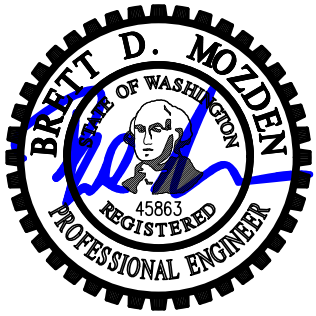
Revised Structural Calculations For:

8480 Residence

8480 85th Ave SE

Mercer Island, WA 98040



Prepared for:	Brandt Design Group
Job #:	01519-2021-09
Date:	October 4, 2022

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Lateral Design

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Main House Lateral

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Criteria Sheet

Codes

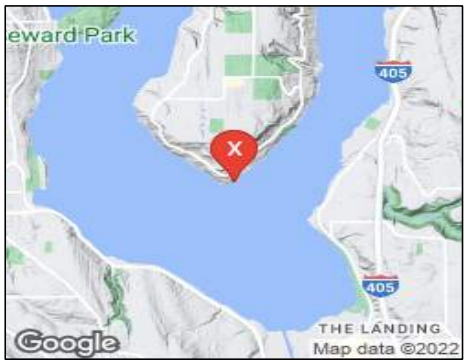
Structural	IBC 2018
Loading	ASCE 7-16
Wood:	NDS 2018
Steel:	AISC 360-16
Concrete:	ACI 318-14
Masonry:	TMS 402/602-16

Project Location

Street & Number	8480 85th Ave SE		
City:	Mercer Island	State:	WA
ZIP:	98040		
Latitude:	47.5249 N		
Longitude:	-122.2254 W		
Ground Elevation	35 ft		

Occupancy Category

Risk Category: II — ASCE 7 Table 1.5-1



Seismic Load Summary:

Analysis Procedure:	Equivalent Lateral Force Procedure		
Lateral System:	Special Reinforced Concrete Shear Walls		
R:	5.00	C_d=	5
Base Shear V =	135 kips	Ω_o=	2
S_S=	1.465	S_1=	0.504
S_{DS}=	1.17	S_{D1}=	0.57
C_s=	0.234	I_E=	1.0

Story Information

# Stories Above Grade (Including Mezzanine Levels)	2

Horizontal and Vertical Irregularities:

Is the building a "Regular Structure"? (No horizontal or vertical irregularities): No

Wind Load Summary:

V= 98 K_{ZT}= 1.00 Exposure = C

Dead Loads:

Roof		Deck		Thin Stone Veneer Wall	
Roofing	4.1 psf	Pavers (3cm)	15.6 psf	2x6 Studs @ 16"o.c.	1.5 psf
3/4" Plywood	2.7 psf	3/4" Plywood	2.7 psf	1/2" plywood	1.8 psf
Rafters @ 24"oc	2.1 psf	Joists @ 16"oc	2.5 psf	Insulation	1.5 psf
Steel Beams (seismic only)	20.0 psf	Misc./Mech.	2.5 psf	5/8" GWB + 1/2" cement	5.3 psf
Ceiling	6.8 psf	Ceiling	4.3 psf	Veneer (1.5" max)+thinset	21.9 psf
(N) Solar Panels & Misc	6.0 psf		27.6	Misc./Mech.	2.0 psf
	41.6 psf	Use	28.0 psf		34.0 psf
Use	42.0 psf			Use	34.0 psf
Main Floor		**Typical Interior Wall**		**Typical Exterior Wall**	
Floor Finish	9.4 psf	2x6 Studs@16"oc	1.5 psf	2x6 Studs @ 16"o.c.	1.5 psf
3.5" Concrete	53.1 psf	1/2" plywood	1.8 psf	1/2" plywood	1.8 psf
1.5" Metal Deck	2.9 psf	(2) 5/8" GWB	5.5 psf	Insulation	1.5 psf
Steel Beams (seismic only)	10.0 psf	Rock Wool Insul.	1.3 psf	5/8" GWB	2.8 psf
Ceiling	4.3 psf	Misc./Mech.	1.5 psf	Siding	5.0 psf
Misc./Mech.	1.5 psf		11.6 psf	Misc./Mech.	1.5 psf
	81.2 psf	Use	12.0 psf		14.1 psf
Use	82.0 psf	**Concrete Wall**		Use	15.0 psf
Trellis Roof		8" Concrete	100.0 psf	**Exterior Glazing**	
Rafters @ 24"oc	2.5 psf	*with thin stone veneer (1.5" max):*		Window glass, frames	15.0 psf
Misc./Mech.	5.0 psf	(1.5" max)+thinset	21.9 psf	**Glass Roof**	
	7.5 psf		121.9 psf	Glass	5.0 psf
Use	8.0 psf	Use	122.0 psf	Steel Beams	25.0 psf
				Use	30.0 psf

Live Loads:

Snow	25	psf	Deck	60	psf
Floor	40	psf			

Soils: Soils Report Provided? Yes

Allowable Bearing	n/a	psf	Active	40 + 10H/40	pcf (Restrained/Unrestrained)
Sliding, μ	n/a		Seismic Surcharge	9H	psf
Passive	165	pcf (includes FS 1.5)	Traffic Surcharge	40 x 2ft	psf

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Seismic Design

ASCE 7-16 Seismic Analysis Equivalent Lateral Force Procedure

Seismic Force Resisting System Per Table 12.2-1		
	System	Bearing Wall Systems
	Type:	Special Reinforced Concrete Shear Walls

Seismic Design Cat.	D
Risk Category	II
Site Class	D
Diaphragm Flexibility	Flexible

I, II, or III, or IV per Table 1.5-1
Per soils report.

Section 12.8.1.3 Exceptions

Regular Structure	No
≤ 5 Stories above grade	Yes
T ≤ 0.5s	Yes
ρ = 1.0	No
Not Site Class E or F	Yes
Risk Category I or II	Yes

If all exceptions are met, S_{DS} may be taken as 1, but not less than 0.7*(Calculated S_{DS})

S_S	1.465 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.504 g	2% in 50 yr, Latitude & Longitude lookup
R	5.00	
C_d	5.0	
Ω_o	2	
I_e	1.00	Table 1.5-2
h_n	22.7 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.21 sec	
T	0.21 sec	Eq. 12.8-7
T_0	0.10 sec	
T_S	0.49 sec	
T_L	6.00 sec	
F_a	1.20	Table 11.4-1
F_v	1.70	Table 11.4-2
S_{MS}	1.76 g	Eq. 11.4-1
S_{M1}	0.86 g	Eq. 11.4-2
S_{DS}	1.172 g	Eq. 11.4-3
S_{D1}	0.571 g	Eq. 11.4-4
C_s	***0.234 Controls***	Eq. 12.8-2
	0.550	Eq. 12.8-3 need not exceed, T < T_L
	0.010	Eq. 12.8-5 or 12.8-6 minimum
C_s, design	0.234	
Bldg. Weight	577.3 k	
V = C_SW	135.3 k	Eq. 12.8-1, Strength Level Base Shear
V = C_{Sasd}W	94.7 k	Eq. 12.8-1 ASD Base Shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	1.2
F_v	

$$T_a = C_t h_n^x \quad \text{Eq. 12.8.7}$$

$$S_{MS} = F_a S_S \quad \text{Eq. 11.4-1}$$
$$S_{M1} = F_v S_1 \quad \text{Eq. 11.4-2}$$
$$S_{DS} = {}^2/_3\, S_{MS} \quad \text{Eq. 11.4-3}$$
$$S_{D1} = {}^2/_3\, S_{M1} \quad \text{Eq. 11.4-4}$$

$$C_S = \frac{S_{DS}}{(R/I_e)} \quad \text{Eq. 12.8-2}$$
$$C_S = \frac{S_{D1}}{T(R/I_e)} \quad \text{Eq. 12.8-3}$$
$$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)} \quad \text{Eq. 12.8-4}$$
$$C_S \geq 0.044 S_{DS} I_e \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.01 \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.5 \frac{S_1}{(R/I_e)} \quad \text{Eq. 12.8-6}$$

$$C_{VX} = w_x h_x^k / \sum_{i=1}^{n} w_x h_i^k \quad \text{Eq. 12.8-12}$$

$$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i} w_{px} \quad \text{Eq. 12.10-1}$$
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-2}$$
$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-3}$$

Vertical Distribution Strength ρ= 1.3 k= 1.000

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear Strength			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	γ=F_{px}/F_x
Roof	22.7	168	22.7	3799	0.455	80.0	**80.0**	61.5	39.3	78.6	**61.5**	0.77
Main Floor	11.1	410	11.1	4560	0.545	96.0	**175.9**	96.0	96.0	192.1	**96.0**	1.00
Σ		577.3		8359		**175.9**						

Wind Design - MWFRS

ASCE 7 Chapter 27 - Directional Procedure

Design Method	Strength

Wind Coefficients

Exposure	C	
V=	98	mph
K_d=	0.85	Table 26.6-1
K_h=	0.93	Table 26.10-1
K_e=	1.00	Table 26.9-1
G=	0.85	26.9.4

Transverse Wind Pressures

L/B = 0.40 h/L = 0.61

Pressure Coefficients from Figure 27.3-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.69 / -0.15
Leeward Roof	-0.54

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.00	
Roof Type	Hip	
Roof Slope - Transverse Dir	16.25	degrees
Roof Slope - Long Dir	16.25	degrees
Ground to top of roof	25.00	ft
Bot of roof to top of roof	4.67	ft
Mean Roof Height, h	22.67	ft
Short Plan Dimension	37.08	ft
Long Plan Dimension	93.00	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft

Velocity Pressure at Mean Roof Height, q_h =	19.3	psf

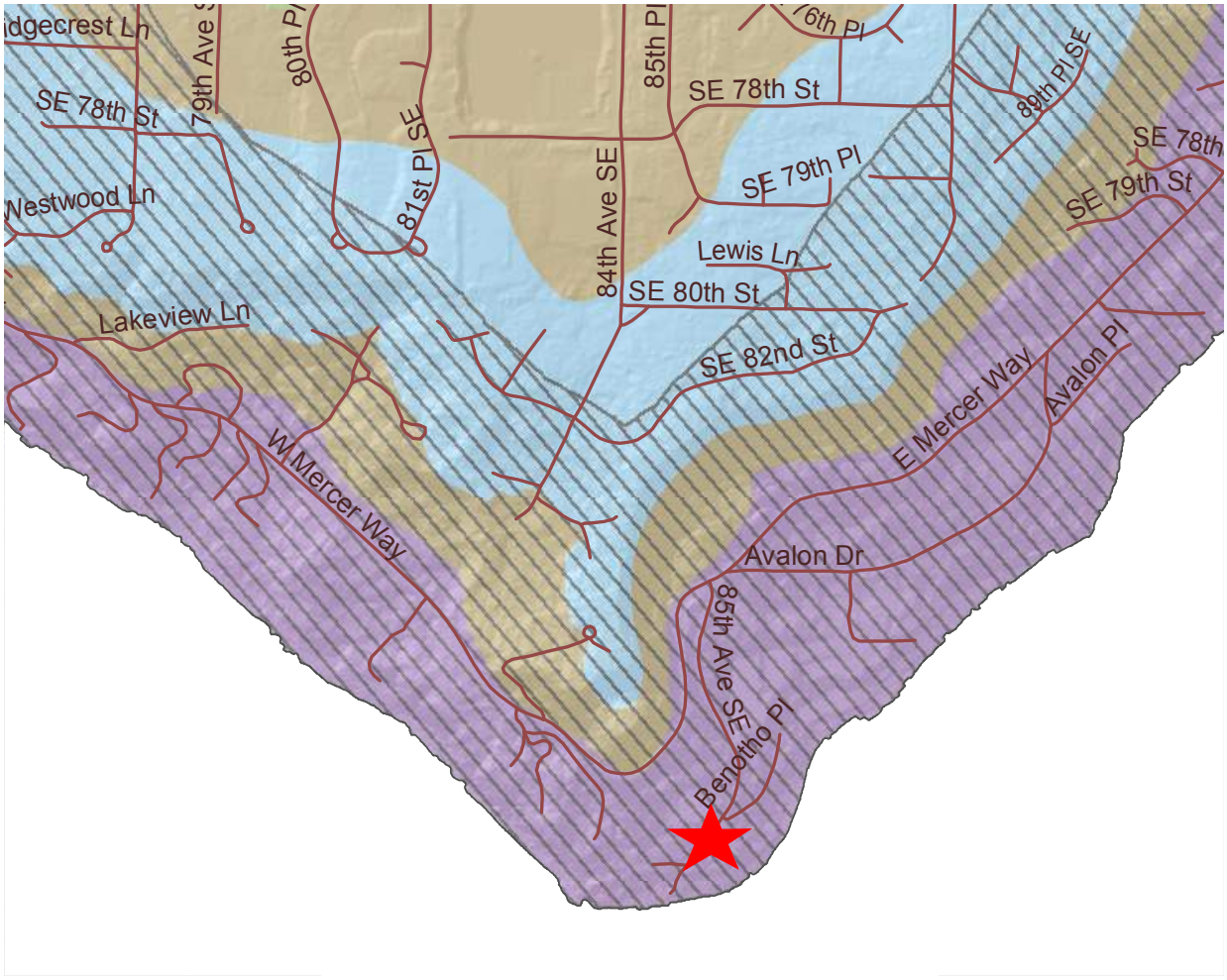
Wall Pressures (Unfactored): Strength

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	17.74	12.06	8.21	**20.3**
15-20	0.9	18.78	12.77	8.21	**21.0**
20-25	0.94	19.62	13.34	8.21	**21.6**
25-30	0.98	20.45	13.91	8.21	**22.1**
30-40	1.04	21.71	14.76	8.21	**23.0**
41-50	1.09	22.75	15.47	8.21	**23.7**
51-60	1.13	23.59	16.04	8.21	**24.3**
61-70	1.17	24.42	16.61	8.21	**24.8**
71-80	1.21	25.26	17.17	8.21	**25.4**
81-90	1.24	25.88	17.60	8.21	**25.8**
91-100	1.26	26.30	17.88	8.21	**26.1**

Roof Pressures (Unfactored) Strength

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.4	-11.4	-8.9	**8.00**

Wind Exposure and Kzt



WIND EXPOSURE CATEGORIES:

Wind Exposure Category	
	Exposure 'C' (1500 feet from Lake)
	Exposure 'B' (all other areas)

WIND SPEED-UP (TOPOGRAPHIC EFFECT) - K_zt Factor :

K_zt Factor	
	$K_zt = 1.0$
	$K_zt = 1.3$
	$K_zt = 1.6$
	$K_zt = 1.9$

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10/20/2022
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SEISMIC WEIGHT - LOADING CALCULATIONS

Level	Area 1	A_{TOTAL} (ft^2)	$W_{AREA\ 1}$ (psf)	Area 2	A_{TOTAL} (ft^2)	$W_{AREA\ 2}$ (psf)
Roof	Roof	2804	48	Glass Roof	221	30
Main Floor	Main Floor	2552	94	Trellis Roof	126	8
			0			0
			0			0

Seismic Weight

Level	Area 3	A_{TOTAL} (ft^2)	$W_{AREA\ 3}$ (psf)	Area 4	A_{TOTAL} (ft^2)	$W_{AREA\ 4}$ (psf)	W_{AREA} (k)
Roof	Chimney (brick)	149	34			0	146.27
Main Floor	Deck	947	28	Chimney (brick)	50	34	269.11
			0			0	0.00
			0			0	0.00

Level	Cladding 1	$W_{CLAD\ 1}$ (psf)	$L_{CLAD\ 1}$ (ft)	$H_{CLAD\ 1}$ (ft)	Cladding 2	$W_{CLAD\ 2}$ (psf)	$L_{CLAD\ 2}$ (ft)	$H_{CLAD\ 2}$ (ft)
Roof	Exterior Glazing	15	203.1	4.8	Typical Exterior Wal	15	58.0	4.8
Main Floor	Exterior Glazing	15	203.1	4.8	Typical Exterior Wal	15	58.0	4.8
0		0				0		0.0
0		0				0		0.0

Level	Cladding 3	$W_{CLAD\ 3}$ (psf)	$L_{CLAD\ 3}$ (ft)	$H_{CLAD\ 3}$ (ft)	Cladding 4	$W_{CLAD\ 4}$ (plf)	$\#Col_{CLAD\ 4}$ (#)	$H_{CLAD\ 4}$ (ft)
Roof		0		4.8	Columns	25	23	4.8
Main Floor	crete Wall w/ Thin S	122	132.6	5.6	Columns	25	23	10.3
0		0		0.0		0		0.0
0		0		0.0		0		0.0

Level	Cladding 5	$W_{CLAD\ 5}$ (psf)	$L_{CLAD\ 5}$ (ft)	$H_{CLAD\ 5}$ (ft)	Cladding 6	$W_{CLAD\ 6}$ (psf)	$L_{CLAD\ 6}$ (ft)	$H_{CLAD\ 6}$ (ft)	W_{CLAD} (k)	TOTAL $W_{DIST\ TO\ LVL}$ (k)
Roof		0		4.8		0		4.8	21.34	168
Main Floor	in Stone Veneer W	34	87	5.6	Exterior Glazing	15	115	5.6	140.56	410
0		0		0.0		0		0.0	0.00	0
0		0		0.0		0		0.0	0.00	0
0		0		0.0		0		0.0	0.00	0

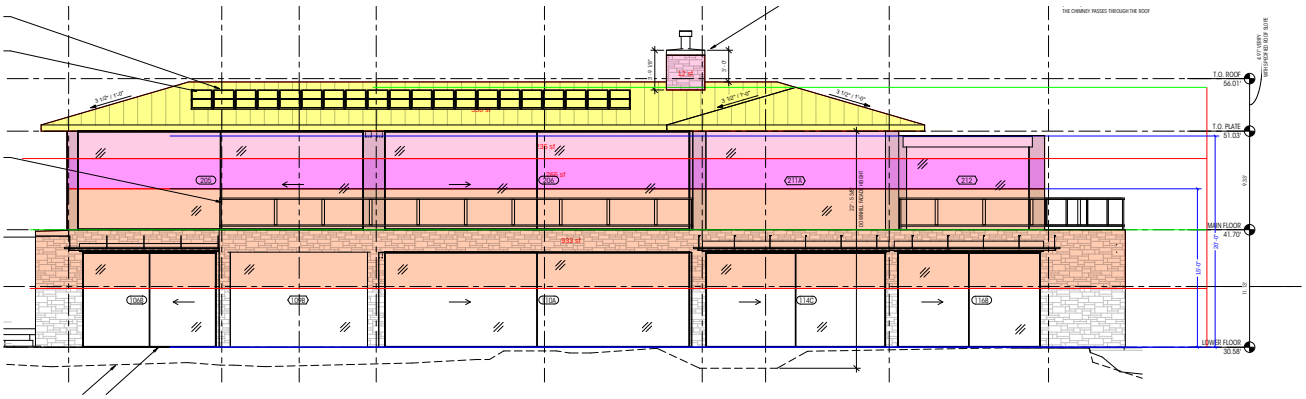
Wall DL		
Typical Interior Wall	12	psf
Typical Exterior Wall	15	psf
Thin Stone Veneer Wall	34	psf
Exterior Glazing	15	psf
Concrete Wall w/ Thin Ston	122	psf
Columns	25	plf
Conc+Siding	105	psf

Area DL		
Roof	48	psf
Main Floor	94	psf
Trellis Roof	8	psf
Deck	28	psf
Glass Roof	30	psf
Chimney (brick)	34	psf

Assuming HSS 4 1/2 x 4 1/2 for columns

roof and main area loads include typ interior wall load
exterior walls, glazing, concrete added as line loads
steel beams included in area loads
steel columns added as line loads

Wind Base Shear Calc - NS Direction



South elevation governs for NS direction

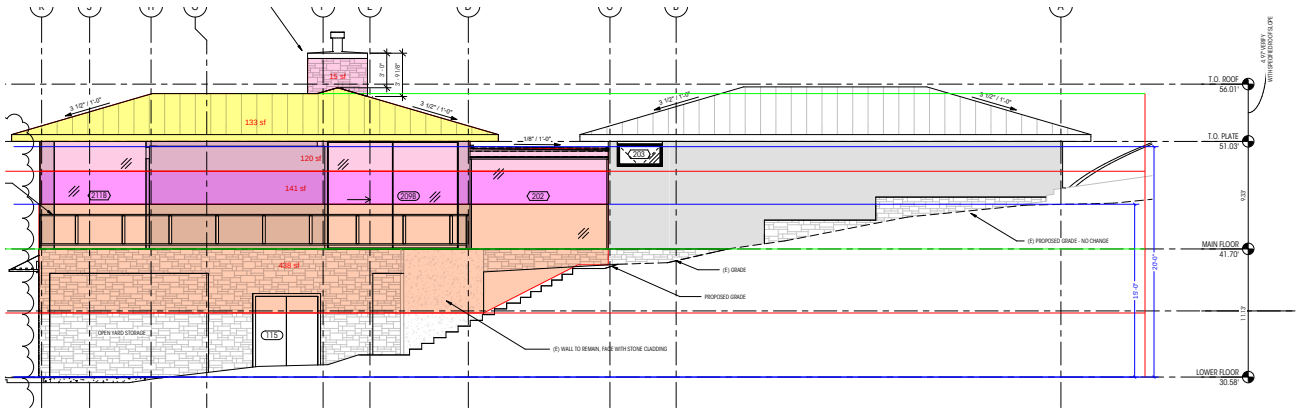
ROOF
roof: 330*8.0 = 2640 lbs
25'-0" < wall < 30'-0": 12*22.1 = 265 lbs
15'-0" < wall < 20'-0": 235*21.0 = 4935 lbs
total: 7840 lbs

FLOOR
15'-0" < wall < 20'-0": 265*21.0 = 5565 lbs
wall < 15'-0": 933*20.3 = 18940 lbs
total: 24505 lbs

TOTAL WIND BASE SHEAR
32.4 kips

seismic governs NS

Wind Base Shear Calc - EW Direction



East elevation governs for EW direction

ROOF
roof: 133*8.0 = 1064 lbs
25'-0" < wall < 30'-0": 15*22.1 = 332 lbs
15'-0" < wall < 20'-0": 120*21.0 = 2520 lbs
total: 3916 lbs

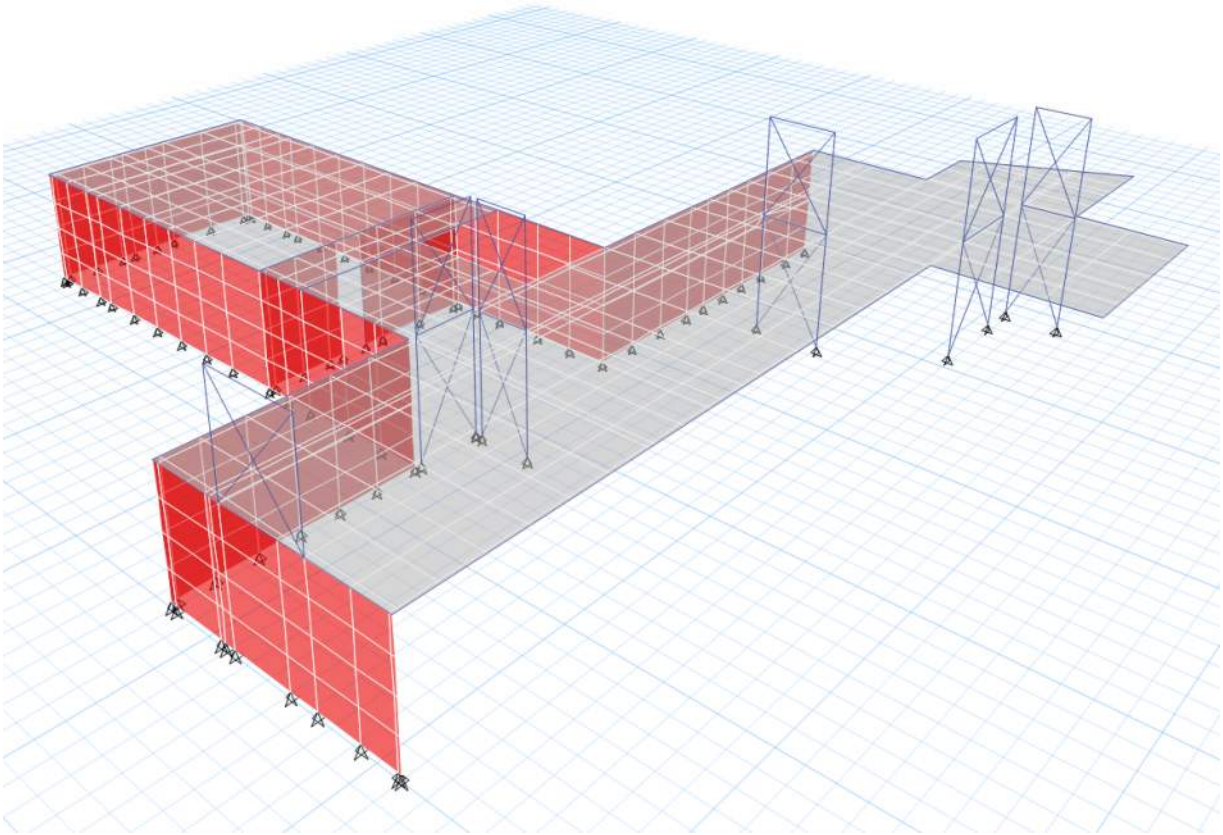
FLOOR
15'-0" < wall < 20'-0": 141*21.0 = 2961 lbs
wall < 15'-0": 438*20.3 = 8892 lbs
total: 11853 lbs

TOTAL WIND BASE SHEAR
15.8 kips

seismic governs EW

ETABS MODEL AND LATERAL ANALYSIS

The lateral system of the main house consists of steel special concentrically braced frames and special concrete reinforced shear walls. All braced frames are two-stories, except one on the west side of the house that lands on a concrete wall. The roof diaphragm is a flexible, plywood diaphragm so it's not modeled in ETABS; tributary widths are used to distribute the loads to the braced frames and those loads are added to ETABS. The main floor diaphragm is concrete on metal deck and is modeled as semi-rigid. The equivalent lateral force procedure was used to analyze the system. Calculated story shears and accidental torsion are added to ETABS at the center of mass. ETABS is utilized to determine design forces in the braced frames, concrete shear walls, and diaphragms. Concrete shear walls and diaphragms are designed in accordance with ACI 318-14, and steel braced frames, drag struts, and connections are designed in accordance with AISC 360-16.



