	M111	HSS6X4X4	3									Gravity
112	M112	HSS4X4X4	15									Gravity
113	M113	HSS6X4X4	3									Gravity
114	M114	HSS4X4X4	15									Gravity
115	M115	HSS6X4X4	3									Gravity
116	M116	HSS12X4...	37	1		Lbyy						Gravity
117	M117	HSS6X4X6	18									Gravity
118	M118	HSS6X4X4	7.5									Gravity
119	M119	HSS6X4X6	18									Gravity
120	M120	HSS6X4X4	7.5									Gravity
121	M121	HSS6X4X6	18									Gravity
122	M122	HSS6X4X4	7.5									Gravity
123	M123	HSS6X4X6	18									Gravity
124	M124	HSS6X4X4	7.5									Gravity
125	M125	HSS6X4X6	18									Gravity
126	M126	HSS6X4X4	7.5									Gravity
127	M127	HSS6X4X6	18									Gravity
128	M128	HSS6X4X4	7.5									Gravity
129	M129	HSS4X4X4	9.4									Lateral
130	M130	HSS4X4X4	9.4									Lateral

Hot Rolled Steel Design Parameters (Continued)

	Label	Shape	Length[...]	Lbyy[ft]	Lbzz[ft]	Lcomp top..	Lcomp bot..	L-torq...	Kyy	Kzz	Cb	Function
131	M131	HSS4X4X4	9.4									Lateral
132	M132	HSS4X4X4	11.026									Lateral
133	M133	HSS4X4X4	11.026									Lateral
134	M134	HSS4X4X4	4.9									Lateral
135	M135	HSS4X4X4	6.4									Lateral
136	M136	HSS4X4X4	15									Gravity
137	M137	HSS6X4X4	3									Gravity
138	M138	HSS4X4X4	15									Gravity
139	M139	HSS6X4X4	3									Gravity
140	M140	HSS4X4X4	15									Gravity
141	M141	HSS6X4X4	3									Gravity
142	M142	HSS4X4X4	4.4									Gravity
143	M143	HSS4X4X4	6.4									Gravity
144	M144	HSS4X4X4	4.3									Gravity
145	M145	HSS4X4X4	7.4									Gravity
146	M146	HSS8X8X6	11.3									Gravity
147	M147	HSS4X4X4	4.9									Gravity
148	M148	HSS4X4X4	6.4									Gravity
149	M149	HSS4X4X4	4.4									Gravity
150	M150	HSS4X4X4	6.4									Gravity
151	M151	HSS4X4X4	4.3									Gravity
152	M152	HSS4X4X4	7.4									Gravity
153	M153	HSS6X4X4	1.7									Gravity
154	M154	HSS6X4X4	3									Gravity
155	M155	HSS6X4X4	4.2									Gravity
156	M156	HSS6X4X4	5.3									Gravity
157	M157	HSS6X4X4	5.3									Gravity
158	M158	HSS6X4X4	5.3									Gravity

Member Area Loads (BLC 1 : Dead)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N48	N51	N55	N52	Y	A-B	-.018
2	N55	N52	N78	N81	Y	A-B	-.018
3	N120	N122	N123	N131	Y	A-B	-.018
4	N131	N125	N128	N136	Y	A-B	-.018
5	N147	N148	N149	N137	Y	A-B	-.018

Member Area Loads (BLC 4 : Snow)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N48	N51	N55	N52	Y	A-B	-.03
2	N55	N52	N78	N81	Y	A-B	-.03
3	N147	N148	N149	N137	Y	A-B	-.03
4	N119	N122	N123	N117	Y	A-B	-.03
5	N125	N131	N132	N126	Y	A-B	-.03

Member Area Loads (BLC 6 : Wind)

	Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
1	N147	N148	N149	N137	Y	A-B	.02
2	N131	N125	N126	N132	Y	A-B	.02
3	N119	N122	N123	N117	Y	A-B	.02
4	N48	N51	N81	N78	Y	A-B	.032

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed Area(Me...	Surface(P...
1	Dead	DL		-1				28	5
2	Floor Live	LL							
3	Roof Live	RLL							
4	Snow	SL				9	8	4	5
5	Rain	RL							
6	Wind	WL						30	4
7	Earthquake	EL							
8	BLC 1 Transient Area L...	None						357	
9	BLC 4 Transient Area L...	None						304	
10	BLC 6 Transient Area L...	None						304	

Load Combinations

	Description	Solve	PDelta	S...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...	BLC Fac...
1	Def Dead	Yes	Y		DL 1									
2	Def S	Yes	Y		SL 1									
3	Def D+S	Yes	Y		DL 1	SL 1								
4	Def 0.6D...	Yes	Y		WL .42	DL .6								
5	Uplift 0.6...	Yes	Y		WL .6	DL .6								
6	1.4D	Yes	Y		DL 1.4									
7	1.2D+1.6...	Yes	Y		DL 1.2	LL 1.6	LLS 1.6	RLL .5						
8	1.2D+1.6...	Yes	Y		DL 1.2	LL 1.6	LLS 1.6	SL .5	SLN .5					
9	1.2D+1.6...	Yes	Y		DL 1.2	RLL 1.6	LL .5	LLS 1						
10	1.2D+1.6...	Yes	Y		DL 1.2	SL 1.6	SLN 1.6	LL .5	LLS 1					
11	1.2D+1.6...	Yes	Y		DL 1.2	RLL 1.6	WL .5							
12	1.2S+1.6...	Yes	Y		DL 1.2	SL 1.6	SLN 1.6	WL .5						
13	1.2D+0.5W	Yes	Y		DL 1.2	WL .5								
14	1.2D+W+	Yes	Y		DL 1.2	WL 1	LL .5	LLS 1	RLL .5					
15	1.2D+W+	Yes	Y		DL 1.2	WL 1	LL .5	LLS 1	SL .5	SLN .5				
16	0.9D+W	Yes	Y		DL .9	WL 1								

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	.216	16	46.096	10	1.202	16	0	16	.889	16	0	16
2		min	-1.07	10	8.078	5	-4.397	10	0	1	-.257	6	0	1
3	N15	max	-.013	1	1.863	6	1.086	16	0	16	0	16	0	16
4		min	-.232	15	-.047	2	-.001	2	0	1	0	1	0	1
5	N40	max	.101	12	5.81	10	.891	6	0	16	.128	16	0	16
6		min	.016	4	.502	5	-.025	2	0	1	-.07	10	0	1
7	N44	max	0	10	-.526	5	0	10	0	16	0	16	0	16
8		min	0	16	-7.316	10	0	15	0	1	0	1	0	1
9	N45	max	0	16	60.215	10	.002	10	0	16	.008	10	0	16
10		min	0	10	3.98	5	0	15	0	1	-.004	16	0	1
11	N46	max	0	5	58.133	10	0	2	0	16	0	10	0	16
12		min	0	10	3.974	5	0	12	0	1	0	5	0	1
13	N43	max	.019	16	4.395	10	.585	6	0	16	.197	16	0	16
14		min	-.015	6	.706	2	.001	2	0	1	-.134	6	0	1
15	N47	max	.184	10	-.126	5	0	16	0	16	.007	10	0	16
16		min	.021	5	-2.319	10	-.002	10	0	1	.001	5	0	1
17	N55	max	.072	14	0	16	.129	10	0	16	0	16	0	16
18		min	.005	2	0	1	-.008	16	0	1	0	1	0	1
19	N48	max	.441	16	0	16	.444	16	0	16	0	16	0	16
20		min	-.412	10	0	1	-1.171	6	0	1	0	1	0	1
21	N51	max	.744	15	0	16	-.003	2	0	16	0	16	0	16

Envelope Joint Reactions (Continued)

Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
22		min	.042	1	0	1	-695	6	0	1	0	0	1
23	N57	max	0	16	0	16	1.754	16	0	16	0	16	16
24		min	0	1	0	1	-.145	10	0	1	0	1	1
25	N74	max	.036	6	6.407	10	.17	10	0	16	0	16	16
26		min	-.193	16	.157	16	-.004	16	0	1	0	1	1
27	N75	max	0	16	28.994	10	0	10	0	16	0	16	16
28		min	0	1	-1.971	16	0	5	0	1	0	1	1
29	N76	max	0	16	29.196	10	0	10	0	16	0	16	16
30		min	0	1	-1.652	16	0	16	0	1	0	1	1
31	N77	max	.058	10	5.592	10	.002	12	0	16	.088	10	16
32		min	.007	5	-.03	16	0	2	0	1	.01	5	1
33	N78	max	.111	6	0	16	.159	16	0	16	0	16	16
34		min	-.589	16	0	1	-.195	10	0	1	0	1	1
35	N79	max	0	16	0	16	.323	16	0	16	0	16	16
36		min	0	1	0	1	-.77	10	0	1	0	1	1
37	N80	max	0	16	0	16	0	6	0	16	0	16	16
38		min	0	1	0	1	0	16	0	1	0	1	1
39	N81	max	.01	10	0	16	.026	16	0	16	0	16	16
40		min	.001	5	0	1	-.293	10	0	1	0	1	1
41	N82	max	0	16	0	16	0	16	0	16	0	16	16
42		min	0	1	0	1	0	1	0	1	0	1	1
43	N84	max	0	16	0	16	1.751	16	0	16	0	16	16
44		min	0	1	0	1	-1.004	10	0	1	0	1	1
45	N85	max	0	16	0	16	1.588	15	0	16	0	16	16
46		min	0	1	0	1	.058	2	0	1	0	1	1
47	N86	max	0	16	0	16	1.39	16	0	16	0	16	16
48		min	0	1	0	1	-.184	10	0	1	0	1	1
49	N87	max	0	16	0	16	.572	16	0	16	0	16	16
50		min	0	1	0	1	-1.228	6	0	1	0	1	1
51	N88	max	0	16	0	16	1.687	14	0	16	0	16	16
52		min	0	1	0	1	-.012	2	0	1	0	1	1
53	N89	max	0	16	0	16	1.175	16	0	16	0	16	16
54		min	0	1	0	1	-.034	10	0	1	0	1	1
55	N91	max	.035	16	12.272	10	1.137	10	0	16	.309	15	16
56		min	-.033	10	1.285	5	.128	2	0	1	-.011	10	1
57	N92	max	.028	15	6.856	10	.187	10	0	16	.079	10	16
58		min	-.008	6	.49	16	.016	2	0	1	-.131	16	1
59	N96A	max	.026	16	9.877	10	.961	6	0	16	.273	16	16
60		min	-.009	6	1.889	5	.06	2	0	1	-.098	10	1
61	N98	max	.035	15	7.551	10	.137	6	0	16	.159	10	16
62		min	0	6	.454	16	.006	2	0	1	-.043	16	1
63	N100	max	-.008	1	3.326	6	-.005	5	0	16	.011	6	16
64		min	-.142	15	-2.148	16	-.035	10	0	1	-.34	15	1
65	N101	max	.089	10	0	16	.487	15	0	16	0	16	16
66		min	-.366	16	0	1	.047	2	0	1	0	1	1
67	N102	max	.009	16	12.323	12	.045	15	0	16	.013	6	16
68		min	-.027	10	3.291	2	.006	2	0	1	-.392	15	1
69	N103	max	.764	15	0	16	-.003	16	0	16	0	16	16
70		min	-.008	6	0	1	-.328	10	0	1	0	1	1
71	N107	max	0	16	0	16	1.371	16	0	16	0	16	16
72		min	0	1	0	1	-1.109	10	0	1	0	1	1
73	N14	max	.142	16	3.222	6	.005	10	0	16	-.004	2	16
74		min	-.069	10	.084	2	-.538	14	0	1	-2.986	15	1
75	N111	max	-.213	5	37.103	10	5.347	12	0	16	.831	16	16
76		min	-.885	12	3.47	5	1.119	4	0	1	-.149	6	1
77	N118	max	1.142	10	2.89	10	.811	16	1.913	16	0	16	16
78		min	-1.732	16	.046	16	-.112	6	-12.01	10	0	1	1

Envelope Joint Reactions (Continued)

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
79	N139	max	.164	10	2.815	10	0	5	0	16	.007	10	0	16
80		min	.019	5	.118	16	-.002	10	0	1	.001	5	0	1
81	N140	max	-.017	5	0	16	.122	10	0	16	0	16	0	16
82		min	-.142	10	0	1	-.01	16	0	1	0	1	0	1
83	N143	max	.157	10	2.759	10	0	12	0	16	.007	10	0	16
84		min	.018	5	.139	16	0	4	0	1	0	5	0	1
85	N144	max	-.015	5	0	16	.013	16	0	16	0	16	0	16
86		min	-.138	10	0	1	-.143	10	0	1	0	1	0	1
87	N152	max	3.089	16	3.98	10	.012	16	-.069	16	0	16	0	16
88		min	-.547	6	.001	16	0	2	-2.84	10	0	1	0	1
89	N155	max	.018	6	3.982	10	0	2	-.17	5	0	16	0	16
90		min	-.169	15	.002	16	0	15	-2.343	10	0	1	0	1
91	N158	max	.005	6	3.982	10	0	2	-.143	4	0	16	0	16
92		min	-.03	15	.002	16	0	16	-.869	12	0	1	0	1
93	N161	max	.39	10	4.034	10	0	2	.93	10	0	16	0	16
94		min	-1.111	16	-.002	16	-.004	16	-.123	16	0	1	0	1
95	N164	max	1.063	16	4.515	10	4.477	16	2.35	10	0	16	0	16
96		min	-.671	10	.065	16	-.178	10	.042	16	0	1	0	1
97	N168	max	1.186	10	0	16	3.798	15	0	16	0	16	0	16
98		min	-1.73	16	0	1	.062	2	0	1	0	1	0	1
99	N164A	max	.154	10	2.703	10	0	5	0	16	.007	10	0	16
100		min	.018	5	.14	16	-.001	10	0	1	0	5	0	1
101	N165A	max	-.017	5	0	16	.101	10	0	16	0	16	0	16
102		min	-.147	10	0	1	-.015	16	0	1	0	1	0	1
103	N168A	max	.155	10	2.684	10	0	12	0	16	.006	10	0	16
104		min	.018	5	.149	16	0	1	0	1	0	5	0	1
105	N169	max	-.022	5	0	16	.014	16	0	16	0	16	0	16
106		min	-.188	10	0	1	-.112	10	0	1	0	1	0	1
107	N172	max	.159	10	3.367	10	0	5	0	16	.019	10	0	16
108		min	.018	5	.194	5	-.004	10	0	1	.002	5	0	1
109	N173	max	-.026	5	0	16	.199	10	0	16	0	16	0	16
110		min	-.23	10	0	1	-.017	16	0	1	0	1	0	1
111	Totals:	max	0	16	363.008	10	26.827	16						
112		min	0	1	30.981	5	0	1						

Envelope AISC 14th(360-10): LRFD Steel Code Checks

	Member	Shape	Code C...	Loc[ft]	LC	Shear ...	Loc[ft]	Dir	LC	phi*Pnc [K]	phi*Pnt [K]	phi*Mn y...	phi*Mn z....	Cb	Eqn
1	M1	HSS6X6X6	.398	0	16	.033	0	z	14	253.655	313.812	54.51	54.51	2....	H1-1b
2	M2	HSS6X6X6	.387	0	10	.218	0	y	12	253.655	313.812	54.51	54.51	4....	H1-1b
3	M3	HSS6X6X6	.193	0	15	.055	0	y	12	253.655	313.812	54.51	54.51	2....	H1-1b
4	M4	HSS6X6X6	.540	0	12	.168	0	y	12	253.655	313.812	54.51	54.51	4....	H1-1b
5	M5	HSS6X6X6	.266	0	15	.020	0	z	15	286.492	313.812	54.51	54.51	1....	H1-1b
6	M6	HSS6X6X6	.155	0	15	.019	0	z	16	286.492	313.812	54.51	54.51	2....	H1-1b
7	M7	HSS6X6X6	.113	0	16	.016	0	z	16	286.492	313.812	54.51	54.51	2....	H1-1b
8	M8	HSS6X6X6	.102	0	16	.016	0	z	16	286.492	313.812	54.51	54.51	1....	H1-1b
9	M9	HSS6X6X6	.100	0	16	.017	0	z	16	286.492	313.812	54.51	54.51	1....	H1-1b
10	M10	HSS6X6X6	.102	0	16	.017	0	z	15	286.492	313.812	54.51	54.51	1....	H1-1b
11	M11	HSS6X6X6	.118	0	15	.018	0	z	16	286.492	313.812	54.51	54.51	2....	H1-1b
12	M12	HSS6X6X6	.153	0	15	.014	0	z	16	286.492	313.812	54.51	54.51	1....	H1-1b
13	M13	HSS6X6X6	.199	16.042	15	.013	0	z	14	127.627	313.812	54.51	54.51	1....	H1-1b
14	M14	HSS6X6X6	.046	4.236	15	.043	0	z	15	294.188	313.812	54.51	54.51	1....	H1-1b
15	M15	HSS6X6X6	.104	4.308	15	.043	0	y	12	294.631	313.812	54.51	54.51	1....	H1-1b
16	M16	HSS6X6X6	.122	0	15	.045	5.846	z	15	294.494	313.812	54.51	54.51	1....	H1-1b
17	M17	HSS6X6X6	.021	3.231	15	.093	0	z	15	290.368	313.812	54.51	54.51	1....	H1-1b
18	M18	HSS6X6X6	.016	2.787	15	.045	5.573	z	16	296.207	313.812	54.51	54.51	1....	H1-1b

Envelope AISC 14th(360-10): LRFD Steel Code Checks (Continued)

	Member	Shape	Code C...	Loc[ft]	LC	Shear ...	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-...	phi*Mn z-...	Cb	Eqn
19	M19	HSS6X6X6	.013	2.512	15	.022	0	z	16	299.423	313.812	54.51	54.51	1....	H1-1b
20	M20	HSS6X6X6	.010	2.205	15	.019	0	y	10	302.669	313.812	54.51	54.51	1....	H1-1b
21	M21	HSS6X6X6	.012	2.35	15	.007	0	z	16	301.187	313.812	54.51	54.51	1....	H1-1b
22	M22	HSS6X6X6	.012	2.352	15	.014	4.704	y	10	301.165	313.812	54.51	54.51	1....	H1-1b
23	M23	HSS6X6X6	.014	2.602	15	.025	0	y	10	298.406	313.812	54.51	54.51	1....	H1-1b
24	M24	HSS6X6X6	.016	2.779	15	.005	5.558	z	16	296.301	313.812	54.51	54.51	1....	H1-1b
25	M25	HSS6X6X6	.061	5.609	15	.011	5.609	z	16	295.987	313.812	54.51	54.51	2....	H1-1b
26	M26	HSS6X6X6	.050	2.947	14	.050	0	y	10	294.188	313.812	54.51	54.51	1....	H1-1b
27	M27	HSS6X6X6	.040	3.231	14	.131	0	y	15	290.368	313.812	54.51	54.51	1....	H1-1b
28	M28	HSS6X6X6	.031	2.787	14	.023	5.573	y	14	296.207	313.812	54.51	54.51	1....	H1-1b
29	M29	HSS6X6X6	.026	2.512	14	.043	5.025	y	10	299.423	313.812	54.51	54.51	1....	H1-1b
30	M30	HSS6X6X6	.021	2.205	14	.035	0	y	10	302.669	313.812	54.51	54.51	1....	H1-1b
31	M31	HSS6X6X6	.023	2.35	14	.014	0	y	10	301.187	313.812	54.51	54.51	1....	H1-1b
32	M32	HSS6X6X6	.023	2.352	14	.026	4.704	y	10	301.165	313.812	54.51	54.51	1....	H1-1b
33	M33	HSS6X6X6	.028	2.602	14	.044	0	y	10	298.406	313.812	54.51	54.51	1....	H1-1b
34	M34	HSS6X6X6	.031	2.779	14	.022	0	y	10	296.301	313.812	54.51	54.51	1....	H1-1b
35	M35	HSS6X6X6	.032	2.804	14	.128	0	y	15	295.987	313.812	54.51	54.51	1....	H1-1b
36	M36	HSS6X6X6	.070	2.912	15	.052	0	y	12	294.631	313.812	54.51	54.51	1....	H1-1b
37	M37	HSS6X4X6	.274	11.25	6	.014	11.406	z	10	103.609	255.852	30.843	41.055	1....	H1-1b
38	M38	HSS4X4X6	.019	15	10	.000	0	z	10	71.818	197.892	22.046	22.046	1	H1-1b
39	M39	HSS4X4X6	.838	0	10	.000	0	y	10	71.818	197.892	22.046	22.046	1	H1-1a
40	M40	HSS4X4X6	.809	0	10	.000	0	y	10	71.818	197.892	22.046	22.046	1	H1-1a
41	M41	HSS6X4X6	.184	11.25	6	.011	0	y	15	103.609	255.852	30.843	41.055	1....	H1-1b
42	M42	HSS4X4X4	.136	11.25	10	.007	11.406	y	10	54.386	139.518	16.181	16.181	1....	H1-1b
43	M43	W18X35	.052	5.5	10	.020	0	y	10	389.024	463.5	30.225	249.375	1	H1-1b
44	M44	W18X35	.031	4.994	10	.015	9.4	y	10	390.899	463.5	30.225	249.375	1	H1-1b
45	M45	W18X35	.029	4.7	10	.011	0	y	10	390.899	463.5	30.225	249.375	1	H1-1b
46	M46	W18X55	.327	9.3	10	.114	9.3	y	10	680.77	729	69.375	420	2....	H1-1b
47	M47	W18X55	.740	0	10	.169	0	y	10	600.521	729	69.375	182.833	1....	H1-1b
48	M48	W18X55	.306	0	10	.134	0	y	10	676.433	729	69.375	420	2....	H1-1b
49	M49	W18X35	.074	6.234	12	.037	10.5	y	10	389.641	463.5	30.225	249.375	1	H1-1b
50	M50	W18X35	.060	7.852	10	.045	11.25	y	10	388.704	463.5	30.225	249.375	1	H1-1b
51	M51	W18X35	.044	7.547	10	.045	10.5	y	10	389.641	463.5	30.225	249.375	1	H1-1b
52	M52	W18X35	.050	7.5	10	.040	11.25	y	10	388.704	463.5	30.225	249.375	1	H1-1b
53	M53	W18X35	.045	7.766	10	.026	10.5	y	12	389.641	463.5	30.225	249.375	1	H1-1b
54	M54	W18X35	.053	7.383	10	.040	11.25	y	10	388.704	463.5	30.225	249.375	1	H1-1b
55	M55	W18X35	.053	7.969	12	.040	0	y	10	388.704	463.5	30.225	249.375	1	H1-1b
56	M56	W18X35	.064	6.453	12	.032	10.5	y	10	389.641	463.5	30.225	249.375	1	H1-1b
57	M57	HSS6X6X4	.089	2.923	15	.075	0	y	12	204.168	216.936	38.64	38.64	1....	H1-1b
58	M58	HSS6X6X6	.060	11.25	12	.007	11.406	y	16	206.553	313.812	54.51	54.51	1....	H1-1b
59	M59	HSS4X4X6	.404	0	10	.000	0	z	10	71.818	197.892	22.046	22.046	1	H1-1a
60	M60	HSS4X4X6	.407	0	10	.000	0	z	10	71.818	197.892	22.046	22.046	1	H1-1a
61	M61	HSS4X4X6	.078	0	10	.015	11.406	z	10	71.818	197.892	22.046	22.046	1....	H1-1b*
62	M62	W24X55	.048	5.522	10	.027	9.3	y	10	601.406	729	49.815	502.5	1	H1-1b
63	M63	W24X55	.312	15.725	10	.088	0	y	10	563.195	729	49.815	502.5	1	H1-1b
64	M64	W24X55	.044	5.768	10	.041	0	y	10	599.395	729	49.815	502.5	1	H1-1b
65	M65	W18X40	.208	16.9	10	.043	33.8	y	10	391.87	531	37.5	294	1	H1-1b
66	M66	W18X40	.185	16.9	10	.040	33.8	y	10	391.87	531	37.5	294	1	H1-1b
67	M67	W18X40	.192	16.9	10	.039	0	y	10	391.87	531	37.5	294	1	H1-1b
68	M68	W18X40	.182	16.9	10	.038	33.8	y	10	391.87	531	37.5	294	1	H1-1b
69	M69	W18X40	.178	16.9	10	.037	33.8	y	10	391.87	531	37.5	294	1	H1-1b
70	M70	W18X40	.182	16.9	10	.038	0	y	10	391.87	531	37.5	294	1	H1-1b
71	M71	W18X40	.201	16.9	10	.042	33.8	y	10	391.87	531	37.5	294	1	H1-1b
72	M72	W18X40	.300	16.9	10	.063	0	y	10	391.87	531	37.5	294	1	H1-1b
73	M73	HSS6X4X6	.387	11.25	10	.020	0	y	15	103.609	255.852	30.843	41.055	1....	H1-1b
74	M74	HSS6X4X6	.084	11.25	10	.006	0	y	14	103.609	255.852	30.843	41.055	1....	H1-1b
75	M75	HSS6X4X6	.307	11.25	6	.018	0	y	15	103.609	255.852	30.843	41.055	1....	H1-1b

Envelope AISC 14th(360-10): LRFD Steel Code Checks (Continued)

	Member	Shape	Code C...	Loc(ft)	LC	Shear ...	Loc(ft)	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y...	phi*Mn z...	Cb	Eqn
76	M76	HSS6X4X6	.073	0	10	.007	0	y	10	103.609	255.852	30.843	41.055	1....	H1-1b*
77	M77	HSS4X4X4	.187	11.406	15	.028	0	y	15	54.386	139.518	16.181	16.181	1....	H1-1b
78	M78	HSS4X4X4	.260	11.25	12	.030	0	z	15	54.386	139.518	16.181	16.181	4....	H1-1a
79	M79	HSS6X6X6	.115	0	15	.016	0	z	16	286.492	313.812	54.51	54.51	1....	H1-1b
80	M80	W18X35	.043	7.547	12	.041	10.5	y	10	389.641	463.5	30.225	249.375	1	H1-1b
81	M81	W18X40	.182	16.9	10	.038	0	y	10	391.87	531	37.5	294	1	H1-1b
82	M82	HSS6X6X6	.009	2.105	15	.019	0	y	10	303.638	313.812	54.51	54.51	1....	H1-1b
83	M83	HSS6X6X6	.019	2.105	14	.039	0	y	10	303.638	313.812	54.51	54.51	1....	H1-1b
84	M84	W18X35	.027	5.19	10	.015	9.4	y	10	390.899	463.5	30.225	249.375	1	H1-1b
85	M85	W18X35	.049	5.5	10	.019	0	y	10	389.024	463.5	30.225	249.375	1	H1-1b
86	M86	HSS6X6X6	.337	16.042	14	.103	15.125	z	15	127.627	313.812	54.51	54.51	1....	H1-1b
87	M87	HSS6X6X6	.115	0	15	.021	0	z	15	303.864	313.812	54.51	54.51	3....	H1-1b
88	M88	HSS6X6X6	.023	2.081	15	.116	0	y	14	303.864	313.812	54.51	54.51	1....	H1-1b
89	M89	HSS8X8X6	.748	11.3	12	.050	0	z	12	378.589	430.56	101.43	101.43	1	H1-1b
90	M90	HSS8X4X4	.502	2.396	15	.219	0	y	15	211.522	216.936	26.719	45.885	1	H1-1b
91	M91	HSS8X8X10	.673	37.5	10	.092	0	y	10	162.88	678.96	154.215	154.215	1....	H1-1b
92	M92	HSS8X8X10	.300	18.063	10	.131	18.063	y	10	334.634	678.96	154.215	154.215	2....	H1-1b
93	M93	HSS8X8X10	.106	.906	15	.245	0	y	15	678.222	678.96	154.215	154.215	1	H3-6
94	M94	HSS8X8X10	.173	5.846	15	.112	0	y	12	654.173	678.96	154.215	154.215	1	H1-1b
95	M95	HSS8X8X10	.166	0	15	.078	5.825	y	10	654.351	678.96	154.215	154.215	1	H1-1b
96	M96	HSS8X8X10	.247	5.894	10	.121	5.894	y	10	653.774	678.96	154.215	154.215	2....	H1-1b
97	M97	HSS8X8X10	.414	0	10	.170	0	y	10	633.226	678.96	154.215	154.215	2....	H1-1b
98	M98	HSS12X4X6	.182	17.797	10	.061	17.797	y	10	291.537	430.56	53.841	126.615	2....	H1-1b
99	M99	HSS8X8X10	.220	0	10	.097	0	y	10	656.976	678.96	154.215	154.215	1....	H1-1b
100	M100	HSS12X4X6	.373	25.254	10	.147	45.917	y	10	86.813	430.56	53.841	126.615	1....	H1-1b
101	M101	HSS8X8X10	.305	0	15	.111	0	y	12	602.207	678.96	154.215	154.215	1....	H1-1b
102	M102	HSS8X4X10	.154	0	6	.032	0	y	10	440.892	484.38	57.27	94.53	1....	H1-1b
103	M103	HSS8X4X10	.222	0	10	.035	0	y	10	456.905	484.38	57.27	94.53	1....	H1-1b
104	M104	HSS8X4X6	.271	0	6	.035	0	y	12	280.192	313.812	39.675	64.86	1....	H1-1b
105	M105	HSS12X4X6	.083	0	15	.008	0	y	6	375.749	430.56	53.841	126.615	1....	H1-1b
106	M106	HSS12X4X6	.066	0	10	.035	0	y	10	350.754	430.56	53.841	126.615	1....	H1-1b
107	M107	HSS12X4X6	.169	0	15	.065	0	y	10	139.548	430.56	53.841	126.615	1....	H1-1b
108	M108	HSS12X4X6	.113	.69	12	.044	11.045	y	12	281.411	430.56	53.841	126.615	1....	H1-1b
109	M109	HSS12X4X6	.054	18.827	10	.014	18.827	y	10	127.918	430.56	53.841	126.615	1....	H1-1b
110	M110	HSS12X4X6	.044	0	10	.031	0	y	10	108.947	430.56	53.841	126.615	1....	H1-1b
111	M111	HSS6X4X4	.099	0	10	.026	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
112	M112	HSS4X4X4	.141	11.25	10	.005	11.406	y	10	54.386	139.518	16.181	16.181	1....	H1-1b
113	M113	HSS6X4X4	.080	0	10	.016	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
114	M114	HSS4X4X4	.136	11.25	10	.005	11.406	z	10	54.386	139.518	16.181	16.181	1....	H1-1b
115	M115	HSS6X4X4	.081	0	10	.014	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
116	M116	HSS12X4X4	.024	31.219	10	.011	5.01	y	10	127.552	293.94	30.641	88.32	2....	H1-1b
117	M117	HSS6X4X6	.414	8.813	10	.142	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
118	M118	HSS6X4X4	.093	3.75	10	.079	7.5	y	10	144.145	178.02	22.252	29.428	1....	H1-1b
119	M119	HSS6X4X6	.412	8.813	10	.125	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
120	M120	HSS6X4X4	.093	3.75	10	.088	0	y	10	144.145	178.02	22.252	29.428	1....	H1-1b
121	M121	HSS6X4X6	.412	8.813	10	.075	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
122	M122	HSS6X4X4	.093	3.75	10	.043	7.5	y	12	144.145	178.02	22.252	29.428	1....	H1-1b
123	M123	HSS6X4X6	.415	8.625	10	.078	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
124	M124	HSS6X4X4	.093	3.75	10	.067	7.5	y	10	144.145	178.02	22.252	29.428	1....	H1-1b
125	M125	HSS6X4X6	.378	6.563	10	.132	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
126	M126	HSS6X4X4	.093	3.75	10	.108	0	y	10	144.145	178.02	22.252	29.428	1....	H1-1b
127	M127	HSS6X4X6	.253	12.375	10	.139	0	y	10	72.147	255.852	30.843	41.055	1....	H1-1b
128	M128	HSS6X4X4	.101	3.828	10	.096	7.5	y	10	144.145	178.02	22.252	29.428	1....	H1-1b
129	M129	HSS4X4X4	.125	0	10	.053	0	y	10	101.456	151.65	17.588	17.588	2....	H1-1b
130	M130	HSS4X4X4	.122	9.4	15	.046	0	y	6	101.456	151.65	17.588	17.588	2....	H1-1b
131	M131	HSS4X4X4	.111	0	6	.047	9.4	y	6	101.456	151.65	17.588	17.588	2....	H1-1b
132	M132	HSS4X4X4	.109	0	6	.036	0	y	6	87.233	151.65	17.588	17.588	2....	H1-1b

Envelope AISC 14th(360-10): LRFD Steel Code Checks (Continued)

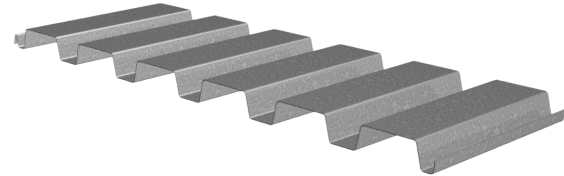
	Member	Shape	Code C...	Loc[ft]	LC	Shear ...	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y...	phi*Mn z...	Cb	Eqn
133	M133	HSS4X4X4	.207	11.026	10	.055	11.026	y	10	87.233	151.65	17.588	17.588	2....	H1-1b
134	M134	HSS4X4X4	.022	0	10	.013	0	y	10	135.959	151.65	17.588	17.588	2....	H1-1b
135	M135	HSS4X4X4	.024	6.4	10	.011	6.4	y	10	125.87	151.65	17.588	17.588	2....	H1-1b
136	M136	HSS4X4X4	.133	11.25	10	.005	11.406	y	10	54.386	139.518	16.181	16.181	1....	H1-1b
137	M137	HSS6X4X4	.081	0	10	.013	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
138	M138	HSS4X4X4	.133	11.25	10	.006	11.406	y	10	54.386	139.518	16.181	16.181	1....	H1-1b
139	M139	HSS6X4X4	.086	0	10	.016	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
140	M140	HSS4X4X4	.145	11.25	10	.012	11.406	y	10	54.386	139.518	16.181	16.181	1....	H1-1b
141	M141	HSS6X4X4	.124	0	10	.028	0	y	10	172.108	178.02	22.252	29.428	1....	H1-1b
142	M142	HSS4X4X4	.014	4.4	10	.007	4.4	y	10	138.866	151.65	17.588	17.588	3....	H1-1b
143	M143	HSS4X4X4	.025	6.4	10	.010	6.4	y	10	125.87	151.65	17.588	17.588	2....	H1-1b
144	M144	HSS4X4X4	.024	4.3	10	.009	4.3	y	10	139.416	151.65	17.588	17.588	3....	H1-1b
145	M145	HSS4X4X4	.055	0	10	.034	0	y	10	118.211	151.65	17.588	17.588	2....	H1-1b
146	M146	HSS8X8X6	.621	11.3	10	.036	0	z	10	406.932	468	110.25	110.25	1....	H1-1b
147	M147	HSS4X4X4	.024	4.9	10	.012	4.9	y	10	135.959	151.65	17.588	17.588	2....	H1-1b
148	M148	HSS4X4X4	.042	6.4	10	.019	0	y	10	125.87	151.65	17.588	17.588	1....	H1-1b
149	M149	HSS4X4X4	.024	4.4	10	.012	4.4	y	10	138.866	151.65	17.588	17.588	3....	H1-1b
150	M150	HSS4X4X4	.038	6.4	10	.018	6.4	y	10	125.87	151.65	17.588	17.588	1....	H1-1b
151	M151	HSS4X4X4	.026	4.3	10	.015	0	y	10	139.416	151.65	17.588	17.588	3....	H1-1b
152	M152	HSS4X4X4	.053	0	10	.028	0	y	10	118.211	151.65	17.588	17.588	1....	H1-1b
153	M153	HSS6X4X4	.298	.336	10	.542	0	y	10	176.1	178.02	22.252	29.428	1....	H3-6
154	M154	HSS6X4X4	.051	1.281	10	.224	3	y	10	172.108	178.02	22.252	29.428	1....	H3-6
155	M155	HSS6X4X4	.004	0	15	.007	0	y	12	166.618	178.02	22.252	29.428	1....	H1-1b*
156	M156	HSS6X4X4	.032	0	16	.198	0	y	10	160.211	178.02	22.252	29.428	1....	H1-1b*
157	M157	HSS6X4X4	.005	2.65	15	.119	0	y	10	160.211	178.02	22.252	29.428	1....	H1-1b
158	M158	HSS6X4X4	.004	2.65	15	.041	0	y	10	160.211	178.02	22.252	29.428	1....	H1-1b

PLB™-36/HSB®-36 ROOF DECKS GRADE 50 STEEL

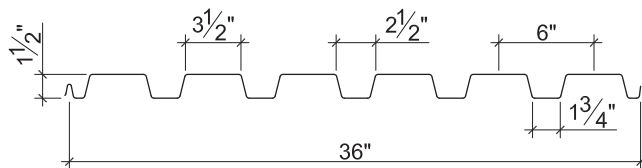
ASD

B ROOF DECKS

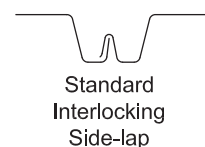
- PLB-36 Deck used with PunchLok® II System
- HSB-36 Deck used with TSWs or BPs
- HSB-36-SS Deck used with Side-lap Screws



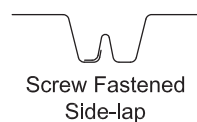
Nominal Dimensions



PLB-36 or HSB-36



HSB-36-SS



Section Properties

Deck Gage	Deck Weight w_{dd} (psf)	Base Metal Thickness t (in.)	Yield Strength F_y (ksi)	Effective Moment of Inertia at Service Load $I_d = (2I_e + I_g)/3$		Effective Section Modulus at $F_y = 50$ ksi		Vertical Web Shear V_n/Ω (lb/ft)
				I_{d+} (in ⁴ /ft)	I_{d-} (in ⁴ /ft)	S_{e+} (in ³ /ft)	S_{e-} (in ³ /ft)	
22	1.9	0.0299	50	0.178	0.192	0.176	0.188	2688
20	2.3	0.0359	50	0.219	0.231	0.230	0.237	3220
18	2.9	0.0478	50	0.302	0.306	0.314	0.331	4264
16	3.5	0.0598	50	0.381	0.381	0.399	0.410	5302

Allowable Reactions at Supports Based on Web Crippling, R_n/Ω (lb/ft)

Deck Gage	Bearing Length of Webs											
	One-Flange Loading						Two-Flange Loading					
	End Bearing				Interior Bearing		End Bearing				Interior Bearing	
	1 1/2"	2"	3"	4"	3"	4"	1 1/2"	2"	3"	4"	3"	4"
22	850	934	1075	1163	1558	1670	893	962	1077	1149	1933	2082
20	1188	1301	1492	1609	2189	2339	1316	1413	1575	1675	2743	2946
18	2001	2182	2485	2667	3714	3949	2388	2550	2822	2986	4713	5038
16	3006	3264	3698	3954	5604	5935	3775	4015	4419	4657	7164	7627

Standard Features

- ASTM A653 SS GR50 Min., with G60 or G90, white or gray primer optional
- ASTM A1008 SS GR50 Min. with gray primer
- Standard lengths – 6'-0" to 40'-0"
- IAPMO UES ER-2018, UL, and FM Listed
- Tables conform to ANSI/SDI RD-2017

Optional Features

- Inquire regarding cost and lead times for:
 - Short cuts < 6'-0"
 - Sheet Lengths > 40'-0"
 - Alternative metallic and painted finishes
- Web and Fully Perforated Acoustical Versions
- HSB-30-NS Deck used with Side-lap screws

PLB™-36/HSB®-36 ROOF DECKS GRADE 50 STEEL

ASD

Install Deck continuous over two or more spans per General Notes

Inward Uniform Allowable Loads, ASD (psf)

Deck Gage	Spans	Criteria	Span (ft.-in.)										
			2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	Single	W_n / Ω	878	390	219	140	98	72	55	43	35	29	24
		L/240	---	---	182	93	54	34	23	16	12	9	7
	Double	W_n / Ω	860	400	229	148	103	76	58	46	37	31	26
		L/240	---	---	---	---	---	---	---	42	30	23	18
	Triple	W_n / Ω	1039	492	283	184	128	95	73	57	47	39	32
		L/240	---	---	---	176	102	64	43	30	22	17	13
20	Single	W_n / Ω	1147	510	287	184	127	94	72	57	46	38	32
		L/240	---	---	224	115	66	42	28	20	14	11	8
	Double	W_n / Ω	1075	503	288	186	130	96	73	58	47	39	33
		L/240	---	---	---	---	---	---	71	50	36	27	21
	Triple	W_n / Ω	1295	617	356	231	162	119	92	72	59	49	41
		L/240	---	---	---	217	125	79	53	37	27	20	16
18	Single	W_n / Ω	1566	696	392	251	174	128	98	77	63	52	44
		L/240	---	---	309	158	92	58	39	27	20	15	11
	Double	W_n / Ω	1486	699	401	259	181	134	102	81	66	54	46
		L/240	---	---	---	---	---	---	94	66	48	36	28
	Triple	W_n / Ω	1785	856	496	322	225	166	128	101	82	68	57
		L/240	---	---	---	299	173	109	73	51	37	28	22
16	Single	W_n / Ω	1992	885	498	319	221	163	124	98	80	66	55
		L/240	---	---	390	200	116	73	49	34	25	19	14
	Double	W_n / Ω	1842	865	497	321	224	165	127	100	81	67	57
		L/240	---	---	---	---	---	---	118	83	60	45	35
	Triple	W_n / Ω	2213	1060	614	399	279	206	158	125	102	84	71
		L/240	---	---	---	377	218	137	92	65	47	35	27

Notes:

1. Table does not account for web crippling. Required bearing should be determined based on specific span conditions.
2. The symbol "----" indicates that the uniform allowable load based on deflection exceeds the allowable load based on stress.

Roof:

Max span = 6'-6" (L/240)

Max load = 18 psf (1.0D) + 72 psf (1.0S Max) = 90 psf < 102 psf OK

Joist Load = 21 psf * 6.5' + 35 psf * 6.5' = 137 plf (1.0D) + 228 plf (1.0S) = 365 plf

Joist uplift = 0.6*(21 psf - 70.5 psf zone 2) = -28.5 psf * 6.5' = -185 plf

Deck uplift = 0.6*(18 psf D - 70.5 psf zone 2) = 32 psf uplift, OK

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General Beam

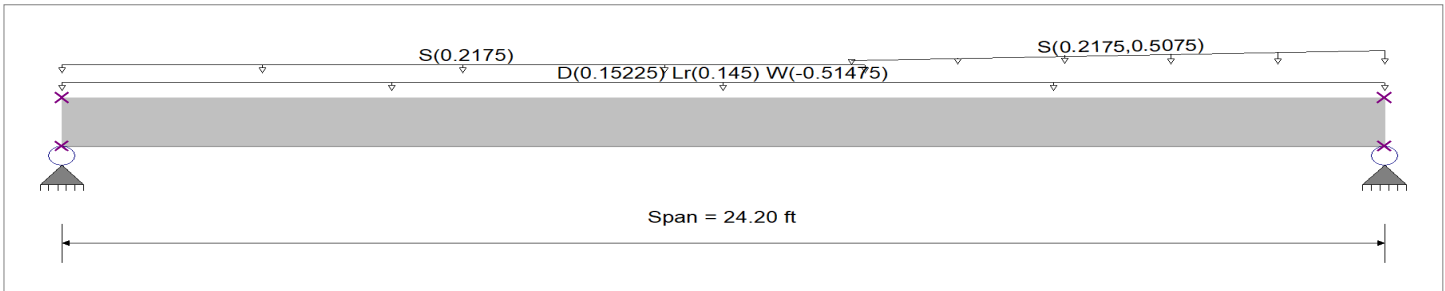
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KPFF Mountain West

DESCRIPTION: J1 - 18K Joists

General Beam Properties

Elastic Modulus 29,000.0 ksi
Span #1 Span Length = 24.20 ft Area = 10.0 in² Moment of Inertia = 100.0 in⁴



Applied Loads

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

Uniform Load : D = 0.0210, Lr = 0.020, W = -0.0710 ksf, Tributary Width = 7.250 ft, (D+Lr)

Uniform Load : S = 0.030 ksf, Extent = 0.0 --> 14.70 ft, Tributary Width = 7.250 ft, (Drift 1)

Varying Uniform Load : S = 0.030->0.070 ksf, Extent = 14.450 --> 24.20 ft, Trib Width = 7.250 ft, (Drift 2)

DESIGN SUMMARY

Maximum Bending =	29.687 k-ft	Maximum Shear =	5.731 k
Load Combination	+D+S	Load Combination	+D+S
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	12.705 ft	Location of maximum on span	24.200 ft
Maximum Deflection			
Max Downward Transient Deflection	0.687 in	422	
Max Upward Transient Deflection	-1.381 in	210	
Max Downward Total Deflection	1.095 in	265	
Max Upward Total Deflection	-0.583 in	497	

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values (k-ft)							Shear Values (k)		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope													
Dsgn. L = 24.20 ft	1			29.69		29.69					5.73		
D Only													
Dsgn. L = 24.20 ft	1			11.15		11.15					1.84		
+D+Lr													
Dsgn. L = 24.20 ft	1			21.76		21.76					3.60		
+D+S													
Dsgn. L = 24.20 ft	1			29.69		29.69					5.73		
+D+0.750Lr													
Dsgn. L = 24.20 ft	1			19.11		19.11					3.16		
+D+0.750S													
Dsgn. L = 24.20 ft	1			25.05		25.05					4.76		
+D+0.60W													
Dsgn. L = 24.20 ft	1				-11.46	11.46					1.89		
+D+0.750Lr+0.450W													
Dsgn. L = 24.20 ft	1			2.15		2.15					0.36		
+D+0.750S+0.450W													
Dsgn. L = 24.20 ft	1			8.20		8.20					1.96		
+0.60D+0.60W													
Dsgn. L = 24.20 ft	1				-15.92	15.92					2.63		
+0.60D													
Dsgn. L = 24.20 ft	1			6.69		6.69					1.11		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-1.3807	

General Beam

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DESCRIPTION: J1 - 18K Joists

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-6.228	-6.228
Overall MINimum	-6.228	-6.228
D Only	1.842	1.842
+D+Lr	3.597	3.597
+D+S	4.685	5.731
+D+0.750Lr	3.158	3.158
+D+0.750S	3.975	4.759
+D+0.60W	-1.895	-1.895
+D+0.750Lr+0.450W	0.355	0.355
+D+0.750S+0.450W	1.172	1.956
+0.60D+0.60W	-2.632	-2.632
+0.60D	1.105	1.105
Lr Only	1.755	1.755
S Only	2.843	3.888
W Only	-6.228	-6.228

General Beam

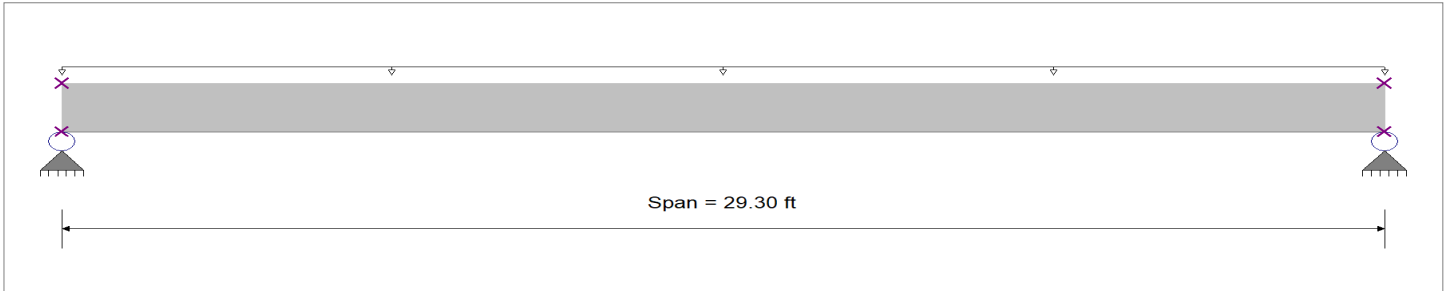
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Lic. # : KW-06008320

DESCRIPTION: J2 - 20K Joists

General Beam Properties

Elastic Modulus 29,000.0 ksi
Span #1 Span Length = 29.30 ft Area = 10.0 in² Moment of Inertia = 100.0 in⁴



Applied Loads

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

DESIGN SUMMARY

Maximum Bending =	41.889 k-ft	Maximum Shear =	6.645 k
Load Combination	+D+S	Load Combination	+D+S
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	15.090 ft	Location of maximum on span	29.300 ft
Maximum Deflection			
Max Downward Transient Deflection	1.388 in	253	
Max Upward Transient Deflection	-2.967 in	118	
Max Downward Total Deflection	2.266 in	155	
Max Upward Total Deflection	-1.254 in	280	

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios		Summary of Moment Values (k-ft)							Shear Values (k)		
			M	V	Mmax +	Mmax -	Ma - Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Overall MAXimum Envelope														
Dsgn. L = 29.30 ft		1			41.89		41.89					6.65		
D Only														
Dsgn. L = 29.30 ft		1			16.34		16.34					2.23		
+D+Lr														
Dsgn. L = 29.30 ft		1			31.90		31.90					4.35		
+D+S														
Dsgn. L = 29.30 ft		1			41.89		41.89					6.65		
+D+0.750Lr														
Dsgn. L = 29.30 ft		1			28.01		28.01					3.82		
+D+0.750S														
Dsgn. L = 29.30 ft		1			35.50		35.50					5.54		
+D+0.60W														
Dsgn. L = 29.30 ft		1				-16.80	16.80					2.29		
+D+0.750Lr+0.450W														
Dsgn. L = 29.30 ft		1			3.15		3.15					0.43		
+D+0.750S+0.450W														
Dsgn. L = 29.30 ft		1			10.70		10.70					2.15		
+0.60D+0.60W														
Dsgn. L = 29.30 ft		1				-23.34	23.34					3.19		
+0.60D														
Dsgn. L = 29.30 ft		1			9.80		9.80					1.34		

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-2.9670	14.797

Vertical Reactions

Load Combination	Support 1	Support 2
Overall MAXimum	-7.541	-7.541
Overall MINimum	-7.541	-7.541
D Only	2.230	2.230
+D+Lr	4.355	4.355
+D+S	5.566	6.645
+D+0.750Lr	3.824	3.824

General Beam

File: Roof Framing.ec6
Software copyright ENERCALC, INC. 1983-2020, Build:12.20.5.31
KPFF

Lic. # : KW-06008320

DESCRIPTION: J2 - 20K Joists

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
+D+0.750S	4.732	5.542
+D+0.60W	-2.294	-2.294
+D+0.750Lr+0.450W	0.430	0.430
+D+0.750S+0.450W	1.338	2.148
+0.60D+0.60W	-3.186	-3.186
+0.60D	1.338	1.338
Lr Only	2.124	2.124
S Only	3.335	4.415
W Only	-7.541	-7.541

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Available now!