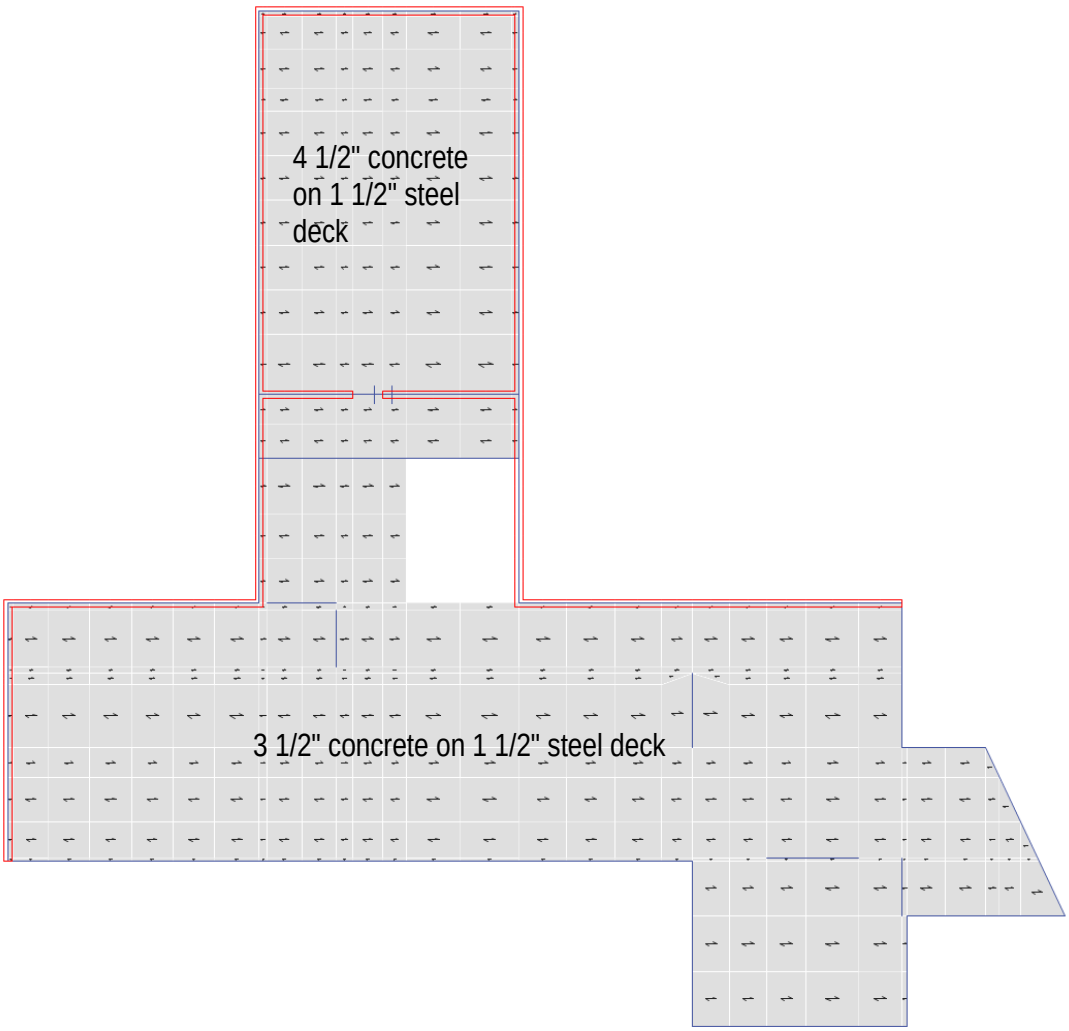
ETABS - Floor Levels

Story Definitions

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: Yes Sort: None Story Definitions
Filter: None

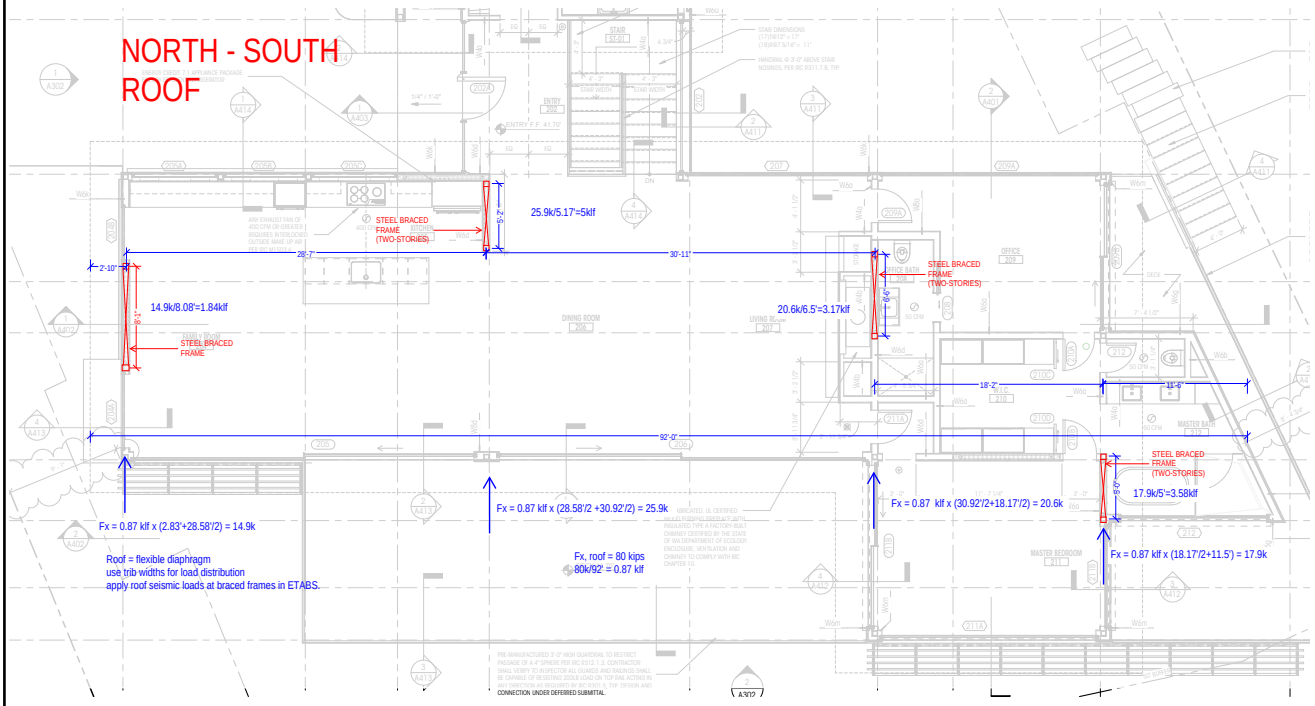
	Name	Height ft	Master Story	Similar To	Splice Story	Splice Height ft	Color
	Upper	9.3333	Yes	None	No		Cyan
	Main	11	No	Upper	No		Magenta

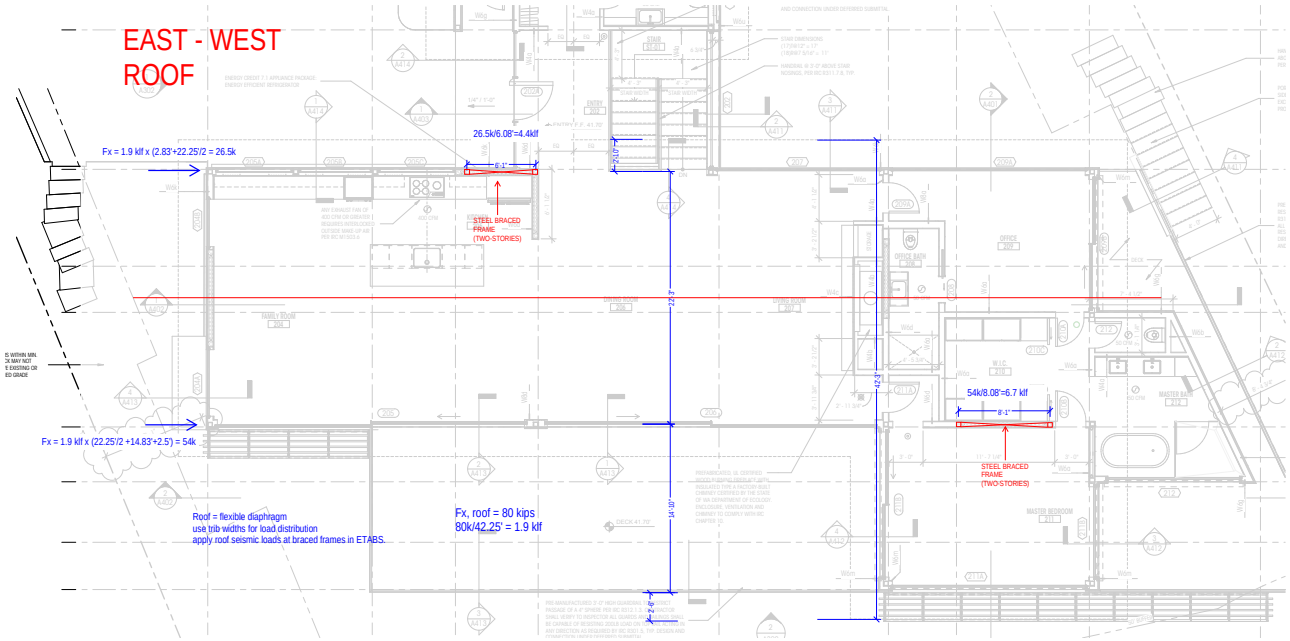


ETABS: ADDED MASS AND SEISMIC LOADS

ETABS: ADDITIONAL MASS								
AREA MASS								
		Total (psf)	Total (lb-s^2/ft^2)					
MH Roof		47	1.46	*includes 20 psf for steel beams and added 5 psf for steel columns				
MH Main Floor		31	0.96	*includes 10 psf for steel beams, added 5 psf steel columns. Doesn't include somd (etabs account for self weight)				
MH Deck		28	0.87					
MH Trellis		8	0.25					
Garage Roof		38	1.18	*includes 10 psf for steel beams, added 5 psf for steel columns, doesn't include somd (etabs accounts for self weight)				
Garage Floor		21.5	0.67	*includes 10 psf steel beams, added 5 psf steel columns				
Glass Roof		25	0.78	*includes 10 psf for steel				
LINE MASS								
		[psf]	Height	[plf]	[lb-s^2/ft^2]			
Exterior Wall/Glazing		15	9.33	139.95	4.35			
	Area [ft2]	[psf]	Weight [lb]	Length to Dist. [ft]	[lb-s^2/ft^2]			
South Deck to Grid G	643	28	18004	45	12.43			
East Deck to Grid 8	304	28	8512	21.25	12.44			
SW Trellis to Grid G	38	8	304	14.5	0.65			
SE Trellis to Grid K	88	8	704	38	0.58			
	thickness	Height	weight [plf]	[lb-s^2/ft^2]				
Concrete wall at garage	0.5	11.5	0.404	0.012546584				
ETABS: Additional Seismic Loads								
Seismic Load from Garage Shearwalls:								
Base Shear	18.8	k (LRFD)						
N/S Direction								
	Force	total Length	[klf]					
West walls	9.4	10.25	0.917					
East Walls	9.4	33.92	0.277					
E/W Direction								
	Force	total Length	[klf]					
North Wall	9.4	23.25	0.404					
South Wall	9.4	18.25	0.515					
Loads from Wood portions getting dragged into House								
Main Floor Weight	410	k						
Main Floor Shear	96	k						
	Area [ft2]	[psf]	Weight [k]	% weight	Force [[k]	Length	[klf]	
SW Trellis:	38	8	0.304	0.001	0.07	14.5	0.00	along grid G between 1 & 2
SE Trellis	88	8	0.704	0.002	0.704	18.5	0.04	along grid F between 6 & 8
South Deck	643	28	18.004	0.044	4.22	45	0.09	along grid G between 2 & 6
East Deck	304	28	8.512	0.021	1.99	21.25	0.09	along grid 8

ETABS: ADDED SEISMIC LOADS FROM ROOF





STRUCTURAL ENGINEERING

8480 Residence
PROJECT
Lateral Design 15

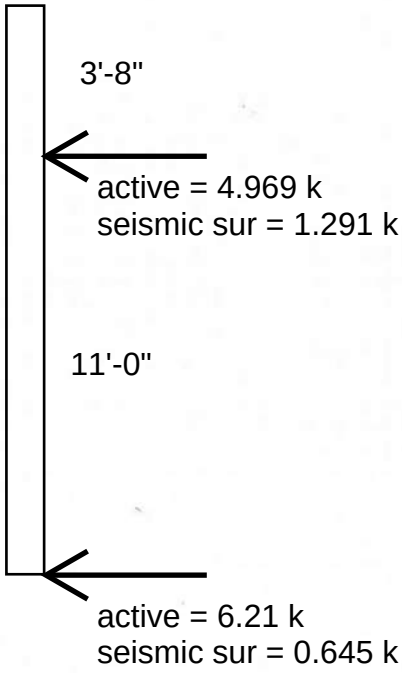
10/24/22
DATE

PROJ. #
SRW
DESIGN

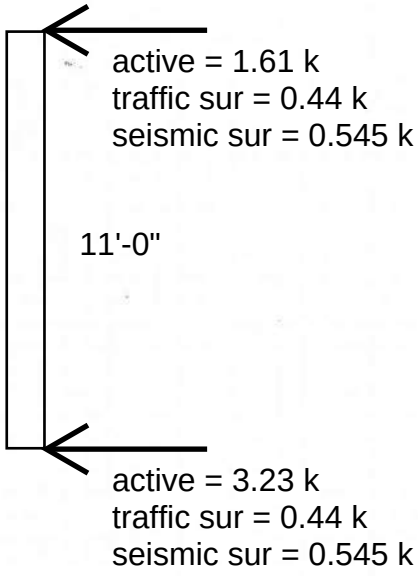
SHEET

Basement Wall Reactions

NORTH AND EAST WALLS



NORTH AND WEST WALLS
AT GARAGE ENTRANCE



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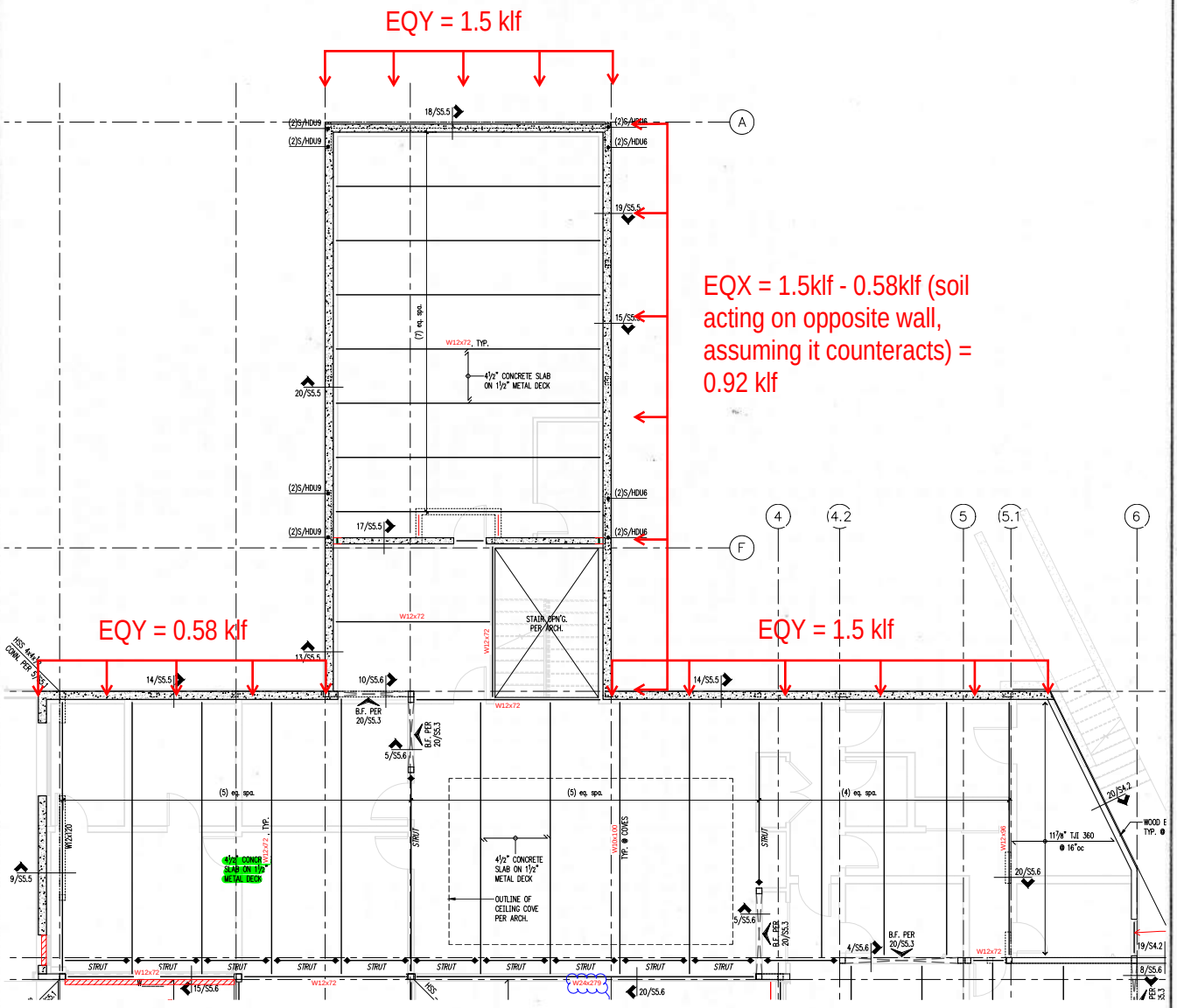
Basement Wall Reactions



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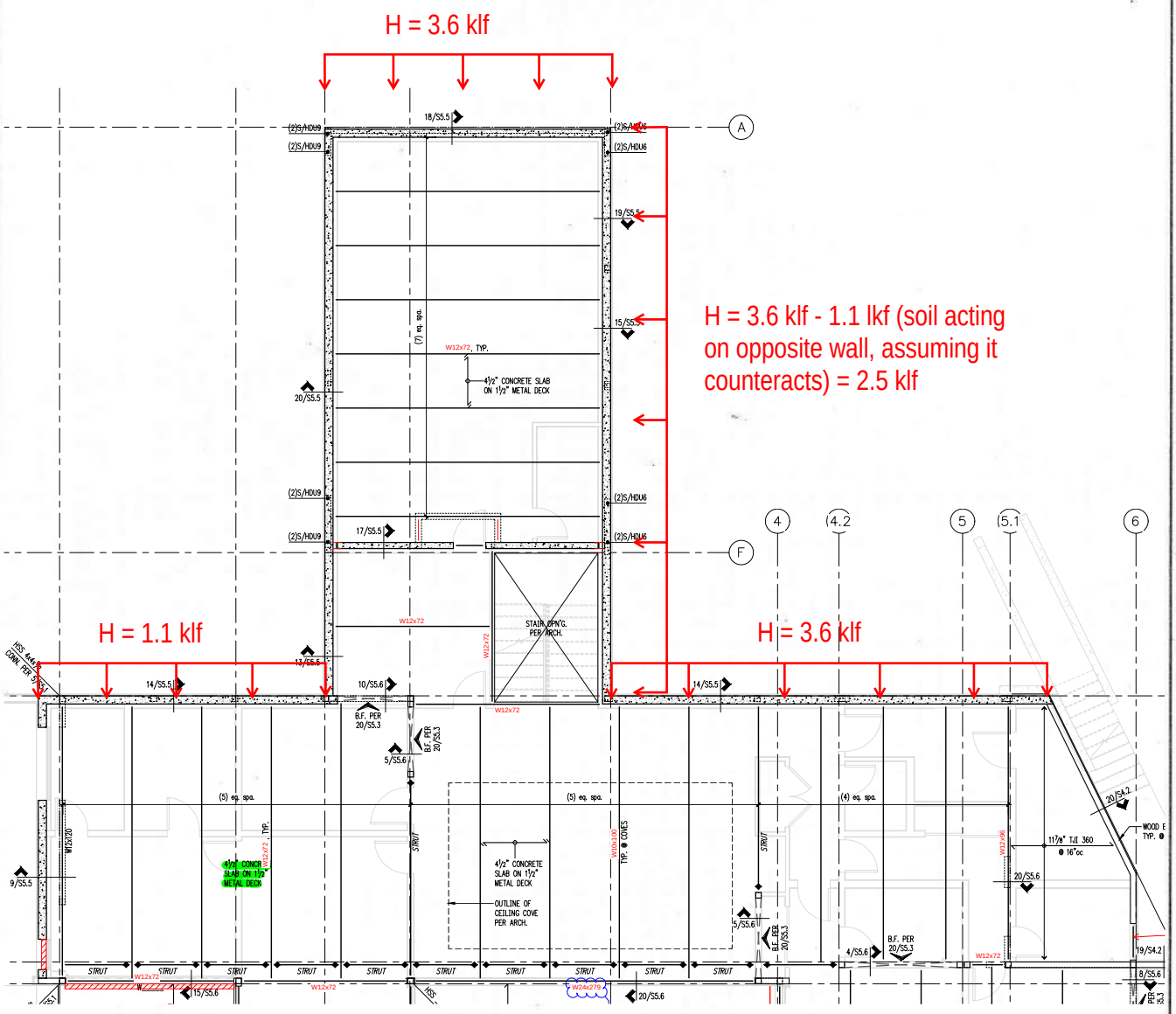
Basement Wall Reactions



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SUMMARY OF CALCULATED DRIFTS

The deflection at Level x (δ_x) (in. or mm) used to compute the design story drift, Δ, shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad (12.8\text{-}15)$$

where

C_d = the deflection amplification factor in Table 12.2-1
δ_{xe} = the deflection at the location required by this section determined by an elastic analysis
I_e = the importance factor determined in accordance with Section 11.5.1

	Elev
Upper	51.0312
Main	41.6979
Base	30.6979

Story	Height [ft]	Allowable Drift, 0.02hx [in]	Max Drift, X [in]	Max Drift, Y [in]
Upper to Main	9.3333	2.24	0.60	0.85
Main to Base	11	2.64	0.69	0.90

Cd	5	
I	1	
ρ	1	** combinations already account for change in rho from 1.3 to 1
% for drift	0.02	

	Max Drift, X	Max Drift, Y
Upper-Main	0.60	0.85

Upper-Main

Unique Node	Ux [in]	Drift, X	Uy [in]	Drift, Y
20	0.00	0.11	0.04	0.20
19	0.02		0.00	
25	0.07	0.16	0.04	0.20
21	0.04		0.00	
109	0.00	0.00	0.00	0.00
104	0.00		0.00	
110	0.00	0.60	0.00	0.64
106	0.12		0.13	
108	0.04	0.10	0.20	0.83
849	0.02		0.04	
107	0.03	0.06	0.20	0.85
848	0.04		0.03	
88	0.08	0.19	0.19	0.66
851	0.05		0.06	
89	0.06	0.14	0.19	0.66
852	0.03		0.06	
82	0.00	0.00	0.00	0.00
853	0.00		0.00	
81	0.00	0.00	0.00	0.00
854	0.00		0.00	
73	0.19	0.43	0.28	0.78
855	0.10		0.13	
74	0.16	0.37	0.28	0.77
856	0.08		0.13	
	0.00	0.00	0.00	0.00
	0.00		0.00	

Cd	5	
I	1	
ρ	1	** combinations already account for change in rho from 1.3 to 1
% for drift	0.02	

	Max Drift, X	Max Drift, Y
Main-Base	0.69	0.90

GR-BASE

Unique Node	Ux [in]	Drift, X	Uy [in]	Drift, Y
857	0.05	0.25	0.00	0.01
29	0.00		0.00	
21	0.04	0.19	0.00	0.02
98	0.00		0.00	
19	0.02	0.11	0.00	0.02
94	0.00		0.00	
858	0.01	0.03	0.00	0.02
1210	0.00		0.00	
859	0.01	0.06	0.01	0.05
1053	0.00		0.00	
867	0.01	0.04	0.00	0.02
64	0.00		0.00	
860	0.00	0.00	0.00	0.01
918	0.00		0.00	
862	0.00	0.01	0.00	0.01
41	0.00		0.00	
869	0.01	0.03	0.00	0.02
71	0.00		0.00	
864	0.04	0.22	0.06	0.31
951	0.00		0.00	
311	0.00	0.01	0.13	0.63
236	0.00		0.00	
105	0.03	0.02	0.01	0.06
1	0.02		0.00	
305	0.01	0.07	0.02	0.12
6	0.00		0.00	
849	0.02	0.11	0.04	0.19
10	0.00		0.00	
848	0.04	0.22	0.03	0.16
9	0.00		0.00	
852	0.03	0.16	0.06	0.30
92	0.00		0.00	
851	0.05	0.23	0.06	0.30
90	0.00		0.00	
156	0.08	0.42	0.08	0.42
148	0.00		0.00	
145	0.09	0.43	0.11	0.56
146	0.00		0.00	
856	0.08	0.42	0.13	0.63
78	0.00		0.00	
855	0.10	0.51	0.13	0.63
77	0.00		0.00	
868	0.01	0.04	0.01	0.05
362	0.00		0.00	
870	0.01	0.03	0.01	0.06
365	0.00		0.00	
162	0.05	0.24	0.13	0.63
	0.00		0.00	
132	0.05	0.24	0.15	0.77
	0.00		0.00	
143	0.10	0.51	0.18	0.90
	0.00		0.00	
646	0.10	0.51	0.13	0.63
	0.00		0.00	
875	0.14	0.68	0.13	0.64
	0.00		0.00	
244	0.14	0.69	0.06	0.30
	0.00		0.00	
241	0.08	0.40	0.06	0.29
	0.00		0.00	

TORSIONAL AMPLIFICATION

	Ux	Unique Nc	Uy	Unique Node Name
Upper	0.331228464	139	0.280119	74
Main	9.5380236	2	2.120764	2

Upper	Unique Node Name	Ux	Avg	Ux Max	Ax
	109	0.00	0.00	0.33	#DIV/0!
	110	0.00			
	81	0.00	0.00	0.33	#DIV/0!
	82	0.00			

Unique Node Name	Uy	Avg	Uy Max	Ax
73	0.28	0.28	0.28	0.69
74	0.28			
88	0.19	0.19	0.28	1.48
89	0.19			

Main	Unique Node Name	Ux	Avg	Ux Max	Ax
	860	0.00	0.00	0.00	1.59
	862	0.00			
	858	0.01	0.01	0.01	1.20
	859	0.01			
	864	0.04	0.02	0.04	2.78
	23	0.00			
	858	0.01	0.00	0.01	2.78
	23	0.00			
	857	0.05	0.02	0.05	2.78
	877	0.00			

Unique Node Name	Uy	Avg	Uy Max	Ax
857	0.00	0.00	0.00	1.02
858	0.00			
859	0.01	0.01	0.01	2.25
860	0.00			
864	0.06	0.03	0.06	2.65
862	0.00			
875	0.13	0.06	0.13	2.78
23	0.00			

Use Ax=3

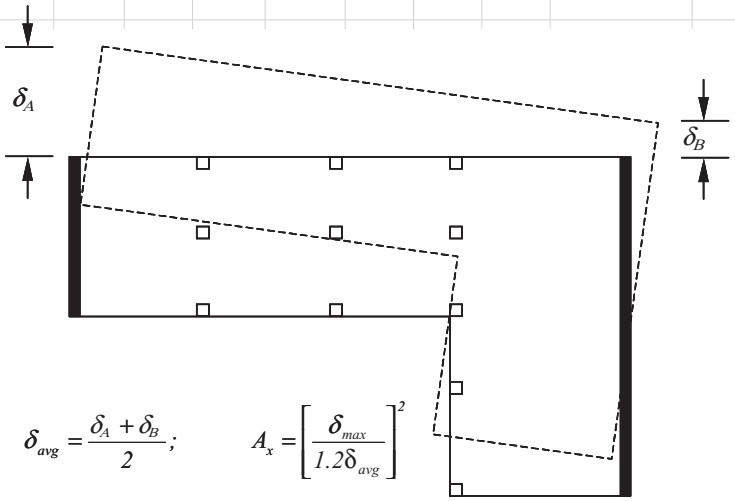


FIGURE 12.8-1 Torsional Amplification Factor, A_x

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = maximum displacement at level x computed assuming $A_x = 1$ [in. (mm)], and

δ_{avg} = average of the displacements at the extreme points of the structure at level x computed assuming $A_x = 1$ [in. (mm)].

BRACED FRAMES

Summary

The braced frames are designed using a custom spreadsheet. Brace demands from ETABS are imported into the spreadsheet, along with gravity beam and column loads calculated by spreadsheet. The spreadsheet designs the braces for the forces from ETABs, then calculates the expected strengths of the braces in tension and compression. The braced frame beams and columns are designed using these expected strength values from the braces, in combination with the calculated gravity loads. The braced frame connections are designed using the expected brace strengths. All of the design checks are shown in the spreadsheet output on the subsequent pages.