SJI 45th edition

The latest load tables for steel joists and joist girders.

[Download](#)

Select Joist Series:

K, LH & DLH-Series

Unit of Measure:

U.S. Units

Design Methodology:

ASD

Required Joist Specifications

Determine steel joist by one of the following:

"Lowest weight" per SJI standard load tables

Span (ft):

33

Dead Load (psf):

25

Live Load (psf):

35

Avg Snow w/ Drifts

Live Load Deflection:

L/360

Spacing Between Steel Joists (ft):

6

Total Design Load (plf):

360

Optional Joist Specifications:

Max. Joist Depth (in):

28

Joist Series:

K

Submit

Results:

Joist Designation:

26K6

Total Bridging Rows Required:

2

OSHA Erection Bridging Rows Required:

0

Minimum Bearing Seat Depth (in):

2.5

Total Safe Load (plf):

364

[View Additional Joist Options](#)

**Additional Joist Options
(ASD)**

Joist Designation:	Self-weight (plf):	Total Bridging Rows Required:	OSHA Erection Bridging Rows Required:	Minimum Bearing Seat Depth (in):	Total Safe Load (plf):
26K6	8.6	2	0	2.5	364
28K6	8.9	2	0	2.5	393
24K7	9.0	2	0	2.5	373
26K7	9.0	2	0	2.5	406
28K7	9.2	2	0	2.5	438
24K8	9.4	2	0	2.5	413
26K8	9.7	2	0	2.5	448
28K8	9.8	2	0	2.5	484
20K9	10.1	2	0	2.5	371
22K9	10.2	2	0	2.5	410

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The latest load tables for steel joists and joist girders.

[Download](#)

Select Joist Series:

K, LH & DLH-Series

Unit of Measure:

U.S. Units

Design Methodology:

ASD

Required Joist Specifications

Determine steel joist by one of the following:

"Lowest weight" per SJI standard load tables

Span (ft):

30

Dead Load (psf):

25

Live Load (psf):

45

[Avg Snow w/ Drifts](#)

Live Load Deflection:

L/360

Spacing Between Steel Joists (ft):

7.25

Total Design Load (plf):

5075

Optional Joist Specifications:

Max. Joist Depth (in):

24

Joist Series:

Select

Submit

Results:

Joist Designation:

24K9

[Use 20K10](#)

Total Bridging Rows Required:

2

OSHA Erection Bridging Rows Required:

0

Minimum Bearing Seat Depth (in):

2.5

Total Safe Load (plf):

544

[View Additional Joist Options](#)

**Additional Joist Options
(ASD)**

Joist Designation:	Self-weight (plf):	Total Bridging Rows Required:	OSHA Erection Bridging Rows Required:	Minimum Bearing Seat Depth (in):	Total Safe Load (plf):
24K9	10.3	2	0	2.5	544
20K10	11.6	2	0	2.5	533
22K10	11.7	2	0	2.5	550
24K10	11.7	2	0	2.5	550
22K11	11.9	2	0	2.5	550
24LH05	13.0	2	0	5.0	509
24K12	13.5	2	0	2.5	550
20LH05	14.0	2	0	5.0	571
20LH06	15.0	2	0	5.0	679
18LH06	15.0	2	0	5.0	605

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Available now!

SJI 45th edition

The latest load tables for steel joists and joist girders.

[Download](#)

Select Joist Series:

K, LH & DLH-Series

Unit of Measure:

U.S. Units

Design Methodology:

ASD

Required Joist Specifications

Determine steel joist by one of the following:

"Lowest weight" per SJI standard load tables

Span (ft):

25

Dead Load (psf):

25

Live Load (psf):

45

[Avg Snow w/ Drifts](#)

Live Load Deflection:

L/360

Spacing Between Steel Joists (ft):

7.25

Total Design Load (plf):

5075

Optional Joist Specifications:

Max. Joist Depth (in):

18

Joist Series:

Select

Submit

Results:

Joist Designation:

18K9

Total Bridging Rows Required:

2

OSHA Erection Bridging Rows Required:

0

Minimum Bearing Seat Depth (in):

2.5

Total Safe Load (plf):

550

[View Additional Joist Options](#)

**Additional Joist Options
(ASD)**

Joist Designation:	Self-weight (plf):	Total Bridging Rows Required:	OSHA Erection Bridging Rows Required:	Minimum Bearing Seat Depth (in):	Total Safe Load (plf):
18K9	10.1	2	0	2.5	550
18LH03	11.0	2	0	5.0	539
18K10	11.6	2	0	2.5	550
18LH04	12.0	2	0	5.0	628
18LH06	15.0	1	0	5.0	839
18LH05	15.0	2	0	5.0	709
18LH07	17.0	1	0	5.0	872
18LH08	19.0	1	0	5.0	908
18LH09	21.0	1	0	5.0	973

Building a better steel experience



project BOA Centerville

location Centerville, UT

client Gensler

by NRP

date 9/22/20

job no.

10211900134

Joist to Bottom Flange

- Joist J2 seat bears on W24x bottom flange, check bending and connection

Rxn ptr Enercalc: $P_{bearing} = 5.6 \text{ K ASD (D+S)}$
 $P_{uplift} = -3.2 \text{ K ASD (0.6 D + 0.6 W)}$

W24x55 $bf = 7.01"$, $tf = 0.505"$

Bearing seat interacts w/ 12" length of flange

$$M = 5.6 \text{ K} \times 1.75" \text{ eccentricity} = 9.8 \text{ K-in}$$

$$M_u / \Omega = F_y Z / \Omega = (50 \text{ Ksi}) (12" \times 0.505"^2 / 4) / 1.67 = 22.9 \text{ K-in}$$

$$M_u / \Omega > M = 9.8 \text{ K-in, OK} \checkmark$$

Full ht stiffeners & bot chord extension prevents torsion in Wx, OK

Weld to prevent uplift: 3/16" x 2" fillet (2) sides

$$P_{uplift} = 3.2 \text{ K}$$

$$r_n / \Omega = (2)(0.928)(3)(2) = 11.1 \text{ K,}$$

$$r_n / \Omega > P_{uplift} = 3.2 \text{ K, OK} \checkmark$$

∴ W24x55 bot flange adequate for bot flange loading

EXTERIOR STUDWALLS

Project Name: BOA Centerville Metal Studs

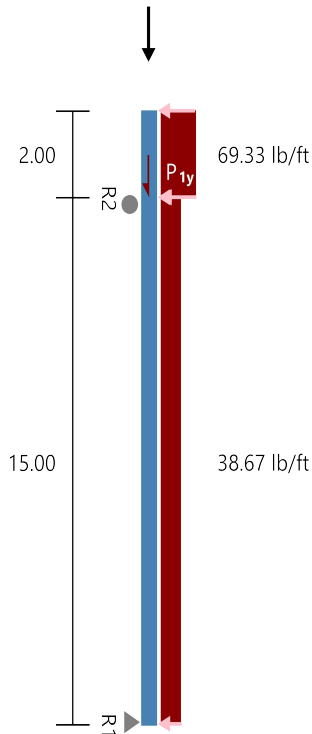
Model: Grid D Stud - Hi Zone 5

Code: 2012 NASPEC [AISI S100-2012]

Page 1 of 2

Date: 12/09/2020

Simpson Strong-Tie® CFS Designer™ 3.4.5.0



Section : 800S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)

Maxo = 3065.9 Ft-Lb **Va =** 2091.3 lb **I =** 5.60 in⁴

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	Flexural, Distortional	Connector	Stress Ratio
Top Cant.	None, None	None, 24.0"	N/A	-
Span	48.0", 48.0"	48.0", 180.0"	LSUBH3.25 (Min)	0.57

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	437.9	--Shear Connection w/ clip--				NO
R1	280.8	--Shear Connection w/ clip--				NO

Gravity Load

Type	Load (lb)
Uniform	21.33plf (Top Cantilever), 21.33plf (Span)
P1y	800lb @ 15ft

	Code Check	Required	Allowed	Interaction	Notes
Top Cant.	Max. Axial, lbs	842.7(c)	7227.6(c)	12%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	138.7	2091.3	7%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	138.7	2734.3	5%	Ma-dist (control), KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	88.5	3065.9	3%	
	Shear/Moment	0.08	1.00	8%	Shear 138.7, Moment 138.7
	Axial/Moment	0.17	1.00	17%	Axial 842.7(c), Moment 138.7
	Deflection Cant., in	0.068	--meets L/701--		
Span	Max. Axial, lbs	1162.7(c)	5517.8(c)	21%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	299.2	2091.3	14%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	1019.3	2734.3	37%	Ma-dist (control), KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	1019.3	2784.3	37%	
	Shear/Moment	0.33	1.00	33%	Shear 0.0, Moment 1019.3
	Axial/Moment	0.57	1.00	57%	Axial 1013.9(c), Moment 1017.7
	Deflection Span, in	0.172	--meets L/1044--		

Project Name: BOA Centerville Metal Studs

Page 2 of 2

Model: Grid D Stud - Hi Zone 5

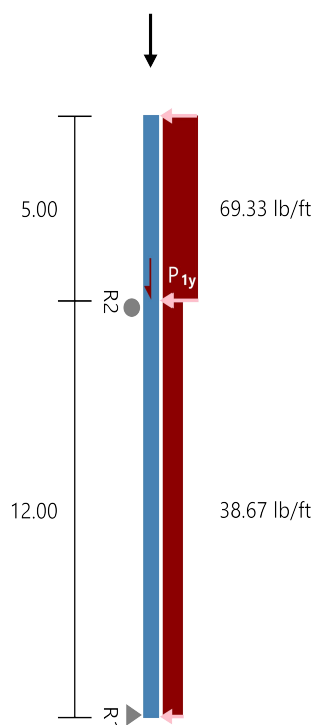
Date: 12/09/2020

Code: 2012 NASPEC [AISI S100-2012]

Simpson Strong-Tie® CFS Designer™ 3.4.5.0

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	437.9	0.0	SCB45.5(3) & (2) #12-24 SST X or XL to A36 Steel	44.23 %	39.27 %
R1	280.8	1162.7	800T125-33 (33) & (2) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	40.81 %	63.75 %

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



Section : 800S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)
Maxo = 3065.9 Ft-Lb **Va =** 2091.3 lb **I =** 5.60 in⁴

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	Flexural, Distortional	Connector	Stress Ratio
Top Cant.	48.0", 48.0"	48.0", 60.0"	LSUBH3.25 (Min)	0.93
Span	48.0", 48.0"	48.0", 144.0"	LSUBH3.25 (Min)	0.54

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	650.9	--Shear Connection w/ clip--				NO
R1	159.8	--Shear Connection w/ clip--				NO

Gravity Load

Type	Load (lb)
Uniform	21.33plf (Top Cantilever), 21.33plf (Span)
P1y	250lb @ 12ft

	Code Check	Required	Allowed	Interaction	Notes
Top Cant.	Max. Axial, lbs	356.7(c)	5517.8(c)	6%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	346.7	2091.3	17%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	866.7	2734.3	32%	Ma-dist (control), KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	553.3	2779.6	20%	
	Shear/Moment	0.33	1.00	33%	Shear 346.7, Moment 866.7
	Axial/Moment	0.38	1.00	38%	Axial 356.7(c), Moment 866.7
	Deflection Cant., in	0.065	--meets L/1856--		
Span	Max. Axial, lbs	612.7(c)	5517.8(c)	11%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	304.2	2091.3	15%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	866.7	2734.3	32%	Ma-dist (control), KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	330.1	2793.8	12%	
	Shear/Moment	0.32	1.00	32%	Shear 304.2, Moment 866.7
	Axial/Moment	0.38	1.00	38%	Axial 356.7(c), Moment 866.7
	Deflection Span, in	0.022	--meets L/6638--		

Project Name: BOA Centerville Metal Studs

Page 2 of 2

Model: Grid D Stud - Lo Zone 5

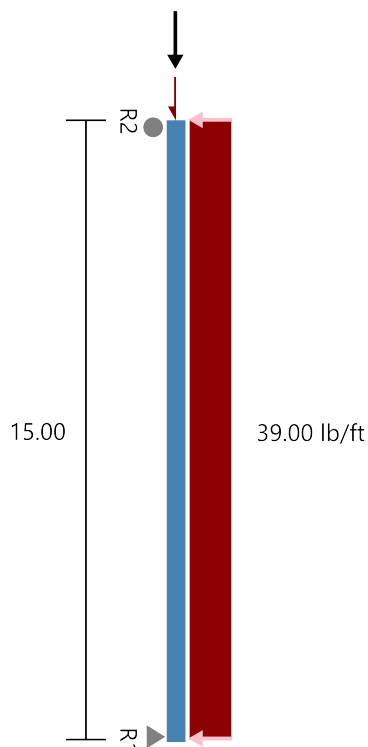
Date: 12/09/2020

Code: 2012 NASPEC [AISI S100-2012]

Simpson Strong-Tie® CFS Designer™ 3.4.5.0

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	650.9	0.0	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	85.64 %	58.38 %
R1	159.8	612.7	800T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	17.85 %	21.74 %

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



Loads have not been modified for strength checks

Loads have been multiplied by 0.70 for deflection calculations

Built-Up Section:

Section Number	Section	Ixx (in ⁴)	% of Total Ixx	Area (in ²)	% of Total Area
1	800S162-54(50)	5.600	33.3 %	0.670	33.3 %
2	800S162-54(50)	5.600	33.3 %	0.670	33.3 %
3	800S162-54(50)	5.600	33.3 %	0.670	33.3 %

Overall Member Inputs:

Span	Bracing (in)	Flexure		Load (lb)	KyLy (in)	Axial		Lm (in)
		K-phi (lb-in/in)	Lm (in)			KtLt (in)	K-phi (lb-in/in)	
Span	48	0	180	10210.0(c)	48.0	48.0	0.0	180.0

Reaction And Point Load Data:

	R1	R2
Load (lb)	292.5	292.5
Brng (in)	1.00	1.50

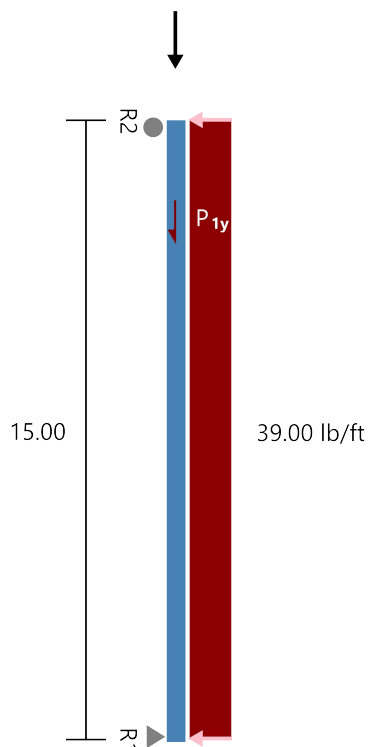
Analysis Summary:

Section	<u>Flexure</u>		<u>Web Crippling</u>	<u>Shear & Bending</u>			<u>Axial</u>	
	Defl	M/Ma	Stiffen Req'd	V/Va	Unstiffened	Stiffened	P/Pa	Combined
800S162-54(50)	L/2869	0.134	No	0.05	0.05	N/A	0.62	0.74
800S162-54(50)	L/2869	0.134	No	0.05	0.05	N/A	0.62	0.74
800S162-54(50)	L/2869	0.134	No	0.05	0.05	N/A	0.62	0.74

Simpson Strong-Tie® Connectors

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	292.5	0	By Others & Anchorage Designed by Engineer	NA	NA
R1	292.5	10210	800T125-33 (33) & (2) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	14.17 %	66.42 %

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



Loads have not been modified for strength checks

Loads have been multiplied by 0.70 for deflection calculations

Built-Up Section:

Section Number	Section	Ixx (in ⁴)	% of Total Ixx	Area (in ²)	% of Total Area
1	800S162-54(50)	5.600	50.0 %	0.670	50.0 %
2	800S162-54(50)	5.600	50.0 %	0.670	50.0 %

Overall Member Inputs:

Span	Bracing (in)	Flexure		Load (lb)	KyLy (in)	Axial		Lm (in)
		K-phi (lb-in/in)	Lm (in)			KtLt (in)	K-phi (lb-in/in)	
Span	48	0	180	7410.0(c)	48.0	48.0	0.0	180.0

Reaction And Point Load Data:

	R1	R2
Load (lb)	292.5	292.5
Brng (in)	1.00	1.50

Analysis Summary:

Section	<u>Flexure</u>		<u>Web Crippling</u>	<u>Shear & Bending</u>			<u>Axial</u>	
	Defl	M/Ma	Stiffen Req'd	V/Va	Unstiffened	Stiffened	P/Pa	Combined
800S162-54(50)	L/1913	0.201	No	0.07	0.07	N/A	0.67	0.86
800S162-54(50)	L/1913	0.201	No	0.07	0.07	N/A	0.67	0.86

Simpson Strong-Tie® Connectors

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	292.5	0	By Others & Anchorage Designed by Engineer	NA	NA
R1	292.5	7410	800T125-33 (33) & (2) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	21.26 %	66.42 %

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



Salt Lake City, Utah
(801) 441-2204

project BOA Centerville

location Centerville, UT

client Gensler

by NRP

date 9/22/20

job no.

10211900134

sheet no.

Uplift @ Built-up stud posts

- Canopy Framing to grid D @ 5.25 ft oc

Rxn per RISA: $0.9D + W = -1.7K$

(2) #12 SMS Ea stud to track x 6 studs = (12) #12 SMS

#12 SMS to 54 mill: $V_{allow} = 569 \text{ lb/screw}$

$V_{str} = 812 \text{ lb/screw}$

$P_{capacity} = (12) 812 \text{ lb} = 9.75 K, OK \checkmark$

(2) $\frac{1}{2}" \phi$ Expansion Anch w/ $3 \frac{5}{8}"$ embed into 8" curb

$P_{capacity} = 6750 \text{ lb}$, see attached Simpson output

- Joist framing @ grid I (J2)

Rxn per Enercalc: $0.9D + W = -5.5 K$

(2) #12 SMS ea stud to track, sim to above

$P_{capacity} = 9.75 K, OK \checkmark$

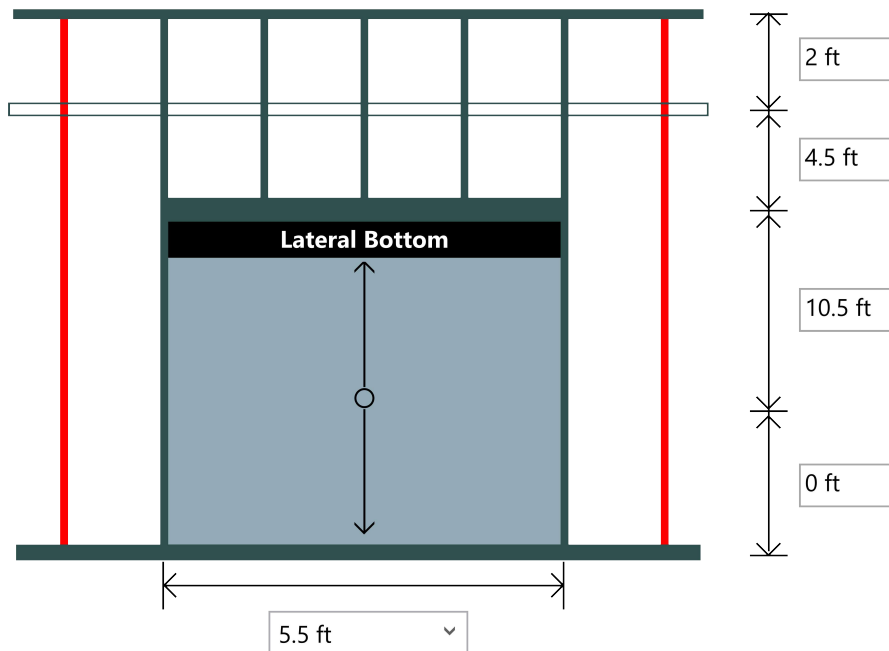
(2) $\frac{1}{2}" \phi$ Expansion anchor sim to above

$P_{capacity} = 6.75 K, OK \checkmark$

(2) $\frac{1}{4}" \times 2"$ fillet weld @ joist seat to built-up post

$\phi_{rn} = 1.392(2)(4)(2) = 22.3 K, OK \checkmark$

$\therefore$ Provide calculated connections on 506.04 dtls

**Design Loads**Wall Lateral Pressure : **25 psf**RO Lateral Pressure : **Head/Sill Only**Parapet Lateral Pressure : **48 psf**

Lateral Element Forces multiplied by 1 for strength checks

Lateral Forces multiplied by 0.7 for deflection determination

Header: **Box (lateral top, bottom)**Gravity Load at Header: **16 psf**

+ Additional load from each cripple of 335 lbs at 16 in o.c.