Indexing		SCBF - 1 story					
0	0	BF Name	# stories	Frame Height	RAM Steel Col 1 #	RAM Steel Col 2 #	RAM Steel Beam #
1	1	BF-1	1	9.33	COL1-1	COL1-2	U-BM1
1	0						
1	0						
1	0						

Indexing		SCBF - 2 story					
1	1	BF Name	# stories	Frame Height	RAM Steel Col 1 #	RAM Steel Col 2 #	RAM Steel Beam #
2	2	BF-4-U	2	9.33	COL4-1U	COL4-2U	U-BM4
3	3	BF-4-M	2	11.00	COL4-1M	COL4-2M	M-BM4
4	4	BF-6-U	2	9.33	COL6-1U	COL6-2U	U-BM6
5	5	BF-6-M	2	11.00	COL6-1M	COL6-2M	M-BM6
6	6	BF-8-U	2	9.33	COL8-1U	COL8-2U	U-BM8
7	7	BF-8-M	2	11.00	COL8-1M	COL8-2M	M-BM8
8	8	BF-D-U	2	9.33	COLD-1U	COLD-2U	U-BMD
9	9	BF-D-M	2	11.00	COLD-1M	COLD-2M	M-BMD
10	10	BF-G-U	2	9.33	COLG-1U	COLG-2U	U-BMG
11	11	BF-G-M	2	11	COLG-1M	COLG-2M	M-BMG
11	0						

Compressive Capacities

phi	0.9
E	29000
Fy	46
K	1

Brace Capacity in Compression

length	HSS4X.250	HSS3X.250	HSS3.5X.250	HSS3.5X.313	HSS6x.500	HSS6x.375	HSS5x.500	HSS4.5x.375	HSS4x.313	HSS5x.375	HSS7.625x.375	HSS8.625X.500	HSS8.625X.500
rx	1.33	0.982	1.16	1.14	1.96	2	1.61	1.47	1.32	1.65	2.58	2.89	2.89
ry	1.33	0.982	1.16	1.14	1.96	2	1.61	1.47	1.32	1.65	2.58	2.89	2.89
Ag	2.76	2.03	2.39	2.93	8.09	6.20	6.62	4.55	3.39	5.10	7.98	11.90	11.90
Length													
6	94	59	76	93	306	235	240	160	115	186	314	473	473
7	87	51	70	84	296	228	228	151	107	177	308	465	465
8	80	44	62	75	285	220	216	141	98	168	301	457	457
9	73	37	55	66	273	211	202	131	89	158	294	448	448
10	66	31	48	58	260	201	189	120	80	148	286	439	439
11	59	25	41	49	247	191	174	110	72	137	277	428	428
12	52	21	35	41	233	181	160	99	63	126	268	417	417
13	45	18	30	35	219	170	146	88	55	116	258	405	405
14	39	16	26	30	204	160	132	78	47	105	248	392	392
15	34	14	22	27	190	149	118	69	41	95	238	380	380
16	30	12	20	23	176	138	105	60	36	85	228	366	366
17	27	11	17	21	162	127	93	53	32	75	217	352	352
18	24	9	16	18	148	117	83	48	29	67	206	338	338
19	21	9	14	17	135	107	75	43	26	60	195	324	324
20	19	8	13	15	122	97	67	39	23	54	185	310	310
21	17	7	11	14	111	88	61	35	21	49	174	295	295
22	16	6	10	12	101	80	56	32	19	45	163	281	281
23	14	6	10	11	92	74	51	29	18	41	153	267	267
24	13	5	9	10	85	68	47	27	16	38	143	253	253
25	12	5	8	10	78	62	43	25	15	35	133	239	239
26	11	5	7	9	72	58	40	23	14	32	123	225	225
27	11	4	7	8	67	53	37	21	13	30	114	212	212
28	10	4	6	8	62	50	34	20	12	28	106	198	198
29	9	4	6	7	58	46	32	18	11	26	99	185	185
30	9	3	6	7	54	43	30	17	10	24	93	173	173
32	7	3	5	6	48	38	26	15	9	21	81	152	152
34	7	3	4	5	42	34	23	13	8	19	72	135	135
36	6	2	4	5	38	30	21	12	7	17	64	120	120
38	5	2	3	4	34	27	19	11	6	15	58	108	108
40	5	2	3	4	30	24	17	10	6	14	52	97	97
	0	0	0	0	0	0	0	0	0	0	0	0	0

Tensile Capacities

Brace Capacity in Tension	
Brace	[k]
HSS4X.250	114
HSS3X.250	84
HSS3.5X.250	99
HSS3.5X.313	121
HSS6x.500	335
HSS6x.375	257
HSS5x.500	274
HSS4.5x.375	188
HSS4x.313	140
HSS5x.375	211
HSS7.625x.375	330
HSS8.625X.500	493
HSS8.625X.500	493

Frame Assignments

Story	Label	niqueNam	esign Typ	Length	Analysis Section	Design Section	A
				in			
Upper	D18	46	Brace	10.5833	HSS3X.250	HSS3X.250	
Upper	D20	74	Brace	12.2944	HSS3X.250	HSS3X.250	
Upper	D23	48	Brace	11.3796	HSS3X.250	HSS3X.250	
Upper	D12	110	Brace	12.3471	HSS3X.250	HSS3X.250	
Upper	D21	44	Brace	5.2917	HSS3X.250	HSS3X.250	
Upper	D22	45	Brace	5.2917	HSS3X.250	HSS3X.250	
Upper	D39	50	Brace	5.6898	HSS3X.250	HSS3X.250	
Upper	D40	51	Brace	5.6898	HSS3X.250	HSS3X.250	
Upper	D45	66	Brace	6.1472	HSS3X.250	HSS3X.250	
Upper	D46	73	Brace	6.1472	HSS3X.250	HSS3X.250	
Upper	D69	107	Brace	6.1735	HSS3X.250	HSS3X.250	
Upper	D70	108	Brace	6.1735	HSS3X.250	HSS3X.250	
Upper	D2	98	Brace	11.1181	HSS3X.250	HSS3X.250	
Upper	D7	92	Brace	5.5591	HSS3X.250	HSS3X.250	
Upper	D9	93	Brace	5.5591	HSS3X.250	HSS3X.250	
Upper	D8	29	Brace	5.2746	HSS3X.250	HSS3X.250	
Upper	D19	30	Brace	5.2746	HSS3X.250	HSS3X.250	
Upper	D1	32	Brace	10.5492	HSS3X.250	HSS3X.250	
Main	D18	41	Brace	12.0787	HSS3X.250	HSS3X.250	
Main	D20	79	Brace	13.603	HSS3X.250	HSS3X.250	
Main	D23	53	Brace	12.7822	HSS3X.250	HSS3X.250	
Main	D14	39	Brace	6.0394	HSS3X.250	HSS3X.250	
Main	D15	40	Brace	6.0394	HSS3X.250	HSS3X.250	
Main	D43	55	Brace	6.3911	HSS3X.250	HSS3X.250	
Main	D44	56	Brace	6.3911	HSS3X.250	HSS3X.250	
Main	D49	76	Brace	6.8015	HSS3X.250	HSS3X.250	
Main	D50	77	Brace	6.8015	HSS3X.250	HSS3X.250	
Main	D2	102	Brace	12.55	HSS3X.250	HSS3X.250	
Main	D10	104	Brace	6.275	HSS3X.250	HSS3X.250	
Main	D11	106	Brace	6.275	HSS3X.250	HSS3X.250	
Main	D3	25	Brace	6.0244	HSS3X.250	HSS3X.250	
Main	D4	26	Brace	6.0244	HSS3X.250	HSS3X.250	
Main	D1	27	Brace	12.0488	HSS3X.250	HSS3X.250	

Brace Design - Max Brace Forces

Level	Label	Unique Name	Group Name	Member Size	Member Length [ft]	Max Force [k]
Upper	D18	46	BF-8-U	HSS3X.250	10.6	18.70094
Upper	D20	74	BF-G-U	HSS3X.250	12.3	41.00614
Upper	D23	48	BF-6-U	HSS3X.250	11.4	18.08823
Upper	D12	110	BF-1	HSS3X.250	12.3	11.24072
Upper	D21	44	BF-8-U	HSS3X.250	5.3	18.74596
Upper	D22	45	BF-8-U	HSS3X.250	5.3	18.7254
Upper	D39	50	BF-6-U	HSS3X.250	5.7	17.75383
Upper	D40	51	BF-6-U	HSS3X.250	5.7	17.74358
Upper	D45	66	BF-G-U	HSS3X.250	6.1	40.8151
Upper	D46	73	BF-G-U	HSS3X.250	6.1	40.80035
Upper	D69	107	BF-1	HSS3X.250	6.2	11.26495
Upper	D70	108	BF-1	HSS3X.250	6.2	11.26082
Upper	D2	98	BF-D-U	HSS3X.250	11.1	24.45218
Upper	D7	92	BF-D-U	HSS3X.250	5.6	23.86459
Upper	D9	93	BF-D-U	HSS3X.250	5.6	23.84855
Upper	D8	29	BF-4-U	HSS3X.250	5.3	26.48486
Upper	D19	30	BF-4-U	HSS3X.250	5.3	26.4525
Upper	D1	32	BF-4-U	HSS3X.250	10.5	25.566
Main	D18	41	BF-8-M	HSS3X.250	12.1	18.63975
Main	D20	79	BF-G-M	HSS3X.250	13.6	6.436752
Main	D23	53	BF-6-M	HSS3X.250	12.8	10.10183
Main	D14	39	BF-8-M	HSS3X.250	6.0	18.62255
Main	D15	40	BF-8-M	HSS3X.250	6.0	18.58897
Main	D43	55	BF-6-M	HSS3X.250	6.4	10.24937
Main	D44	56	BF-6-M	HSS3X.250	6.4	10.23675
Main	D49	76	BF-G-M	HSS3X.250	6.8	6.286636
Main	D50	77	BF-G-M	HSS3X.250	6.8	6.253979
Main	D2	102	BF-D-M	HSS3X.250	12.6	7.070624
Main	D10	104	BF-D-M	HSS3X.250	6.3	0
Main	D11	106	BF-D-M	HSS3X.250	6.3	0.045901
Main	D3	25	BF-4-M	HSS3X.250	6.0	14.67423
Main	D4	26	BF-4-M	HSS3X.250	6.0	14.71287
Main	D1	27	BF-4-M	HSS3X.250	12.0	15.57208

Brace Design

Group Name	Member Size	Max Length [ft]	Override Max Length [ft]	Max Force [k]		Compressive Capacity [k]	Tensile Capacity [k]	DCR
BF-1	HSS3X.250	12.3	6.2	11.3		51	84	0.22
BF-4-U	HSS3X.250	10.5	6.0	26.5		59	84	0.45
BF-6-U	HSS3X.250	11.4	5.7	18.1		59	84	0.31
BF-D-U	HSS3X.250	11.1	5.6	24.5		59	84	0.42
BF-G-U	HSS3X.250	12.3	6.1	41.0		51	84	0.80
BF-G-M	HSS3X.250	13.6	6.8	6.4		51	84	0.13
BF-D-M	HSS3X.250	12.6	6.3	7.1		51	84	0.14
BF-6-M	HSS3X.250	12.8	6.4	10.2		51	84	0.20
BF-4-M	HSS3X.250	12.0	6.0	15.6		51	84	0.30
BF-8-U	HSS3X.250	10.6	5.3	18.7		59	84	0.32
BF-8-M	HSS3X.250	12.1	6.0	18.6		51	84	0.36

GRAVITY LOADS - COLUMNS SUMMARY

*MAIN ARE TOTAL FORCE. Including snow&live for live

			DL [k]	Self Wt DL [k	SL [k]	LL	Total DL [k]	Total SL/LL [	1.2D+1.6L
COL4-1U	1U	COL4	2.2	0.33	1.6		2.53	1.6	5.59
COL4-1M	1M	COL4	2.8	0.39	1.6	0.24	3.19	1.84	6.77
COL4-2U	2U	COL4	11.6	0.33	8.7		11.93	8.7	28.23
COL4-2M	2M	COL4	16.9	0.39	8.7	2.8	17.29	11.5	39.14
COL6-1U	1U	COL6	1.6	0.33	1.2		1.93	1.2	4.23
COL6-1M	1M	COL6	5.4	0.39	1.2	1.95	5.79	3.15	11.98
COL6-2U	2U	COL6	5.9	0.33	4.1		6.23	4.1	14.03
COL6-2M	2M	COL6	9.7	0.39	4.1	1.95	10.09	6.05	21.78
COL8-1U	1U	COL8	3.5	0.33	2.8		3.83	2.8	9.07
COL8-1M	1M	COL8	10.1	0.39	2.8	2.8	10.49	5.6	21.54
COL8-2U	2U	COL8	3.5	0.33	3.4		3.83	3.4	10.03
COL8-2M	2M	COL8	8.3	0.39	3.4	4.8	8.69	8.2	23.54
COLD-1U	1U	COLD	2.4	0.33	2.1		2.73	2.1	6.63
COLD-1M	1M	COLD	3.7	0.39	2.1	0.64	4.09	2.74	9.29
COLD-2U	2U	COLD	5.3	0.33	4.3		5.63	4.3	13.63
COLD-2M	2M	COLD	20.4	0.39	4.3	6.9	20.79	11.2	42.86
COLG-1U	1U	COLG	4.3	0.33	3.9		4.63	3.9	11.79
COLG-1M	1M	COLG	17.2	0.39	3.9	6.5	17.59	10.4	37.74
COLG-2U	2U	COLG	1.9	0.33	1.5		2.23	1.5	5.07
COLG-2M	2M	COLG	12.5	0.39	1.5	5.6	12.89	7.1	26.82
COL1-1	1	COL1	3.2	0.33	2.2		3.53	2.2	7.75
COL1-2	2	COL1	3.2	0.33	2.2		3.53	2.2	7.75

GRAVITY LOADS - BEAMS SUMMARY

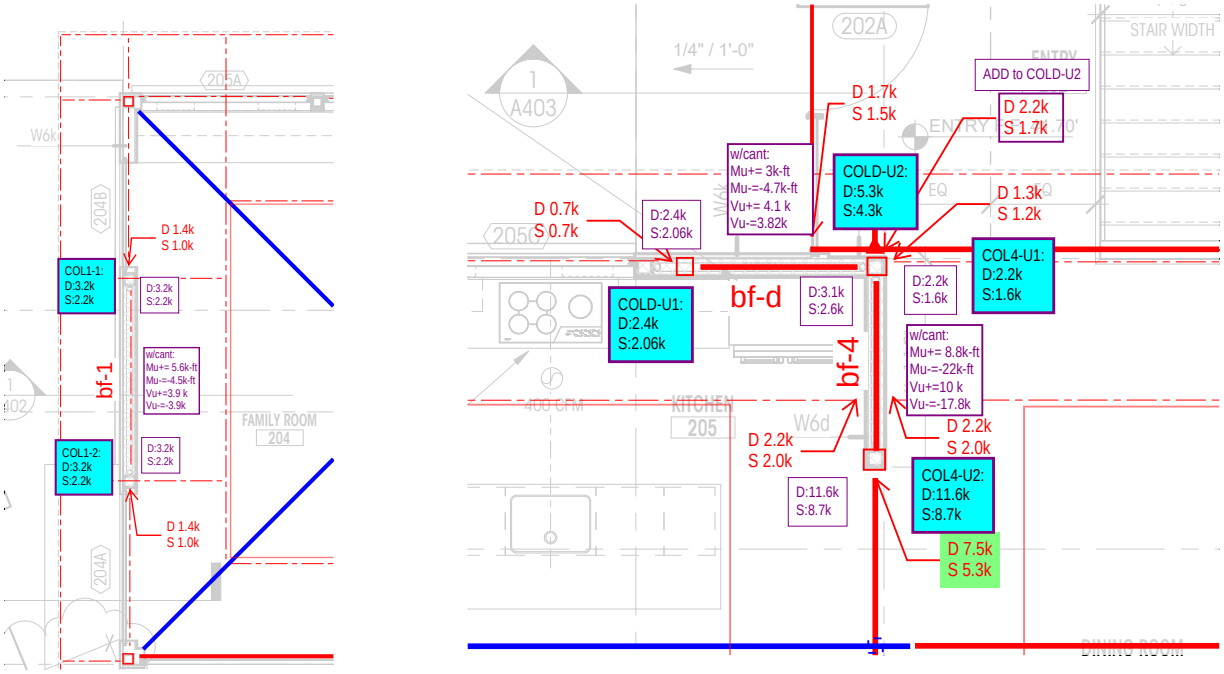
	kips		kip-ft	
Beam #	Vu	phiVn	Mu	phiMn
U-BM1	3.9	147	5.6	320
U-BM4	17.8	147	22	320
U-BM6	7.4	147	9.1	320
U-BM8	6.1	147	7	320
U-BMD	4.1	153.9	4.7	263
U-BMG	10.5	147	27	320

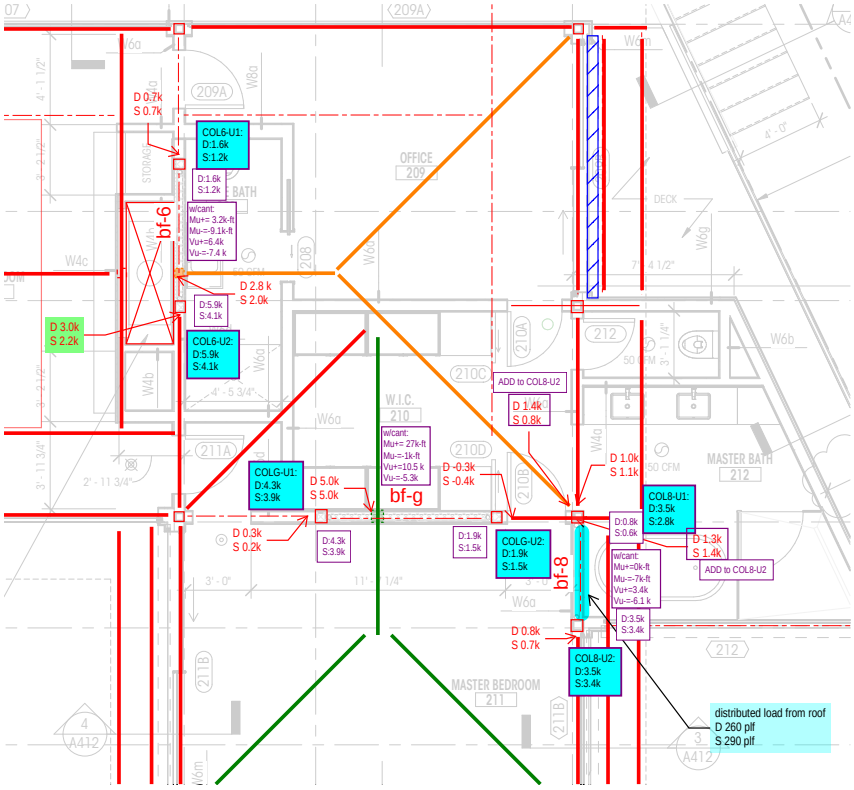
	kips		kip-ft	
Beam #	Vu	phiVn	Mu	phiMn
M-BM4	7.1	135	8.2	269.6
M-BM6	4.9	135	5.5	269.6
M-BM8	7.1	135	8.5	269.6
M-BMD	18.62	135	23.2	269.6
M-BMG	13.1	135	23	269.6

NOTE: THESE GRAVITY LOADS ARE USED IN COMBINATION WITH THE EXPECTED STRENGTH COMPONENTS OF THE BRACES TO CHECK THE BRACED FRAME BEAMS AND COLUMN FOR COMBINED GRAVITY AND LATERAL LOADING. THE FOLLOWING PAGES SHOW THE CALCULATED GRAVITY LOADS AT THE BRACED FRAME BEAMS AND COLUMNS AT BOTH THE ROOF LEVEL AND MAIN FLOOR LEVEL.

GRAVITY LOADS AT BRACED FRAMES

ROOF BRACED FRAME BEAM/COLUMN LOADS





STRUCTURAL ENGINEERING

8480 Residence
PROJECT

Lateral Design 30

10/24/22
DATE

PROJ. # SRW
DESIGN

SHEET

Beam Analysis

Beam:		BF-D ROOF				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.202**		**0.153**	0.395	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**1.7**		**1.5**	3.540	**4**
	P_2	**0.7**		**0.7**	1.540	**0.00**
	P_3	**1.3**		**1.2**	2.760	**8.50**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**8.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	4.884	**1.25**
R_2	6.317	**7.25**
R_3	0.000	**7.25**
R_4	0.000	**7.25**
R_5	0.000	**7.25**
R_6	0.000	**7.25**
R_7	0.000	**7.25**
R_8	0.000	**7.25**
R_9	0.000	**7.25**
R_{10}	0.000	**7.25**

Demand Output		
Location, ft		**6.92**
Shear, k	V =	-2.93
Moment, k-ft	M =	-2.77
Deflection, in	Δ =	0.00
Δ/Span		L/481169

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.00**

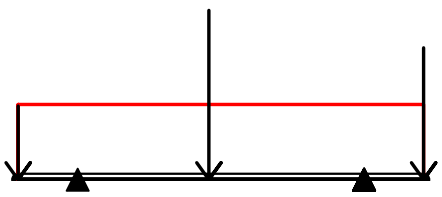
Stresses @ Input Location	
f_v (psi)	-572
f_b (psi)	-550

Max/Min Stresses	
f_{v_MAX} (psi)	797
f_{v_MIN} (psi)	-743
f_{b_MAX} (psi)	1009
f_{b_MIN} (psi)	-940

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**272**
S (in^3)	**60.4**
A (in^2)	**5.13**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W8x67		
F_y, ksi	50		
Beam Weight (plf)	67		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	5.13 in^2	V_n/Ω_v	103 k
S	60.4 in^3	$\phi_v V_n$	154 k
Z	70.1 in^3	M_n/Ω_b	175 k-ft
I	272 in^4	$\phi_b M_n$	263 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-2.57	-2.83	-	0.001 (↑)	0	L/29784	0	-	L/∞
Span 1	3.55	-3.81	-4.73	5.08	-0.002 (↓)	4.05	L/35904	0	-	L/∞
Right Cantilever	4.09	-	-4.73	-	0	-	L/∞	0	-	L/∞

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PROJECT 8480

DATE *10/24/2022*

BF Beams

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		BF-4 ROOF				
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.112**		**0.05**	0.214	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**4.4**		**4**	11.680	**3.25**
	P_2	**0.8**		**0.5**	1.760	**10.00**
	P_3	**7.5**		**5.3**	17.480	**0.00**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**10.00**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	27.811	**1.25**
R_2	5.253	**6.00**
R_3	0.000	**6.00**
R_4	0.000	**6.00**
R_5	0.000	**6.00**
R_6	0.000	**6.00**
R_7	0.000	**6.00**
R_8	0.000	**6.00**
R_9	0.000	**6.00**
R_{10}	0.000	**6.00**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	-0.01
Δ/Span		L/4796

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

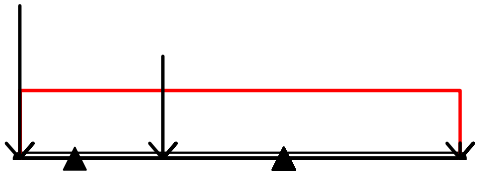
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	2058
f_{v_MIN} (psi)	-3629
f_{b_MAX} (psi)	0
f_{b_MIN} (psi)	-3490

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**394**
S (in^3)	**75.7**
A (in^2)	**4.89**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W10x68		
F_y, ksi	50		
Beam Weight (plf)	68		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.89 in^2	V_n/Ω_v	97.8 k
S	75.7 in^3	$\phi_v V_n$	147 k
Z	85.3 in^3	M_n/Ω_b	213 k-ft
I	394 in^4	$\phi_b M_n$	320 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-17.75	-22.02	-	-0.005 (↓)	0	L/5952	0	-	L/∞
Span 1	10.06	-2.64	-22.02	-	0.002 (↑)	3.2	L/28440	0	-	L/∞
Right Cantilever	2.62	-	-8.76	-	-0.012 (↓)	10	L/7960	0	-	L/∞

PROJECT 8480

DATE *10/24/2022*

BF Beams

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		bf-1 ROOF				
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.203**		**0.153**	0.488	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**1.5**		**0.8**	3.080	**5.25**
	P_2	**1.4**		**1**	3.280	**0.00**
	P_3	**1.4**		**1**	3.280	**10.50**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**10.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	7.384	**1.25**
R_2	7.384	**9.25**
R_3	0.000	**9.25**
R_4	0.000	**9.25**
R_5	0.000	**9.25**
R_6	0.000	**9.25**
R_7	0.000	**9.25**
R_8	0.000	**9.25**
R_9	0.000	**9.25**
R_{10}	0.000	**9.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/53157

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

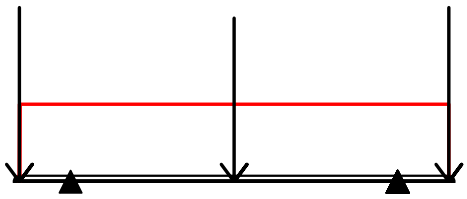
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	796
f_{v_MIN} (psi)	-796
f_{b_MAX} (psi)	885
f_{b_MIN} (psi)	-710

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**394**
S (in^3)	**75.7**
A (in^2)	**4.89**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W10x68	
F_y, ksi		50	
Beam Weight (plf)		68	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.89 in^2	V_n/Ω_v	97.8 k
S	75.7 in^3	$\phi_v V_n$	147 k
Z	85.3 in^3	M_n/Ω_b	213 k-ft
I	394 in^4	$\phi_b M_n$	320 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-3.89	-4.48	-	0	-	L/∞	0	-	L/∞
Span 1	3.49	-3.49	-4.48	5.59	-0.003 (↓)	5.25	L/31920	0	-	L/∞
Right Cantilever	3.89	-	-4.48	-	0	-	L/∞	0	-	L/∞

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PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		bf-6 ROOF				
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.112**		**0.05**	0.214	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**2.8**		**2**	6.560	**3.25**
	P_2	**3**		**2.2**	7.120	**0.00**
	P_3	**0.7**		**0.7**	1.960	**9.50**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**9.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	13.746	**1.25**
R_2	3.931	**8.25**
R_3	0.000	**8.25**
R_4	0.000	**8.25**
R_5	0.000	**8.25**
R_6	0.000	**8.25**
R_7	0.000	**8.25**
R_8	0.000	**8.25**
R_9	0.000	**8.25**
R_{10}	0.000	**8.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/25741

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

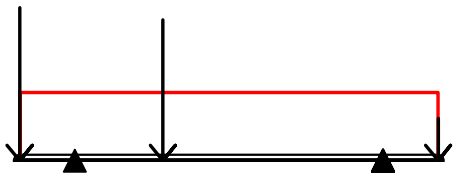
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	1300
f_{v_MIN} (psi)	-1511
f_{b_MAX} (psi)	509
f_{b_MIN} (psi)	-1437

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**394**
S (in^3)	**75.7**
A (in^2)	**4.89**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W10x68		
F_y, ksi	50		
Beam Weight (plf)	68		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.89 in^2	V_n/Ω_v	97.8 k
S	75.7 in^3	$\phi_v V_n$	147 k
Z	85.3 in^3	M_n/Ω_b	213 k-ft
I	394 in^4	$\phi_b M_n$	320 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-7.39	-9.07	-	-0.001 (↓)	0	L/29640	0	-	L/∞
Span 1	6.36	-1.70	-9.07	3.21	-0.001 (↓)	4.45	L/83904	0	-	L/∞
Right Cantilever	2.23	-	-2.62	-	0	-	L/∞	0	-	L/∞

PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		BF-G ROOF				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.112**		**0.05**	0.214	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**5**		**5**	14.000	**4**
	P_2	**0.3**		**0.2**	0.680	**0.00**
	P_3	**-0.3**		**-0.4**	-1.000	**10.75**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**10.75**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	11.420	**1.25**
R_2	4.565	**9.50**
R_3	0.000	**9.50**
R_4	0.000	**9.50**
R_5	0.000	**9.50**
R_6	0.000	**9.50**
R_7	0.000	**9.50**
R_8	0.000	**9.50**
R_9	0.000	**9.50**
R_{10}	0.000	**9.50**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.01
Δ/Span		L/2555

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

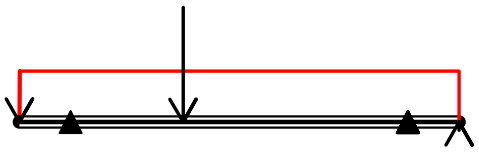
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	2142
f_{v_MIN} (psi)	-1083
f_{b_MAX} (psi)	4274
f_{b_MIN} (psi)	-161

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**394**
S (in^3)	**75.7**
A (in^2)	**4.89**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W10x68		
F_y, ksi	50		
Beam Weight (plf)	68		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.89 in^2	V_n/Ω_v	97.8 k
S	75.7 in^3	$\phi_v V_n$	147 k
Z	85.3 in^3	M_n/Ω_b	213 k-ft
I	394 in^4	$\phi_b M_n$	320 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-0.95	-1.02	-	0.008 (↑)	0	L/3742	0	-	L/∞
Span 1	10.47	-5.30	-1.02	26.96	-0.017 (↓)	5.07	L/5797	0	-	L/∞
Right Cantilever	-0.73	-	-	1.08	0.007 (↑)	10.75	L/4276	0	-	L/∞

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PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		BF-8 ROOF				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.328**		**0.29**	0.858	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**0.8**		**0.7**	2.080	**0**
	P_2	**1**		**1.1**	2.960	**0.00**
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**7.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	9.516	**1.25**
R_2	1.956	**6.25**
R_3	0.000	**6.25**
R_4	0.000	**6.25**
R_5	0.000	**6.25**
R_6	0.000	**6.25**
R_7	0.000	**6.25**
R_8	0.000	**6.25**
R_9	0.000	**6.25**
R_{10}	0.000	**6.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/15053

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

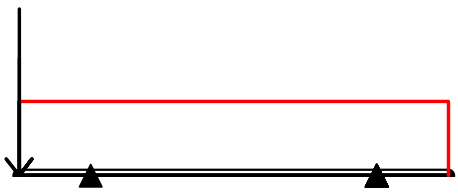
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	696
f_{v_MIN} (psi)	-1250
f_{b_MAX} (psi)	0
f_{b_MIN} (psi)	-1105

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**394**
S (in^3)	**75.7**
A (in^2)	**4.89**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W10x68	
F_y, ksi		50	
Beam Weight (plf)		68	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.89 in^2	V_n/Ω_v	97.8 k
S	75.7 in^3	$\phi_v V_n$	147 k
Z	85.3 in^3	M_n/Ω_b	213 k-ft
I	394 in^4	$\phi_b M_n$	320 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-6.11	-6.97	-	-0.001 (↓)	0	L/29880	0	-	L/∞
Span 1	3.40	-0.88	-6.97	-	0.001 (↑)	2.97	L/59760	0	-	L/∞
Right Cantilever	1.07	-	-0.67	-	0	-	L/∞	0	-	L/∞

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PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

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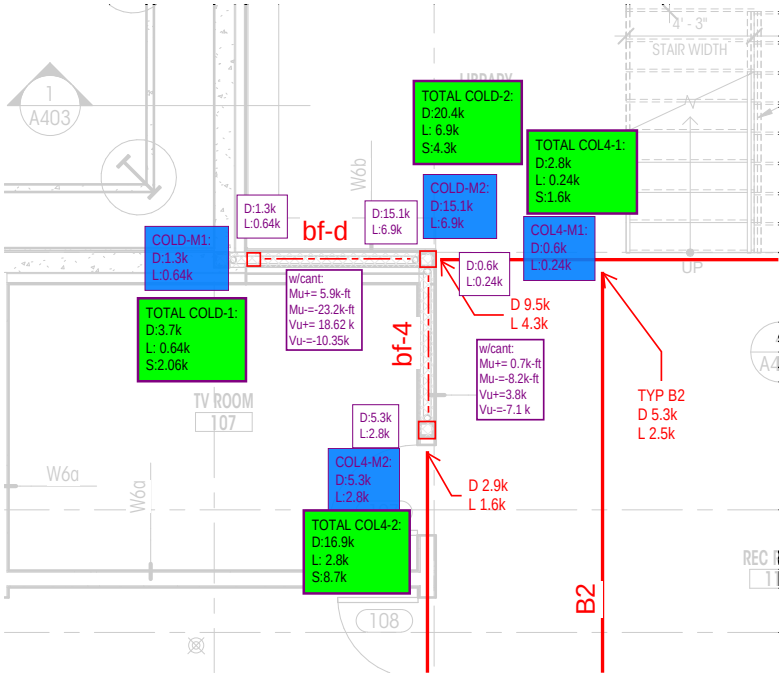
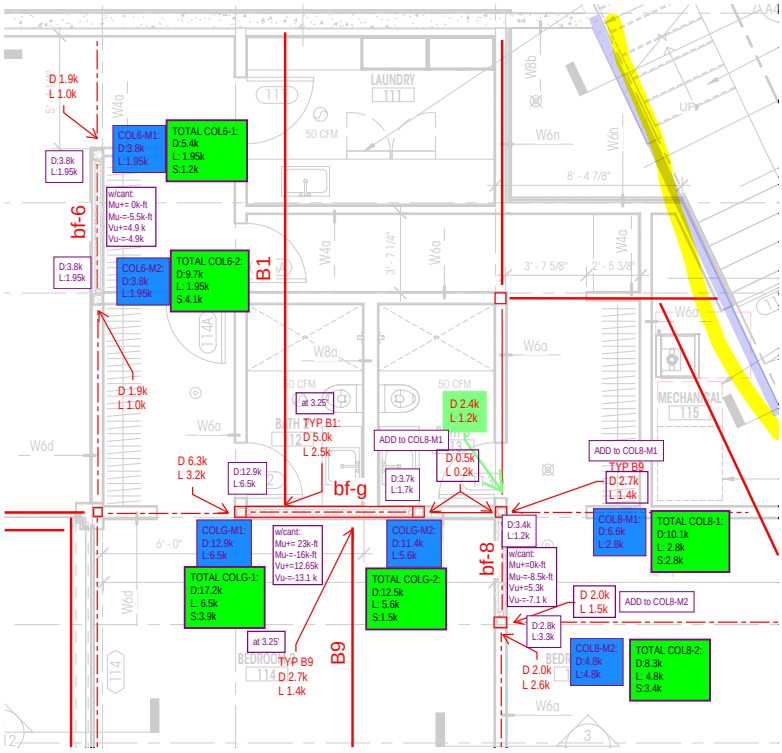
STRUCTURAL ENGINEERING

GRAVITY LOADS AT BRACED FRAMES

MAIN FLOOR BRACED FRAME BEAM/COLUMN LOADS





Beam Analysis

Beam:		BF-D MAIN FLOOR				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.230**	**0.100**		0.436	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**5.3**	**2.5**		10.360	**3**
	P_2	**9.5**	**4.3**		18.280	**7.25**
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**7.25**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	2.623	**0.00**
R_2	29.178	**6.00**
R_3	0.000	**6.00**
R_4	0.000	**6.00**
R_5	0.000	**6.00**
R_6	0.000	**6.00**
R_7	0.000	**6.00**
R_8	0.000	**6.00**
R_9	0.000	**6.00**
R_{10}	0.000	**6.00**

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

Stresses @ Input Location	
f_v (psi)	292
f_b (psi)	1104

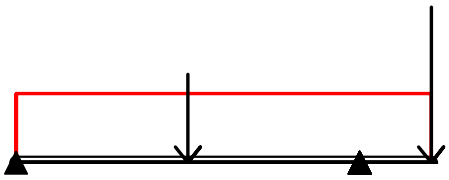
Max/Min Stresses	
f_{v_MAX} (psi)	4174
f_{v_MIN} (psi)	-2296
f_{b_MAX} (psi)	1102
f_{b_MIN} (psi)	-4335

Demand Output		
Location, ft		**3.00**
Shear, k	V =	1.31
Moment, k-ft	M =	5.91
Deflection, in	Δ =	0.00
Δ/Span		L/262682

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**391**
S (in^3)	**64.2**
A (in^2)	**4.51**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	2.62	-10.35	-23.19	5.89	0.001 (↑)	4.92	L/71862	0	-	L/∞
Right Cantilever	18.82	-	-23.19	-	-0.004 (↓)	7.25	L/7482	-0.001 (↓)	7.25	L/23700

PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

STRUCTURAL ENGINEERING

Beam Analysis

Beam:		BF-4 MAIN FLOOR				
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.410**	**0.200**		0.812	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**2.9**	**1.6**		6.040	
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**7.25**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	10.855	**1.25**
R_2	1.072	**7.25**
R_3	0.000	**7.25**
R_4	0.000	**7.25**
R_5	0.000	**7.25**
R_6	0.000	**7.25**
R_7	0.000	**7.25**
R_8	0.000	**7.25**
R_9	0.000	**7.25**
R_{10}	0.000	**7.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/12696

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

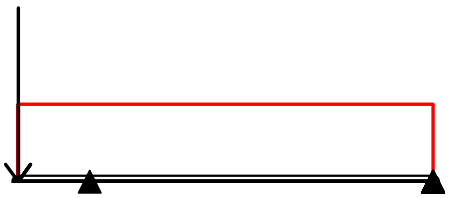
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	843
f_{v_MIN} (psi)	-1564
f_{b_MAX} (psi)	132
f_{b_MIN} (psi)	-1530

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**391**
S (in^3)	**64.2**
A (in^2)	**4.51**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-7.06	-8.18	-	-0.002 (↓)	0	L/14964	-0.001 (↓)	0	L/30000
Span 1	3.80	-1.07	-8.18	0.71	0.001 (↑)	2.93	L/71862	0	-	L/∞

PROJECT 8480 DATE *10/24/2022*

BF Beams PROJ. #

DESIGN SRW

SHEET

STRUCTURAL ENGINEERING

Beam Analysis

Beam:		bf-6 MAIN FLOOR				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.410**	**0.200**		0.812	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**1.9**	**1**		3.880	**0**
	P_2	**1.9**	**1**		3.880	**9.50**
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**9.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	7.737	**1.25**
R_2	7.737	**8.25**
R_3	0.000	**8.25**
R_4	0.000	**8.25**
R_5	0.000	**8.25**
R_6	0.000	**8.25**
R_7	0.000	**8.25**
R_8	0.000	**8.25**
R_9	0.000	**8.25**
R_{10}	0.000	**8.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/16054

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

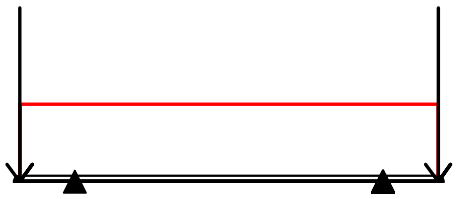
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	1085
f_{v_MIN} (psi)	-1085
f_{b_MAX} (psi)	0
f_{b_MIN} (psi)	-1025

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**391**
S (in^3)	**64.2**
A (in^2)	**4.51**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-4.90	-5.48	-	-0.001 (↓)	0	L/29640	-0.001 (↓)	0	L/30000
Span 1	2.84	-2.84	-5.48	-	0.001 (↑)	4.75	L/83904	0	-	L/∞
Right Cantilever	4.89	-	-5.48	-	-0.001 (↓)	9.5	L/29640	-0.001 (↓)	9.5	L/59808

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PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

Beam Analysis

Beam:		BF-G MAIN FLOOR				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.194**	**0.080**		0.361	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**7.7**	**3.9**		15.480	**4.5**
	P_2	**6.3**	**3.2**		12.680	**0.00**
	P_3	**0.5**	**0.2**		0.920	**10.75**
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**10.75**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	25.783	**1.25**
R_2	7.176	**9.50**
R_3	0.000	**9.50**
R_4	0.000	**9.50**
R_5	0.000	**9.50**
R_6	0.000	**9.50**
R_7	0.000	**9.50**
R_8	0.000	**9.50**
R_9	0.000	**9.50**
R_{10}	0.000	**9.50**

Demand Output		
Location, ft		**8.25**
Shear, k	V =	-5.35
Moment, k-ft	M =	5.54
Deflection, in	Δ =	-0.01
Δ/Span		L/12936

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

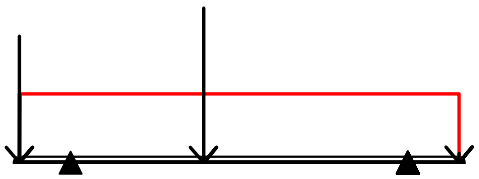
Stresses @ Input Location	
f_v (psi)	-1187
f_b (psi)	1036

Max/Min Stresses	
f_{v_MAX} (psi)	2805
f_{v_MIN} (psi)	-2912
f_{b_MAX} (psi)	4303
f_{b_MIN} (psi)	-3015

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**391**
S (in^3)	**64.2**
A (in^2)	**4.51**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-13.13	-16.13	-	0.003 (↑)	0	L/9976	0.001 (↑)	0	L/30000
Span 1	12.65	-5.80	-16.13	23.02	-0.013 (↓)	5.29	L/7581	-0.004 (↓)	5.31	L/24639
Right Cantilever	1.37	-	-1.43	-	0.006 (↑)	10.75	L/4988	0.002 (↑)	10.75	L/15708

SWENSON SAY FAGET
SEATTLE 2124 Third Ave, Suite 100, Seattle, WA 98121 | O 206.443.6212
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PROJECT 8480 — DATE *10/24/2022*

BF Beams — PROJ. #

DESIGN SRW

SHEET

STRUCTURAL ENGINEERING

Beam Analysis

Beam:		BF-8 MAIN FLOOR				
Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	**0.230**	**0.100**		0.436	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	**2**	**2.6**		6.560	**0**
	P_2	**2.4**	**1.2**		4.800	**7.50**
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		**7.50**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	8.635	**1.25**
R_2	5.995	**6.25**
R_3	0.000	**6.25**
R_4	0.000	**6.25**
R_5	0.000	**6.25**
R_6	0.000	**6.25**
R_7	0.000	**6.25**
R_8	0.000	**6.25**
R_9	0.000	**6.25**
R_{10}	0.000	**6.25**

Demand Output		
Location, ft		**0.00**
Shear, k	V =	0.00
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/7582

Load Factors	
Dead	**1.20**
Live	**1.60**
Snow	**1.60**

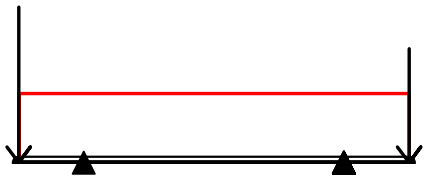
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	0

Max/Min Stresses	
f_{v_MAX} (psi)	1185
f_{v_MIN} (psi)	-1575
f_{b_MAX} (psi)	0
f_{b_MIN} (psi)	-1596

Beam Properties	
E (ksi)	**29000**
b (in)	**5.125**
d (in)	**12**
I (in^4)	**391**
S (in^3)	**64.2**
A (in^2)	**4.51**
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbraced Length (L_b), ft		0	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-7.11	-8.54	-	-0.003 (↓)	0	L/9960	-0.001 (↓)	0	L/30000
Span 1	1.53	-0.65	-8.54	-	0.002 (↑)	3.69	L/29880	0.001 (↑)	3.59	L/59760
Right Cantilever	5.34	-	-6.34	-	-0.003 (↓)	7.5	L/9960	-0.001 (↓)	7.5	L/28468

PROJECT 8480

BF Beams

DATE *10/24/2022*

PROJ. #

DESIGN SRW

SHEET

STRUCTURAL ENGINEERING

ONE-STORY BRACED FRAME: COLUMN & BEAM DESIGN

Start at ID	0	
End at ID	1	
Brace Fy	46	ksi
Brace Fu	62	ksi
Beam Fy	50	ksi
W Column Fy	50	ksi
HSS Column Fy	50	ksi
E	29000	ksi
Sds	1.172	
ϕ	0.9	

						Expected Tensile Strength			Expected Compressive Strength								
MaxID	ID	BF Name	# Stories	Brace Size	Length [ft]	Ry	Ag [in2]	$P_{tension}$ [k]	K	ry	KL/r	4.71√(E/RyFy)	F_e [ksi]	F_{cre} [ksi]	P_{comp} [k]	$P_{post\text{-}buckling}$ [k]	Angle
						AISC 341: Table A3.1		AISC 341 F2.3: RyFyAg	assuming pin-pin				AISC 360 Eqn E3-4: $\pi^2E/(L/c)^2$	AISC 360 Eqn E3-2/E3-3, Include Ry with Fy	AISC 341 F2.3: min(RyFyAg, 1.14FcreAg)	AISC 341 F2.3: 0.3Pcomp	
1	1	BF-1	1	HSS3X.250	6.17	1.4	2.03	131	1	0.982	75	100	50	38	87	26	49
	0																

	Summary of Expected Strengths										Column Length		Ram Steel Col ID		Loads					
	Tensile Strength			Compressive Strength			Post-Buckling Strength													
BF Name	Diag [k]	Vert [k]	Horiz [k]	Diag [k]	Vert [k]	Horiz [k]	Diag [k]	Vert [k]	Horiz [k]	BF Name	Hframe [ft]	Lb Col	Col 1	Col 2	Pu,E (T) [k]	Pu,E (C)[k]	Col 1, Pu, Dead [k]	Col 2, Pu, Dead [k]	Col 1, Pu, Live [k]	Col 2, Pu, Live [k]
												Reenter length. Assuming full height			For seismic - Pu,E is vertical component of expected brace strength. Gravity reactions ~~from RAM Steel~~ are at foundation.					
BF-1	131	99	86	87	66	57	26	20	17	BF-1	9.33	10	COL1-1	COL1-2	66	99	3.5	3.5	2.2	2.2

	Columns															
	Load Combinations				Design Force				Ductility Check			Capacities		DCR		
BF Name	1.4D [k]	1.2D+1.6L [k]	(1.2+0.2Sds)D+ 0.5L+E [k]	(0.9-0.2Sds) D+E [k]	Design Force, C [k]	Design Force, T [k]	Column Size	Ry	bf/2tf (or b/t)	λhd	Highly ductile?	ϕcPn [k]	ϕtPn [k]	DCR Comp	DCR Tens
							Pick size	AISC 341: Table A3.1		AISC 341 Table D1.1: λhd=0.32√(E/RyFy)				Pu/ϕcPn	Pu/ϕtPn
BF-1	5	8	105	68	105	68	HSS6X6X5/8	1.4	7.33	13.23	OK	421.01	526.5	✓ 0.25	✓ 0.13

ONE-STORY BRACED FRAME: COLUMN & BEAM DESIGN

Start at ID	0	
End at ID	1	
Brace Fy	46	ksi
Brace Fu	62	ksi
Beam Fy	50	ksi
W Column Fy	50	ksi
HSS Column Fy	50	ksi
E	29000	ksi
Sds	1.172	
ϕ	0.9	

		Beam Length [ft]					Loads				
BF Name	BF Name	Beam Length [ft]	ry	Lb, calc [ft]	Lb - use [ft]	Ram Steel Beam ID	Pu,E [k]	Vu [k]	Mu [k-ft]	Beam Size	Ry
				AISC 341 D1.2a. Beam bracing, Max spacing	AISC 341 D1.2a. Assuming braced at midspan					Pick size	AISC 341: Table A3.1
BF-1	BF-1	8.09	2.59	9.6	8	U-BM1	143	4	6	W10x68	1.1

	Beams															
	Ductility Check - Highly Ductile								Capacities			DCR				
BF Name	Ag	bf/2tf	Flanges, λhd	Check Flanges	h/tw	Ca	Webs, λhd	Check Web	ϕPn [k]	ϕMn [k-ft]	ϕVn [k]	Pu/ϕPn		Mu/ϕMn		Vu/ϕVn		Combined
			AISC 341 Table D1.1: λhd=0.32√(E/RyFy)			AISC 341 Table D1.1	AISC 341 Table D1.1: λhd, webs											AISC 360: Eqn H1-1a/H1-1b
BF-1	19.9	6.6	7	OK	17	0.14	51.2	OK	809.9	320	147	✓	0.18	✓	0.02	✓	0.03	✓	0.11

TWO-STORY BRACED FRAME: COLUMN & BEAM DESIGN

Start at ID	1	
End at ID	11	
Brace Fy	46	ksi
Brace Fu	62	ksi
Beam Fy	50	ksi
W Column Fy	50	ksi
HSS Column Fy	50	ksi
E	29000	ksi
Sds	1.172	
ϕ	0.9	

						Expected Tensile Strength			Expected Compressive Strength								
MaxID	ID	BF Name	# Stories	Brace Size	Length [ft]	Ry	Ag [in2]	$P_{tension}$ [k]	K	ry	KL/r	4.71√(E/RyFy)	F_e [ksi]	F_{cre} [ksi]	P_{comp} [k]	$P_{post\text{-}buckling}$ [k]	Angle
						AISC 341: Table A3.1		AISC 341 F2.3: RyFyAg	assuming pin-pin				AISC 360 Eqn E3-4: $\pi^2E/(L/c)^2$	AISC 360 Eqn E3-2/E3-3, Include Ry with Fy	AISC 341 F2.3: min(RyFyAg, 1.14FcreAg)	AISC 341 F2.3: 0.3Pcomp	
2	2	BF-4-U	2	HSS3X.250	6.00	1.4	2.03	131	1	0.982	73	100	53	39	90	27	51
3	3	BF-4-M	2	HSS3X.250	6.02	1.4	2.03	131	1	0.982	74	100	53	39	89	27	66
4	4	BF-6-U	2	HSS3X.250	5.69	1.4	2.03	131	1	0.982	70	100	59	41	95	28	55
5	5	BF-6-M	2	HSS3X.250	6.39	1.4	2.03	131	1	0.982	78	100	47	36	84	25	59
6	6	BF-8-U	2	HSS3X.250	5.29	1.4	2.03	131	1	0.982	65	100	68	43	101	30	62
7	7	BF-8-M	2	HSS3X.250	6.04	1.4	2.03	131	1	0.982	74	100	53	39	89	27	66
8	8	BF-D-U	2	HSS3X.250	5.56	1.4	2.03	131	1	0.982	68	100	62	42	97	29	57
9	9	BF-D-M	2	HSS3X.250	6.28	1.4	2.03	131	1	0.982	77	100	49	37	86	26	61
10	10	BF-G-U	2	HSS3X.250	6.15	1.4	2.03	131	1	0.982	75	100	51	38	88	26	49
11	11	BF-G-M	2	HSS3X.250	6.80	1.4	2.03	131	1	0.982	83	100	41	34	78	23	54
	0																

	Summary of Expected Strengths										Column Length		Ram Steel Col ID		Loads				
	Tensile Strength			Compressive Strength			Post-Buckling Strength												
BF Name	Diag [k]	Vert [k]	Horiz [k]	Diag [k]	Vert [k]	Horiz [k]	Diag [k]	Vert [k]	Horiz [k]	BF Name	Hframe [ft]	Lb Col	Col 1	Col 2	Pu,E (T) [k]	Pu,E (C)[k]	Col 1, Pu, Dead [k]	Col 2, Pu, Dead [k]	Col 1, Pu, Live [k]	Col 2, Pu, Live [k]
												Reenter length. Assuming full height			Loads tabulated for bottom column only. For seismic - Pu,E is sum of vertical components of expected brace strength for both levels. Gravity reactions ~~from DWM Steel~~ are at foundation.					
BF-4-U	131	102	82	90	70	56	27	21	17	BF-4-U	9.33	10	COL4-1U	COL4-2U			2.5	11.9	1.6	8.7
BF-4-M	131	119	53	89	82	37	27	25	11	BF-4-M	11.00	11	COL4-1M	COL4-2M	253	291	3.2	17.3	1.8	11.5
BF-6-U	131	107	75	95	78	54	28	23	16	BF-6-U	9.33	10	COL6-1U	COL6-2U			1.9	6.2	1.2	4.1
BF-6-M	131	113	67	84	72	43	25	22	13	BF-6-M	11.00	11	COL6-1M	COL6-2M	257	297	5.8	10.1	3.2	6.1
BF-8-U	131	115	62	101	89	47	30	27	14	BF-8-U	9.33	10	COL8-1U	COL8-2U			3.8	3.8	2.8	3.4
BF-8-M	131	119	54	89	81	37	27	24	11	BF-8-M	11.00	11	COL8-1M	COL8-2M	285	323	10.5	8.7	5.6	8.2
BF-D-U	131	110	71	97	81	52	29	24	16	BF-D-U	9.33	10	COLD-1U	COLD-2U			2.7	5.6	2.1	4.3
BF-D-M	131	115	63	86	75	41	26	23	12	BF-D-M	11.00	11	COLD-1M	COLD-2M	266	305	4.1	20.8	2.7	11.2
BF-G-U	131	99	85	88	66	57	26	20	17	BF-G-U	9.33	10	COLG-1U	COLG-2U			4.6	2.2	3.9	1.5
BF-G-M	131	106	77	78	63	46	23	19	14	BF-G-M	11.00	11	COLG-1M	COLG-2M	229	271	17.6	12.9	10.4	7.1

	Columns															
	Load Combinations				Design Force				Ductility Check			Capacities		DCR		
BF Name	1.4D [k]	1.2D+1.6 L [k]	(1.2+0.2Sds) D+0.5L+E [k]	(0.9-0.2Sds) D+E [k]	Design Force, C [k]	Design Force, T [k]	Column Size	Ry	bf/2tf (b/t)	λhd	Highly ductile?	ϕcPn [k]	ϕtPn [k]	DCR Comp		DCR Tens	
							Pick size	AISC 341: Table A3.1		AISC 341 Table D1.1: λhd=0.32√(E/RyFy)				Pu/ϕcPn		Pu/ϕtPn	
BF-4-U	17	28	21	2	28	2	HSS6X6X5/8	1.1	7.33	14.93	OK	421	526.5	✓	0.07	✓	0.00
BF-4-M	24	39	321	251	321	251	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✓	0.80	✓	0.48
BF-6-U	9	14	11	1	14	1	HSS6X6X5/8	1.1	7.33	14.93	OK	421	526.5	✓	0.03	✓	0.00
BF-6-M	14	22	315	253	315	253	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✓	0.78	✓	0.48
BF-8-U	5	10	7	3	10	3	HSS6X6X5/8	1.1	7.33	14.93	OK	421	526.5	✓	0.02	✓	0.00
BF-8-M	15	26	342	279	342	279	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✓	0.85	✓	0.53
BF-D-U	8	14	10	2	14	2	HSS6X6X5/8	1.1	7.33	14.93	OK	421	526.5	✓	0.03	✓	0.00
BF-D-M	29	43	341	263	341	263	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✓	0.85	✓	0.50
BF-G-U	6	12	9	1	12	1	HSS6X6X5/8	1.1	7.33	14.93	OK	421	526.5	✓	0.03	✓	0.00
BF-G-M	25	38	302	220	302	220	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✓	0.75	✓	0.42

TWO-STORY BRACED FRAME: COLUMN & BEAM DESIGN

Start at ID	1	
End at ID	11	
Brace Fy	46	ksi
Brace Fu	62	ksi
Beam Fy	50	ksi
W Column Fy	50	ksi
HSS Column Fy	50	ksi
E	29000	ksi
Sds	1.172	
ϕ	0.9	

		Beam Length [ft]					Loads							Beam	
BF Name	BF Name	Beam Length [ft]	ry	Lb, calc [ft]	Lb, Beam	Ram Steel Beam ID	Case 1 Pu,E [k]		Case 2 Pu,E [k]		Design Pu,E [k]	Vu [k]	Mu [k-ft]	Beam Size	Ry
				AISC 341 D1.2b. Beam bracing, Max spacing	AISC 341 D1.2b. Brace at midspan where req'd		BM 2, Case 1: Fstory/2	BM 2: Case 1	BM 2, Case 2: Fstory/2	BM2: Case 2	Design			Pick size	AISC 341: Table A3.1
BF-4-U	BF-4-U	7.55	2.59	11.9	8	U-BM4					139	17.8	22	W10x68	1.1
BF-4-M	BF-4-M	4.92	1.96	9.0	5	M-BM4	24	28	17	54	54	7.1	8.2	W12x50	1.1
BF-6-U	BF-6-U	6.52	2.59	11.9	7	U-BM6					129	7.4	9.1	W10x68	1.1
BF-6-M	BF-6-M	6.51	1.96	9.0	7	M-BM6	10	22	6	56	56	4.9	5.5	W12x50	1.1
BF-8-U	BF-8-U	5.00	2.59	11.9	5	U-BM8					109	6.1	7	W10x68	1.1
BF-8-M	BF-8-M	4.99	1.96	9.0	5	M-BM8	9	16	5	45	45	7.1	8.5	W12x50	1.1
BF-D-U	BF-D-U	6.05	2.12	9.7	7	U-BMD					124	4.1	4.7	W8X67	1.1
BF-D-M	BF-D-M	6.04	1.96	9.0	7	M-BMD	10	20	6	53	53	18.6	23.2	W12x50	1.1
BF-G-U	BF-G-U	8.01	2.59	11.9	9	U-BMG					142	10.5	27	W10x68	1.1
BF-G-M	BF-G-M	8.00	1.96	9.0	8	M-BMG	10	30	6	66	66	13.1	23	W12x50	1.1

	Beams																	
	Ductility Check - Highly Ductile								Capacities			DCR						
BF Name	Ag	bf/2tf	Flanges, λhd	Check Flanges	h/tw	Ca	Webs, λhd	Check Web	ϕPn [k]	ϕMn [k-ft]	ϕVn [k]	Pu/ϕPn		Mu/ϕMn		Vu/ϕVn		Combined
			AISC 341 Table D1.1: λhd=0.32√(E/RyFy)			AISC 341 Table D1.1	AISC 341 Table D1.1: λhd, webs											AISC 360: Eqn H1-1a/H1-1b
BF-4-U	19.9	6.6	7	OK	17	0.14	51.3	OK	810	320	147	✓	0.17	✓	0.07	✓	0.12	✓ 0.07
BF-4-M	14.6	6.3	7	OK	27	0.07	54.4	OK	613	269.6	135	✓	0.09	✓	0.03	✓	0.05	✓ 0.05
BF-6-U	20	7	7	OK	17	0.13	51.5	OK	829	320	147	✓	0.16	✓	0.03	✓	0.05	✓ 0.03
BF-6-M	15	6	7	OK	27	0.08	54.2	OK	574	269.6	135	✓	0.10	✓	0.02	✓	0.04	✓ 0.04
BF-8-U	20	7	7	OK	17	0.11	52.2	OK	861	320	147	✓	0.13	✓	0.02	✓	0.04	✓ 0.02
BF-8-M	15	6	7	OK	27	0.06	55.2	OK	613	269.6	135	✓	0.07	✓	0.03	✓	0.05	✓ 0.04
BF-D-U	20	4	7	OK	11	0.13	51.6	OK	790	263	154	✓	0.16	✓	0.02	✓	0.03	✓ 0.02
BF-D-M	15	6	7	OK	27	0.07	54.5	OK	574	269.6	135	✓	0.09	✓	0.09	✓	0.14	✓ 0.10
BF-G-U	20	7	7	OK	17	0.14	51.2	OK	789	320	147	✓	0.18	✓	0.08	✓	0.07	✓ 0.08
BF-G-M	15	6	7	OK	27	0.09	53.4	OK	551	269.6	135	✓	0.12	✓	0.09	✓	0.10	✓ 0.11