Lateral Pressure to: **Head/Sill Only** ?**Brace Settings**

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	800S162-54(50), Single@16 in o/c	48 in	48 in	48 in	0	None	N/A
Jamb Studs	800S162-54(50), Boxed	48 in	48 in	48 in	0	None	12 in
Vertical Header	800S162-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	800T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	800S162-54(50), Single@16 in o/c	697.7	874.6	258.5	241.5	386.5
Jamb Studs	800S162-54(50), Boxed	1270.3	1788.4	449.1	618.8	513.1
Vertical Header	800S162-54(50), Boxed	N/A	1343.3	976.9	N/A	976.9
Lat. Top Head	800T125-54(50), Single	N/A	132.0	96.0	N/A	96.0

Summary Design Results

Component(s)	Members(s)	Deflection		Bending +Axial Interaction	Shear Interaction	Web Stiffeners	Design OK
		Span	Parapet				
Wall Studs	800S162-54(50), Single@16 in o/c	L/1218	L/823	0.446	0.13	NA	Yes
Jamb Studs	800S162-54(50), Boxed	L/1376	L/808	0.38	0.31	NA	Yes

Vertical Header	800S162-54(50), Boxed	L/2981	NA	0.22	0.23	No	Yes
Lat. Top Head	800T125-54(50), Single	L/17123	NA	0.06	0.05	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	386.53	0.00	SCB45.5(2) & (4) #12-24 SST X or XL to A36 Steel	50.86 %	17.33 %
R1	241.47	697.67	800T125-54 (50) & (2) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	26.98 %	29.45 %

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

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	513.14	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	67.52 %	46.02 %
R1	618.76	1270.27	800T125-54 (50) & (2) 1/2" x 4" Titen HD to 2500 psi min concrete	46.10 %	42.09 %

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

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

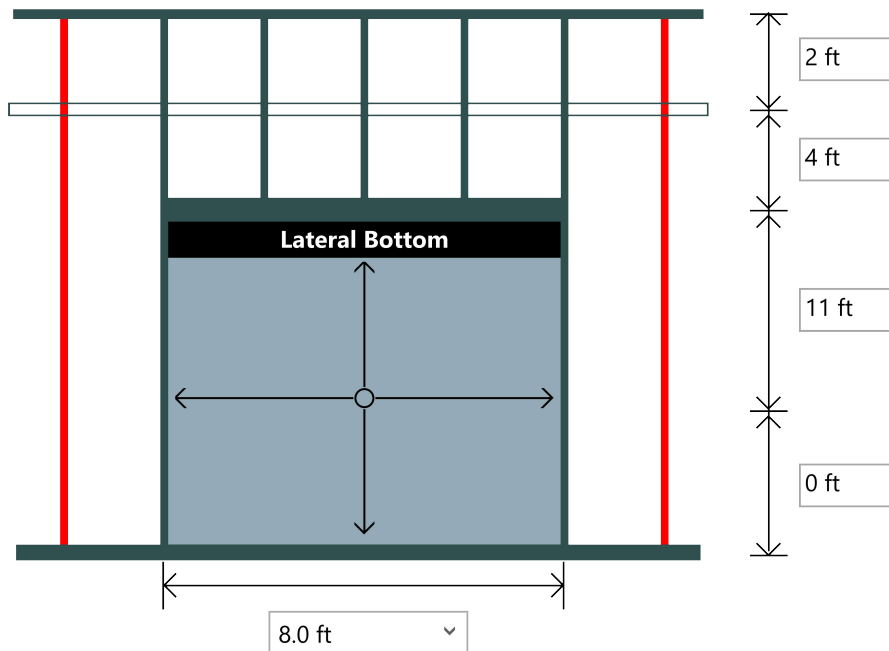
Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min) ¹	LSUBH (Max) ¹	SUBH (Min) ¹	SUBH (Max) ¹	MSUBH (Min) ¹	MSUBH (Max) ¹
Top CantiLever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	4	9932.0	OK (0.48)	OK (0.34)	OK (0.39)	OK (0.22)	OK (0.24)	OK (0.19)

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/CantiLever	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
Top CantiLever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	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**Wall Lateral Pressure : **29 psf**RO Lateral Pressure : **4-Ways**Parapet Lateral Pressure : **52 psf**

Lateral Element Forces multiplied by 1 for strength checks

Lateral Forces multiplied by 0.7 for deflection determination

Header: **Box (lateral top, bottom)**Gravity Load at Header: **16 psf**Lateral Pressure to: **4-Ways****Brace Settings**

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	800S162-54(50), Single@16 in o/c	48 in	48 in	48 in	0	None	N/A
Jamb Studs	800S162-54(50), Boxed	48 in	48 in	48 in	0	None	12 in
Vertical Header	800S162-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	800T125-54(50), Back-To-Back	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	800S162-54(50), Single@16 in o/c	362.7	1019.3	299.2	280.8	437.9
Jamb Studs	800S162-54(50), Boxed	682.7	3258.2	750.6	982.6	780.7
Vertical Header	800S162-54(50), Boxed	N/A	768.0	384.0	N/A	384.0
Lat. Top Head	800T125-54(50), Back-To-Back	N/A	256.0	128.0	N/A	128.0

Summary Design Results

Component(s)	Members(s)	Deflection		Bending +Axial Interaction	Shear Interaction	Web Stiffeners	Design OK
		Span	Parapet				
Wall Studs	800S162-54(50), Single@16 in o/c	L/1044	L/701	0.438	0.15	NA	Yes

Jamb Studs	800S162-54(50), Boxed	L/649	L/416	0.58	0.53	NA	Yes
Vertical Header	800S162-54(50), Boxed	L/3585	NA	0.13	0.09	No	Yes
Lat. Top Head	800T125-54(50), Back- To-Back	L/12143	NA	0.06	0.03	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	437.91	0.00	SCB45.5(2) & (4) #12-24 SST X or XL to A36 Steel	57.62 %	19.64 %
R1	280.76	362.67	800T125-54 (50) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	31.37 %	68.48 %

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

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	780.69	0.00	(2) SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	51.36 %	35.01 %
R1	982.64	682.67	800T125-54 (50) & (2) 1/2" x 4" Titen HD to 2500 psi min concrete	73.21 %	66.85 %

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

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

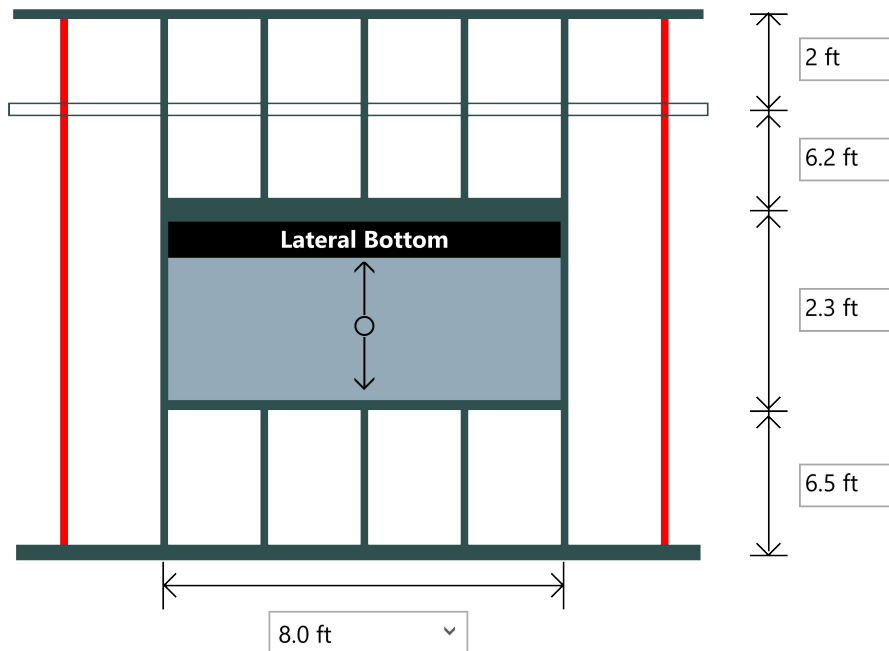
Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min) ¹	LSUBH (Max) ¹	SUBH (Min) ¹	SUBH (Max) ¹	MSUBH (Min) ¹	MSUBH (Max) ¹
Top CantiLever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	4	9932.0	OK (0.53)	OK (0.38)	OK (0.44)	OK (0.24)	OK (0.27)	OK (0.17)

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/CantiLever	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
Top CantiLever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	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**Wall Lateral Pressure : **25 psf**RO Lateral Pressure : **Head/Sill Only**Parapet Lateral Pressure : **48 psf**

Lateral Element Forces multiplied by 1 for strength checks

Lateral Forces multiplied by 0.7 for deflection determination

Header: **Box (lateral top, bottom)**Gravity Load at Header: **16 psf**

+ Additional load from each cripple of 735 lbs at 16 in o.c.

Lateral Pressure to: **Head/Sill Only** ?**Built-Up Members**

Components	Section 1	Section 2	Section 3	Section 4
Jamb Studs	800S162-54(50)	800S162-54(50)	800S162-54(50)	
Vertical Header	800S162-54(50)	800S162-54(50)	800S162-54(50)	

Brace Settings

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	800S162-54(50), Single@16 in o/c	48 in	48 in	48 in	0	None	N/A
Jamb Studs	Built-Up	48 in	48 in	48 in	0	None	N/A
Vertical Header	Built-Up	Full	N/A	N/A	0	None	N/A
Lat. Top Head	800T125-54(50), Back-To-Back	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	800T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Sill	600T125-54(50), Back-To-Back	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	800S162-54(50), Single@16 in o/c	1097.7	874.6	258.5	241.5	386.5
Jamb Studs	Built-Up	3005.0	3028.9	532.9	520.1	596.9

Vertical Header	Built-Up	N/A	5459.6	2729.8	N/A	2729.8
Lat. Top Head	800T125-54(50), Back-To-Back	N/A	496.1	248.1	N/A	248.1
Lat. Bottom Head	800T125-54(50), Single	N/A	230.0	115.0	N/A	115.0
Sill	600T125-54(50), Back-To-Back	N/A	880.0	440.0	N/A	440.0

Summary Design Results

Component(s)	Members(s)	Deflection		Bending +Axial Interaction	Shear Interaction	Web Stiffeners	Design OK
		Span	Parapet				
Wall Studs	800S162-54(50), Single@16 in o/c	L/1218	L/823	0.532	0.13	NA	Yes
Jamb Studs	Built-Up	L/1140	L/754	0.56	0.34	NA	Yes
Vertical Header	Built-Up	L/756		0.666	0.44	R1, R2	Yes
Lat. Top Head	800T125-54(50), Back-To-Back	L/6266	NA	0.12	0.06	No	Yes
Lat. Bottom Head	800T125-54(50), Single	L/6758	NA	0.11	0.06	No	Yes
Sill	600T125-54(50), Back-To-Back	L/1789	NA	0.30	0.08	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	386.53	0.00	SCB45.5(2) & (4) #12-24 SST X or XL to A36 Steel	50.86 %	17.33 %
R1	241.47	1097.67	800T125-54 (50) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	26.98 %	58.89 %

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

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	596.93	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	78.54 %	53.54 %
R1	520.13	3005.00	800T125-54 (50) & (2) 1/2" x 4" Titen HD to 2500 psi min concrete	19.37 %	35.38 %

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

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

Span/CantiLever	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
Top CantiLever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	4	9932.0	OK (0.50)	OK (0.35)	OK (0.41)	OK (0.31)	OK (0.31)	OK (0.31)

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/CantiLever	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
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Project Name: BOA Centerville Metal Studs

Page 3 of 3

Model: 8 ft Window - Grid D, Zone 4

Date: 12/09/2020

Code: 2012 NASPEC [AISI S100-2012]

Simpson Strong-Tie® CFS Designer™ 3.4.5.0

Top Cantilever	Span	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Span	48	4	9932.0	OK (0.36)	OK (0.26)	OK (0.33)	OK (0.28)	OK (0.28)	OK (0.28)

Notes:

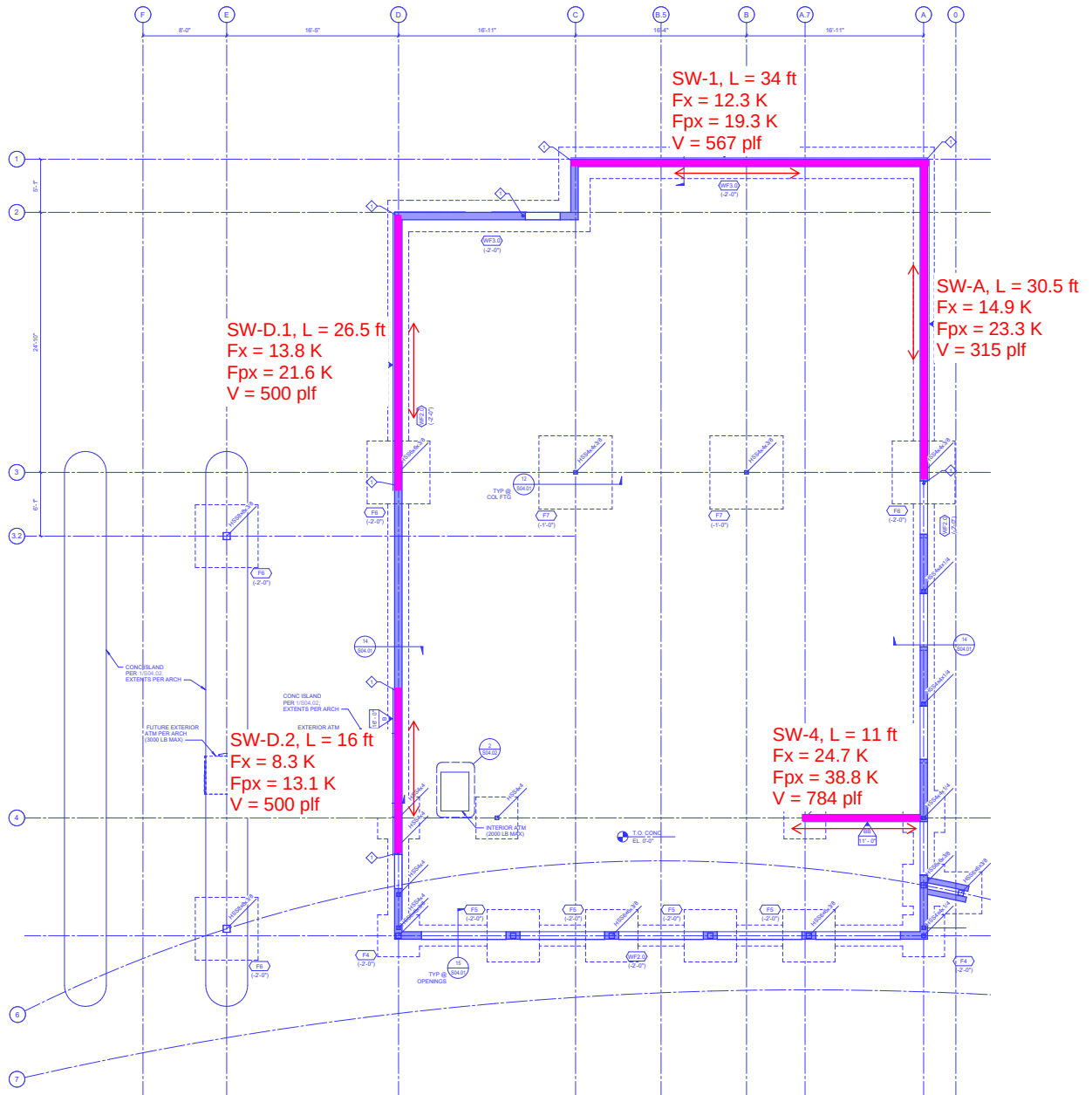
- 1) Values in parentheses are stress ratios.
- 2) Bridging connectors are not designed for back-back, box, or built-up sections.
- 3) Reference www.strongtie.com for latest load data, important information, and general notes
- 4) CFS Designer will not select bridging connectors unless all flexural and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

LATERAL SYSTEM DESIGN

[illegible]

SEISMIC FORCE DISTRIBUTION (LRFD)

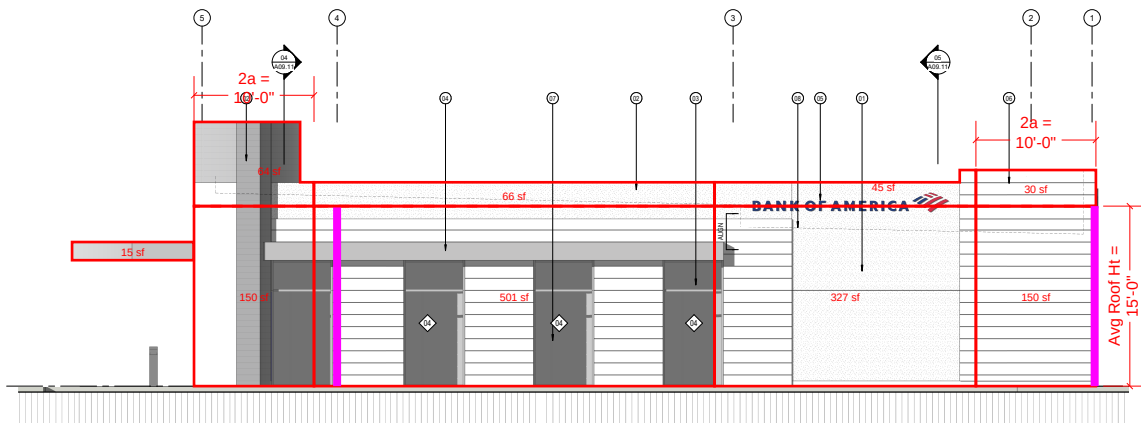
($V = F_{px}/\text{diaphragm length}$)



WIND FORCE DISTRIBUTION (LRFD)

Zone A = 37.6 psf

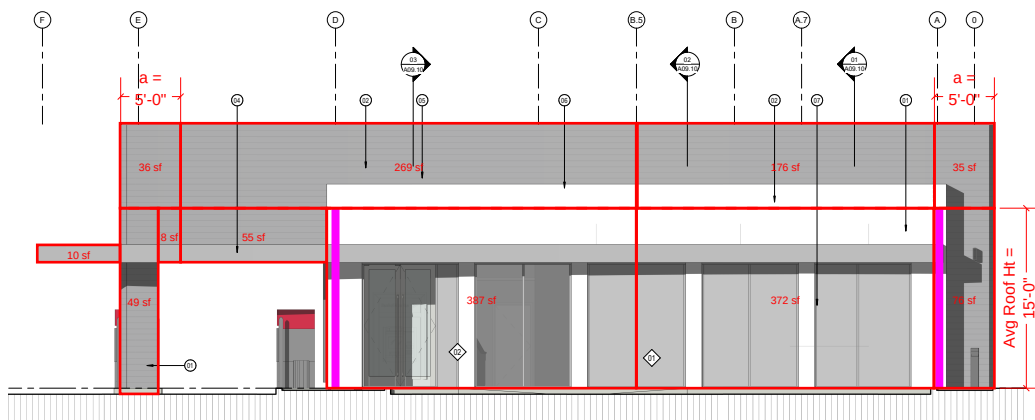
Zone C = 24.9 psf



EAST/WEST

SW 1 = 9.1 K

SW 4 = 13.7 K



NORTH/SOUTH

SW D = 15.8 K

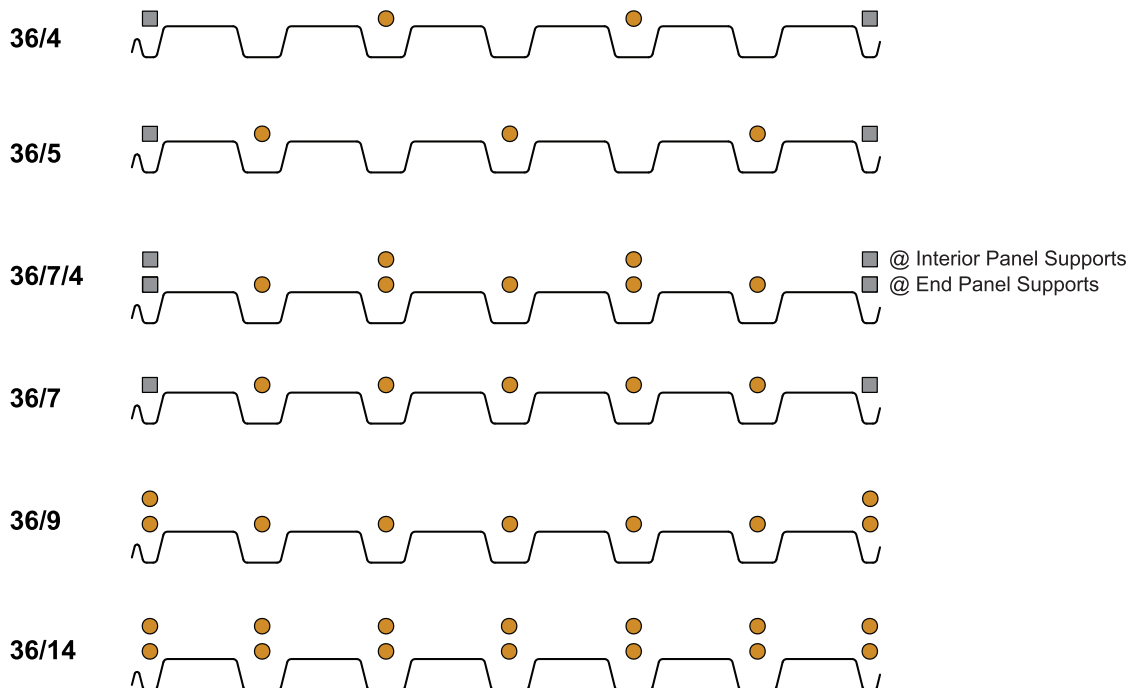
SW A = 11.8 K

Type PLB™-36 or HSB®-36



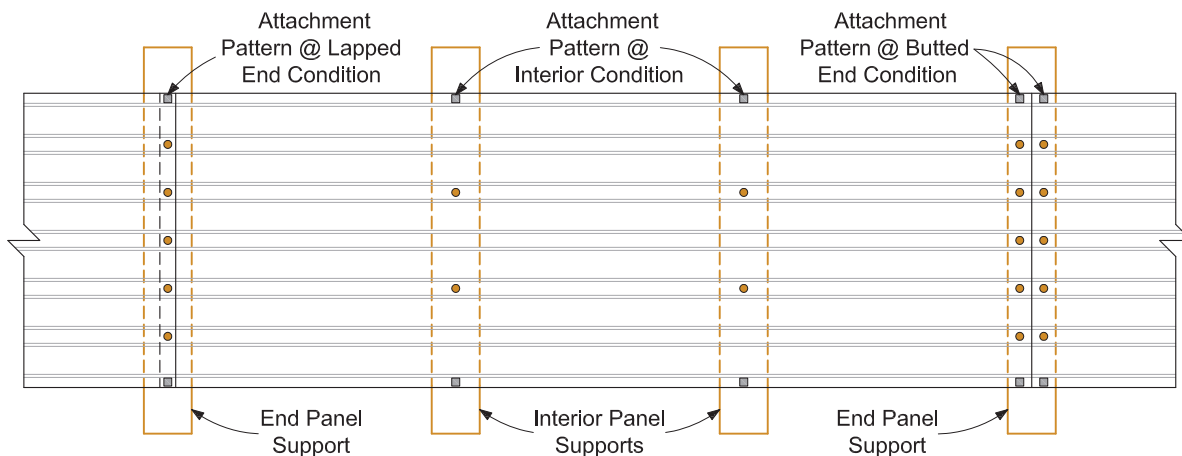
DECK SHEAR CAPACITY

Attachment Patterns to Supports



36/7/4 Attachment Pattern

The 36/7/4 pattern requires a 36/7 attachment pattern at end panel supports and a 36/4 attachment pattern at interior panel supports.



Footnotes for Allowable Uniform Load Tables

1. Stress = Allowable uniform load based on maximum allowable flexural stress in deck.
2. L/360, L/240 or L/180 = Uniform load which produces selected deflection in deck.
3. The symbol ♦♦ indicates allowable uniform load based on deflection exceeds allowable uniform load based on stress.
4. Nominal uniform loads governed by stress may be determined by multiplying the allowable values in the table by $\Omega_b = 1.67$.
LRFD loads may be determined by multiplying nominal loads by $\Phi_b = 0.95$.

Type PLB™ -36

- 36/4 Screw Pattern at Supports
#12 or #14 SDI Recognized Screws
to Supports 0.0385" and thicker



- Sidelaps Connected with PunchLok II Tool

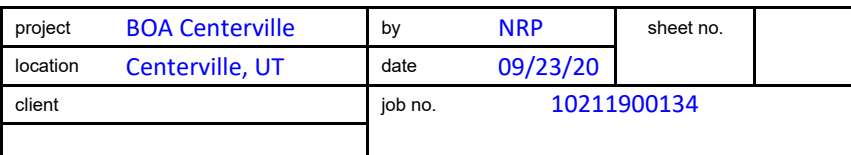
Max Shear, V = 784 plf LRFD = 549 plf ASD < Vallow = 739 plf, ok

Span = 7 ft max

Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10⁶)

DECK GAGE	SIDELAP ATTACHMENT	SPAN (ft-in.)								
		2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
22	VSC2 @ 24"	q 535	548	476	495	444	464	426	445	415
		F -24.1+542R	-12.1+360R	-4.5+269R	-0.9+215R	3+178R	4.5+153R	7.1+133R	7.8+118R	9.8+106R
	VSC2 @ 18"	q 632	548	555	555	508	516	522	489	498
		F -25.2+542R	-12.1+360R	-5.9+270R	-2.2+216R	1.5+179R	3.2+153R	4.5+134R	6.5+119R	7.2+107R
	VSC2 @ 12"	q 632	616	606	598	592	588	585	582	580
		F -25.2+542R	-13.2+361R	-6.9+270R	-3.1+216R	-0.5+180R	1.4+154R	2.8+135R	4+120R	4.9+108R
	VSC2 @ 8"	q 678	681	665	669	658	663	655	659	653
		F -26+543R	-14.5+361R	-8.2+271R	-4.8+217R	-2.2+180R	-0.6+155R	0.9+135R	1.8+120R	2.7+108R
	VSC2 @ 6"	q 701	697	694	692	691	690	689	688	688
		F -26.5+543R	-14.9+362R	-9.1+271R	-5.5+217R	-3.2+181R	-1.5+155R	-0.2+135R	0.8+120R	1.6+108R
	VSC2 @ 4"	q 723	722	721	720	719	719	719	719	718
		F -27.1+543R	-15.7+362R	-10+271R	-6.6+217R	-4.3+181R	-2.7+155R	-1.4+136R	-0.5+121R	0.3+109R
20	VSC2 @ 24"	q 668	689	606	632	572	599	554	578	542
		F -13.4+342R	-5.7+228R	-0.6+170R	1.6+136R	4.3+113R	5.1+96R	7+84R	7.3+75R	8.7+67R
	VSC2 @ 18"	q 781	689	699	702	648	659	667	630	640
		F -14.4+343R	-5.7+228R	-1.9+171R	0.5+136R	3+113R	4+97R	4.8+85R	6.2+75R	6.6+68R
	VSC2 @ 12"	q 781	765	757	749	743	739	736	734	732
		F -14.4+343R	-6.7+228R	-2.7+171R	-0.3+137R	1.4+114R	2.5+97R	3.4+85R	4.2+76R	4.7+68R
	VSC2 @ 8"	q 829	833	818	823	812	817	809	813	807
		F -15.1+343R	-7.8+229R	-3.8+171R	-1.7+137R	0+114R	1+98R	1.9+86R	2.5+76R	3.1+68R
	VSC2 @ 6"	q 853	850	848	846	845	844	843	842	842
		F -15.5+344R	-8.2+229R	-4.5+172R	-2.3+137R	-0.8+114R	0.3+98R	1.1+86R	1.7+76R	2.2+69R
	VSC2 @ 4"	q 875	874	873	872	872	871	871	871	871
		F -16.1+344R	-8.9+229R	-5.3+172R	-3.1+137R	-1.6+115R	-0.6+98R	0.2+86R	0.8+76R	1.3+69R
18	VSC2 @ 24"	q 926	960	856	894	818	854	797	830	784
		F -4.7+167R	-1.2+111R	1.6+83R	2.5+66R	4+55R	4.2+47R	5.2+41R	5.2+37R	5.9+33R
	VSC2 @ 18"	q 1068	960	974	980	916	930	941	897	910
		F -5.7+168R	-1.2+111R	0.6+83R	1.7+67R	3+55R	3.4+47R	3.7+42R	4.5+37R	4.6+33R
	VSC2 @ 12"	q 1068	1052	1043	1036	1030	1026	1023	1021	1019
		F -5.7+168R	-1.9+112R	0+84R	1.2+67R	1.9+56R	2.5+48R	2.9+42R	3.2+37R	3.5+33R
	VSC2 @ 8"	q 1123	1129	1113	1118	1107	1112	1104	1109	1103
		F -6.2+168R	-2.7+112R	-0.7+84R	0.3+67R	1.1+56R	1.5+48R	2+42R	2.3+37R	2.6+34R
	VSC2 @ 6"	q 1149	1146	1144	1143	1141	1141	1140	1140	1139
		F -6.5+168R	-2.9+112R	-1.1+84R	-0.1+67R	0.6+56R	1.2+48R	1.5+42R	1.8+37R	2.1+34R
	VSC2 @ 4"	q 1172	1171	1170	1170	1169	1169	1169	1169	1169
		F -6.8+168R	-3.3+112R	-1.6+84R	-0.5+67R	0.2+56R	0.7+48R	1+42R	1.3+37R	1.6+34R
16	VSC2 @ 24"	q 1182	1227	1104	1152	1061	1106	1037	1078	1022
		F -1.2+95R	0.8+63R	2.7+47R	3.1+38R	4.2+31R	4.2+27R	5+23R	4.9+21R	5.5+19R
	VSC2 @ 18"	q 1352	1227	1246	1254	1179	1197	1210	1158	1174
		F -2+96R	0.8+63R	1.8+47R	2.4+38R	3.4+31R	3.6+27R	3.7+24R	4.3+21R	4.3+19R
	VSC2 @ 12"	q 1352	1336	1327	1319	1313	1309	1306	1303	1301
		F -2+96R	0.2+64R	1.3+48R	2+38R	2.5+32R	2.8+27R	3+24R	3.2+21R	3.4+19R
	VSC2 @ 8"	q 1416	1423	1405	1412	1400	1406	1397	1402	1395
		F -2.5+96R	-0.5+64R	0.7+48R	1.2+38R	1.7+32R	2+27R	2.3+24R	2.4+21R	2.6+19R
	VSC2 @ 6"	q 1445	1442	1440	1438	1437	1437	1436	1436	1435
		F -2.8+96R	-0.7+64R	0.3+48R	1+38R	1.4+32R	1.7+27R	1.9+24R	2.1+21R	2.2+19R
	VSC2 @ 4"	q 1470	1469	1469	1468	1468	1467	1467	1467	1467
		F -3.1+96R	-1.1+64R	0+48R	0.6+38R	1+32R	1.2+27R	1.5+24R	1.6+21R	1.8+19R

See footnotes on page 28.