BRACED FRAME CONNECTION DESIGN - GUSSET

					Brace Properties										
Max ID	ID	BF Name	Column Size	Beam Size	Brace Size	Max Brace Compression Expected	Max Brace Tension Expected	Brace Width, b	Brace Area, Ag [in2]	Brace Wall thickness, t	Brace Length [ft] assumes braced at midspan by other brace	Brace X dimension [in] beam length	Brace Y dimension [in] column height	Brace angle to horiz based on column height / beam length
1	1	BF-1	HSS6X6X5/8	W10x68	HSS3X.250	87	131	3.00	2.03	0.23	6.17	8.09	9.33	49.1
2	2	BF-4-U	HSS6X6X5/8	W10x68	HSS3X.250	90	131	3.00	2.03	0.23	6.00	7.55	9.33	51.0
3	3	BF-4-M	HSS6X6X5/8	W12x50	HSS3X.250	89	131	3.00	2.03	0.23	6.02	4.92	11.00	65.9
4	4	BF-6-U	HSS6X6X5/8	W10x68	HSS3X.250	95	131	3.00	2.03	0.23	5.69	6.52	9.33	55.1
5	5	BF-6-M	HSS6X6X5/8	W12x50	HSS3X.250	84	131	3.00	2.03	0.23	6.39	6.51	11.00	59.4
6	6	BF-8-U	HSS6X6X5/8	W10x68	HSS3X.250	101	131	3.00	2.03	0.23	5.29	5.00	9.33	61.8
7	7	BF-8-M	HSS6X6X5/8	W12x50	HSS3X.250	89	131	3.00	2.03	0.23	6.04	4.99	11.00	65.6
8	8	BF-D-U	HSS6X6X5/8	W8X67	HSS3X.250	97	131	3.00	2.03	0.23	5.56	6.05	9.33	57.1
9	9	BF-D-M	HSS6X6X5/8	W12x50	HSS3X.250	86	131	3.00	2.03	0.23	6.28	6.04	11.00	61.2
10	10	BF-G-U	HSS6X6X5/8	W10x68	HSS3X.250	88	131	3.00	2.03	0.23	6.15	8.01	9.33	49.4
11	11	BF-G-M	HSS6X6X5/8	W12x50	HSS3X.250	78	131	3.00	2.03	0.23	6.80	8.00	11.00	54.0

			Gusset Properties		Design Parameters				Brace Block Shear Rupture					Brace to Gusset Weld Fracture	
Max ID	ID	BF Name	Fy, Gusset [ksi]	Fu, Gusset [ksi]	Gusset Thk, tg [in]	Gusset to Brace Weld	Brace to Gusset Weld	ϕ (Whitmore angle at gusset)	ϕ	Rt	Anv [in2]	ϕRn [k]	DCR	ϕRn [k]	DCR
1	1	BF-1	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
2	2	BF-4-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
3	3	BF-4-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
4	4	BF-6-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
5	5	BF-6-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
6	6	BF-8-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
7	7	BF-8-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
8	8	BF-D-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
9	9	BF-D-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
10	10	BF-G-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39
11	11	BF-G-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406 ✓	0.32	334 ✓	0.39

			Block Shear Rupture of Gusset								Tensile Yielding of Gusset Plate				Buckling of Gusset Plate							
Max ID	ID	BF Name	ϕ	Agv=Anv [in2]	Ant [in2]	0.6FuAnv [k]	FuAnt [k]	0.6FyAgv [k]	ϕRn [k]	DCR	ϕ	Whitmore length [in]	ϕRn [k]	DCR	K	Lb [in]	r [in]	KL/r	Fe [ksi]	Fcr [ksi]	ϕPn [k]	DCR
1	1	BF-1	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	8.81	0.14	73.21	53.40	33.79	256.30 ✓	0.51
2	2	BF-4-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	8.28	0.14	68.87	60.34	35.35	268.11 ✓	0.49
3	3	BF-4-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.03	0.14	41.78	163.98	44.01	333.83 ✓	0.39
4	4	BF-6-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	7.29	0.14	60.60	77.95	38.23	289.97 ✓	0.45
5	5	BF-6-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.33	0.14	52.63	103.35	40.83	309.74 ✓	0.42
6	6	BF-8-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.82	0.14	48.40	122.17	42.13	319.56 ✓	0.41
7	7	BF-8-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.08	0.14	42.27	160.16	43.88	332.81 ✓	0.39
8	8	BF-D-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.84	0.14	56.84	88.60	39.48	299.48 ✓	0.44
9	9	BF-D-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.95	0.14	49.44	117.11	41.82	317.21 ✓	0.41
10	10	BF-G-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	8.73	0.14	72.56	54.36	34.02	258.07 ✓	0.51
11	11	BF-G-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	7.55	0.14	62.78	72.61	37.48	284.30 ✓	0.46

BRACED FRAME CONNECTION DESIGN - GUSSET

													Brace Net Section Fracture											
Max ID	ID	BF Name	tgap [in]	An=Ae [in2]	An>Ag?	Fy, cover pl [ksi]	Fu, cover pl [ksi]	Cover plate thk, tcp [in]	Length cover plate [in]	# plates	Apl [in2]	Total Apl [in2]	U assumpt.	Arn [in2]	r1 [in]	r2 [in]	Xbar, brace [in]	Xbar, pl [in]	Xbar, comb [in]	U calc	An [in2]	Ae, calc [in2]	Check Ae>Ag
				Assuming u=1																				
1	1	BF-1	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
2	2	BF-4-U	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
3	3	BF-4-M	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
4	4	BF-6-U	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
5	5	BF-6-M	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
6	6	BF-8-U	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
7	7	BF-8-M	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
8	8	BF-D-U	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
9	9	BF-D-M	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
10	10	BF-G-U	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK
11	11	BF-G-M	1/16	1.74	NG-Add Plates	50	65	1/4	4	2	1.00	2.00	0.8	0.80	1.38	1.63	0.88	1.63	1.28	0.89	3.74	3.34	OK

					Welds - Cover Plates to Brace				
Max ID	ID	BF Name	ϕ	Ry	ϕRyFyAp [k]	Weld Size (tpl-1/16)	Lw [in]	ϕRn [k]	DCR
1	1	BF-1	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
2	2	BF-4-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
3	3	BF-4-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
4	4	BF-6-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
5	5	BF-6-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
6	6	BF-8-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
7	7	BF-8-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82
8	8	BF-D-U	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62
9	9	BF-D-M	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62
10	10	BF-G-U	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62
11	11	BF-G-M	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62

					Beam to Column Connection						
				Forces		Beam Shear		Beam Axial		Combined	
Max ID	ID	BF Name	Pu,E Beam Nu [k]		Vu [k]	ϕVn [k]	Vu/ϕVn	ϕPn [k]	Pu/ϕPn	Mu/ϕMn	Combine
1	1	BF-1	143	143	48	147 ✓	0.33	810 ✓	0.18 ✓	0.02 ✓	0.11
2	2	BF-4-U	139	139	63	147 ✓	0.43	810 ✓	0.17 ✓	0.07 ✓	0.15
3	3	BF-4-M	54	55	64	135 ✓	0.48	613 ✓	0.09 ✓	0.03 ✓	0.08
4	4	BF-6-U	129	130	55	147 ✓	0.38	829 ✓	0.16 ✓	0.03 ✓	0.11
5	5	BF-6-M	56	58	60	135 ✓	0.45	574 ✓	0.10 ✓	0.02 ✓	0.07
6	6	BF-8-U	109	111	57	147 ✓	0.39	861 ✓	0.13 ✓	0.02 ✓	0.09
7	7	BF-8-M	45	47	64	135 ✓	0.48	613 ✓	0.08 ✓	0.03 ✓	0.07
8	8	BF-D-U	124	125	49	154 ✓	0.32	790 ✓	0.16 ✓	0.02 ✓	0.10
9	9	BF-D-M	53	54	75	135 ✓	0.55	574 ✓	0.09 ✓	0.09 ✓	0.13
10	10	BF-G-U	142	144	55	147 ✓	0.37	789 ✓	0.18 ✓	0.08 ✓	0.18
11	11	BF-G-M	66	67	65	135 ✓	0.48	526 ✓	0.13 ✓	0.09 ✓	0.15

BRACED FRAME CONNECTION DESIGN - GUSSET

Gusset-to-Beam Interface

Max ID	ID	BF Name	Forces Nu [k]	Vu [k]	Ru [k] Resultant	L_A [in]	Shear Yielding ϕ	ϕRn [k]	DCR	Tension Yielding ϕ	ϕRn [k]	DCR	Weld Load angle (deg)	Directional strength Inc	Ductility Factor	Weld Size	ϕRn [k]	DCR
1	1	BF-1	44	60	75	13	1	186 ✓	0.32	0.9	280 ✓	0.16	36.29	1.23	1.25	5/16	212 ✓	0.44
2	2	BF-4-U	46	56	72	13	1	177 ✓	0.32	0.9	265 ✓	0.17	39.18	1.25	1.25	5/16	205 ✓	0.44
3	3	BF-4-M	57	25	62	10	1	129 ✓	0.20	0.9	193 ✓	0.30	66.11	1.44	1.25	5/16	172 ✓	0.45
4	4	BF-6-U	48	47	67	12	1	159 ✓	0.30	0.9	239 ✓	0.20	45.58	1.30	1.25	5/16	192 ✓	0.44
5	5	BF-6-M	55	39	68	11	1	151 ✓	0.26	0.9	227 ✓	0.24	54.46	1.37	1.25	5/16	192 ✓	0.44
6	6	BF-8-U	51	32	60	10	1	135 ✓	0.24	0.9	202 ✓	0.25	57.56	1.39	1.25	5/16	174 ✓	0.43
7	7	BF-8-M	57	26	63	10	1	130 ✓	0.20	0.9	195 ✓	0.29	65.53	1.43	1.25	5/16	173 ✓	0.45
8	8	BF-D-U	45	41	61	11	1	145 ✓	0.29	0.9	217 ✓	0.21	47.29	1.31	1.25	5/16	177 ✓	0.43
9	9	BF-D-M	56	35	66	11	1	144 ✓	0.25	0.9	217 ✓	0.26	57.64	1.39	1.25	5/16	186 ✓	0.44
10	10	BF-G-U	44	60	74	13	1	185 ✓	0.32	0.9	277 ✓	0.16	36.70	1.23	1.25	5/16	211 ✓	0.44
11	11	BF-G-M	52	51	73	13	1	174 ✓	0.29	0.9	260 ✓	0.20	45.58	1.30	1.25	5/16	210 ✓	0.44

Gusset-to-Beam Interface

Max ID	ID	BF Name	Web Local Yielding ϕ	tw [in]	Kdes	ϕRn [k]	DCR	Web Local Crippling ϕ	tf [in]	ϕRn [k]	DCR
1	1	BF-1	1	0.47	1.27	367 ✓	0.12	0.75	0.77	553 ✓	0.08
2	2	BF-4-U	1	0.47	1.27	352 ✓	0.13	0.75	0.77	536 ✓	0.09
3	3	BF-4-M	1	0.37	1.14	212 ✓	0.27	0.75	0.64	251 ✓	0.23
4	4	BF-6-U	1	0.47	1.27	324 ✓	0.15	0.75	0.77	503 ✓	0.10
5	5	BF-6-M	1	0.37	1.14	239 ✓	0.23	0.75	0.64	272 ✓	0.20
6	6	BF-8-U	1	0.47	1.27	286 ✓	0.18	0.75	0.77	457 ✓	0.11
7	7	BF-8-M	1	0.37	1.14	213 ✓	0.27	0.75	0.64	252 ✓	0.23
8	8	BF-D-U	1	0.57	1.33	370 ✓	0.12	0.75	0.935	761 ✓	0.06
9	9	BF-D-M	1	0.37	1.14	231 ✓	0.24	0.75	0.64	265 ✓	0.21
10	10	BF-G-U	1	0.47	1.27	364 ✓	0.12	0.75	0.77	551 ✓	0.08
11	11	BF-G-M	1	0.37	1.14	267 ✓	0.20	0.75	0.64	293 ✓	0.18

BRACED FRAME CONNECTION DESIGN - GUSSET

Gusset-to-Column Interface

Max ID	ID	BF Name	Forces							Weld						Shear Yielding			Tension Yielding		
			Nu [k]	Vu [k]	Muc [k-in]	Nu,equiv [k]	R [k] Resultant	L_B [in]	Fy [ksi]	Load angle (deg)	Directional strength	Ductility Factor	Weld Size	ϕRn [k]	DCR	ϕ	ϕRn [k]	DCR	ϕ	ϕRn [k]	DCR
1	1	BF-1	25	55	0	25	60	12	50	25	1.14	1.25	5/16	172	0.44	1	163	0.34	0.90	244	0.10
2	2	BF-4-U	26	56	0	26	62	12	50	25	1.14	1.25	5/16	171	0.45	1	162	0.35	0.90	243	0.11
3	3	BF-4-M	28	62	0	28	68	12	50	24	1.13	1.25	5/16	178	0.48	1	170	0.37	0.90	255	0.11
4	4	BF-6-U	28	59	0	28	65	12	50	25	1.14	1.25	5/16	171	0.48	1	162	0.37	0.90	243	0.11
5	5	BF-6-M	27	57	0	27	63	12	50	25	1.14	1.25	5/16	169	0.47	1	160	0.36	0.90	240	0.11
6	6	BF-8-U	29	64	0	29	71	12	50	25	1.13	1.25	5/16	176	0.50	1	167	0.38	0.90	251	0.12
7	7	BF-8-M	28	62	0	28	68	12	50	24	1.13	1.25	5/16	178	0.48	1	169	0.37	0.90	254	0.11
8	8	BF-D-U	30	65	0	30	71	12	50	25	1.13	1.25	5/16	175	0.51	1	166	0.39	0.90	249	0.12
9	9	BF-D-M	28	59	0	28	65	12	50	25	1.14	1.25	5/16	171	0.47	1	162	0.36	0.90	243	0.11
10	10	BF-G-U	26	55	0	26	61	12	50	25	1.14	1.25	5/16	172	0.44	1	163	0.34	0.90	244	0.10
11	11	BF-G-M	26	53	0	26	59	11	50	26	1.14	1.25	5/16	167	0.44	1	157	0.34	0.90	236	0.11

Gusset-to-Column Interface

Max ID	ID	BF Name	Column Shear (HSS only)						Web Local Yielding					Web Local Crippling			
			h [in]	tw [in]	h/tw	Cv	ϕVn [k]	DCR	ϕ	tw [in]	Kdes	ϕRn [k]	DCR	ϕ	tf [in]	ϕRn [k]	DCR
1	1	BF-1	4.26	0.58	7.33	1.00	133.65	0.19	1.0					0.75			
2	2	BF-4-U	4.26	0.58	7.33	1.00	133.65	0.20									
3	3	BF-4-M	4.26	0.58	7.33	1.00	133.65	0.21									
4	4	BF-6-U	4.26	0.58	7.33	1.00	133.65	0.21									
5	5	BF-6-M	4.26	0.58	7.33	1.00	133.65	0.20									
6	6	BF-8-U	4.26	0.58	7.33	1.00	133.65	0.22									
7	7	BF-8-M	4.26	0.58	7.33	1.00	133.65	0.21									
8	8	BF-D-U	4.26	0.58	7.33	1.00	133.65	0.22	1.0					0.75			
9	9	BF-D-M	4.26	0.58	7.33	1.00	133.65	0.21	1.0					0.75			
10	10	BF-G-U	4.26	0.58	7.33	1.00	133.65	0.19									
11	11	BF-G-M	4.26	0.58	7.33	1.00	133.65	0.19									

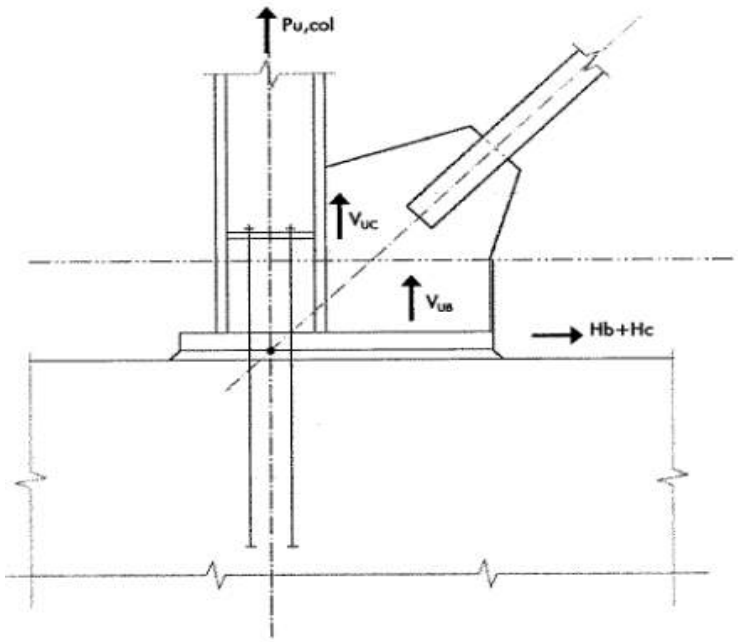
BRACED FRAME CONNECTION DESIGN - BASE CONNECTION

1-Story Braced Frame Base Connection Loading

		Vertical Component - Load Combinations									Horizontal Component		
		$P_{U\ COL}$		Tension Component		Total Load to Foundation - Strength Combinations							
BF Name	Column Size	(1.2+0.2Sds)D+0.5L+E [k]	(0.9-0.2Sds) D+E [k]	V_{UC}	V_{UB}	1.4D [k]	1.2D+1.6L [k]	(1.2+0.2Sds)D+0.5L+E [k]	Max Compression	Max Tension: (0.9-0.2Sds) D+E [k]	Tension [k]	Compression [k]	Maximum [k]
BF-1	HSS6X6X5/8	105	68	55	44	5	8	171	171	167	86	57	86

2-Story Braced Frame Base Connection Loading

		Vertical Component - Load Combinations									Horizontal Component		
		$P_{U\ COL}$		Tension Component		Total Load to Foundation - Strength Combinations							
BF Name	Column Size	(1.2+0.2Sds)D+0.5L+E [k]	(0.9-0.2Sds) D+E [k]	V_{UC}	V_{UB}	1.4D [k]	1.2D+1.6L [k]	(1.2+0.2Sds)D+0.5L+E [k]	Max Compression	Max Tension: (0.9-0.2Sds) D+E [k]	Tension [k]	Compression [k]	Maximum [k]
BF-4-M	HSS6X6X5/8	321	251	62	57	24	39	403	403	370	53	37	53
BF-6-M	HSS6X6X5/8	315	253	57	55	14	22	387	387	366	67	43	67
BF-8-M	HSS6X6X5/8	342	279	62	57	15	26	423	423	398	54	37	54
BF-D-M	HSS6X6X5/8	341	263	59	56	29	43	416	416	378	63	41	63
BF-G-M	HSS6X6X5/8	302	220	53	52	25	38	365	365	326	77	46	77



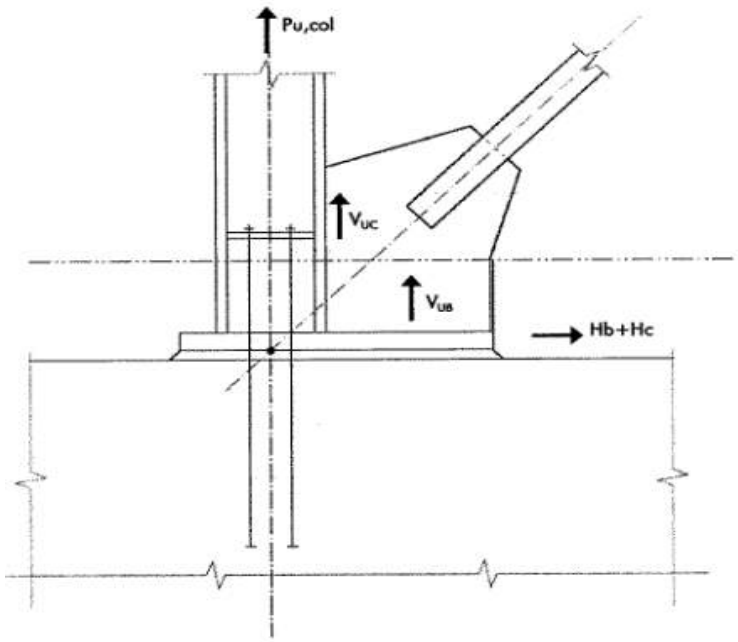
BRACED FRAME CONNECTION DESIGN - BASE CONNECTION

1-Story Braced Frame Base Connection Loading

		Vertical Component - Load Combinations									Horizontal Component		
		$P_{U\ COL}$		Tension Component		Total Load to Foundation - Strength Combinations							
BF Name	Column Size	(1.2+0.2Sds)D+0.5L+E [k]	(0.9-0.2Sds) D+E [k]	V_{UC}	V_{UB}	1.4D [k]	1.2D+1.6L [k]	(1.2+0.2Sds)D+0.5L+E [k]	Max Compression	Max Tension: (0.9-0.2Sds) D+E [k]	Tension [k]	Compression [k]	Maximum [k]
BF-1	HSS6X6X5/8	105	68	55	44	5	8	171	171	167	86	57	86

2-Story Braced Frame Base Connection Loading

		Vertical Component - Load Combinations									Horizontal Component		
		$P_{U\ COL}$		Tension Component		Total Load to Foundation - Strength Combinations							
BF Name	Column Size	(1.2+0.2Sds)D+0.5L+E [k]	(0.9-0.2Sds) D+E [k]	V_{UC}	V_{UB}	1.4D [k]	1.2D+1.6L [k]	(1.2+0.2Sds)D+0.5L+E [k]	Max Compression	Max Tension: (0.9-0.2Sds) D+E [k]	Tension [k]	Compression [k]	Maximum [k]
BF-4-M	HSS6X6X5/8	321	251	62	57	24	39	403	403	370	53	37	53
BF-6-M	HSS6X6X5/8	315	253	57	55	14	22	387	387	366	67	43	67
BF-8-M	HSS6X6X5/8	342	279	62	57	15	26	423	423	398	54	37	54
BF-D-M	HSS6X6X5/8	341	263	59	56	29	43	416	416	378	63	41	63
BF-G-M	HSS6X6X5/8	302	220	53	52	25	38	365	365	326	77	46	77



BRACED FRAME CONNECTION DESIGN - BASE CONNECTION - BF-1

HSS Base Connection			
Braced Frame ID:	**BF-1**		
Braced Frame Type:	**1-Story**		

Expected Forces on Base Plate

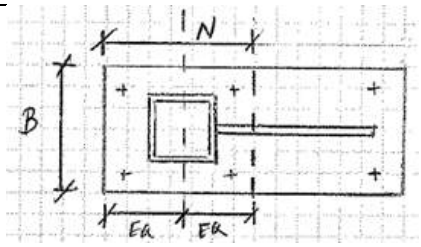
Vertical (P_U compression - column only) =	105	k	
Vertical (P_U tension - column only) =	68	k	
Vertical Tension Component, V_{UC} =	55	k	
Vertical Tension Component, V_{UB} =	44	k	
Total Vertical (compression) =	171	k	
Total Vertical (tension) =	167	k	
Horizontal =	86	k	

Column to Base Plate Weld

Column Size:	HSS6X6X5/8		
Column / Connection Width:	6	in.	
Column Workable Flat =	3.1875	in.	
PJP Weld Thickness (effective throat) =	0.5	in.	
PJP Weld Length Required =	7.801	in.	
Fillet Weld Thickness =	0.625	in.	
Fillet Weld Length Required =	11.033	in.	

Size Base Plate (For Compression)

Fy =	50	ksi	
B =	8	in.	
N =	22	in.	
m = (N-0.95d)/2 =	8.15	in.	
n = (B-0.95b_f)/2 =	1.15	in.	
λn = √(d*b_f)/4	1.50	in.	
ℓ =	8.15	in.	
T_{min} = ℓ √(2*P_u/(0.9F_yBN)) =	1.69	in.	
Use T =	2.00	in.	OK

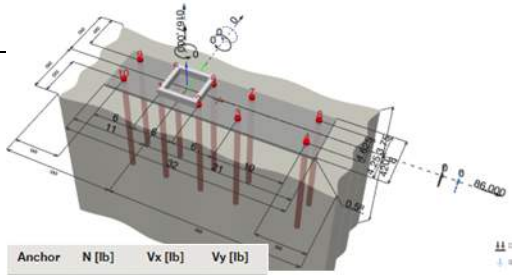


Size Base Plate (For Tension)

e =	6	in	max. distance from anchor to face of column/gusset
Plate trib width =	4	in	
Mu =	41.1	k-in/in	
T_{min} = ℓ √(4*M_u/(0.9F_y)) =	1.91	k-in	OK

Weldable Rebar - Tension

Number of Anchors =	8		(number of anchors adjacent to column)
A706 Rebar Grade:	60	ksi	
Tension Force per anchor:	20.9	k	
Tension Force per anchor (override):	27.4	k	(load distribution at anchor varies - Hilti Profis reactions)
Weldable Rebar Diameter =	#7		
Rebar Area =	0.6	in^2	
Rebar Edge Distance =	2	in	
DCR:	0.85		
Check Anchors	OK		



Anchor	N [lb]	Vx [lb]	Vy [lb]
1	19,278	8,555	149
2	15,217	8,555	18
3	11,156	8,555	-114
4	4,388	8,555	-333
5	23,327	8,645	149
6	19,266	8,645	18
7	15,205	8,645	-114
8	8,436	8,645	-333
9	27,388	8,645	281
10	23,339	8,555	281

Weldable Rebar - Shear

Number of Anchors =	10		(total number of anchors)
A706 Rebar Grade:	60	ksi	
Tension Force per anchor:	8.56	k	
Tension Force per anchor (override):	8.645	k	(load distribution at anchor varies - Hilti Profis reactions)
Weldable Rebar Diameter =	#7		
Rebar Area =	0.6	in^2	
Rebar Edge Distance =	2	in	
DCR:	0.32		
Check Anchors	OK		

Weldable Rebar - Shear and Tension

DCR (Sum of Square Roots):	0.90 < 1, OK	
For weld required - See following spreadsheet results		

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

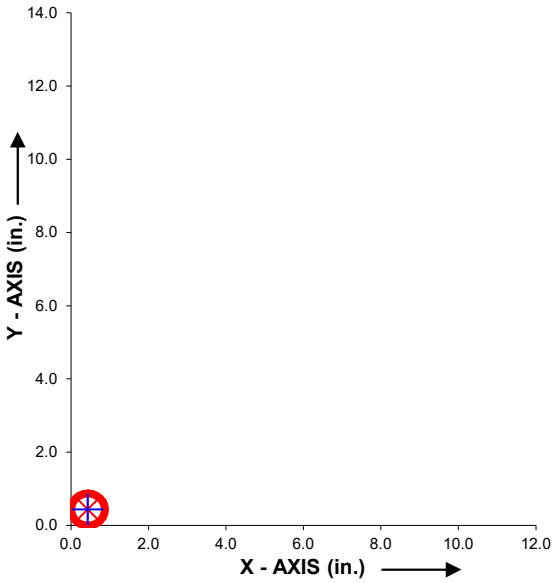
Job Name:		Subject:			
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 24

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	0.000	0.438	0.015	0.324
Weld #2	0.015	0.324	0.059	0.219
Weld #3	0.059	0.219	0.128	0.128
Weld #4	0.128	0.128	0.219	0.059
Weld #5	0.219	0.059	0.324	0.015
Weld #6	0.324	0.015	0.438	0.000
Weld #7	0.438	0.000	0.551	0.015
Weld #8	0.551	0.015	0.656	0.059
Weld #9	0.656	0.059	0.747	0.128
Weld #10	0.747	0.128	0.816	0.219
Weld #11	0.816	0.219	0.860	0.324
Weld #12	0.860	0.324	0.875	0.438
Weld #13	0.875	0.438	0.860	0.551
Weld #14	0.860	0.551	0.816	0.656
Weld #15	0.816	0.656	0.747	0.747
Weld #16	0.747	0.747	0.656	0.816
Weld #17	0.656	0.816	0.551	0.860
Weld #18	0.551	0.860	0.438	0.875
Weld #19	0.438	0.875	0.324	0.860
Weld #20	0.324	0.860	0.219	0.816
Weld #21	0.219	0.816	0.128	0.747
Weld #22	0.128	0.747	0.059	0.656
Weld #23	0.059	0.656	0.015	0.551
Weld #24	0.015	0.551	0.000	0.438

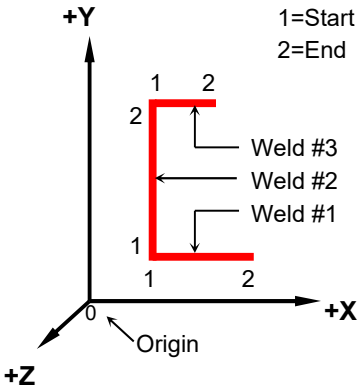


WELD GROUP PLOT

No. of Load Points = 1

Load Point Data:

	Point #1			
X-Coordinate (in.) =	0.438			
Y-Coordinate (in.) =	0.438			
Z-Coordinate (in.) =	0.000			
Axial Load, Pz (k) =	27.40			
Shear Load, Px (k) =	8.65			
Shear Load, Py (k) =	0.00			
Moment, Mx (in-k) =	0.00			
Moment, My (in-k) =	0.00			
Moment, Mz (in-k) =	0.00			



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	2.741	in.
Xc =	0.438	in.
Yc =	0.437	in.
Ix =	0.26	in^3
Iy =	0.26	in^3
J =	0.52	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	27.40	kips
Σ Px =	8.65	kips
Σ Py =	0.00	kips
Σ Mx =	0.00	in-k
Σ My =	0.00	in-k
Σ Mz =	0.00	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	10.482	10.482
Weld #2	10.482	10.482
Weld #3	10.482	10.482
Weld #4	10.482	10.482
Weld #5	10.482	10.482
Weld #6	10.482	10.482
Weld #7	10.482	10.482
Weld #8	10.482	10.482
Weld #9	10.482	10.482
Weld #10	10.482	10.482
Weld #11	10.482	10.482
Weld #12	10.482	10.482
Weld #13	10.482	10.482
Weld #14	10.482	10.482
Weld #15	10.482	10.482
Weld #16	10.482	10.482
Weld #17	10.482	10.482
Weld #18	10.482	10.482
Weld #19	10.482	10.482
Weld #20	10.482	10.482
Weld #21	10.482	10.482
Weld #22	10.482	10.482
Weld #23	10.482	10.482
Weld #24	10.482	10.482

Required E70XX Weld Size:

			Strength	
Fw(max) =	10.482	kips/in.		
Fillet (leg) =	0.706	in.	0.471	in.
Throat (eff) =	0.499	in.	0.333	in.

BRACED FRAME CONNECTION DESIGN - BASE CONNECTION - WORST CASE BF (EXCEPT BF-1)

HSS Base Connection

Braced Frame ID:	**BF-8-M**	
Braced Frame Type:	**2-Story**	

Expected Forces on Base Plate

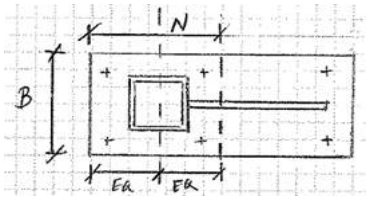
Vertical (P_U compression - column only) =	342	k
Vertical (P_U tension - column only) =	279	k
Vertical Tension Component, V_{UC} =	62	k
Vertical Tension Component, V_{UB} =	57	k
Total Vertical (compression) =	423	k
Total Vertical (tension) =	398	k
Horizontal =	54	k

Column to Base Plate Weld

Column Size:	HSS6X6X5/8		
Column / Connection Width:	6	in.	
Column Workable Flat =	3.1875	in.	
PJP Weld Thickness (effective throat) =	0.5	in.	
PJP Weld Length Required =	21.675	in.	Need more weld than workable flat. Check weld in spreadsheet for all around weld
Fillet Weld Thickness =	0.625	in.	
Fillet Weld Length Required =	30.656	in.	Need more weld than workable flat. Check weld in spreadsheet for all around weld

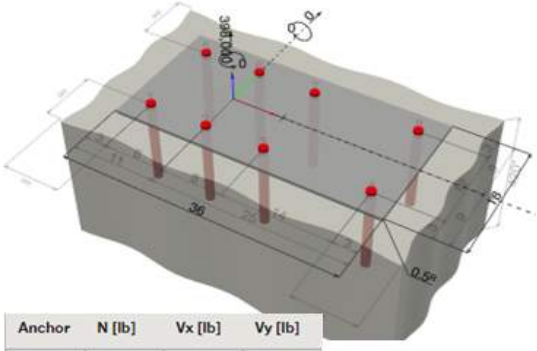
Size Base Plate (For Compression)

Fy =	50	ksi	
B =	18	in.	
N =	22	in.	
m = (N-0.95d)/2 =	8.15	in.	
n = (B-0.95b_f)/2 =	6.15	in.	
λn = √(d*b_f)/4	1.50	in.	
ℓ =	8.15	in.	
T_{min} = ℓ √(2*P_u/(0.9F_yBN)) =	1.78	in.	
Use T =	2.00	in.	OK



Size Base Plate (For Tension)

e =	5	in	max. distance from anchor to face of column/gusset
Plate trib width =	9	in	
Mu =	44.4	k-in/in	
T_{min} = ℓ √(4*M_u/(0.9F_y)) =	1.99	k-in	OK



Anchor Rods - Tension

Number of Anchors =	6		(number of anchors adjacent to column)
F1554 Grade:	55	ksi	
Fu =	75	ksi	
Tension Force per anchor:	66.4	k	
Tension Force per anchor (override):	79.8	k	(load distribution at anchor varies - Hilti Profis reactions)
Anchor Diameter =	1.75	in.	(Reference Table 3.1 for AISC Design Guide)
Anchor Area =	2.41	in^2	(Reference Table 3.1 for AISC Design Guide)
Min Anchor Bolt Spacing =	7	in.	(ACI 318: 17.7.1 - no torque: 4 x anchor diameter)
Anchor spacing used =	8	in.	OK
Anchor Edge Distance =	3	in	
Check Anchors	OK		

Anchor	N [lb]	Vx [lb]	Vy [lb]
1	79,843	0	0
2	62,010	0	0
3	44,177	0	0
4	12,969	0	0
5	79,843	0	0
6	62,010	0	0
7	44,177	0	0
8	12,969	0	0

Size Plate Washer for Yield Strength of Anchor Rod

Concrete Strength, f'c =	4	ksi	
FyAg =	132.55	k	
A_{brg} =	5.92	in^2	
A_{WASHER} =	8.33	in^2	
Min square washer dim =	2.9in. x 2.9in.		

Shear Lug Size

Number of shear lugs:	2		
Grout thickness =	1	in.	
Shear Lug Plate Fy =	50	ksi	
Shear Lug Width =	18	in.	
Shear lug depth, d = ΦR_n/(0.8f'$_c$b) =	0.5	in.	
Use d =	2.0	in.	OK
Total d (includes grout thk.)=	3.0	in.	
Concrete pier width =	12	in.	
Clear dist from lug to edge of conc. =	24	in.	
Depth of concrete failure plane =	26.0	in.	
Av =	276.0	in^2	
Vu = Φ4√(f'$_c$)A_v =	64.1	k	OK
Mu =	54.0	k-in	
t_{min} = √(M_u*4/(ΦF_yb)) =	0.52	in.	
Use t =	1.0	in.	OK
Check base plate T >= t/√(2) =	0.71	in.	OK
Shear lug dimensions:	1 thk. x 3 dp		

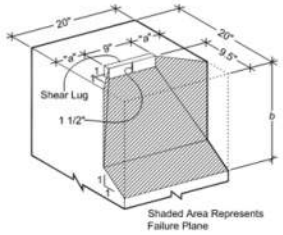
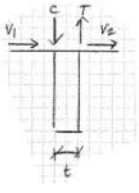


Figure 4.9.3. Lug failure plane.

Shear Lug Welds

C = T = Mu/t =	54.0	k	
V_1 = V_2 =	13.50074484	k	
R_n =	3.1	k/in	
Min. fillet weld size (each side) =	0.069	in.	



WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

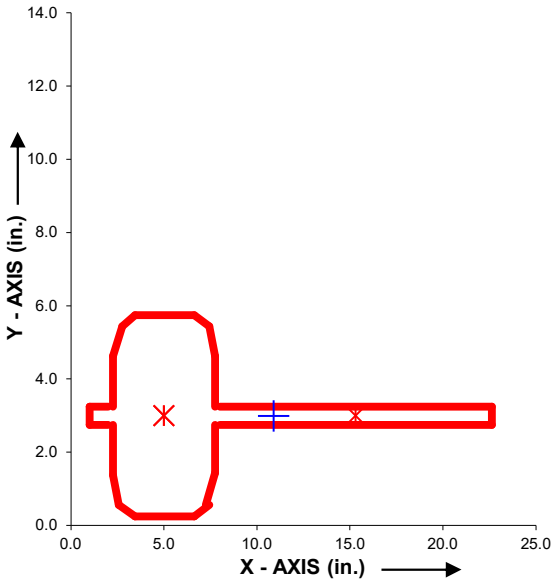
Job Name:		Subject:	BF-1		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	1.000	2.750	2.000	2.750
Weld #2	1.000	3.250	2.000	3.250
Weld #3	2.250	1.375	2.250	2.750
Weld #4	2.250	3.250	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.750
Weld #11	7.750	3.250	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.750	22.625	2.750
Weld #18	8.000	3.250	22.625	3.250
Weld #19	1.000	2.750	1.000	3.250
Weld #20	22.625	2.750	22.625	3.250

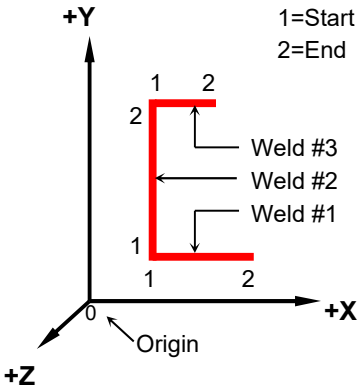


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	15.313	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	68.00	55.00	44.00	
Shear Load, Px (k) =	0.00	0.00	86.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	51.191	in.
Xc =	10.897	in.
Yc =	2.992	in.
Ix =	94.16	in^3
Iy =	2124.47	in^3
J =	2218.63	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	167.00	kips
Σ Px =	86.00	kips
Σ Py =	0.00	kips
Σ Mx =	1.40	in-k
Σ My =	530.99	in-k
Σ Mz =	-0.72	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	5.973	5.734
Weld #2	5.981	5.741
Weld #3	5.655	5.674
Weld #4	5.681	5.701
Weld #5	5.701	5.593
Weld #6	5.593	5.434
Weld #7	5.434	4.683
Weld #8	4.683	4.490
Weld #9	4.490	4.406
Weld #10	4.362	4.380
Weld #11	4.387	4.406
Weld #12	4.362	4.465
Weld #13	4.422	4.606
Weld #14	5.356	4.606
Weld #15	5.568	5.356
Weld #16	5.568	5.655
Weld #17	4.323	1.712
Weld #18	4.329	1.713
Weld #19	5.973	5.981
Weld #20	1.712	1.713

Required E70XX Weld Size:

			Strength	
Fw(max) =	5.981	kips/in.		
Fillet (leg) =	0.403	in.	0.269	in.
Throat (eff) =	0.285	in.	0.190	in.

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

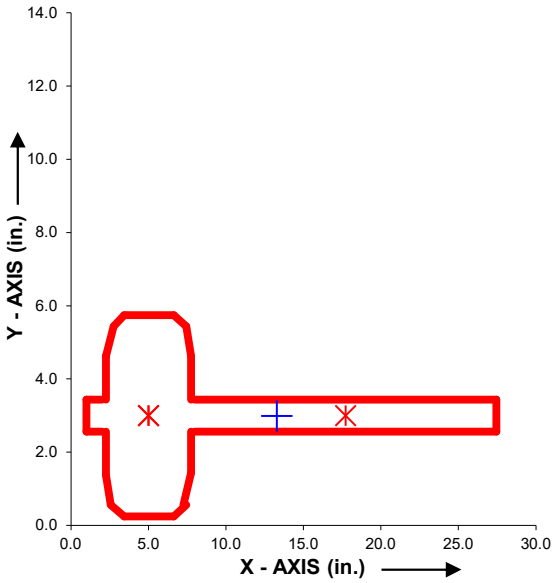
Job Name:		Subject:	BF-4-M		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	1.000	2.563	2.000	2.563
Weld #2	1.000	3.438	2.000	3.438
Weld #3	2.250	1.375	2.250	2.563
Weld #4	2.250	3.438	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.563
Weld #11	7.750	3.438	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.563	27.438	2.563
Weld #18	8.000	3.438	27.438	3.438
Weld #19	1.000	2.563	1.000	3.438
Weld #20	27.438	2.563	27.438	3.438

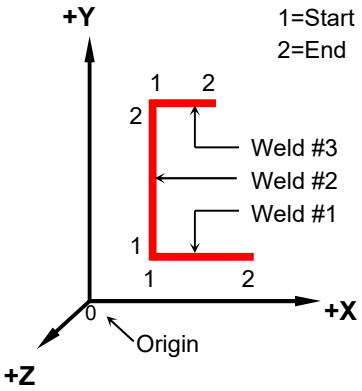


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	17.719	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	251.00	62.00	57.00	
Shear Load, Px (k) =	0.00	0.00	53.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	60.816	in.
Xc =	13.287	in.
Yc =	2.993	in.
Ix =	100.03	in^3
Iy =	3894.14	in^3
J =	3994.17	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	370.00	kips
Σ Px =	53.00	kips
Σ Py =	0.00	kips
Σ Mx =	2.61	in-k
Σ My =	2341.19	in-k
Σ Mz =	-0.37	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	13.488	12.888
Weld #2	13.511	12.911
Weld #3	12.707	12.738
Weld #4	12.761	12.792
Weld #5	12.792	12.513
Weld #6	12.513	12.109
Weld #7	12.109	10.198
Weld #8	10.198	9.704
Weld #9	9.704	9.495
Weld #10	9.413	9.442
Weld #11	9.465	9.495
Weld #12	9.413	9.689
Weld #13	9.577	10.055
Weld #14	11.966	10.055
Weld #15	12.499	11.966
Weld #16	12.499	12.707
Weld #17	9.292	2.586
Weld #18	9.315	2.565
Weld #19	13.488	13.511
Weld #20	2.586	2.565

Required E70XX Weld Size:

			Strength	
Fw(max) =	13.511	kips/in.		
Fillet (leg) =	0.910	in.	0.607	in.
Throat (eff) =	0.643	in.	0.429	in.

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

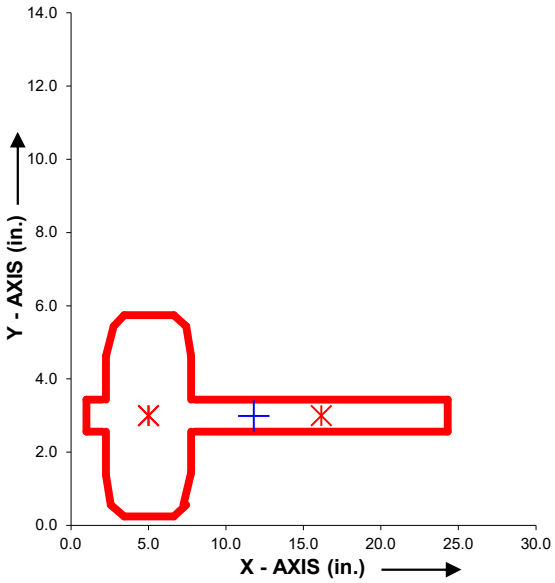
Job Name:		Subject:	BF-6-M		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	1.000	2.563	2.000	2.563
Weld #2	1.000	3.438	2.000	3.438
Weld #3	2.250	1.375	2.250	2.563
Weld #4	2.250	3.438	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.563
Weld #11	7.750	3.438	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.563	24.313	2.563
Weld #18	8.000	3.438	24.313	3.438
Weld #19	1.000	2.563	1.000	3.438
Weld #20	24.313	2.563	24.313	3.438

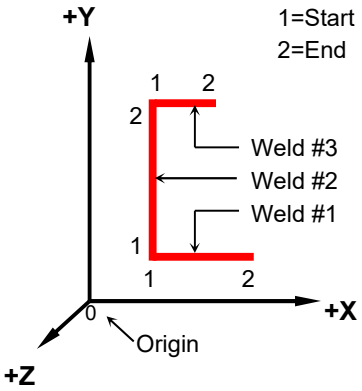


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	16.156	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	253.05	57.30	55.20	
Shear Load, Px (k) =	0.00	0.00	67.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	54.566	in.
Xc =	11.795	in.
Yc =	2.992	in.
Ix =	98.83	in^3
Iy =	2708.38	in^3
J =	2807.21	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	365.55	kips
Σ Px =	67.00	kips
Σ Py =	0.00	kips
Σ Mx =	2.88	in-k
Σ My =	1868.10	in-k
Σ Mz =	-0.53	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	14.186	13.499
Weld #2	14.211	13.524
Weld #3	13.293	13.327
Weld #4	13.353	13.387
Weld #5	13.387	13.067
Weld #6	13.067	12.604
Weld #7	12.604	10.418
Weld #8	10.418	9.853
Weld #9	9.853	9.616
Weld #10	9.523	9.556
Weld #11	9.581	9.616
Weld #12	9.523	9.840
Weld #13	9.712	10.259
Weld #14	12.445	10.259
Weld #15	13.054	12.445
Weld #16	13.054	13.293
Weld #17	9.385	2.302
Weld #18	9.410	2.280
Weld #19	14.186	14.211
Weld #20	2.302	2.280

Required E70XX Weld Size:

			Strength	
Fw(max) =	14.211	kips/in.		
Fillet (leg) =	0.957	in.	0.638	in.
Throat (eff) =	0.677	in.	0.451	in.

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

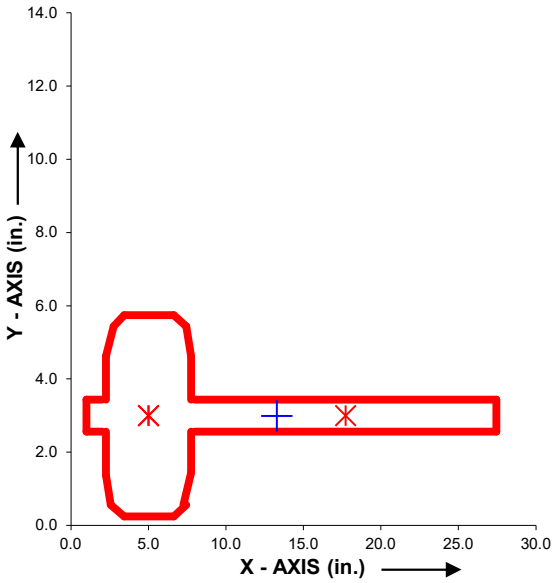
Job Name:		Subject:	BF-8-M		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	1.000	2.563	2.000	2.563
Weld #2	1.000	3.438	2.000	3.438
Weld #3	2.250	1.375	2.250	2.563
Weld #4	2.250	3.438	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.563
Weld #11	7.750	3.438	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.563	27.438	2.563
Weld #18	8.000	3.438	27.438	3.438
Weld #19	1.000	2.563	1.000	3.438
Weld #20	27.438	2.563	27.438	3.438

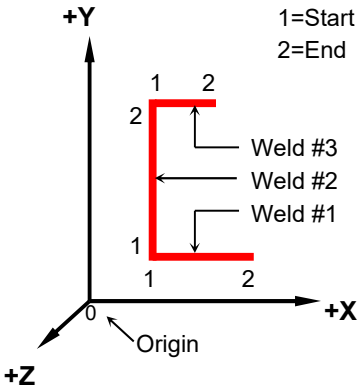


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	17.719	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	279.35	62.03	57.03	
Shear Load, Px (k) =	0.00	0.00	54.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	60.816	in.
Xc =	13.287	in.
Yc =	2.993	in.
Ix =	100.03	in^3
Iy =	3894.14	in^3
J =	3994.17	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	398.41	kips
Σ Px =	54.00	kips
Σ Py =	0.00	kips
Σ Mx =	2.81	in-k
Σ My =	2576.29	in-k
Σ Mz =	-0.38	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	14.695	14.034
Weld #2	14.719	14.059
Weld #3	13.836	13.869
Weld #4	13.894	13.927
Weld #5	13.927	13.620
Weld #6	13.620	13.175
Weld #7	13.175	11.072
Weld #8	11.072	10.527
Weld #9	10.527	10.298
Weld #10	10.209	10.241
Weld #11	10.265	10.298
Weld #12	10.209	10.514
Weld #13	10.391	10.918
Weld #14	13.021	10.918
Weld #15	13.607	13.021
Weld #16	13.607	13.836
Weld #17	10.076	2.959
Weld #18	10.100	2.936
Weld #19	14.695	14.719
Weld #20	2.959	2.936

Required E70XX Weld Size:

			Strength	
Fw(max) =	14.719	kips/in.		
Fillet (leg) =	0.991	in.	0.661	in.
Throat (eff) =	0.701	in.	0.467	in.

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

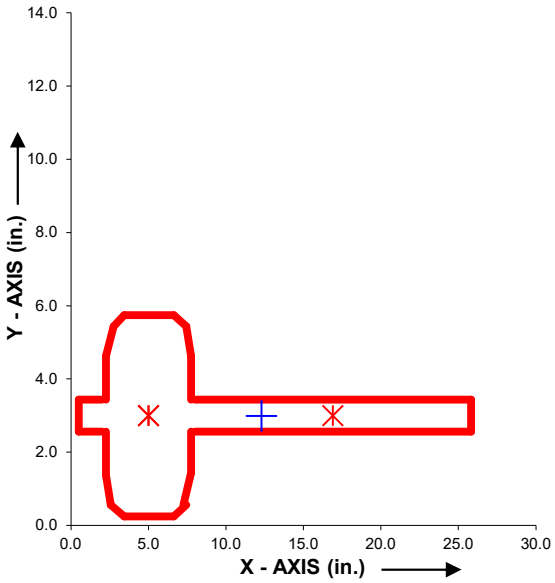
Job Name:		Subject:	BF-D-M		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	0.500	2.563	2.000	2.563
Weld #2	0.500	3.438	2.000	3.438
Weld #3	2.250	1.375	2.250	2.563
Weld #4	2.250	3.438	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.563
Weld #11	7.750	3.438	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.563	25.813	2.563
Weld #18	8.000	3.438	25.813	3.438
Weld #19	0.500	2.563	0.500	3.438
Weld #20	25.813	2.563	25.813	3.438

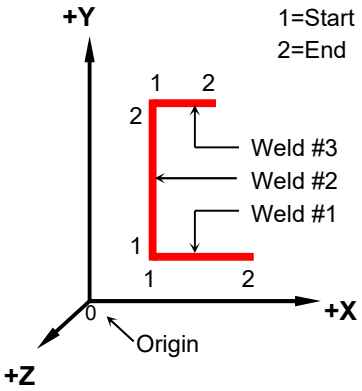


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	16.906	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	263.06	58.67	55.92	
Shear Load, Px (k) =	0.00	0.00	63.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	58.566	in.
Xc =	12.301	in.
Yc =	2.993	in.
Ix =	99.60	in^3
Iy =	3388.53	in^3
J =	3488.13	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	377.64	kips
Σ Px =	63.00	kips
Σ Py =	0.00	kips
Σ Mx =	2.77	in-k
Σ My =	2091.35	in-k
Σ Mz =	-0.46	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	13.762	12.839
Weld #2	13.786	12.863
Weld #3	12.652	12.685
Weld #4	12.709	12.742
Weld #5	12.742	12.457
Weld #6	12.457	12.043
Weld #7	12.043	10.086
Weld #8	10.086	9.578
Weld #9	9.578	9.364
Weld #10	9.276	9.307
Weld #11	9.332	9.364
Weld #12	9.276	9.559
Weld #13	9.444	9.933
Weld #14	11.891	9.933
Weld #15	12.438	11.891
Weld #16	12.438	12.652
Weld #17	9.154	2.186
Weld #18	9.178	2.165
Weld #19	13.762	13.786
Weld #20	2.186	2.165

Required E70XX Weld Size:

			Strength	
Fw(max) =	13.786	kips/in.		
Fillet (leg) =	0.928	in.	0.619	in.
Throat (eff) =	0.656	in.	0.438	in.

WELD GROUP ANALYSIS

Using the Elastic Method for up to 24 Total Welds

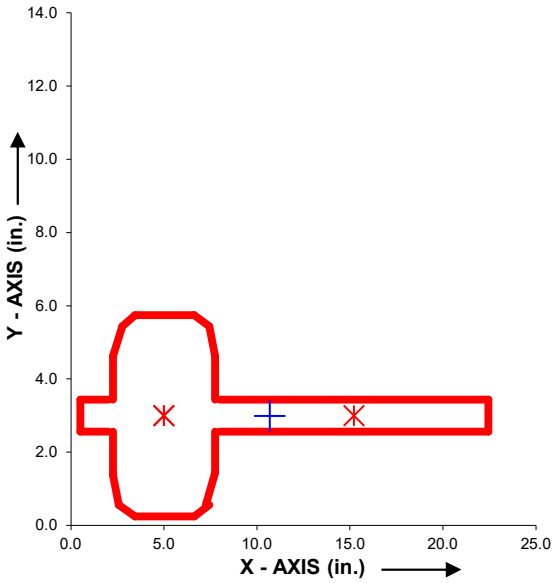
Job Name:		Subject:	BF-G-M		
Job Number:		Originator:		Checker:	

Input Data:

Number of Welds, Nw = 20

Weld Coordinates:

	Start		End	
	X1 (in.)	Y1 (in.)	X2 (in.)	Y2 (in.)
Weld #1	0.500	2.563	2.000	2.563
Weld #2	0.500	3.438	2.000	3.438
Weld #3	2.250	1.375	2.250	2.563
Weld #4	2.250	3.438	2.250	4.625
Weld #5	2.250	4.625	2.750	5.438
Weld #6	2.750	5.438	3.438	5.750
Weld #7	3.438	5.750	6.625	5.750
Weld #8	6.625	5.750	7.438	5.438
Weld #9	7.438	5.438	7.750	4.625
Weld #10	7.750	1.438	7.750	2.563
Weld #11	7.750	3.438	7.750	4.625
Weld #12	7.750	1.438	7.250	0.563
Weld #13	7.438	0.563	6.625	0.250
Weld #14	3.438	0.250	6.625	0.250
Weld #15	2.563	0.563	3.438	0.250
Weld #16	2.563	0.563	2.250	1.375
Weld #17	8.000	2.563	22.438	2.563
Weld #18	8.000	3.438	22.438	3.438
Weld #19	0.500	2.563	0.500	3.438
Weld #20	22.438	2.563	22.438	3.438

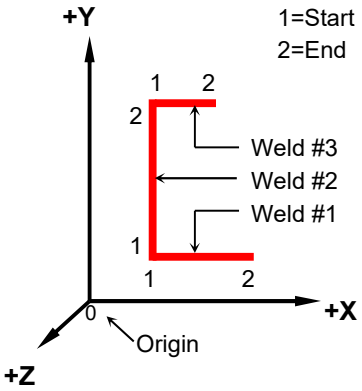


WELD GROUP PLOT

No. of Load Points = 3

Load Point Data:

	Point #1	Point #2	Point #3	
X-Coordinate (in.) =	5.000	5.000	15.219	
Y-Coordinate (in.) =	3.000	3.000	3.000	
Z-Coordinate (in.) =	0.000	0.000	0.000	
Axial Load, Pz (k) =	219.99	53.46	52.26	
Shear Load, Px (k) =	0.00	0.00	77.00	
Shear Load, Py (k) =	0.00	0.00	0.00	
Moment, Mx (in-k) =	0.00	0.00	0.00	
Moment, My (in-k) =	0.00	0.00	0.00	
Moment, Mz (in-k) =	0.00	0.00	0.00	



NOMENCLATURE

(continued)

Results:

Weld Group Properties:

Lw =	51.816	in.
Xc =	10.704	in.
Yc =	2.992	in.
Ix =	98.31	in^3
Iy =	2236.38	in^3
J =	2334.69	in^3

Σ Loads @ C.G. of Weld Group:

Σ Pz =	325.71	kips
Σ Px =	77.00	kips
Σ Py =	0.00	kips
Σ Mx =	2.70	in-k
Σ My =	1323.70	in-k
Σ Mz =	-0.64	in-k

	Weld Forces (k/in.)	
	Fw(1)	Fw(2)
Weld #1	12.403	11.522
Weld #2	12.427	11.546
Weld #3	11.343	11.375
Weld #4	11.399	11.431
Weld #5	11.431	11.160
Weld #6	11.160	10.766
Weld #7	10.766	8.901
Weld #8	8.901	8.419
Weld #9	8.419	8.215
Weld #10	8.128	8.159
Weld #11	8.182	8.215
Weld #12	8.128	8.396
Weld #13	8.287	8.752
Weld #14	10.616	8.752
Weld #15	11.137	10.616
Weld #16	11.137	11.343
Weld #17	8.013	1.630
Weld #18	8.037	1.621
Weld #19	12.403	12.427
Weld #20	1.630	1.621

Required E70XX Weld Size:

			Strength	
Fw(max) =	12.427	kips/in.		
Fillet (leg) =	0.837	in.	0.558	in.
Throat (eff) =	0.592	in.	0.395	in.