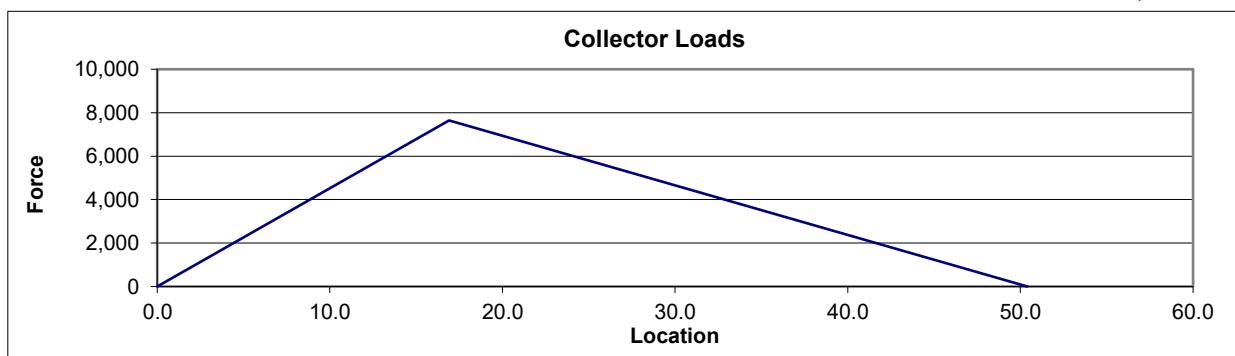
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<u>FLOOR</u>	Roof

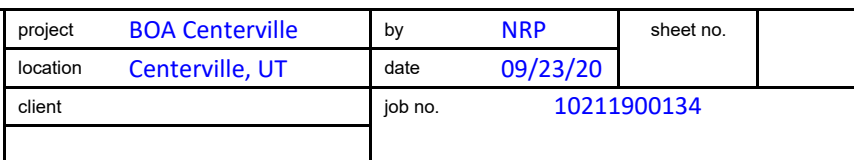
Diaphragm Point Load

P1	=	0.0 k
Dia Scale	=	1.3
Q	=	2.5

[illegible]

Total Length =	50.4	33.5	50.4	0.0	0.0	0.0	ft
Wall/V Shear =		680.6	452.4	0.0	0.0	0.0	plf
Total % =	100.00	OK					
Max Collector =							7,645 #



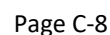


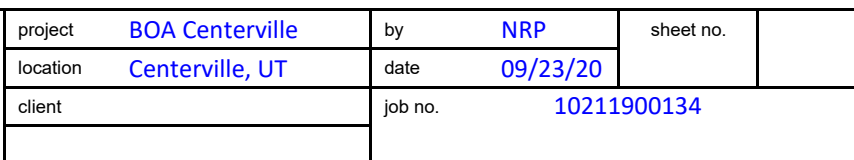
<u>LINE ID</u>	4
<u>FLOOR</u>	Roof

V1	=	29.80	k
V2	=	0.00	k
V3	=	0.00	k
V4	=	0.00	
Total	=	29.8	k

P1	=	0.0 k
Dia Factor	=	1.3
Q	=	2.5

Total Length =	51.0	12.5	51.0	0.0	0.0	0.0	ft
Wall/V Shear =		2384.0	584.3	0.0	0.0	0.0	plf
Total % =	100.00	OK					
Max Collector =							22,496 #



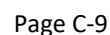


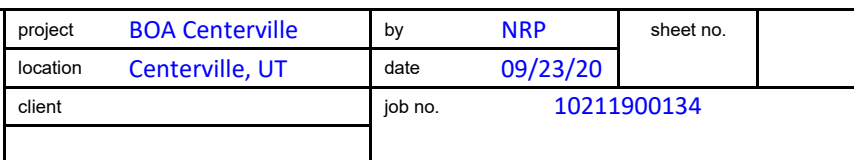
<u>LINE ID</u>	A
<u>FLOOR</u>	Roof

Diaphragm Point Load

P1	=	0.0 k
Dia Factor	=	1.3
Q	=	2.5

Total Length =	73.8	31.0	73.8	0.0	0.0	0.0	ft
Wall/V Shear =		696.8	292.7	0.0	0.0	0.0	plf
	Total % = 100.00 OK			Max Collector =			12,527 #



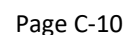


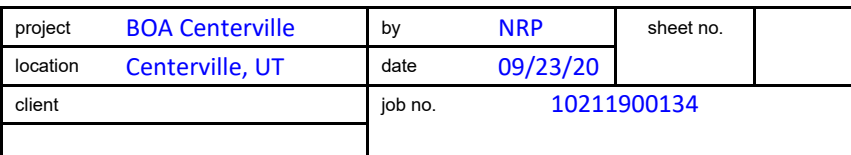
<u>LINE ID</u>	D
<u>FLOOR</u>	Roof

Diaphragm Point Load

P1	=	0.0 k
Dia Factor	=	1.3
Q	=	2.5

Total Length =	69.4	42.5	69.4	0.0	0.0	0.0	ft
Wall/V Shear =		585.9	358.8	0.0	0.0	0.0	plf
Total % =	100.00 OK			Max Collector =			6.018 #





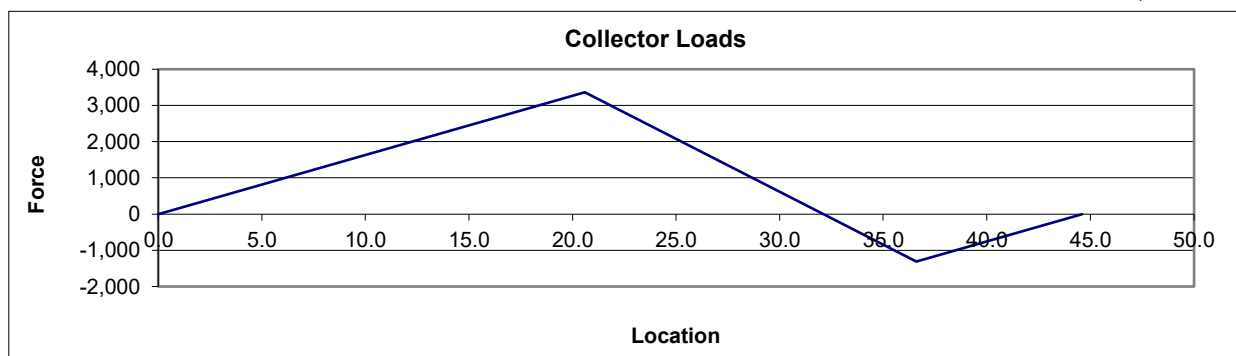
<u>LINE ID</u>	D
<u>FLOOR</u>	Canopy

Diaphragm Point Load

P1	=	0.0 k
Dia Factor	=	1.3
Q	=	2.5

[illegible]

Total Length =	44.6	16.0	44.6	0.0	0.0	0.0	ft
Wall/V Shear =		456.3	163.7	0.0	0.0	0.0	plf
Total % =	100.00 OK						Max Collector = 3,372 #



METAL STUD SHEAR WALL CAPACITY

Table B5.2.2.3-2

Unit Nominal Shear Strength [Resistance] (v_n)^{1, 2}

For Shear Walls with Wood Structural Panel Sheathing on One Side of Wall

United States and Mexico (Pounds Per Foot)						
Sheathing	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (inches)				Designation Thickness of Stud, Track and Blocking (mils)
		6/12	4/12	3/12	2/12	
15/32" structural 1 (4-ply)	2:1	1065	1410	1735	1910	43 (min.)
7/16" rated sheathing (OSB)	2:1	910	1410	1735	1910	33 (min.)
7/16" rated sheathing (OSB) oriented perpendicular to framing	2:1	1020	-	-	-	33 (min.)
7/16" rated sheathing (OSB)	4:1 ³	-	1025	1425	1825	33 (min.)

B5.2.2.3.6 Combined Systems

For a *Type I shear wall* sheathed with the same material and fastener spacing on opposite faces of the same wall, the *nominal strength [resistance]* of the complete wall based on Tables B5.2.2.3-1 through B5.2.2.3-4 shall be determined by adding the *nominal strength [resistance]* from the two opposite faces together.

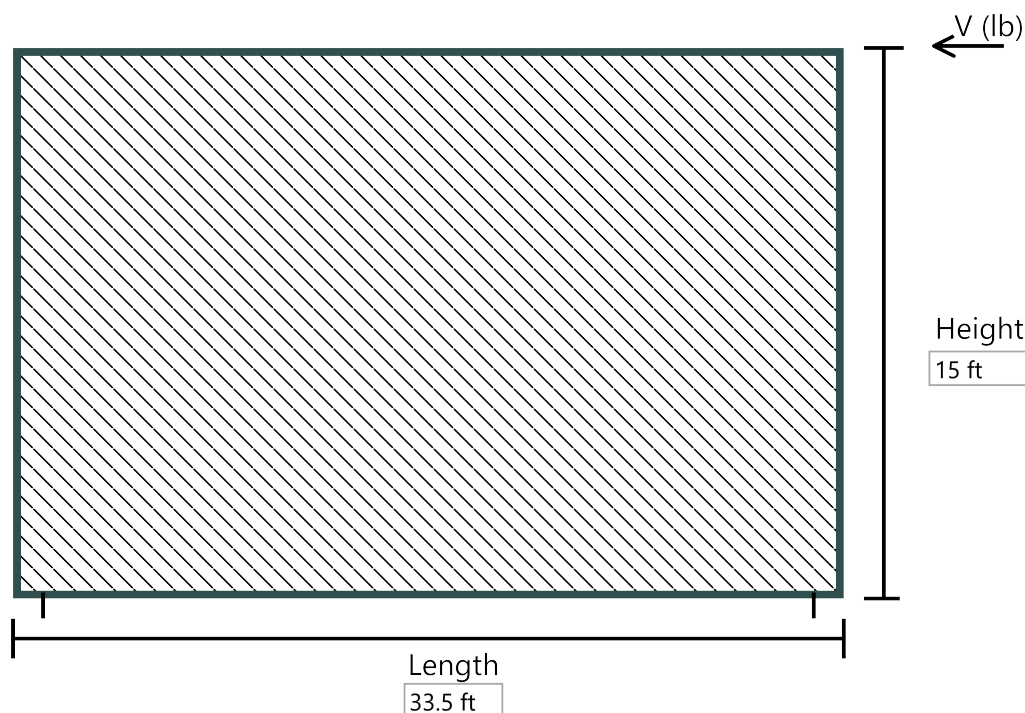
B5.2.3 Available Strength [Factored Resistance]

The *available strength [factored resistance]* ($\phi_v V_n$ for LRFD and LSD or V_n/Ω_v for ASD) shall be determined from the *nominal strength [resistance]* using the applicable *safety factors* (Ω) or *resistance factors* (ϕ) given in this section in accordance with the applicable design method – ASD, LRFD, or LSD as follows:

$$\Omega_v = 2.00 \text{ for ASD}$$

$$\phi_v = 0.65 \text{ for LRFD}$$

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's Stacked Shear Wall design module.

**Chord Data**

Chords: 800S162-54 (50) Back-To-Back
 2012 NASPEC [AISI S100-2012]
 Chord Fastener Spacing, $a =$ 12 in
 Shearwall Chord Force = 3884 lb
 Includes Anchor Offset = 3.0 in
 Additional Axial Loads = 0 lb
 Total Axial Loads = 3884 lb
 $K_y L_y$, $K_t L_t$ for Axial Capacity = 48 in
 Maximum $KL/r =$ 78
 Allowable Axial Load = 12349 lb
 Input Chord Moment = 0 Ft-Lb
 Flexural Bracing = 48 in
 Distortional Buckling Inputs for Moment and Axial
 $K-\phi =$ 0 lb-in/in
 $L_m =$ None
 Allowable Moment = 5469 Ft-Lb
 Chord Interaction = 0.315

Overturning Uplift Data

Anchor offset Each End = 3.0 in
 Uplift at Anchor - Wind = 2463 lb
 Uplift at Anchor - Seismic = 3884 lb

Load Data (Factored ASD)

$V(\text{wind}) =$ 5460 lb
 $V(\text{seismic}) =$ 8610 lb

Sheathing Data

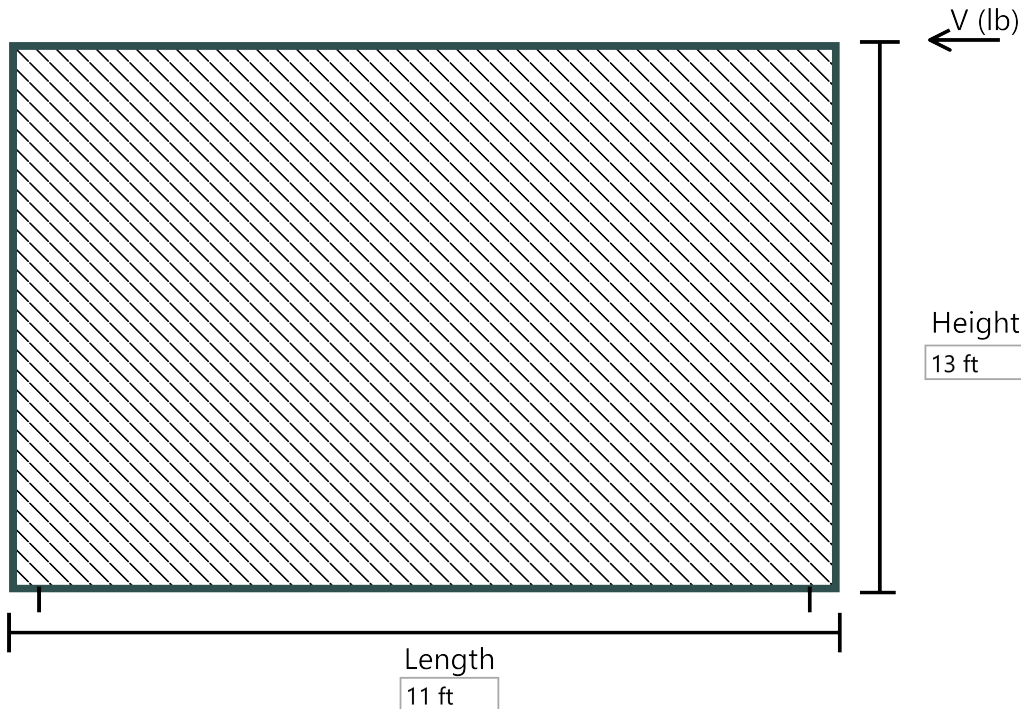
Shear values per IBC 2018 (AISI S240-15)
 See AISI S240-15 for additional information

Stud Thickness = 54 mils
 Sheathing: 7/16 Rated Sheathing (OSB) 1 side
 Fasteners: 6-inches oc edges, 12-inches oc field **Type A**

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio = 0.45:1
 Allowable Aspect Ratio for Seismic = 2.00:1
 Allowable Aspect Ratio for Wind = 2.00:1
 Unit Shear (Wind) = 163 lb/ft
 Allowable Unit Shear (Wind) = 455 lb/ft
 Unit Shear (Seismic) = 257 lb/ft
 Allowable Unit Shear (Seismic) = 455 lb/ft

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's Stacked Shear Wall design module.

**Chord Data**

Chords: 600S162-97 (50) Boxed
 2012 NASPEC [AISI S100-2012]
 Chord Fastener Spacing, a = 12 in
 Shearwall Chord Force = 21765 lb
 Includes Anchor Offset = 8 in
 Additional Axial Loads = 0 lb
 Total Axial Loads = 21765 lb
 KyLy, KtLt for Axial Capacity = 48 in
 Maximum KL/r = 70
 Allowable Axial Load = 27541 lb
 Input Chord Moment = 0 Ft-Lb
 Flexural Bracing = 48 in
 Allowable Moment = 7980 Ft-Lb
 Chord Interaction = 0.790

Overturning Uplift Data

Anchor offset Each End = 8 in
 Uplift at Anchor - Wind = 10341 lb
 Uplift at Anchor - Seismic = 21765 lb

Load Data (Factored ASD)

V(wind) = 8220 lb
 V(seismic) = 17300 lb

Sheathing Data

Shear values per IBC 2018 (AISI S240-15)
 See AISI S240-15 for additional information

Stud Thickness = 54 mils
 Sheathing: 7/16 Rated Sheathing (OSB) 1 side
 Fasteners: 3-inches oc edges, 12-inches oc field

Type CC (2 sides)

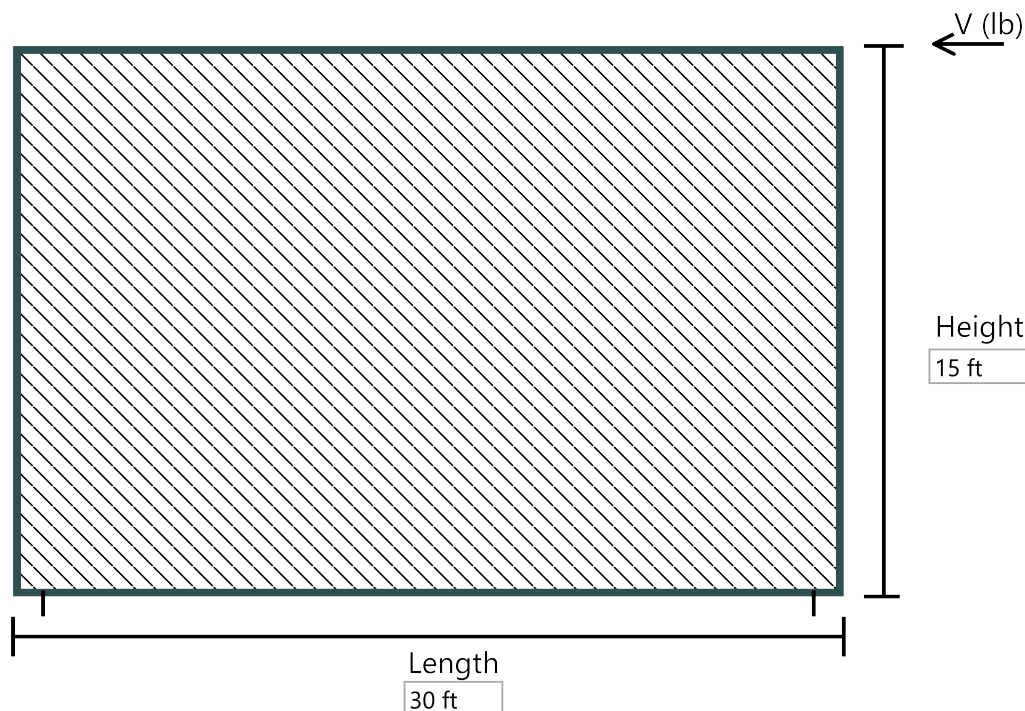
Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio = 1.18:1
 Allowable Aspect Ratio for Seismic = 2.00:1
 Allowable Aspect Ratio for Wind = 2.00:1
 Unit Shear (Wind) = 747 lb/ft
 Allowable Unit Shear (Wind) = 868 lb/ft
 Unit Shear (Seismic) = 1573 lb/ft
 Allowable Unit Shear (Seismic) = 868 lb/ft

Provide sheathing on (2) sides:
 Vallow = 1736 lb/ft, ok

Warning - chord thickness does not match 'Framing Thickness'
 97 MIL, OK

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's Stacked Shear Wall design module.

**Chord Data**

Chords: 800S162-54 (50) Back-To-Back
 2012 NASPEC [AISI S100-2012]
 Chord Fastener Spacing, a = 12 in
 Shearwall Chord Force = 5259 lb
 Includes Anchor Offset = 3.0 in
 Additional Axial Loads = 0 lb
 Total Axial Loads = 5259 lb
 $K_y L_y$, $K_t L_t$ for Axial Capacity = 48 in
 Maximum KL/r = 78
 Allowable Axial Load = 12349 lb
 Input Chord Moment = 0 Ft-Lb
 Flexural Bracing = 48 in
 Distortional Buckling Inputs for Moment and Axial
 $K\phi$ = 0 lb-in/in
 L_m = None
 Allowable Moment = 5469 Ft-Lb
 Chord Interaction = 0.426

Overturning Uplift Data

Anchor offset Each End = 3.0 in
 Uplift at Anchor - Wind = 3570 lb
 Uplift at Anchor - Seismic = 5259 lb

Load Data (Factored ASD)

$V(\text{wind})$ = 7080 lb
 $V(\text{seismic})$ = 10430 lb

Sheathing Data

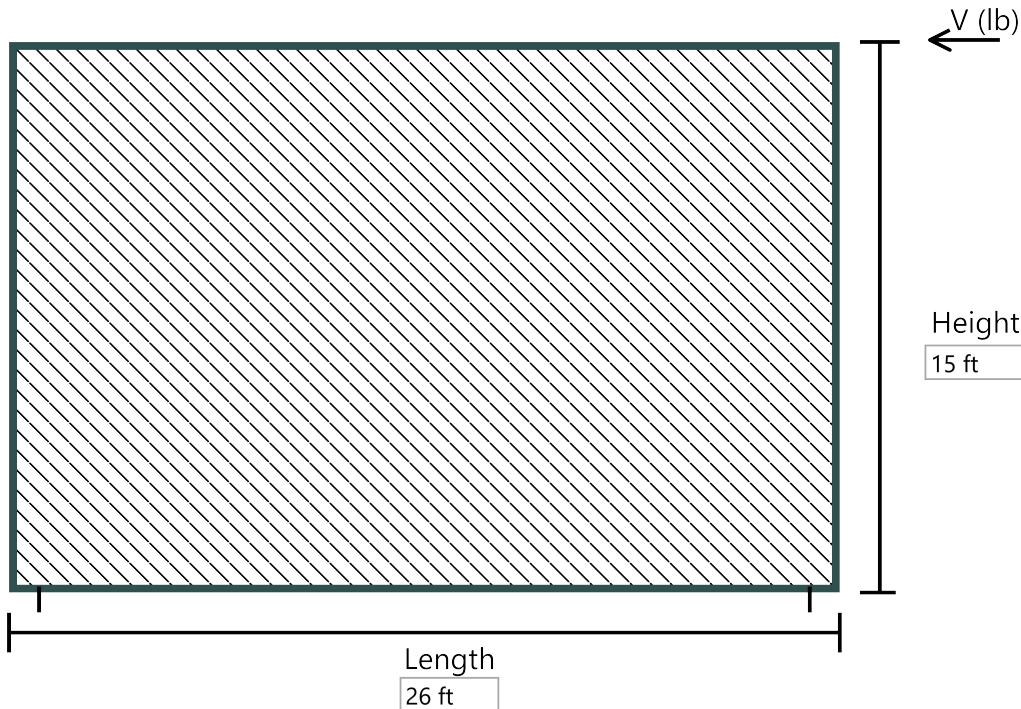
Shear values per IBC 2018 (AISI S240-15)
 See AISI S240-15 for additional information

Stud Thickness = 54 mils
 Sheathing: 7/16 Rated Sheathing (OSB) 1 side
 Fasteners: 6-inches oc edges, 12-inches oc field **Type A**

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio = 0.50:1
 Allowable Aspect Ratio for Seismic = 2.00:1
 Allowable Aspect Ratio for Wind = 2.00:1
 Unit Shear (Wind) = 236 lb/ft
 Allowable Unit Shear (Wind) = 455 lb/ft
 Unit Shear (Seismic) = 348 lb/ft
 Allowable Unit Shear (Seismic) = 455 lb/ft

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's Stacked Shear Wall design module.