BRACED FRAME FOUNDATIONS - LOADS

Strength	
(0.9-0.2Sds)D+E	0.6656
(1.2+0.2Sds)D+E+L+0.2S	1.4344
ASD	
(1+0.14Sds)D + 0.7E	1.1641
(1+0.1Sds)D+0.525E +0.75L+0.	1.1172
(0.6-0.14Sds)D+0.7E	0.4359

				Strength		ASD			Max Load for
	EQ [k]	DL [k]	LL [k]	1.434D+E+L+0.2S	0.6656D+E	1.1641D+0.7E	1.1172D+0.525E+0.75L	0.4359D+0.7E	Piles [k]
BF4	35.1	17.3	11.5	71.4	46.6	44.69	46.36	32.10	46.36
BF6	33.8	10.1	6.1	54.3	40.5	35.40	33.55	28.06	35.40
BF8	40.3	10.5	8.2	63.5	47.3	40.42	39.02	32.78	40.42
BFD	40.5	20.8	11.2	81.5	54.3	52.55	52.88	37.41	52.88
BFG	67.3	17.6	10.4	102.9	79.0	67.58	62.78	54.78	67.58
BF1	8.5	3.53	2.20	15.8	10.8	10.06	10.05	7.49	10.06

Footing Size 1		Grade Beam 1		Slab		Concrete Stem	
Width [ft]	5	Width [ft]	1.5	Thickness [ft	0.5	Thickness [ft	0.66666667
Depth [ft]	2.5	Depth [ft]	2	Weight [ksf]	0.075	Weight [ksf]	0.1
Weight [klf]	1.875	Weight [klf]	0.45				
Footing Size 2		Grade Beam 2					
Width [ft]	4	Width [ft]	1.5				
Depth [ft]	2.5	Depth [ft]	1				
Weight [klf]	1.5	Weight [klf]	0.225				

BRACED FRAME FOUNDATIONS - UPLIFT CHECK

				Footing			AdjacentFooting		
		EQ [k]	Col DL	Ftg [klf]	trib to Col	Ftg [k]	Ftg [klf]	trib to Col	Ftg [k]
BF4	Col 1	35.1	3.2	1.5	5.8	8.8	1.875	4.9	9.2
	Col 2	35.1	17.3	1.5	17.5	26.3	1.875		0.0
BF6	Col 1	33.8	5.8	1.5	8.6	12.9	1.875		0.0
	Col 2	33.8	10.1	1.5	12.0	18.0	1.875	2.8	5.3
BF8	Col 1	40.3	10.5	1.875	5.3	10.0	1.875	7.2	13.5
	Col 2	40.3	8.7	1.875	10.3	19.4	1.875		0.0
BFD	Col 1	40.5	4.1	1.875	5.0	9.4	1.875		0.0
	Col 2	40.5	20.8	1.875	5.0	9.4	1.5	6.7	10.0
BFG	Col 1	67.3	17.6	1.875	9.7	18.2	1.5	8.7	13.0
	Col 2	67.3	12.9	1.875	13.4	25.1	1.875	10.7	20.0

		Adjacent Grade Beam 1			Adjacent Grade Beam 2			Slab on grade			
		Grade Beam	trib to Col	Grade Bm DL [k]	Grade Beam	trib to Col	Grade Bm DL	Slab DL [ksf]	Trib width [f	Trib length [	Slab
BF4	Col 1	0.45	6.5	2.9	0.225		0.0	0.075	6.5	16.5	8.0
	Col 2	0.45		0.0	0.225		0.0	0.075	17.17	11.7	15.0
BF6	Col 1	0.45		0.0	0.225	16.5	3.7	0.075	16.5	8.6	10.6
	Col 2	0.45		0.0	0.225		0.0	0.075	16.5	12.0	14.9
BF8	Col 1	0.45	5.2	2.3	0.225	9.2	2.1	0.075	15.67	10.5	12.3
	Col 2	0.45		0.0	0.225	15.7	3.5	0.075	15.67	10.3	12.1
BFD	Col 1	0.45	5.0	2.3	0.225	24.0	5.4	0.075	6.25	5.0	2.3
	Col 2	0.45	6.5	2.9	0.225		0.0	0.075	6.5	16.5	8.0
BFG	Col 1	0.45	14.8	6.7	0.225	11.915	2.7	0.075	10	9.7	7.3
	Col 2	0.45	5.2	2.3	0.225	8.5	1.9	0.075	10	13.4	10.0

		Slab on grade 2				Concrete Stem/Wall				Addition Pt Load					
		Slab DL [ksf]	Trib width [f	Trib length [	Slab	Weight [ksf]	Height [ft]	Trib length [	Wall DL [k]	Tag	DL [k]	Total DL	0.4359D	0.7E [k]	
BF4	Col 1	0.075	17.17	2.5	3.2	0.10			0	BFD-COL2	20.8	56.1	24.5	24.6	1.00
	Col 2	0.075			0.0	0.10			0			58.6	25.5	24.6	0.96
BF6	Col 1	0.075			0.0	0.10	11.25	16.5	18.56	Col @6/D	10.8	62.4	27.2	23.7	0.87
	Col 2	0.075			0.0	0.10			0	Col @ 6/G	39.4	87.7	38.2	23.7	0.62
BF8	Col 1	0.075			0.0	0.10	1.5	9.165	1.37475	BFG-COl2	12.9	65.0	28.3	28.2	1.00
	Col 2	0.075	15.67	2.0	2.4	0.10	1.5	15.67	2.3505	Col @8/K	13.9	62.3	27.2	28.2	1.04
BFD	Col 1	0.075	5	5.0	1.9	0.10	11.25	24	27	typ beam	15	67.3	29.3	28.4	0.97
	Col 2	0.075	17.17	6.7	8.6	0.10			0	BF4-COL1	2.8	62.5	27.3	28.4	1.04
BFG	Col 1	0.075			0.0	0.10	1.5	11.915	1.78725	Col @ 6/G	39.4	106.6	46.5	47.1	1.01
	Col 2	0.075			0.0	0.10	1.5	8.5	1.275	BF8-Col	33.1	106.6	46.5	47.1	1.01

all within 5%, ok

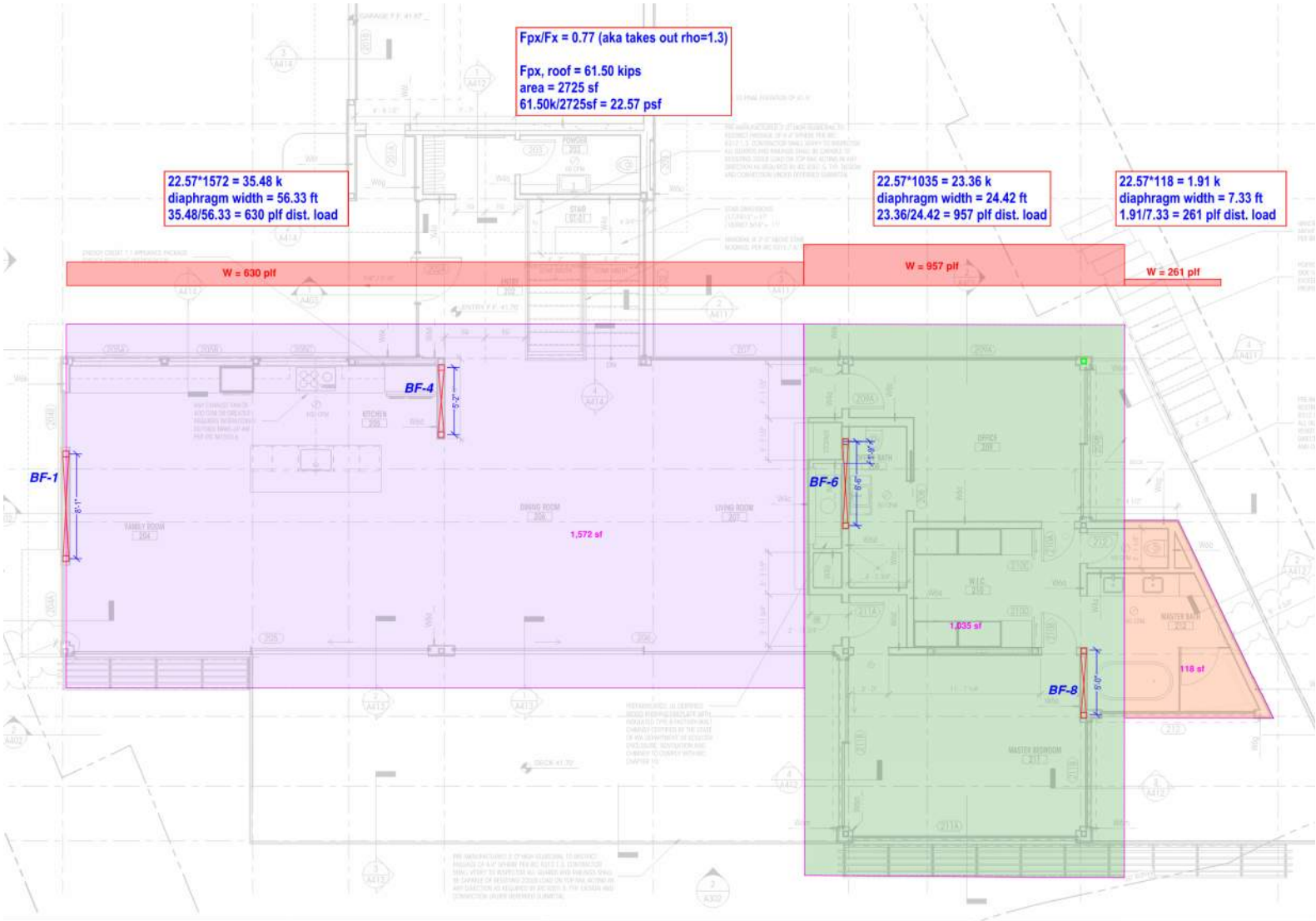
NOTE: SEE FOUNDATIONS SECTION FOR ADDITIONAL DESIGN AND CALCULATIONS

WOOD DIAPHRAGMS

The wood diaphragms are designed using a custom spreadsheet. The load to each braced framed was determined based on the Fpx governing load. The amount and blocking nailing spacing were determined in this spreadsheet as well as the load in the strut (includes Ω) and the chord forces. Where the braced framed is dropped below the top of the roof by more than a few inches, a cripple wall is used to bring the load from the diaphragm into the braced frame.

Main House Roof Diaphragm -- North-South Direction

Distributed loads



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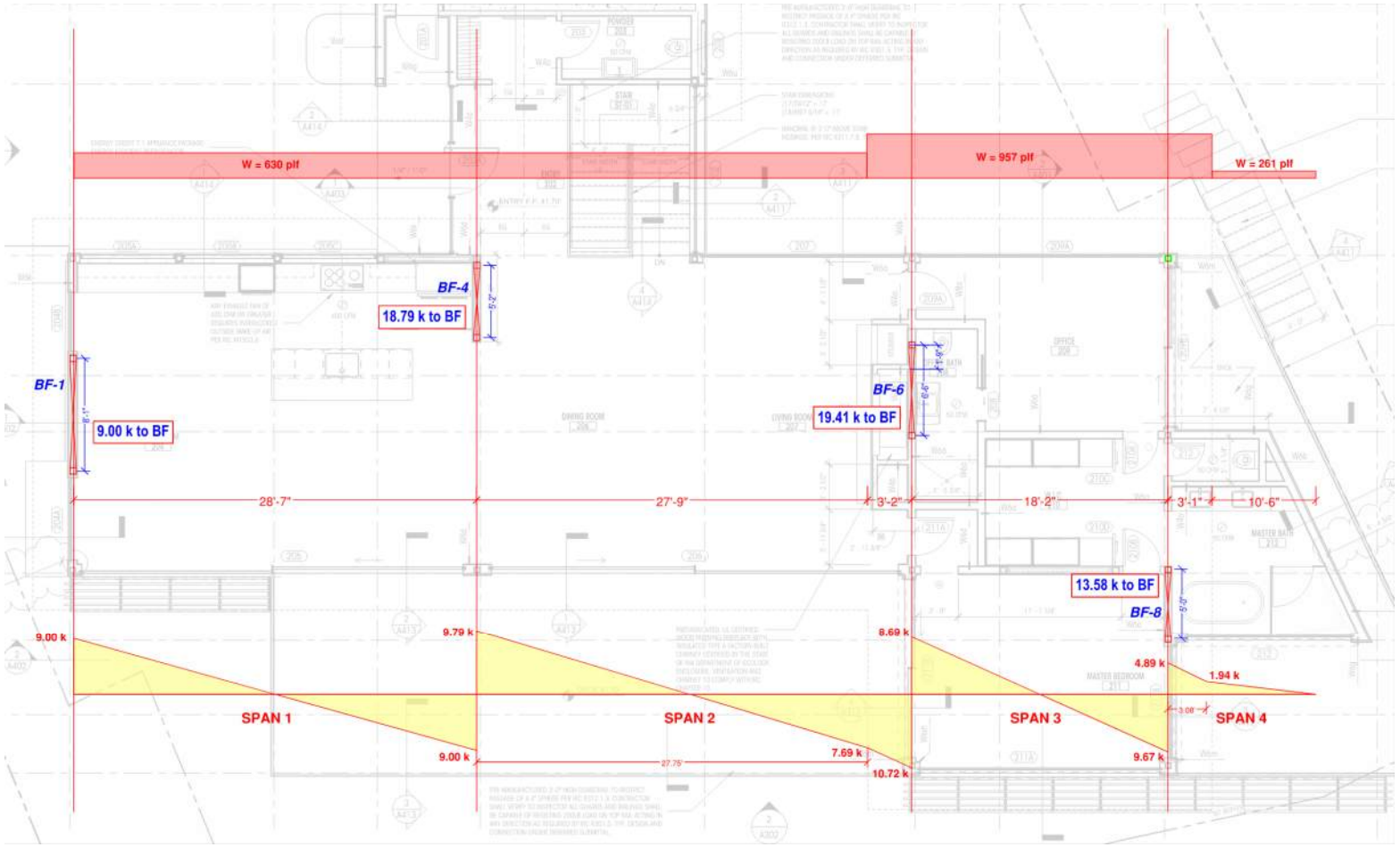
Main House Roof Diaphragm -- North-South Direction

Shear diaphragm



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Main House Roof Diaphragm -- North-South Direction

Ω =	2	
Nail size =	8d	
Nominal w =	2	in, of nailed faced at panel edges
Diaphragm t =	15/32"	PLY sheating on DF framing
Capacity =	288	plf, LRFD for unblocked
	432	plf, LRFD for 6" spacing blkg
	576	plf, LRFD for 4" spacing blkg
	848	plf, LRFD for 2.5" spacing blkg
	960	plf, LRFD for 2" spacing blkg

BF ID	Vpx (kips)	BF L (Ft)	BF V (plf)	V*1.25 (plf)	Strut L have (ft)	Total L have (ft)	V w/ Strut (plf)	Dia Check	Blkg Nailing Req'd
BF-1	9.0	8.1	1113	1392	14.2	22.2	506	add blkg	4in spacing
BF-4	18.8	5.2	-	-	-	-	1080	add blkg	(2) rows 4in spacing
BF-4 L	*9.0*	*5.2*	*1742*	*2177*	*16.6*	*21.8*	*517*	*add blkg*	*4in spacing*
BF-4 R	*9.8*	*5.2*	*1895*	*2369*	*16.6*	*21.8*	*563*	*add blkg*	*4in spacing*
BF-6	19.4	6.5	-	-		6.5	1150	add blkg	(2) rows 4in spacing
BF-6 L	*10.7*	*1.8*	*6126*	*7657*	*14.0*	*15.8*	*851*	*add blkg*	*2.5in spacing*
BF-6 R	*8.7*	*6.5*	*1337*	*1671*	*29.8*	*36.3*	*300*	*OK*	-
BF-8	13.6	5.0	-	-	-	-	864	add blkg	2.5in spacing
BF-8 L	*9.7*	*5.0*	*1934*	*2418*	*22.2*	*27.2*	*445*	*add blkg*	*6in spacing*
BF-8 R	*4.9*	*5.0*	*978*	*1223*	*9.6*	*14.6*	*419*	*add blkg*	*6in spacing*

Span	Grids Btwn	Vpx (plf)	Vpx*1.25 (plf)	Span L** (ft)	Span Type	Moment (k-ft)	Moment Arm (ft)	Chord T/C (kips)	Chord
1	1-4	630	788	28.6	s.s.	80.4	22.3	3.6	stl beam
2	4-6	630	788	30.9	s.s.	95.1	22.3	4.3	stl beam
2 (red)	4-6	957	788	28.4	s.s.	29.8	7.8	3.8	stl beam
3	6-8	957	1196	18.2	s.s.	49.4	36.5	1.4	stl beam
4	8-9	261	326	10.5	cant.	22.1	14.6	1.5	stl beam

** Span L is the location checking moment at for (red) ones

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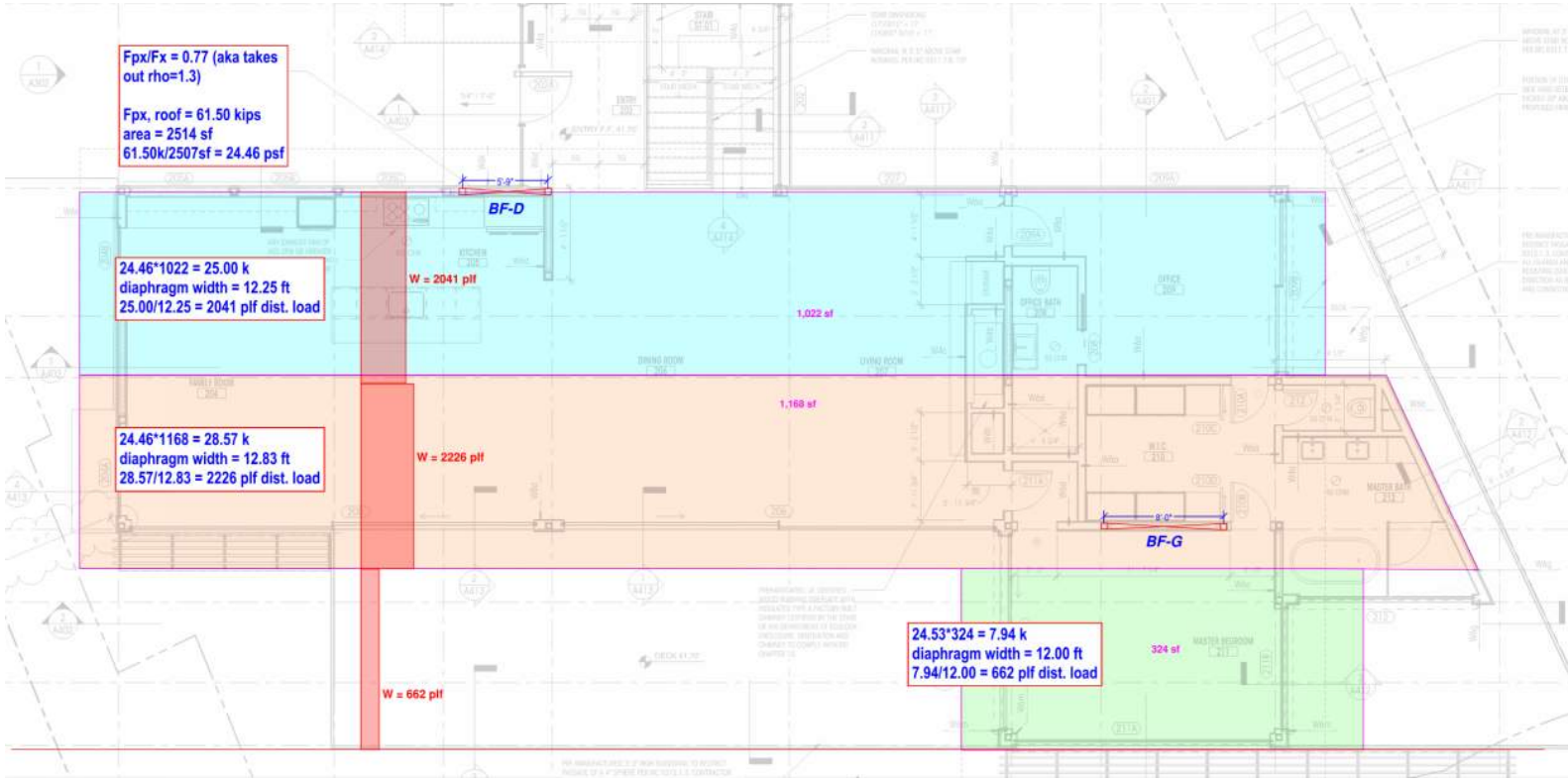
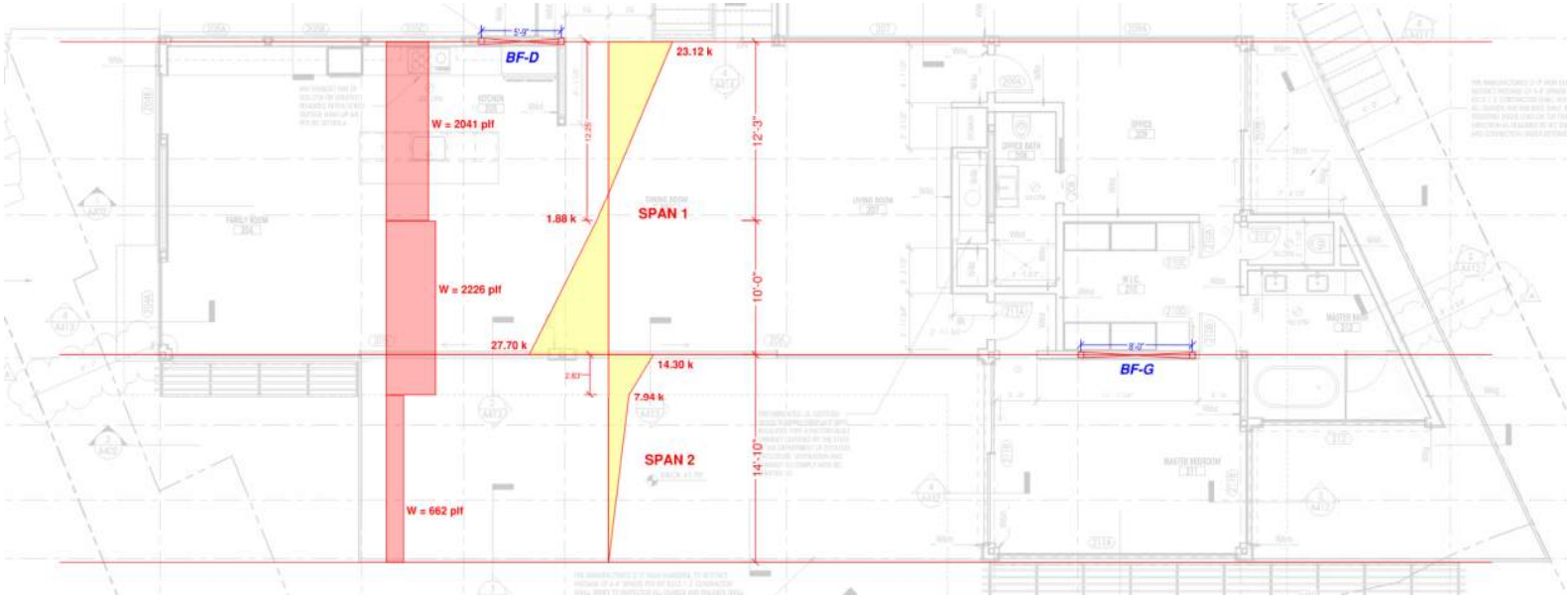
Main House Roof Diaphragm -- East-West Direction

Distributed loads



Main House Roof Diaphragm -- East-West Direction

Shear diaphragm



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Main House Roof Diaphragm -- East-West Direction

Ω =	2	
Nail size =	8d	
Nominal w =	2	in, of nailed faced at panel edges
Diaphragm t =	15/32"	PLY sheating
Capacity =	288	plf, LRFD for unblocked
	432	plf, LRFD for 6" spacing blkg
	576	plf, LRFD for 4" spacing blkg
	848	plf, LRFD for 2.5" spacing blkg
	960	plf, LRFD for 2" spacing blkg

BF ID	Vpx (kips)	BF L (Ft)	BF V (plf)	V*1.25 (plf)	Strut L have (ft)	Total L have (ft)	V w/ Strut (plf)	Dia Check	Blkg Nailing Req'd
BF-D	23.1	5.8	4021	5026	71.9	77.7	372	add blkg	6in spacing
BF-G	42.0	8.0	-	-	-	-	1403	add blkg	(2) rows 2.5in spacing
BF-G L	*27.7*	*8.0*	*3463*	*4328*	*69.8*	*77.8*	*445*	*add blkg*	*6in spacing*
BF-G R	*14.3*	*8.0*	*1788*	*2234*	*10.7*	*18.7*	*958*	*add blkg*	*2in spacing*

Span	Grids Btwn	Vpx (plf)	Vpx*1.25 (plf)	Span L** (ft)	Span Type	Moment (k-ft)	Moment Arm (ft)	Chord T/C (kips)	Chord
1	D-G	2041	2551	22.3	s.s.	163.8	77.7	2.1	stl beam
2	G-K	2226	2783	14.8	s.s.	98.9	18.2	5.4	stl beam

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PROJ. #

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Main House Roof Diaphragm -- Strut Designs (NS)

Ω = 2

BF ID	Blkg Nailing	Capacity (plf)	BF L (ft)	Load over BF (kips)	Vpx*1.25 (kips)	Left for Struts (kips)	Strut Load * Ω (lbs)	Total Strut L (ft)
BF-1	2in spacing	960	8.1	7.8	11.3	3.5	5.6	14.2
BF-4	(2) rows of 2in spacing	1920	5.2	9.9	23.5	-	-	-
BF-4 L	*2in spacing*	*960*	*5.2*	*4.8*	*11.3*	*6.5*	*10.4*	*16.6*
BF-4 R	*2in spacing*	*960*	*5.2*	*5.2*	*12.2*	*7.1*	*11.3*	*16.6*
BF-6	(2) rows of 2in spacing	1920	6.5	12.5	24.3	-	-	0.0
BF-6 L	*2in spacing*	*960*	*1.8*	*1.7*	*13.4*	*11.7*	*18.8*	*14.0*
BF-6 R	*2in spacing*	*960*	*6.5*	*10.8*	*10.9*	*0.1*	*0.1*	*29.8*
BF-8	2in spacing	960	5.0	4.8	17.0	-	-	-
BF-8 L	*2in spacing*	*960*	*5.0*	*3.4*	*12.1*	*8.7*	*13.9*	*22.2*
BF-8 R	*2in spacing*	*960*	*5.0*	*1.4*	*6.1*	*4.7*	*7.6*	*9.6*