**Chord Data**

Chords: 800S162-54 (50) Back-To-Back
 2012 NASPEC [AISI S100-2012]
 Chord Fastener Spacing, a = 12 in
 Shearwall Chord Force = 5627 lb
 Includes Anchor Offset = 3.0 in
 Additional Axial Loads = 0 lb
 Total Axial Loads = 5627 lb
 KyLy, KtLt for Axial Capacity = 48 in
 Maximum KL/r = 78
 Allowable Axial Load = 12349 lb
 Input Chord Moment = 0 Ft-Lb
 Flexural Bracing = 48 in
 Distortional Buckling Inputs for Moment and Axial
 K-phi = 0 lb-in/in
 Lm = None
 Allowable Moment = 5469 Ft-Lb
 Chord Interaction = 0.456

Overturning Uplift Data

Anchor offset Each End = 3.0 in
 Uplift at Anchor - Wind = 2878 lb
 Uplift at Anchor - Seismic = 5627 lb

Load Data (Factored ASD)

V(wind) = 4940 lb
 V(seismic) = 9660 lb

Sheathing Data

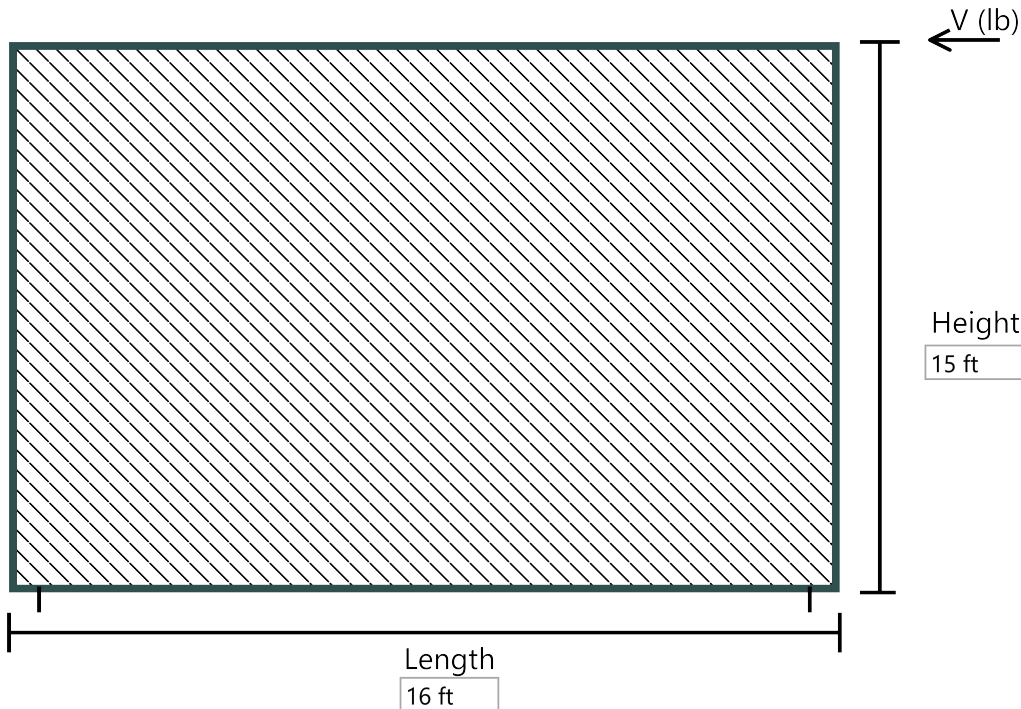
Shear values per IBC 2018 (AISI S240-15)
 See AISI S240-15 for additional information

Stud Thickness = 54 mils
 Sheathing: 7/16 Rated Sheathing (OSB) 1 side
 Fasteners: 6-inches oc edges, 12-inches oc field **Type A**

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio = 0.58:1
 Allowable Aspect Ratio for Seismic = 2.00:1
 Allowable Aspect Ratio for Wind = 2.00:1
 Unit Shear (Wind) = 190 lb/ft
 Allowable Unit Shear (Wind) = 455 lb/ft
 Unit Shear (Seismic) = 372 lb/ft
 Allowable Unit Shear (Seismic) = 455 lb/ft

Simple Shear Wall design is intended for use with element and component design or other designs where special seismic considerations are not required. For design of building LFRS shear walls, use CFS Designer's Stacked Shear Wall design module.

**Chord Data**

Chords: 800S162-54 (50) Back-To-Back
 2012 NASPEC [AISI S100-2012]
 Chord Fastener Spacing, a = 12 in
 Shearwall Chord Force = 5533 lb
 Includes Anchor Offset = 3.0 in
 Additional Axial Loads = 0 lb
 Total Axial Loads = 5533 lb
 $K_y L_y$, $K_t L_t$ for Axial Capacity = 48 in
 Maximum KL/r = 78
 Allowable Axial Load = 12349 lb
 Input Chord Moment = 0 Ft-Lb
 Flexural Bracing = 48 in
 Distortional Buckling Inputs for Moment and Axial
 $K\phi$ = 0 lb-in/in
 L_m = None
 Allowable Moment = 5469 Ft-Lb
 Chord Interaction = 0.448

Overturning Uplift Data

Anchor offset Each End = 3.0 in
 Uplift at Anchor - Wind = 3371 lb
 Uplift at Anchor - Seismic = 5533 lb

Load Data (Factored ASD)

$V(\text{wind})$ = 3540 lb
 $V(\text{seismic})$ = 5810 lb

Sheathing Data

Shear values per IBC 2018 (AISI S240-15)
 See AISI S240-15 for additional information

Stud Thickness = 54 mils
 Sheathing: 7/16 Rated Sheathing (OSB) 1 side
 Fasteners: 6-inches oc edges, 12-inches oc field **Type A**

Screws attaching panels to CFS steel framing shall comply with ASTM C1513. Also, for framing members that are 54 mil and thinner, use minimum #8. For 68 mil and thicker, use min. #10.

Aspect Ratio = 0.94:1
 Allowable Aspect Ratio for Seismic = 2.00:1
 Allowable Aspect Ratio for Wind = 2.00:1
 Unit Shear (Wind) = 221 lb/ft
 Allowable Unit Shear (Wind) = 455 lb/ft
 Unit Shear (Seismic) = 363 lb/ft
 Allowable Unit Shear (Seismic) = 455 lb/ft

HOLDOWN CAPACITIES

TABLE 2 – TENSION LOADS AND DISPLACEMENTS FOR S/HDU SERIES HOLD-DOWNS

Model	Height (in.)	Fasteners		Framing Member(s) ⁵ No.-mil (ga)	ASD		LRFD		Nominal Tension Load ^{7,8} (lbs)
		Anchor Bolt Dia ^{1,2} (in)	Framing Fasteners ⁴		Tension Load (lbs)	Displacement at ASD Load ⁶ (in)	Tension Load (lbs)	Displacement at LRFD Load ⁶ (in)	
S/HDU4	7 ⁷ / ₈	5 ⁵ / ₈	6-#14	2-33 (2-20ga)	2320	0.093	3705	0.149	5685
				2-43 (2-18ga)	3825	0.115	6105	0.190	9365
				2-54 (2-16ga)	3970	0.093	6345	0.156	9730
				Steel Fixture	4470	0.063	7165	0.103	12120
S/HDU6	10 ³ / ₈	5 ⁵ / ₈	12-#14	2-33 (2-20ga)	4895	0.125	8495	0.250	10470
				2-43 (2-18ga)	6125	0.119	9690	0.250	15460
				2-54 (2-16ga)	6125	0.108	9785	0.234	15005
				Steel Fixture	5995	0.060	9580	0.136	14695
S/HDU9	12 ⁷ / ₈	7 ⁷ / ₈	18-#14	2-33 (2-20ga)	6965	0.103	11125	0.189	13165
				2-43 (2-18ga)	9255	0.125	15485	0.250	21810
				2-54 (2-16ga)	9990	0.106	15960	0.225	24480
				Steel Fixture	12715	0.125	20510	0.177	31455
S/HDU11	16 ⁵ / ₈	7 ³ / ₈	27-#14	2-33 (2-20ga)	6965	0.103	11125	0.189	13165

For SI: 1 inch = 25.4 mm, 1 lb = 4.45 N.

- The Designer shall specify the foundation anchor material type, embedment and configuration. Some of the tabulated hold-down tension loads exceed the tension strength of typical ASTM F1554 Grade 36 or A307 anchor bolts.
- A foundation anchor bolt washer is not required.
- A heavy hex nut for the anchor bolt is required to achieve the tabulated loads for S/HDU11.
- 1/4-inch self-tapping screws may be substituted for #14 self-tapping screws.
- The Designer shall specify and detail the connection of the back-to-back full height framing members.
- Hold-down displacement at tabulated ASD and LRFD loads is the difference in the displacement measured between the anchor bolt and back of the hold-down that's attached to the framing member(s) when loaded to the ASD and LRFD static test load, respectively. Deflection fastener slip, hold-down elongation and anchor bolt elongation (L=4").
- The Nominal Tension Load is the average ultimate (peak) load from tests in accordance with AISI S100 Chapter F. When hold-downs are used in CFS framed shear walls or diagonal strap braced walls with an R-coefficient greater than 3, the AISI S213 Lateral Design Section C5 requires hold-downs in shear walls have the nominal strength to resist the lesser of the amplified seismic load or the load the system can deliver and hold-downs in diagonal strap braced walls have the nominal strength to resist the lesser of the amplified seismic load or the expected yield strength of the diagonal strap bracing member.
- When used in lateral force resisting systems, hold-downs shall be designed for the expected strength of designated seismic force resisting systems as specified in Section B3 and Chapter E of AISI S400.

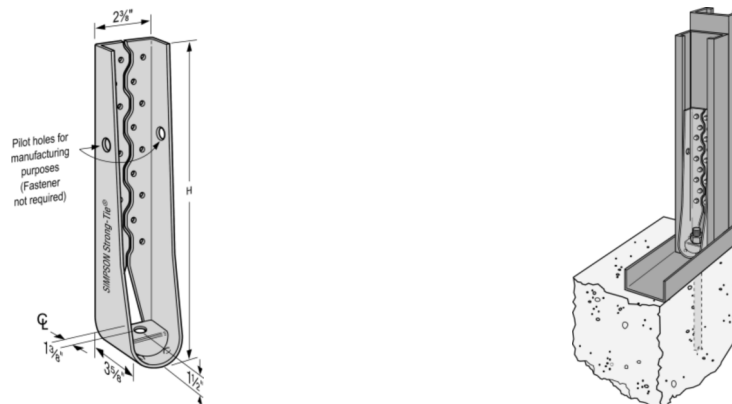


FIGURE 2 – S/HDU HOLD-DOWN

BOT TRACK ANCHORS

www.hilti.com

Company:
Address:
Phone | Fax:
Design:
Fastening point:

Page:
Specifier:
E-Mail:
Date:

1

4/1/2020

Specifier's comments:

1 Input data

Anchor type and diameter:

Kwik Bolt TZ - CS 1/2 (3 1/4)

Item number:

not available

Effective embedment depth:

$h_{ef,act} = 3.250$ in., $h_{nom} = 3.625$ in.

Material:

Carbon Steel

Evaluation Service Report:

ESR-1917

Issued | Valid:

5/1/2019 | 5/1/2021

Proof:

Design Method ACI 318-14 / Mech

Stand-off installation:

Profile:

Base material:

cracked concrete, 4000, $f'_c = 4,000$ psi; $h = 12.000$ in.

Installation:

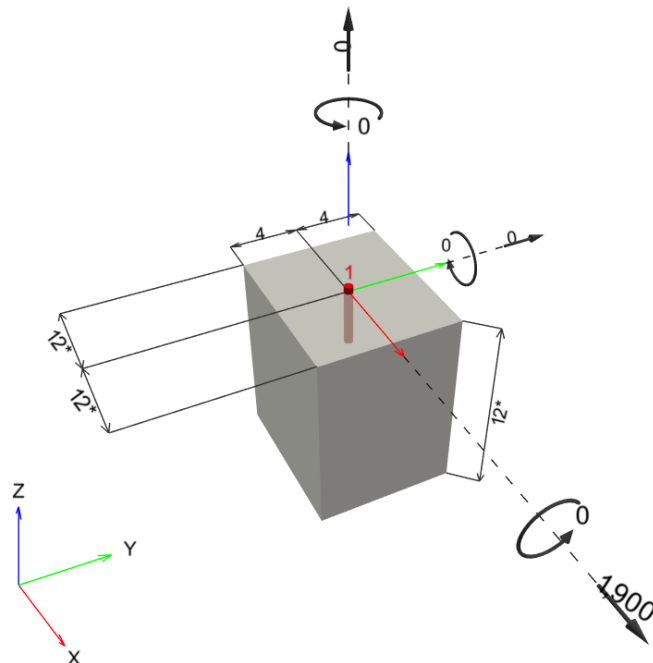
hammer drilled hole, Installation condition: Dry

Reinforcement:

tension: condition B, shear: condition B; no supplemental splitting reinforcement present
edge reinforcement: none or < No. 4 bar



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



www.hilti.com

Company:	Page:	2
Address:	Specifier:	
Phone Fax:	E-Mail:	
Design:	Date:	4/1/2020
Fastening point:		

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V _x = 1,900; V _y = 0; M _x = 0; M _y = 0; M _z = 0;	no	99

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb]		Utilization	
		Load	Capacity	β_N / β_V [%]	Status
Tension	-	-	-	- / -	N/A
Shear	Concrete edge failure in direction x+	1,900	1,923	- / 99	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	-	-	-	-	N/A

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

(1) S/HDU9 ANCHOR



Anchor Designer™
Software
Version 2.8.7094.0

Company:		Date:	3/30/2020
Engineer:		Page:	1/6
Project:	Bank of America		
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: AB
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
 h_{min} (inch): 10.38
 C_{min} (inch): 5.25
 S_{min} (inch): 5.25

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
 $\Psi_{c,v}$: 1.2
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Recommended Anchor

Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB7 (7/8"Ø)



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com



Company:		Date:	3/30/2020
Engineer:		Page:	2/6
Project:	Bank of America		
Address:			
Phone:			
E-mail:			

Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: Not applicable

Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied

Ductility section for shear: 17.2.3.5.3 (a) is satisfied

Ω_0 factor: not set

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

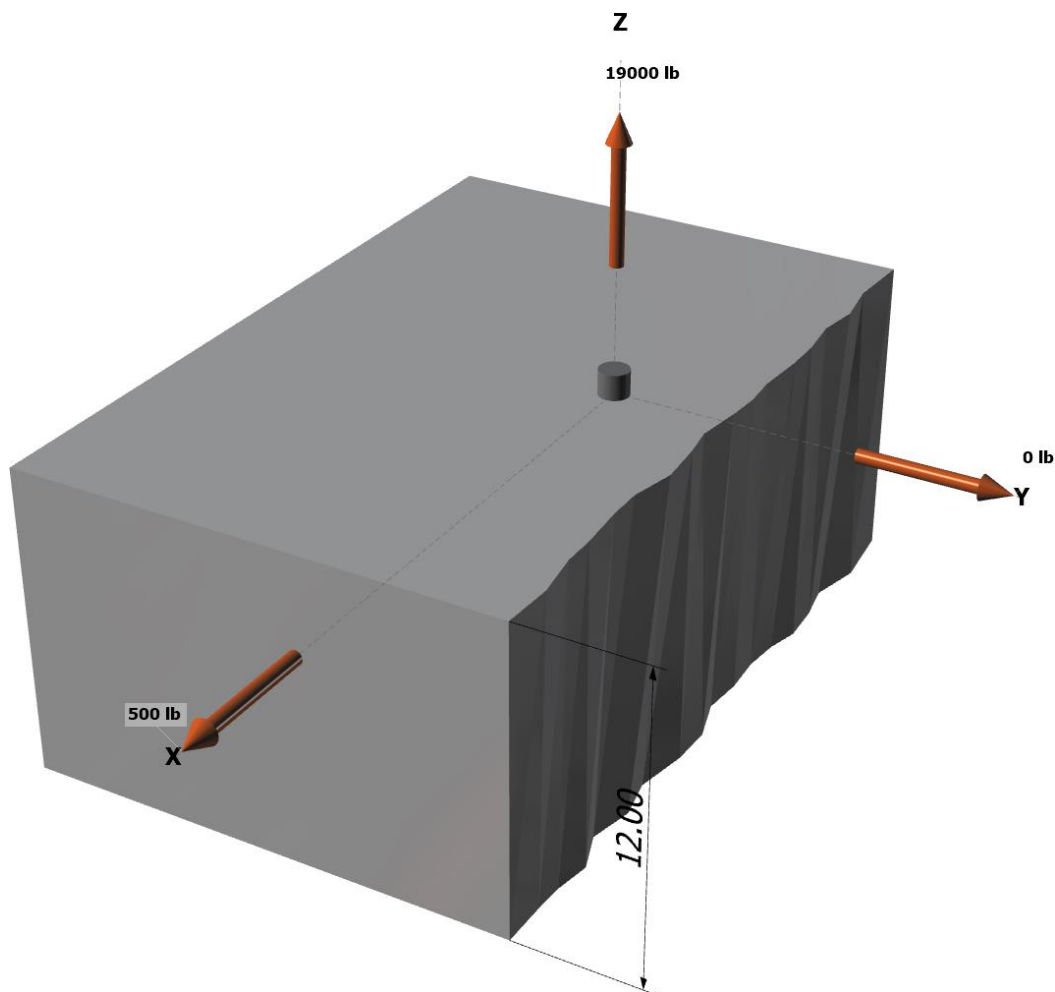
Strength level loads:

N_{ua} [lb]: 19000

V_{uax} [lb]: 500

V_{uay} [lb]: 0

<Figure 1>



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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<Figure 2>





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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)
1	19000.0	500.0	0.0	500.0
Sum	19000.0	500.0	0.0	500.0

Maximum concrete compression strain (%): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 19000
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
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26795	0.75	20096

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	8.000	34346

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

A_{Nc} (in ²)	A_{Nco} (in ²)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75 \phi N_{cb}$ (lb)
605.25	576.00	12.00	1.000	1.00	1.000	34346	0.70	18947

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

$$0.75 \phi N_{pn} = 0.75 \phi \Psi_{c,P} N_p = 0.75 \phi \Psi_{c,P} 8 A_{brg} f'_c \text{ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)}$$

$\Psi_{c,P}$	A_{brg} (in ²)	f'_c (psi)	ϕ	$0.75 \phi N_{pn}$ (lb)
1.0	4.07	4000	0.70	68292



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8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
16080	1.0	0.65	10452

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$$V_{bx} = \min[7(l_e/d_a)^{0.2}\sqrt{d_a\lambda_a}\sqrt{f'_c}C_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}C_{a1}^{1.5}] \text{ (Eq. 17.5.2.2a \& Eq. 17.5.2.2b)}$$

l_e (in)	d_a (in)	λ_a	f'_c (psi)	C_{a1} (in)	V_{bx} (lb)
7.00	0.875	1.00	4000	12.00	23662

$$\phi V_{cbx} = \phi(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx} \text{ (Sec. 17.3.1 \& Eq. 17.5.2.1a)}$$

A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
360.00	648.00	0.900	1.200	1.225	23662	0.70	12171

Shear parallel to edge in x-direction:

$$V_{by} = \min[7(l_e/d_a)^{0.2}\sqrt{d_a\lambda_a}\sqrt{f'_c}C_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}C_{a1}^{1.5}] \text{ (Eq. 17.5.2.2a \& Eq. 17.5.2.2b)}$$

l_e (in)	d_a (in)	λ_a	f'_c (psi)	C_{a1} (in)	V_{by} (lb)
7.00	0.875	1.00	4000	8.00	12880

$$\phi V_{cbx} = \phi(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by} \text{ (Sec. 17.3.1, 17.5.2.1(c) \& Eq. 17.5.2.1a)}$$

A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
288.00	288.00	1.000	1.200	1.000	12880	0.70	21638

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b \text{ (Sec. 17.3.1 \& Eq. 17.5.3.1a)}$$

k_{cp}	A_{Nc} (in ²)	A_{Nco} (in ²)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	605.25	576.00	1.000	1.000	1.000	34346	0.70	50526

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status	
Steel	19000	20096	0.95	Pass	
Concrete breakout	19000	18947	1.00	Pass (Governs)	
Pullout	19000	68292	0.28	Pass	
Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status	
Steel	500	10452	0.05	Pass (Governs)	
T Concrete breakout x+	500	12171	0.04	Pass	
Concrete breakout y-	500	21638	0.02	Pass	
Pryout	500	50526	0.01	Pass	
Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	1.00	0.00	100.3%	1.0	Pass

PAB7 (7/8"Ø) with hef = 8.000 inch meets the selected design criteria.

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	19000	32154	59.1%	Governs
Concrete	Factored Load, N_{ua} (lb)	Nominal Strength, N_n (lb)	Ratio	
Concrete breakout	19000	36090	52.6%	
Pullout	19000	130080	14.6%	

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) satisfied since steel ratio governs and the steel element is ductile.

12. Warnings

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.

(2) S/HDU9 ANCHORS



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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: AB
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
 h_{min} (inch): 14.38
 C_{min} (inch): 5.25
 S_{min} (inch): 5.25

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 16.00
State: Cracked
Compressive strength, f'_c (psi): 4000
 $\Psi_{c,v}$: 1.2
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate

Length x Width x Thickness (inch): 8.00 x 10.00 x 0.25

Recommended Anchor

Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB7 (7/8"Ø)



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: Yes

Anchors subjected to sustained tension: Not applicable

Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied

Ductility section for shear: 17.2.3.5.3 (a) is satisfied

Ω_0 factor: not set

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 39000

V_{uax} [lb]: 500

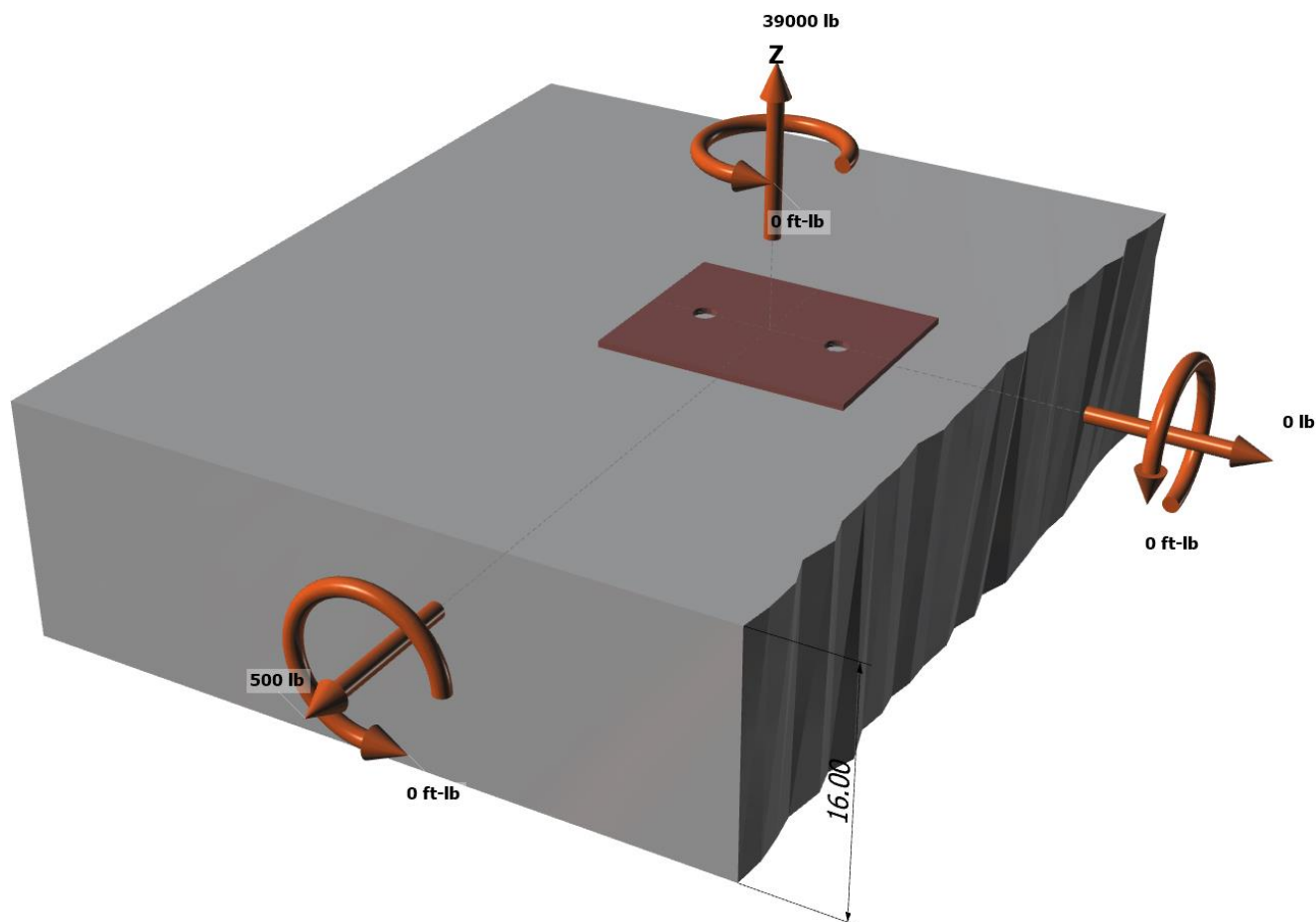
V_{uay} [lb]: 0

M_{ux} [ft-lb]: 0

M_{uy} [ft-lb]: 0

M_{uz} [ft-lb]: 0

<Figure 1>



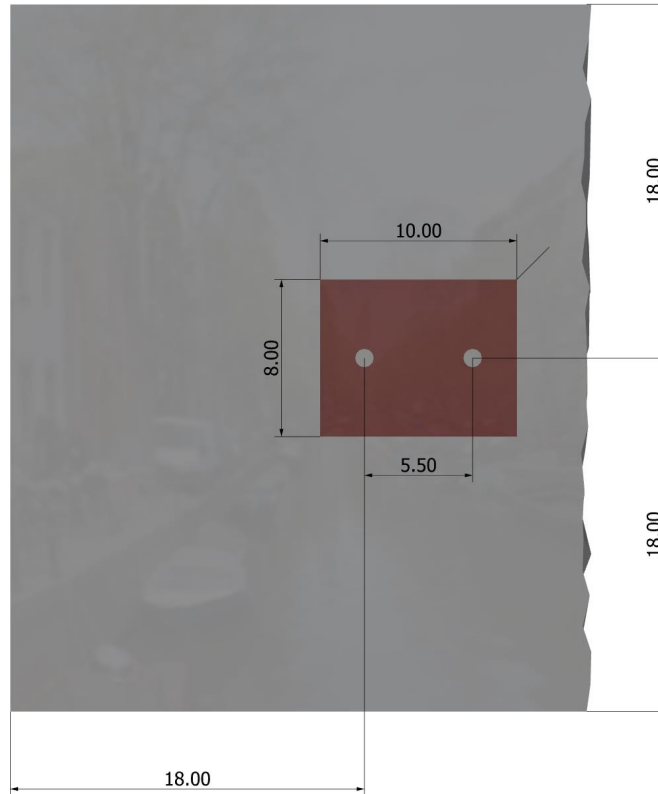
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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<Figure 2>

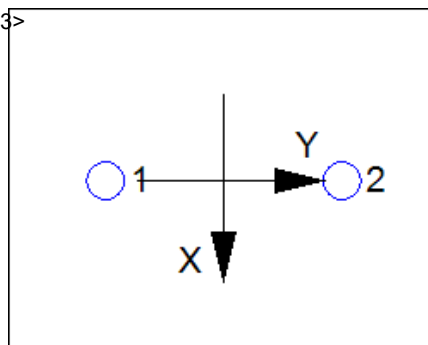


3. Resulting Anchor Forces

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	19500.0	250.0	0.0	250.0
2	19500.0	250.0	0.0	250.0
Sum	39000.0	500.0	0.0	500.0

Maximum concrete compression strain (%): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 39000
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
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



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

N _{sa} (lb)	φ	φN _{sa} (lb)
26795	0.75	20096

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

$$N_b = 16\lambda_a \sqrt{f_c} h_{ef}^{5/3} \text{ (Eq. 17.4.2.2b)}$$

λ _a	f _c (psi)	h _{ef} (in)	N _b (lb)
1.00	4000	12.000	63648

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

A _{Nc} (in ²)	A _{Nco} (in ²)	c _{a,min} (in)	Ψ _{ec,N}	Ψ _{ed,N}	Ψ _{c,N}	Ψ _{cp,N}	N _b (lb)	φ	0.75φN _{cbg} (lb)
1537.88	1296.00	18.00	1.000	1.000	1.00	1.000	63648	0.70	39652

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

$$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8A_{brg} f_c \text{ (Sec. 17.3.1, Eq. 17.4.3.1 \& 17.4.3.4)}$$

Ψ _{c,P}	A _{brg} (in ²)	f _c (psi)	φ	0.75φN _{pn} (lb)
1.0	4.07	4000	0.70	68292



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8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
16080	1.0	0.65	10452

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min[7(l_e/d_a)^{0.2}\sqrt{d_a\lambda_a}\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}]$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
7.00	0.875	1.00	4000	18.00	43469

$\phi V_{cbgx} = \phi (A_{Vc} / A_{Vco}) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
808.00	1458.00	1.000	0.900	1.200	1.299	43469	0.70	23658

Shear parallel to edge in x-direction:

$V_{by} = \min[7(l_e/d_a)^{0.2}\sqrt{d_a\lambda_a}\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}]$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
7.00	0.875	1.00	4000	12.00	23662

$\phi V_{cbx} = \phi (2)(A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in ²)	A_{Vco} (in ²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
576.00	648.00	1.000	1.200	1.061	23662	0.70	37478

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cbg} = \phi k_{cp} (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in ²)	A_{Nco} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	1537.88	1296.00	1.000	1.000	1.000	1.000	63648	0.70	105738

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N _{ua} (lb)	Design Strength, ϕN _n (lb)	Ratio	Status	
Steel	19500	20096	0.97	Pass	
Concrete breakout	39000	39652	0.98	Pass (Governs)	
Pullout	19500	68292	0.29	Pass	
Shear	Factored Load, V _{ua} (lb)	Design Strength, ϕV _n (lb)	Ratio	Status	
Steel	250	10452	0.02	Pass (Governs)	
T Concrete breakout x+	500	23658	0.02	Pass	
Concrete breakout y-	250	37478	0.01	Pass	
Pryout	500	105738	0.00	Pass	
Interaction check	N _{ua} /ϕN _n	V _{ua} /ϕV _n	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.98	0.00	98.4%	1.0	Pass

PAB7 (7/8"Ø) with hef = 12.000 inch meets the selected design criteria.

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	19500	32154	60.6%	Governs
Concrete	Factored Load, N_{ua} (lb)	Nominal Strength, N_n (lb)	Ratio	
Concrete breakout	39000	75527	51.6%	
Pullout	19500	130080	15.0%	

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) satisfied since steel ratio governs and the steel element is ductile.

12. Warnings

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.
- Designer must exercise own judgement to determine if this design is suitable.

FOUNDATION DESIGN



412 E Parkcenter Blvd, Suite 200
Boise, ID 83706
(208) 336-6985

project	BOA Centerville	by	NRP	sheet no.
location	Centerville, UT	date	09/22/20	
client	Gensler	job no.	10211900134	

Concentric Spread Footings Calculations

Allowable Bearing Pressure = 2.0 ksf

Concrete Strength = 4.0 ksi

Steel Reinforcing Strength = 60 ksi

Average Load Factor (LF) = 1.4

$$P_{allow} = Q_{allow} \times W \times L$$

$$P_u = LF \times P_{allow}$$

$$M_u = P_u/2 \times L/4$$

$$A_{s,req} = M_u/4d$$

Tag	W/L	T	P _u	φV _n	M _u	A _{s,req}	Reinforcing	
F2.0	2.0'	12"	11 k	85 k	3 k-ft	0.3	(2)	#4 bars
F3.0	3.0'	12"	25 k	85 k	9 k-ft	0.5	(3)	#4 bars
F3.5	3.5'	12"	34 k	85 k	15 k-ft	0.6	(4)	#4 bars
F4.0	4.0'	12"	45 k	85 k	22 k-ft	0.7	(4)	#4 bars
F4.5	4.5'	15"	57 k	142 k	32 k-ft	1.0	(5)	#4 bars
F5.0	5.0'	15"	70 k	142 k	44 k-ft	1.1	(5)	#5 bars
F5.5	5.5'	18"	85 k	213 k	58 k-ft	1.4	(6)	#5 bars
F6.0	6.0'	18"	101 k	213 k	76 k-ft	1.6	(6)	#5 bars
F6.5	6.5'	21"	118 k	297 k	96 k-ft	2.0	(7)	#5 bars
F7.0	7.0'	21"	137 k	297 k	120 k-ft	2.1	(7)	#5 bars
F7.5	7.5'	24"	158 k	395 k	148 k-ft	2.6	(8)	#6 bars
F8.0	8.0'	24"	179 k	395 k	179 k-ft	2.8	(8)	#6 bars
F8.5	8.5'	27"	202 k	506 k	215 k-ft	3.3	(9)	#6 bars
F9.0	9.0'	27"	227 k	506 k	255 k-ft	3.5	(9)	#6 bars
F9.5	9.5'	30"	253 k	631 k	300 k-ft	4.1	(10)	#6 bars
F10	10.0'	30"	280 k	631 k	350 k-ft	4.3	(10)	#6 bars
F11	11.0'	33"	339 k	770 k	466 k-ft	5.2	(11)	#7 bars
F12	12.0'	36"	403 k	923 k	605 k-ft	6.2	(12)	#7 bars
F13	13.0'	39"	473 k	1089 k	769 k-ft	7.3	(13)	#7 bars
F14	14.0'	42"	549 k	1269 k	960 k-ft	8.5	(14)	#8 bars
F15	15.0'	45"	630 k	1462 k	1181 k-ft	9.7	(15)	#8 bars
F16	16.0'	48"	717 k	1670 k	1434 k-ft	11.1	(16)	#8 bars

COLUMN UPLIFT ANCHORAGE



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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): 0.500
Effective Embedment depth, h_{ef} (inch): 9.000
Anchor category: -
Anchor ductility: Yes
 h_{min} (inch): 10.25
 C_{min} (inch): 3.00
 S_{min} (inch): 3.00

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 4500
 $\Psi_{c,v}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate

Length x Width x Thickness (inch): 10.00 x 10.00 x 0.75
Yield stress: 36000 psi

Profile type/size: HSS4X4X3/8

Recommended Anchor

Anchor Name: Heavy Hex Bolt - 1/2"Ø Heavy Hex Bolt, F1554 Gr. 36



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc. 5956 W. Las Positas Boulevard Pleasanton, CA 94588 Phone: 925.560.9000 Fax: 925.847.3871 www.strongtie.com



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Address:			
Phone:			
E-mail:			

Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: No

Anchors subjected to sustained tension: Not applicable

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 20200

V_{uax} [lb]: 0

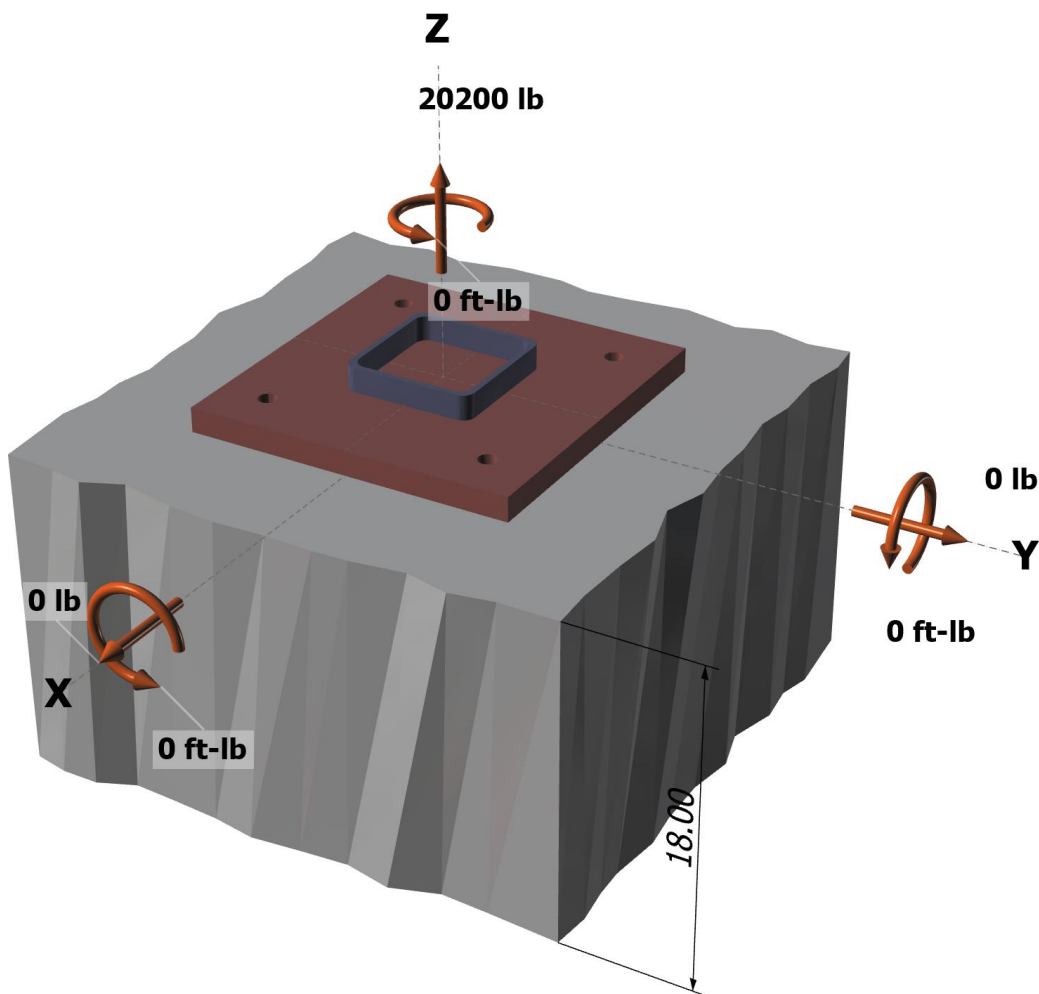
V_{uay} [lb]: 0

M_{ux} [ft-lb]: 0

M_{uy} [ft-lb]: 0

M_{uz} [ft-lb]: 0

<Figure 1>

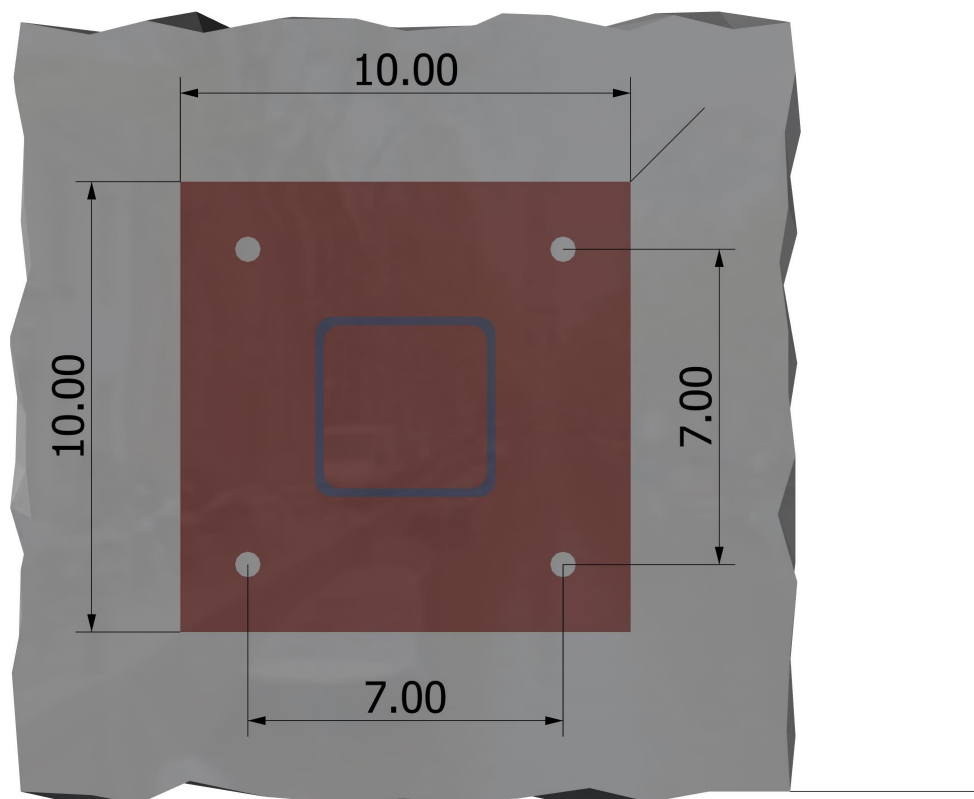


Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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Address:			
Phone:			
E-mail:			

<Figure 2>

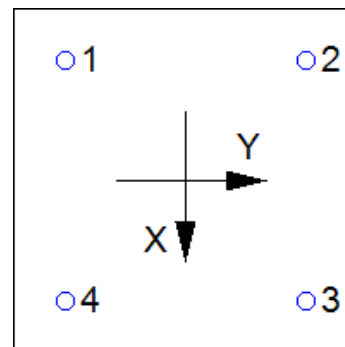


3. Resulting Anchor Forces

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	5050.0	0.0	0.0	0.0
2	5050.0	0.0	0.0	0.0
3	5050.0	0.0	0.0	0.0
4	5050.0	0.0	0.0	0.0
Sum	20200.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 20200
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

<Figure 3>



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

N _{sa} (lb)	φ	φN _{sa} (lb)
8235	0.75	6176

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

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

k _c	λ _a	f' _c (psi)	h _{ef} (in)	N _b (lb)
24.0	1.00	4500	9.000	43469

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

A _{Nc} (in ²)	A _{Nco} (in ²)	C _{a,min} (in)	Ψ _{ec,N}	Ψ _{ed,N}	Ψ _{c,N}	Ψ _{cp,N}	N _b (lb)	φ	φN _{cbg} (lb)
1156.00	729.00	-	1.000	1.000	1.00	1.000	43469	0.70	48251

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

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

Ψ _{c,P}	A _{brg} (in ²)	f' _c (psi)	φ	φN _{pn} (lb)
1.0	0.47	4500	0.70	11768



Company:		Date:	9/22/2020
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Project:			
Address:			
Phone:			
E-mail:			

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	5050	6176	0.82	Pass (Governs)
Concrete breakout	20200	48251	0.42	Pass
Pullout	5050	11768	0.43	Pass

1/2"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 9.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.636 inch

12. Warnings

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

CURB UPLIFT ANCHORAGE



Anchor Designer™
Software
Version 2.9.7376.2

Company:		Date:	9/22/2020
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: Torque controlled expansion anchor
Material: Carbon Steel
Diameter (inch): 0.500
Nominal Embedment depth (inch): 3.750
Effective Embedment depth, h_{ef} (inch): 3.250
Code report: ICC-ES ESR-3037
Anchor category: 1
Anchor ductility: Yes
 h_{min} (inch): 5.78
 c_{ac} (inch): 7.33
 c_{min} (inch): 4.25
 s_{min} (inch): 3.96

Base Material

Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 4500
 $\Psi_{c,v}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: Yes

Base Plate

Length x Width x Thickness (inch): 8.00 x 10.00 x 0.38
Yield stress: 36000 psi

Profile type/size: HSS6X4X3/8

Recommended Anchor

Anchor Name: Strong-Bolt® 2 - 1/2"Ø CS Strong-Bolt 2, h_{nom} : 3.75" (95mm)
Code Report: ICC-ES ESR-3037



Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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Company:		Date:	9/22/2020
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Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry

Load factor source: ACI 318 Section 5.3

Load combination: not set

Seismic design: No

Anchors subjected to sustained tension: Not applicable

Apply entire shear load at front row: No

Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 6000

V_{uax} [lb]: 0

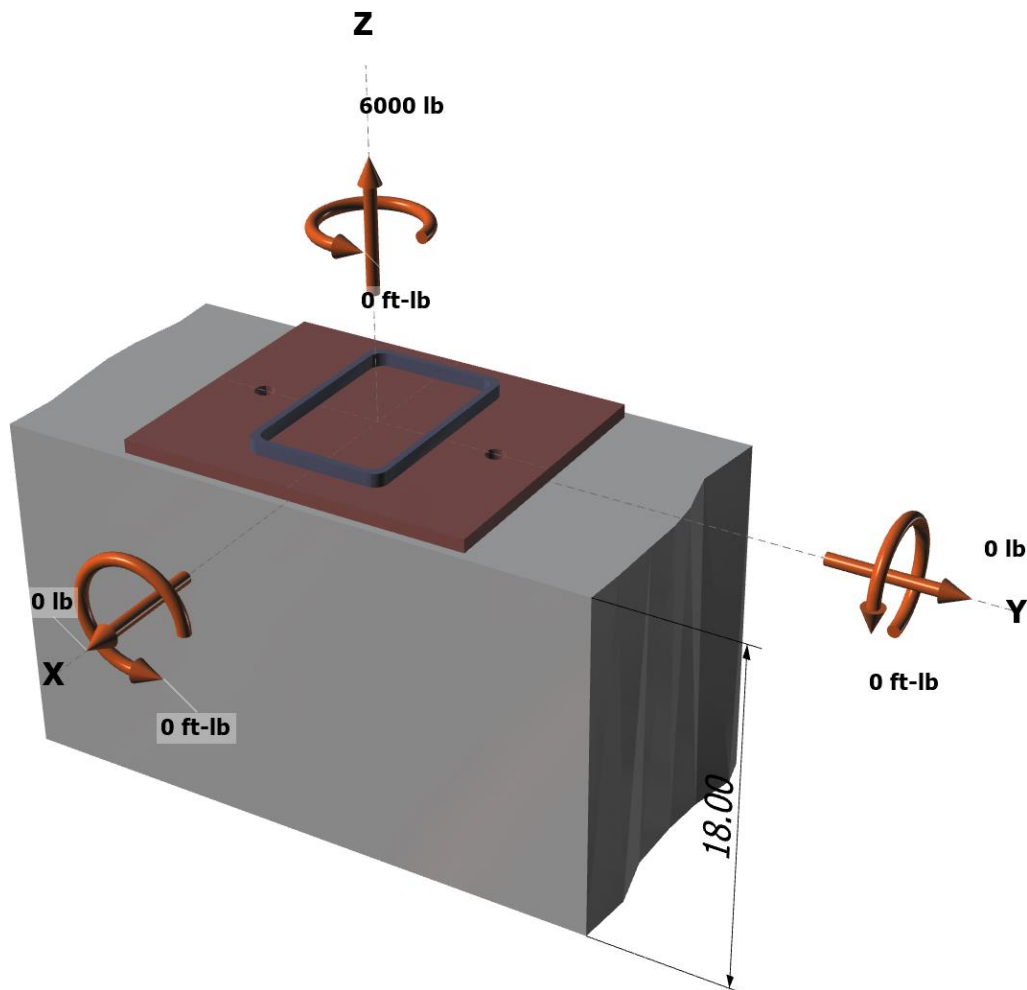
V_{uay} [lb]: 0

M_{ux} [ft-lb]: 0

M_{uy} [ft-lb]: 0

M_{uz} [ft-lb]: 0

<Figure 1>



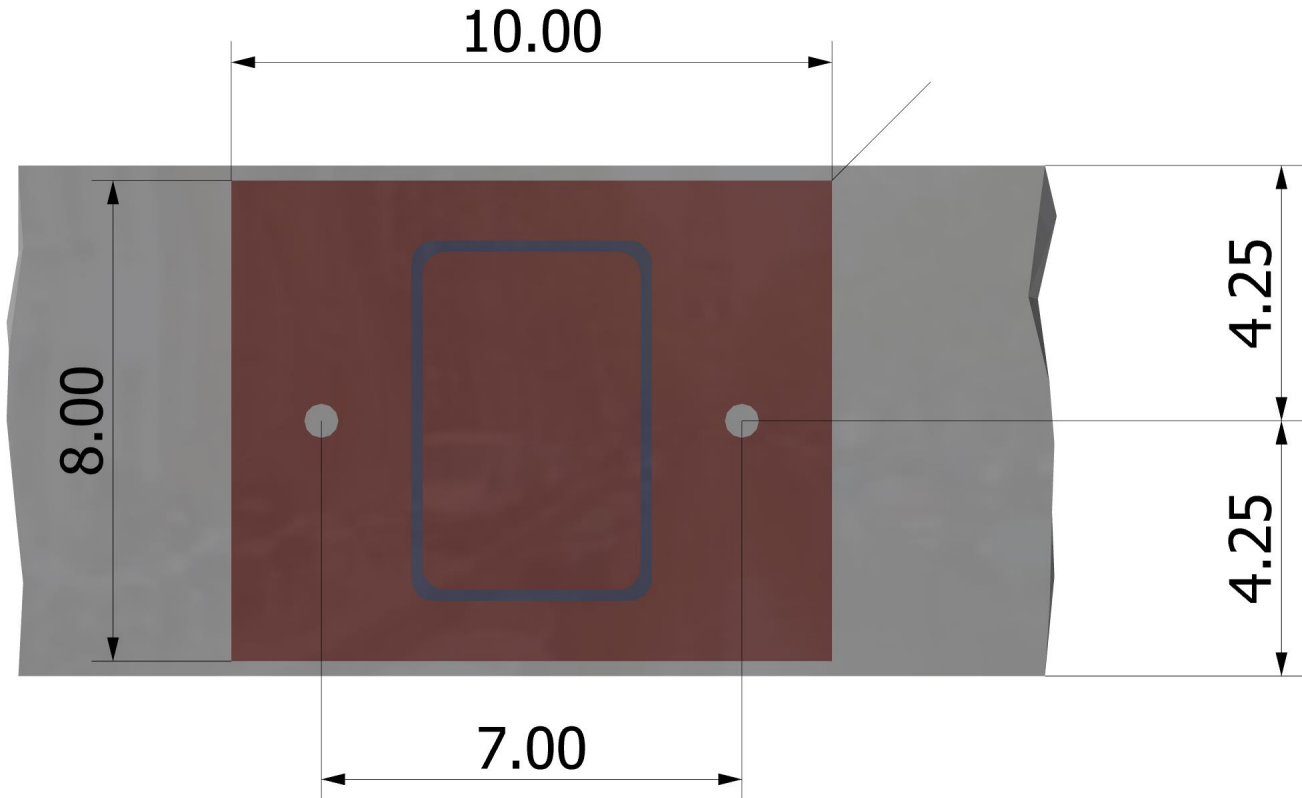
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

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Company:		Date:	9/22/2020
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Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>



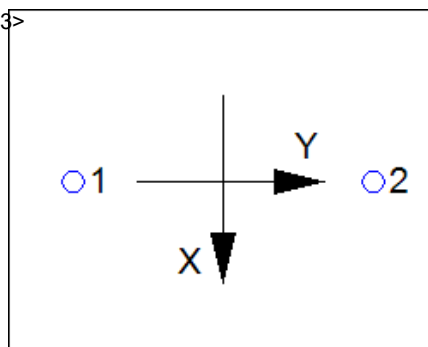
Company:		Date:	9/22/2020
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Address:			
Phone:			
E-mail:			

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	3000.0	0.0	0.0	0.0
2	3000.0	0.0	0.0	0.0
Sum	6000.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 6000
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

<Figure 3>



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

N _{sa} (lb)	φ	φN _{sa} (lb)
12100	0.75	9075

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

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

k _c	λ _a	f' _c (psi)	h _{ef} (in)	N _b (lb)
17.0	1.00	4500	3.250	6682

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

A _{Nc} (in ²)	A _{Nco} (in ²)	c _{a,min} (in)	ψ _{ec,N}	ψ _{ed,N}	ψ _{c,N}	ψ _{cp,N}	N _b (lb)	φ	φN _{cbg} (lb)
142.38	95.06	4.25	1.000	0.962	1.00	1.000	6682	0.65	6254

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

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

ψ _{c,P}	λ _a	N _p (lb)	f' _c (psi)	n	φ	φN _{pn} (lb)
1.0	1.00	4750	4500	0.50	0.65	4142



Company:		Date:	9/22/2020
Engineer:		Page:	5/5
Project:			
Address:			
Phone:			
E-mail:			

11. Results

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

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	3000	9075	0.33	Pass
Concrete breakout	6000	6254	0.96	Pass (Governs)
Pullout	3000	4142	0.72	Pass

1/2"Ø CS Strong-Bolt 2, hnom:3.75" (95mm) meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.308 inch

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.
- Refer to manufacturer's product literature for hole cleaning and installation instructions.

MISCELLANEOUS DESIGNS

$$S_{ps} = 1.300 \text{ (conservative)}$$

Assume the ATM to be described as a rigid, low-deformable element

$$W_p = 1790 \text{ lb}$$

$$a_p = 1$$

$$R_p = 1.5$$

$$I_p = 1$$

$$z = 0$$

$$h = 63.7 \text{ in}$$

$$F_p = \frac{0.4 a_p S_{ps} W_p}{\frac{R_p}{I_p}} \left(1 + 2 \left(\frac{z}{h} \right) \right)$$

$$F_p = \frac{0.4 (1) (1.300) (1790)}{\frac{1.5}{1}} \left(1 + 2 \left(\frac{0}{h} \right) \right)$$

$$F_p = 621 \text{ lb}$$

F_p shall not be taken as less than

$$F_p = 0.3 S_{ps} I_p W_p$$

$$F_p = 0.3 (1.300) (1) (1790)$$

$$F_p = 698 \text{ lb}$$

$$M_p = F_p \left(\frac{2}{3} h \right)$$

$$M_p = 698 \text{ lb} \left(\frac{2}{3} \right) (63.7)$$

$$M_p = 29,642 \text{ in-lb}$$

$$V_{\text{demand}} = \frac{F_p}{\# \text{ of bolts}}$$

$$V_{\text{demand}} = \frac{698 \text{ lb}}{4 \text{ bolts}}$$

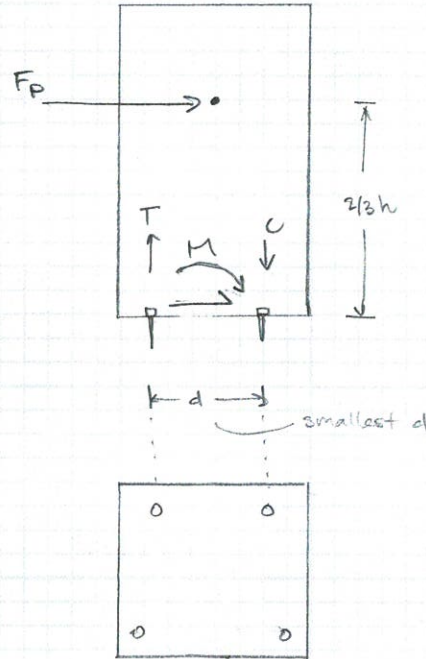
$$V = 175 \text{ lb}$$

$$N_{\text{demand}} = \frac{M_p}{2d}$$

$$d = 19.375 \text{ in}$$

$$N_{\text{demand}} = \frac{29,642 \text{ in-lb}}{2 (19.375) \text{ in}}$$

$$N = 765 \text{ lb}$$



$$V = F_p / 4 \text{ bolts}$$

$$F_c = N = \frac{M_p}{2d}$$

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Company:
Address:
Phone | Fax: |
Design: Utah ATM Anchorage
Fastening point:

Page: 1
Specifier:
E-Mail:
Date: 6/12/2020

Specifier's comments:

1 Input data

Anchor type and diameter:

Kwik Bolt TZ - CS 1/2 (2)

Item number:

not available

Effective embedment depth:

$h_{ef,act} = 2.000$ in., $h_{nom} = 2.375$ in.

Material:

Carbon Steel

Evaluation Service Report:

ESR-1917

Issued | Valid:

1/1/2020 | 5/1/2021

Proof:

Design Method ACI 318-14 / Mech

Stand-off installation:

Profile:

Base material:

cracked concrete, 3000, $f'_c = 3,000$ psi; $h = 4.000$ in.

Installation:

hammer drilled hole, Installation condition: Dry

Reinforcement:

tension: condition B, shear: condition B; no supplemental splitting reinforcement present

Seismic loads (cat. C, D, E, or F)

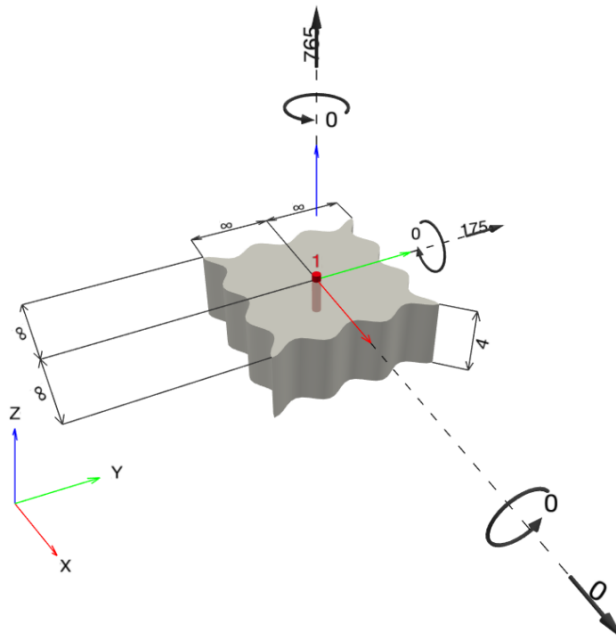
edge reinforcement: none or < No. 4 bar

Tension load: yes (17.2.3.4.3 (c))

Shear load: yes (17.2.3.5.3 (b))



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



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Company:		Page:	2
Address:		Specifier:	
Phone Fax:		E-Mail:	
Design:	Utah ATM Anchorage	Date:	6/12/2020
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 765; V _x = 0; V _y = 175; M _x = 0; M _y = 0; M _z = 0;	yes	60

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb]		Utilization	
		Load	Capacity	β_N / β_V [%]	Status
Tension	Concrete Breakout Failure	765	1,284	60 / -	OK
Shear	Pryout Strength	175	1,844	- / 10	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.596	0.095	5/3	45	OK

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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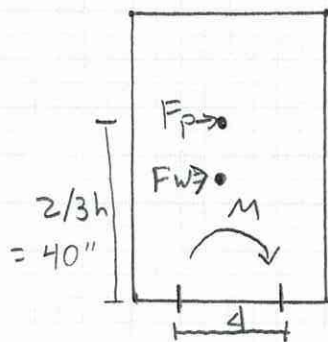
Exterior ATM Anchorage

Seismic $S_Ds = 1.30$

Wind $V_u = 140$ mph

Diebold 7790 Drive up ATM, $W_p = 2850$ lb

$H = 60"$, $W = 44"$, $D = 36"$, $d_{min} = 9.25"$



(4) Hilti $3/4" \times 8"$ HUS-EZ Anchor

$$F_p = \frac{0.4 a_p S_Ds W_p}{R_p / I_p} \left(1 + 2 \frac{z}{h}\right) = \frac{0.4 (1) (1.3) (2850)}{1.5 / 1} (1 + 0)$$

$$= 988 \text{ lb} \leftarrow \text{controls}$$

$$F_w = q_h (G C_F) A_f = 31.2 \text{ psf} (0.85) (1.45) (18.3 \text{ sf}) = 704 \text{ lb}$$

$$M = 988 \text{ lb} \times 40" = 39.5 \text{ K-in}$$

$$T/C = M/d = 4.27 \text{ K over (2) anchors} = \uparrow 2.14 \text{ K / anchor}$$

$$V = 988 \text{ lb} / (4) \text{ anchors} = 247 \text{ lb / anchor}$$

Input loading into Profis, see attached

$\therefore$ Use $3/4" \phi \times 6 1/4"$ embed HUS-EZ

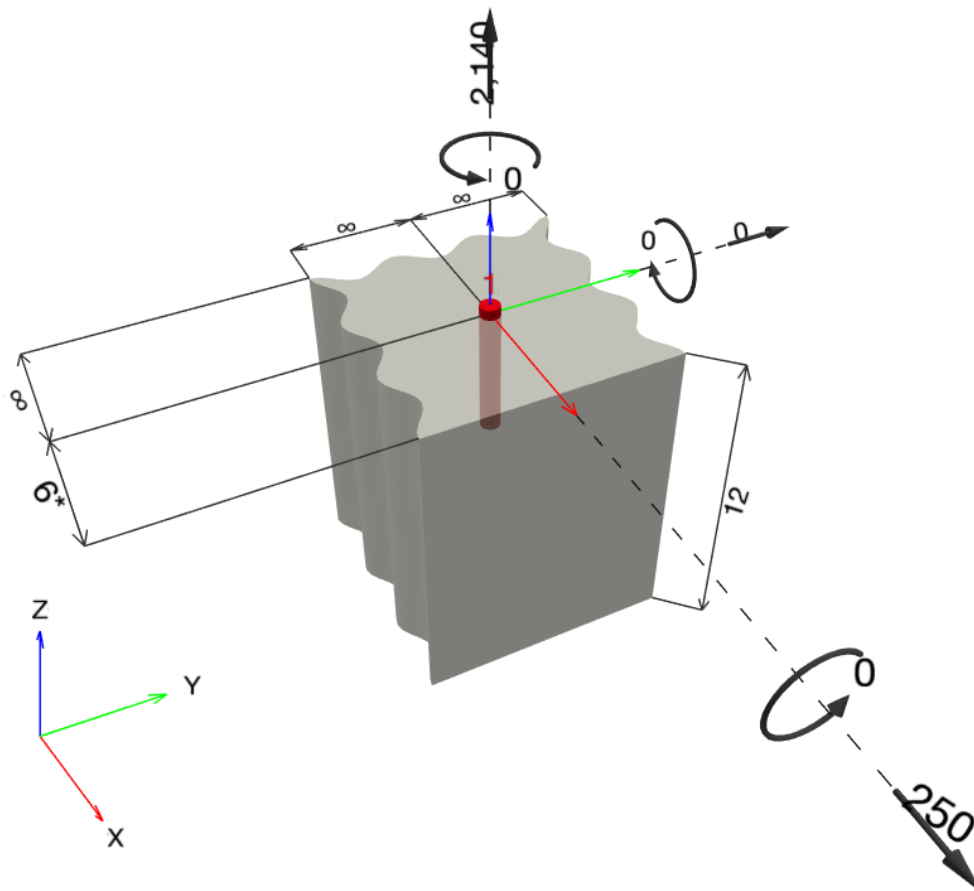
Specifier's comments:

1 Input data

Anchor type and diameter:	KWIK HUS-EZ (KH-EZ) 3/4 (6 1/4)
Item number:	418085 KH-EZ 3/4"x7"
Effective embedment depth:	$h_{ef,act} = 4.840$ in., $h_{nom} = 6.250$ in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	7/1/2020 12/1/2021
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	
Profile:	
Base material:	cracked concrete, 3000, $f_c' = 3,000$ psi; $h = 12.000$ in.
Installation:	hammer drilled hole, Installation condition: Dry
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar



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



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 2,140; V _x = 250; V _y = 0; M _x = 0; M _y = 0; M _z = 0;	no	39

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb]		Utilization	
		Load	Capacity	β_N / β_V [%]	Status
Tension	Concrete Breakout Failure	2,140	5,579	39 / -	OK
Shear	Concrete edge failure in direction x+	250	4,960	- / 6	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.384	0.050	5/3	21	OK

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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Salt Lake City, UT 84102
(801) 441-2204

project		BOA Centerville		by	NRP	sheet no.
location		Centerville, UT		date	12/15/20	
client		Gensler		job no. 10211900134		
		Misc Connections				

Cantilever Column Connection (REF 2/S05.02)

Worst case reactions per RISA output: 10.7 ft tall HSS6x6x3/8

$$M_u = 14.4 \text{ K-ft} = 173 \text{ K-in}$$

$$V_u = 13 \text{ K}$$

$$b = 6 \text{ in} \quad d = 6 \text{ in}$$

$$S = 48 \text{ in}^2$$

$$f_m = M/S = 3.6 \text{ K/in}$$

$$f_v = V/L = 0.5 \text{ K/in}$$

$$f_m + f_v = 4.1 \text{ K/in}$$

$$\text{Weld} = 1/4$$

$$\phi R_n = 5.6 \text{ K/in} \quad \text{OK}$$

Cantilever Beam Connection (REF 1/S05.02)

Worst case reactions per RISA output:

$$M_u = 55.2 \text{ K-ft} = 662 \text{ K-in}$$

$$V_u = 6 \text{ K}$$

$$d = 8 \text{ in}$$

$$T/C = M_u/d = 83 \text{ K}$$

$$\text{Weld} = 5/16$$

$$L = 25 \text{ in}$$

$$\phi R_n = 174 \text{ K} \quad \text{OK}$$

$$PL = 3/4$$

$$W = 9 \text{ in}$$

$$\phi T_n = 304 \text{ K} \quad \text{OK}$$

Cantilever Beam Connection (REF 3/S05.02)

Worst case reactions per RISA output:

$$M_u = 12.8 \text{ K-ft} = 154 \text{ K-in}$$

$$V_u = 2 \text{ K}$$

$$b = 6 \text{ in} \quad d = 9 \text{ in} \quad \text{x 2 sides}$$

$$S = 135 \text{ in}^2$$

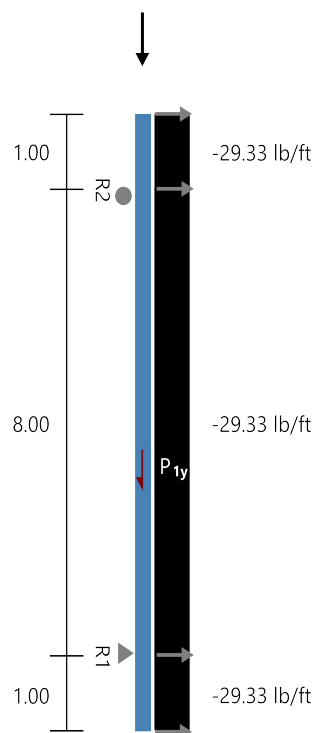
$$f_m = M/S = 1.1 \text{ K/in}$$

$$f_v = V/L = 0.1 \text{ K/in}$$

$$f_m + f_v = 1.2 \text{ K/in}$$

$$\text{Weld} = 1/4$$

$$\phi R_n = 5.6 \text{ K/in} \quad \text{OK}$$



Section : 600S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)

Maxo = 2527.1 Ft-Lb **Va =** 2822.9 lb **I =** 2.86 in⁴

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	Flexural, Distortional	Connector	Stress Ratio
Top Cant.	None, None	None, 12.0"	N/A	-
Span	None, None	None, 96.0"	N/A	-
Bottom Cant.	None, None	None, 12.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	-146.7	1.00	598.9	14.7	0.13	NO
R1	-146.7	--Shear Connection w/ clip--				NO

"*" after support means punched near support

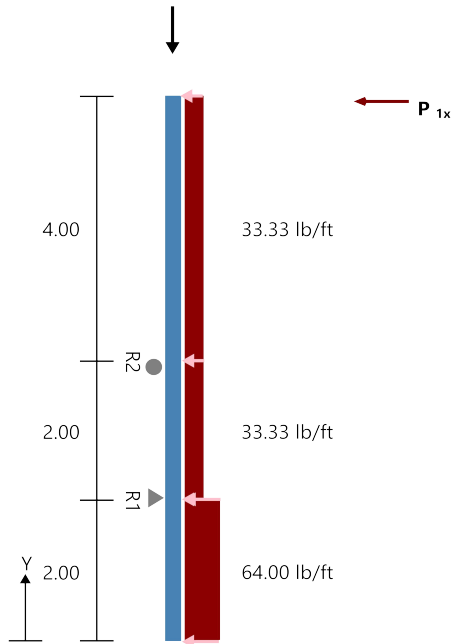
Gravity Load

Type	Load (lb)
Uniform	24.00plf
P1y	922lb @ 3.83ft

	Code Check	Required	Allowed	Interaction	Notes
Top Cant.	Max. Axial, lbs	24.0(c)	8186.6(c)	0%	K Φ =0.00 lb-in/in
	Max. Shear, lbs	29.3	1947.4	2%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	14.7	2180.0	1%	Ma-dist (control), K Φ =0.00 lb-in/in
	Moment Stability, ft-lbs	9.4	2313.4	0%	
	Shear/Moment	0.02	1.00	2%	Shear 29.3, Moment 14.7
	Axial/Moment	0.01	1.00	1%	Axial 24.0(c), Moment 14.7
	Deflection Cant., in	0.008	--meets L/2971--		
Span	Max. Axial, lbs	1138.0(c)	2063.5(c)	55%	K Φ =0.00 lb-in/in
	Max. Shear, lbs	117.3	1947.4	6%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	220.0	2158.3	10%	Ma-dist (control), K Φ =0.00 lb-in/in
	Moment Stability, ft-lbs	220.0	956.0	23%	
	Shear/Moment	0.09	1.00	9%	Shear 0.0, Moment 220.0
	Axial/Moment	0.73	1.00	73%	Axial 1070.2(c), Moment 199.7
	Deflection Span, in	0.021	--meets L/4628--		
Bottom Cant.	Max. Axial, lbs	24.0(t)	14114.8(t)	0%	K Φ =0.00 lb-in/in
	Max. Shear, lbs	29.3	1947.4	2%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	14.7	2180.0	1%	Ma-dist (control), K Φ =0.00 lb-in/in
	Moment Stability, ft-lbs	9.4	2313.4	0%	
	Shear/Moment	0.02	1.00	2%	Shear 29.3, Moment 14.7

		Axial/Moment	0.01	1.00	1%	Axial 24.0(t), Moment 14.7	
		Deflection Cant., in	0.008	--meets L/2971--			
Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector		Connector Interaction	Anchor Interaction	
R2	-146.7	0.0	By Others & Anchorage Designed by Engineer		NA	NA	
R1	-146.7	1162.0	FCB43.5 Min(4#12-14) & (4) #12-24 SST X or XL to A36 Steel		93.03 %	99.17 %	

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



Section : 800S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)
Maxo = 3065.9 Ft-Lb **Va =** 2091.3 lb **I =** 5.60 in⁴

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	Flexural, Distortional	Connector	Stress Ratio
Top Cant.	48.0", 48.0"	48.0", 48.0"	N/A	-
Span	None, None	None, 24.0"	N/A	-
Bottom Cant.	NA	None, 24.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
P1x	150.0	1.50	651.4	0.0	0.12	NO
R2	686.0	1.00	1271.8	866.7	0.47	NO
R1	-208.0	--Shear Connection w/ clip--				NO

"*" after support means punched near support

Gravity Load

Type	Load (lb)
Uniform	21.33plf (Top Cantilever), 21.33plf (Span), 21.33plf (Bottom)

Point Loads P1
Load(lb) 150
X-Dist.(ft) 8.00

	Code Check	Required	Allowed	Interaction	Notes
Top Cant.	Max. Axial, lbs	85.3(c)	5517.8(c)	2%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	283.3	2091.3	14%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	866.7	2734.3	32%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	649.6	2953.4	22%	
	Shear/Moment	0.31	1.00	31%	Shear 283.3, Moment 866.7
	Axial/Moment	0.33	1.00	33%	Axial 85.3(c), Moment 866.7
	Deflection Cant., in	0.049	--meets L/1956--		
Span	Max. Axial, lbs	128.0(c)	7227.6(c)	2%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	402.7	2091.3	19%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	866.7	2734.3	32%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	707.5	3065.9	23%	
	Shear/Moment	0.34	1.00	34%	Shear 402.7, Moment 866.7
	Axial/Moment	0.33	1.00	33%	Axial 85.3(c), Moment 866.7
	Deflection Span, in	0.002	--meets L/13349--		
Bottom Cant.	Max. Axial, lbs	42.7(t)	17504.1(t)	0%	KΦ=0.00 lb-in/in

Project Name: BOA Centerville Metal Studs
Model: Grid 5 Stud over storefront - Zone 4
Code: 2012 NASPEC [AISI S100-2012]

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Date: 12/09/2020

Simpson Strong-Tie® CFS Designer™ 3.4.5.0

Max. Shear, lbs	128.0	2091.3	6%	Shear (Punched)
Max. Moment (MaFy, Ma-dist), ft-lbs	128.0	2734.3	5%	Ma-dist (control), $K\Phi=0.00$ lb-in/in
Moment Stability, ft-lbs	81.7	3065.9	3%	
Shear/Moment	0.07	1.00	7%	Shear 128.0, Moment 128.0
Axial/Moment	0.04	1.00	4%	Axial 42.7(t), Moment 128.0
Deflection Cant., in	0.006	--meets L/7674--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	686.0	0.0	By Others & Anchorage Designed by Engineer	NA	NA
R1	-208.0	170.7	FC32-5/97(4#10) & (4) #12-24 SST X or XL to A36 Steel	18.09 %	14.88 %

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