Strut ID	Axial, Pr (kips)	Strut L (ft)	Beam Size	Unbraced L (ft)	Mu (kip-ft)	ϕMn (kip-ft)	ϕPc (kips)	Pr/Pc	Interact.	Total L of 5/16" fillet need (in)
1A	5.6	7.1	W10x26	7.1	10.0	107	257	0.02	0.10	1
1B	5.6	7.1	W10x26	7.1	10.0	107	257	0.02	0.10	1
4A	21.7	16.6	W14x90	16.6	97.0	563	965	0.02	0.18	4
6A	18.9	6.2	W10x26	6.2	10.0	111	276	0.07	0.12	3
6B	18.9	9.6	W12x26	9.6	38.0	97	197	0.10	0.44	3
6C	7.9	14.0	W10x26	14.0	10.0	77	113	0.07	0.16	2
8A	13.9	12.6	W10x26	12.6	10.0	84	139	0.10	0.17	2
8B	21.4	9.6	W10x26	9.6	10.0	97	203	0.11	0.16	4

** see next page for example ϕPc calc for 1A

** see gravity calcs for Mu values

Main House Roof Diaphragm -- Cripple Wall Designs (NS)

BF ID	Vpx (kips), LRFD	Vx (kips), LRFD	Vx (kips), ASD	Wall L (ft)	V (plf), ASD	SW type	Wall H (ft)	OT (k), ASD	HD
BF-4	18.8	24.4	17.1	21.8	786	2W3	2.9	5.1	HDU5
BF-6	14.5	18.8	13.2	22.3	593	W2	3.1	3.4	HDU4

Main House Roof Diaphragm -- W.T.S. Designs (NS)

BF ID	Vpx (kips), LRFD	Vx (kips), LRFD	Vx (plf), ASD	Max Spacing (in)
BF-1	9.0	11.7	368	36
BF-4 (cripple)	18.8	24.4	786	16
BF-6 (cripple)	14.5	18.8	838	16
BF-6 R (6C)	4.9	6.4	319	36
BF-8 Left	9.7	12.6	324	36
BF-8 Right	4.9	6.4	305	36

Strut Compression Capacity
AISC Specification Chapter E

Beam =	W10X26	
L =	7.08	ft
Fy =	50	ksi
b =	5.77	in
tf =	0.44	in
h =	8.84	in
tw =	0.26	in
Ag =	7.61	in^2
rx =	4.35	in
KL/rx =	19.5	
ry =	1.4	in
KL/ry =	62.5	
KL/r max =	62.5	
bf/2tf =	6.56	
h/tw =	34	
slender flange limit =	13.49	Non-slender Flange
slender web limit =	35.88	Non-slender Web
ϕPn =	**257**	**kips**

COMPRESSIVE STRENGTH (withOUT slender elements):

COMPRESSIVE FLEXURAL BUCKLING (withOUT slender elements):

4.71*SQRT(E/Fy) =	113.43	
Fe =	73.27	ksi
Fcr =	37.58	ksi
Pn =	286	kips
ϕPn =	**257**	**kips**

COMPRESSIVE FLEXURAL TORSIONAL BUCKLING (withOUT slender elements):

Cw =	345	in^6
J =	0.40	in^4
Ix =	144	in^4
Iy =	14	in^4
Fe =	114.92	ksi
4.71*SQRT(E/Fy) =	113.43	
Fcr =	41.68	ksi
Pn =	317	kips
ϕPn =	**285**	**kips**

Main House Roof Diaphragm -- Strut Designs (EW)

Ω = 2

BF ID	Blkg Nailing	Capacity (plf)	BF L (ft)	Load over BF (kips)	Vpx*1.25 (kips)	Left for Struts (kips)	Strut Load * Ω (lbs)	Total Strut L (ft)
BF-D	2in spacing	960	5.8	5.5	28.9	23.4	37.4	71.9
BF-G	(2) rows of 2in spacing	1920	8.0	15.4	52.5	-	-	-
BF-G L	*2in spacing*	*960*	*8.0*	*7.7*	*34.6*	*26.9*	*43.1*	*69.8*
BF-G R	*2in spacing*	*960*	*8.0*	*7.7*	*17.9*	*10.2*	*16.3*	*10.7*

Strut ID	Axial, Pr (kips)	Strut L (ft)	Beam Size	Unbraced L (ft)	Mu (kip-ft)	ϕMn (kip-ft)	ϕPc (kips)	Pr/Pc	Interact.	Total L of 5/16" fillet need (in)
D1	4.1	7.5	W10x26	7.5	10.0	105.7	249	0.02	0.10	1
D2	4.1	7.0	W10x26	7.0	10.0	107.8	259	0.02	0.10	1
D3	4.1	8.3	W10x26	8.3	10.0	102.0	231	0.02	0.11	1
D4	33.3	15.5	W10x26	15.5	10.0	69.5	92	0.36	0.49	5
D5	33.3	15.4	W10x54	15.4	38.0	70.0	93	0.36	0.84	5
D6	16.6	18.2	W10x54	18.2	26.0	56.1	67	0.25	0.66	3
G1	43.1	28.7	BUILT UP	28.7	57.0	473.9	1257	0.03	0.14	7
G2	43.1	30.9	BUILT UP	30.9	57.0	683.3	1257	0.03	0.10	7
G3	59.4	6.4	W10x26	6.4	10.0	110.3	271	0.22	0.30	9
G4	59.4	3.8	W10x26	3.8	10.0	117.2	316	0.19	0.18	9

** see next page for example ϕPc calc for G1

** see gravity calcs for Mu values

Main House Roof Diaphragm -- Cripple Wall Designs (EW)

BF ID	Vpx (kips)	Vx (kips), LRFD	Vx (kips), ASD	Wall L (ft)	V (plf), ASD	SW type	Wall H (ft)	OT (k), ASD	HD
BF-G	21.0	27.2	19.1	18.3	1040	2W2	3.1	5.3	HDU5

Main House Roof Diaphragm -- W.T.S. Designs (NS)

BF ID	Vpx (kips), LRFD	Vx (kips), LRFD	Vx (plf), ASD	Max Spacing (in)
BF-D	23.1	30.1	271	48
BF-G Left	27.7	36.0	324	36
BF-G cripple total	21.0	27.2	1021	12

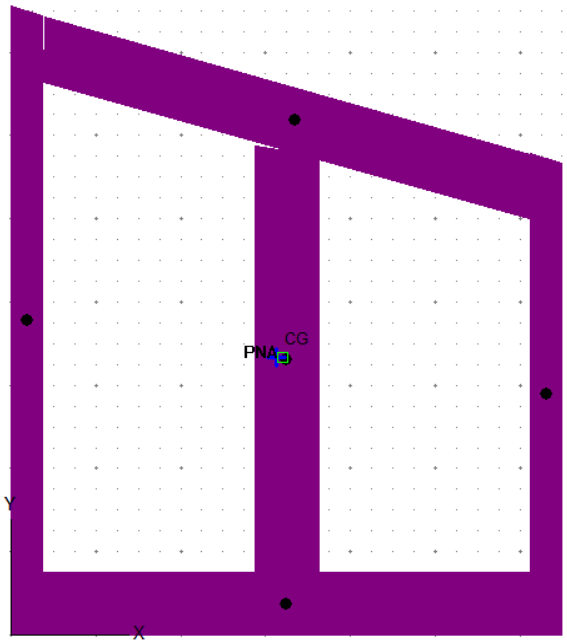
Strut Compression Capacity (BUILT UP MEMBER)

AISC Specification Chapter E

SHAPE HAVE ---

Overall Properties

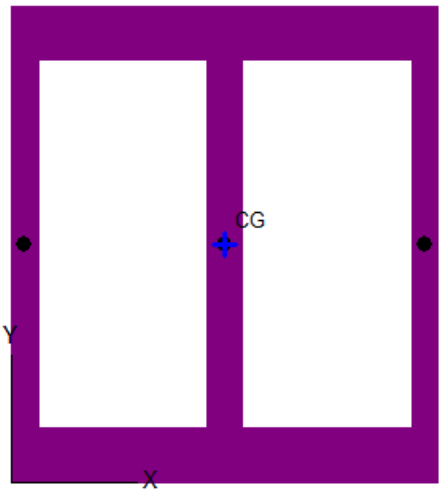
Name: Shape1	Sy left: 180.1 in^3
Area: 70.13 in^2	Sy right: 167.7 in^3
CG: (6.269,6.670)	rx: 4.875 in
Ix: 1667 in^4	ry: 4.013 in
Iy: 1129 in^4	rp: 6.314 in
Ixy: -162.2 in^4	r1: 4.941 in
Ipolar: 2796 in^4	r2: 3.932 in
I1: 1712 in^4	Zx: 311.1 in^3
I2: 1084 in^4	Zy: 227.7 in^3
Theta: 15.55 deg	PNA: (6.397,6.670)
Sx top: 197.1 in^3	J: 37.09 in^4
Sx bottom: 249.9 in^3	



SHAPE APPROXIMATE AS ---

Overall Properties

Name: Shape1	Sy: 153.3 in^3
Area: 61.63 in^2	rx: 4.890 in
CG: (6,6.625)	ry: 3.864 in
Ix: 1474 in^4	rp: 6.232 in
Iy: 920.0 in^4	Zx: 277.2 in^3
Ixy: 0 in^4	Zy: 197.0 in^3
Ipolar: 2394 in^4	J: 31.33 in^4
Sx: 222.4 in^3	



Beam =	BUILT UP		tw left =	0.75	in
L =	30.83	ft	h middle =	10.25	in
Fy =	50	ksi	tw middle =	1.00	in
b top =	12.00	in	h right =	10.25	in
tf top =	1.50	in	tw right =	0.75	in
b bot =	12.00	in	Ag =	61.63	in^2
tf bot =	1.50	in	Ix =	1474	in^4
h left =	10.25	in	Iy =	920	in^4
			rx =	4.89	in
			ry =	3.864	in

top flange

A =	18.00	in^2
Ix =	3.38	in^4
rx =	0.43	in
Iy =	216.00	in^4
ry =	3.46	in
a =	5.50	in
a/r in x =	12.70	
a/r in y =	1.59	

bottom flange

A =	18.00	in^2
Ix =	3.38	in^4
rx =	0.43	in
Iy =	216.00	in^4
ry =	3.46	in
a =	5.50	in
a/r in x =	12.70	
a/r in y =	1.59	

left web

A =	7.69	in^2
Ix =	67.31	in^4
rx =	2.96	in
Iy =	0.36	in^4
ry =	0.22	in
a =	10.25	in
a/r in x =	3.46	
a/r in y =	47.34	

right web

A =	7.69	in^2
Ix =	67.31	in^4
rx =	2.96	in
Iy =	0.36	in^4
ry =	0.22	in
a =	10.25	in
a/r in x =	3.46	
a/r in y =	47.34	

middle web

A =	10.25	in^2
Ix =	89.74	in^4
rx =	2.96	in
Iy =	0.85	in^4
ry =	0.29	in
a =	10.25	in
a/r in x =	3.46	
a/r in y =	35.51	

governing a/r

a/r in x =	12.70	--> since this is < 40, use E6-2a
a/r in y =	47.34	--> since this is > 40, use E6-2b

governing KL/r

rx overall =	4.89	in
KL/rx original =	75.66	
KL/rx modified =	75.66	
ry overall =	3.86	in
Ki =	0.86	
governing a/ry =	47.34	
KL/ry original =	95.76	
KL/ry modified =	104.05	
KL/r max =	104.05	

Slender checks

slender limit =	35.88

top flange

bf/2tf =	4.00
slender?	No

bottom flange

bf/2tf =	4.00
slender?	No

left web

h/tw =	13.67
slender?	No

right web

h/tw =	13.67
slender?	No

middle web

h/tw =	10.25
slender?	No

OVERALL

slender?	NO

Governing compressive strength

ϕPn =	**1257**	**kips**

COMPRESSIVE STRENGTH (withOUT slender elements):

COMPRESSIVE FLEXURAL BUCKLING (withOUT slender elements):

4.71*SQRT(E/Fy) =	113.43	
Fe =	26.44	ksi
Fcr =	22.66	ksi
Pn =	1396	kips
ϕPn =	**1257**	**kips**

COMPRESSIVE FLEXURAL TORSIONAL BUCKLING (withOUT slender elements):

ho =	11.75	in
Cw =	31754	in^6
G =	11200	ksi
J =	31.33	in^4
KLz =	370	in
Fe =	174.30	ksi
4.71*SQRT(E/Fy) =	113.43	
Fcr =	44.34	ksi
Pn =	2733	kips
ϕPn =	**2460**	**kips**

CONCRETE DIAPHRAGMS

The concrete diaphragms are designed using a custom spreadsheet. Section cuts were created in ETABS and section cut forces were extracted from the software program and imported into the spreadsheet. These forces were compared against the calculated Fpx forces and if the ETABS forces were less, they were scaled up to the Fpx forces. The diaphragm capacity is based on the shear capacity of the concrete slab. Where the diaphragms connect to braced frames, the capacity is limited by the shear stud connection value. Where the diaphragms connect to concrete walls, the shear friction capacity is compared to the diaphragm capacity and the smaller value controls. All of the design checks are shown in the spreadsheet output on the subsequent pages.

Fx - Fpx SCALING

Cs	0.234
V (ELF), ρ=1.0	135.3
K	1
Sds	1.172
I	1

ELF

Vertical Distribution

Strength ρ= 1.3

Level	hx (ft)	Wx	hx^k (ft)	$Wxhx^k$	Cvx (%)	Story Shear Strength Fx (k)	ΣV (k)	Diaphragm Min Fpx (k)	Diaphragm Force (ρ not included) Max Fpx (k)	Fpx (k)	γ=Fpx/Fx	ELF Scaling Fpx/Fx
Roof	22.7	168.0	22.7	3813.6	0.456	80.19	80.2	39.4	78.8	61.7	0.77	1.00
Main	11.1	410.0	11.1	4551.0	0.544	95.70	175.9	96.1	192.2	96.1	1.00	1.00
	Σ	578.0		8364.6		175.89						

Diaphragm Capacity

Summary	ϕVn [lb, per ft]	
Decking	11184	
Studs (3/4" at 16" oc)	12931	
Main Floor Conc	11184	3.5" conc w/#3@12"oc

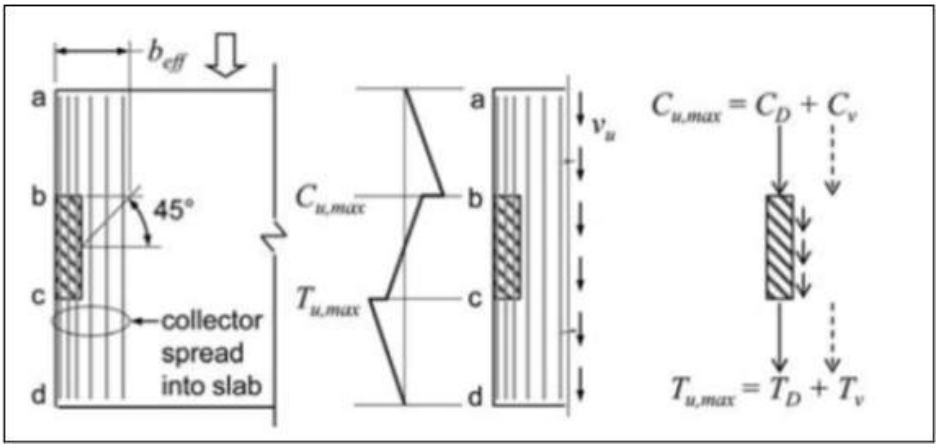
Concrete shear strength

	phiϕ	concrete thk [in]	length [in	area [in2]	fc' [psi]	ϕVc [lb]	Bar size	spacing	Av [in2]	# layers of bars	Fy [psi]	ϕVs [lb]	ϕVn [lb] [per ft]	ϕVn,limit [lb] [per ft]	ϕVn [lb] [per ft]
Main Floor	0.75	3.5	12	42	4000	3984.47	#4	12	0.2	1	60000	7200	11184	15938	11184

Qn		
Stud Dia	0.75	in
Area	0.441786	in2
Ec	3604997	psi
Qn	26.52558	k
Phi*Qn	17.24162	k/stud
Studs @ 16" oc	12931.22	plf

CONCRETE DIAPHRAGMS

Constants		
Fy	60	ksi
Fc'	4	ksi
ϕb	0.9	
ϕv	0.6	
μ, at shear wall	0.6	
μ, at collectors	0.6	
Irregularity Factor	1.25	
ρ, for Fx	1.3	
ρ, for Fpx forces	1	
Ω	2	
Start At ID	0	
End At ID	29	
Fu	65	ksi



MaxID	ID	Section Cut	X or Y?	Wall or Brace?	Fx (from ETABS, ρ=1.3) [k]	Fx*1.25 [k]	Fpx Scale Factor	Fpx (ρ=1.0)	Design Vu [k]	L(B-C) Wall/BF Length [ft]	Design Vu [klf]
1	1	SC-WA-R	X	Wall	74.51	93.13	1.00	57.56	93.13	21.23	4.39
2	2	SC-WB-1-R	X	Wall	61.34	76.67	1.00	47.38	76.67	7.43	10.32
3	3	SC-WB-2-R	X	Wall	63.86	79.83	1.00	49.33	79.83	11.13	7.17
4	4	SC-WB-1-L	X	Wall	50.40	62.99	1.00	38.93	62.99	7.43	8.48
5	5	SC-WB-2-L	X	Wall	54.35	67.94	1.00	41.98	67.94	11.13	6.10
6	6	SC-WD-1-R	X	Wall	204.11	255.13	1.00	157.67	255.13	21.33	11.96
7	7	SC-WD-2-R	X	Wall	270.48	338.10	1.00	208.94	338.10	33.23	10.17
8	8	SC-BF-D-R	X	Brace	7.44	9.30	1.00	5.75	9.30	6.04	1.54
9	9	SC-BF-D-L	X	Brace	50.76	63.44	1.00	39.21	63.44	6.04	10.50
10	10	SC-BF-G-R	X	Brace	10.07	12.59	1.00	7.78	12.59	8.00	1.57
11	11	SC-BF-G-L	X	Brace	25.45	31.81	1.00	19.66	31.81	8.00	3.98
17	17	SC-W1-R	Y	Wall	78.86	98.57	1.00	60.92	98.57	21.75	4.53
18	18	SC-W3-R	Y	Wall	319.47	399.34	1.00	246.79	399.34	50.94	7.84
19	19	SC-W5-L	Y	Wall	127.41	159.26	1.00	98.42	159.26	38.31	4.16
20	20	SC-BF-4-L	Y	Brace	44.20	55.26	1.00	34.15	55.26	4.92	11.24
21	21	SC-BF-4-R	Y	Brace	66.09	82.61	1.00	51.05	82.61	4.92	16.80
22	22	SC-BF-6-L	Y	Brace	52.63	65.79	1.00	40.66	65.79	6.51	10.11
23	23	SC-BF-6-R	Y	Brace	44.38	55.48	1.00	34.29	55.48	6.51	8.52
24	24	SC-BF-8-L	Y	Brace	9.30	11.63	1.00	7.19	11.63	4.99	2.33
25	25	SC-BF-8-R	Y	Brace	8.75	10.93	1.00	6.76	10.93	4.99	2.19
0	0										

CONCRETE DIAPHRAGMS

Section Cut	X or Y?	Conc	slab shear Capacity [klf]	Collector reqd?	Collector Length L(A-B) [ft]	Collector Length L(C-D) [ft]	Check Diaphragm Shear	Total Collector force [k]	Collector, Vu (A-B) [k]	Collector, Vu (C-D) [k]	Deck type at collectors	Collector A-B klf	Check Collector A-B	Collector C-D klf	Check Collector C-D
SC-WA-R	X	Main Floo									Main Floor Conc				
SC-WB-1-R	X	Main Floo									Main Floor Conc				
SC-WB-2-R	X	Main Floo									Main Floor Conc				
SC-WB-1-L	X	Main Floo									Main Floor Conc				
SC-WB-2-L	X	Main Floo									Main Floor Conc				
SC-WD-1-R	X	Main Floo									Main Floor Conc				
SC-WD-2-R	X	Main Floo									Main Floor Conc				
SC-BF-D-R	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-D-L	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-G-R	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-G-L	X	Main Floo	11.18	no							Main Floor Conc				
SC-W1-R	Y	Main Floo									Main Floor Conc				
SC-W3-R	Y	Main Floo									Main Floor Conc				
SC-W5-L	Y	Main Floo									Main Floor Conc				
SC-BF-4-L	Y	Main Floo	11.18	yes	5		ok	0.33	0.33	0	Main Floor Conc				
SC-BF-4-R	Y	Main Floo	11.18	yes	5		ok	34	34	0	Main Floor Conc				
SC-BF-6-L	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-6-R	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-8-L	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-8-R	Y	Main Floo	11.18	no							Main Floor Conc				
											Main Floor Conc				

			Conc to Wall (limit capacity to shear friction)					*checks min of shear friction, concrete shear capacity, and limit						
Section Cut	X or Y?	Conc	Bar size	Bar Spg	shear capacity, ϕVn [klf]	ϕVn (shear friction) [klf]	ϕVn limit [klf]	Collector reqd?	Collector Length L(A-B) [ft]	Collector Length L(C-D) [ft]	Diaphragm Shear	Total Collector force [k]	Vu (A-B) [k]	Vu (C-D) [k]
SC-WA-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-1-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-2-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-1-L	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-2-L	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WD-1-R	X	Main Floo	#5	6	11.18	13.39	33.60	yes		5	ok	20.35	20.35	0.00
SC-WD-2-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-BF-D-R	X	Main Floo												
SC-BF-D-L	X	Main Floo												
SC-BF-G-R	X	Main Floo												
SC-BF-G-L	X	Main Floo												
SC-W1-R	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-W3-R	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-W5-L	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-BF-4-L	Y	Main Floo												
SC-BF-4-R	Y	Main Floo												
SC-BF-6-L	Y	Main Floo												
SC-BF-6-R	Y	Main Floo												
SC-BF-8-L	Y	Main Floo												
SC-BF-8-R	Y	Main Floo												

CONCRETE DIAPHRAGMS

Section Cut	X or Y?	Conc	slab shear Capacity [klf]	Collector reqd?	Collector Length L(A-B) [ft]	Collector Length L(C-D) [ft]	Check Diaphragm Shear	Total Collector force [k]	Collector, Vu (A-B) [k]	Collector, Vu (C-D) [k]	Deck type at collectors	Collector A-B klf	Check Collector A-B	Collector C-D klf	Check Collector C-D
SC-WA-R	X	Main Floo									Main Floor Conc				
SC-WB-1-R	X	Main Floo									Main Floor Conc				
SC-WB-2-R	X	Main Floo									Main Floor Conc				
SC-WB-1-L	X	Main Floo									Main Floor Conc				
SC-WB-2-L	X	Main Floo									Main Floor Conc				
SC-WD-1-R	X	Main Floo									Main Floor Conc				
SC-WD-2-R	X	Main Floo									Main Floor Conc				
SC-BF-D-R	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-D-L	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-G-R	X	Main Floo	11.18	no							Main Floor Conc				
SC-BF-G-L	X	Main Floo	11.18	no							Main Floor Conc				
SC-W1-R	Y	Main Floo									Main Floor Conc				
SC-W3-R	Y	Main Floo									Main Floor Conc				
SC-W5-L	Y	Main Floo									Main Floor Conc				
SC-BF-4-L	Y	Main Floo	11.18	yes	5		ok	0.33	0.33	0	Main Floor Conc				
SC-BF-4-R	Y	Main Floo	11.18	yes	5		ok	34	34	0	Main Floor Conc				
SC-BF-6-L	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-6-R	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-8-L	Y	Main Floo	11.18	no							Main Floor Conc				
SC-BF-8-R	Y	Main Floo	11.18	no							Main Floor Conc				
											Main Floor Conc				

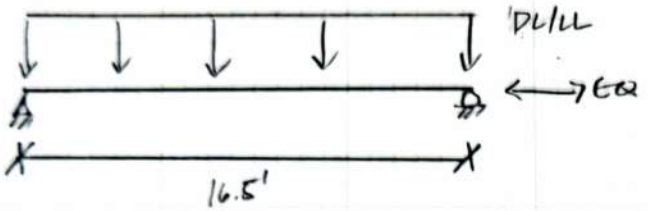
			Conc to Wall (limit capacity to shear friction)					*checks min of shear friction, concrete shear capacity, and limit						
Section Cut	X or Y?	Conc	Bar size	Bar Spg	shear capacity, ϕVn [klf]	ϕVn (shear friction) [klf]	ϕVn limit [klf]	Collector reqd?	Collector Length L(A-B) [ft]	Collector Length L(C-D) [ft]	Diaphragm Shear	Total Collector force [k]	Vu (A-B) [k]	Vu (C-D) [k]
SC-WA-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-1-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-2-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-1-L	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WB-2-L	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-WD-1-R	X	Main Floo	#5	6	11.18	13.39	33.60	yes		5	ok	20.35	20.35	0.00
SC-WD-2-R	X	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-BF-D-R	X	Main Floo												
SC-BF-D-L	X	Main Floo												
SC-BF-G-R	X	Main Floo												
SC-BF-G-L	X	Main Floo												
SC-W1-R	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-W3-R	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-W5-L	Y	Main Floo	#5	6	11.18	13.39	33.60	no						
SC-BF-4-L	Y	Main Floo												
SC-BF-4-R	Y	Main Floo												
SC-BF-6-L	Y	Main Floo												
SC-BF-6-R	Y	Main Floo												
SC-BF-8-L	Y	Main Floo												
SC-BF-8-R	Y	Main Floo												

MAIN FLOOR DRAGS

BEAM @ BT-4

TOTAL COLLECTOR FORCE = 35K (INCLUDES Ω=2)

W12X50 BEAM



GRAVITY LOADS: TRIB = $8.67'/_2 = 4.33'$

DL = 75 PSF x 4.33' = 325 PLF

LL = 40 PSF x 4.33' = 173 PLF

EQ = 35K +/-

SEE VA PRINTOUT

COMBINED DCR = 0.13 (AXIAL + FLEXURE)

DRAG CONNECTION:

PL1/2: ϕR_n = 0.9(50 KSI)(0.5IN x 6IN) = 135 K OK
IN TENSION

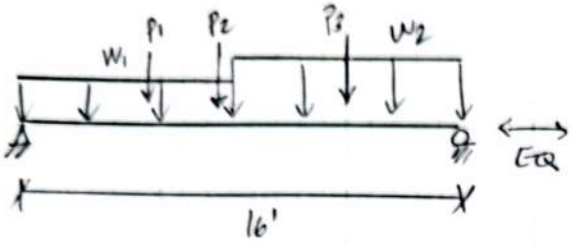
ϕR_{nw} = 1.392(5) = 6.96 K/IN x 6IN (2) = 83.5 K OK

SWENSON SAY FAGET
SEATTLE 2124 Third Ave Suite 100 Seattle, WA 98121 O 206.443.6212
TACOMA 934 Broadway Suite 100 Tacoma, WA 98402 O 253.284.9470

BF-D DRAG

W12x72 BEAM

TOTAL COLLECTOR FORCE = 20.4K → MAX 25K



W_1 DL = 238 PLF 0'to7'
LL = 100 PLF

W_2 DL = 288 PLF 7'to16'
LL = 160 PLF

P_1 DL = 5.3K @X = 5.33'
LL = 2.5K

P_2 DL = 2.66K @X = 7'
LL = 1.12K

P_3 DL = 5.3K @X = 10.67'
LL = 2.5K

EQ = +/- 25K

SEE VA PRINTOUT DCR (COMBINED AXIAL & FLEXURE) = 0.23

DRAG CONNECTION W/PL 1/2 & 5/16" FILLET OK
(PER PREVIOUS CALC W/EQ = 35K)

STRUCTURAL ENGINEERING

B480
PROJECT LATERAL

10/25/22
DATE

PROJ # SRW
DESIGN
SHEET

DRAG @ BF-D

ASSUMING 25K COLLECTOR FORCE
DRAGGED F/ W12X72 INTO BF BEAM & INTO
CONCRETE WALL.

BF BEAM ALREADY CHECKED FOR COMBINED LOADING
W/ AXIAL LOAD = EXPECTED STRENGTH OF BRACE
PER BF SPREADSHEET $P_{u,E} = 53K$ (>25K SO BEAM OK)

CONNECT BEAM TO CONCRETE WALL
PER TYP DRAG STRUT @ CONC WALL
USE SHEAR TAB FOR GRAVITY ONLY
T & B PLATES FOR AXIAL

$\phi R_n = 0.9(50KSI)(0.5IN \times 6IN) = 135K$ PER PLATE — TENSILE YIELDING OK

WELDS: $1.392(5) = 6.96 K/IN \times (6IN \times 2) = 83.5K$ EA PLATE TO BEAM OK
$1.392(5) = 6.96 K/IN \times 6IN = 41.8K$ EA PLATE TO EMBED OK

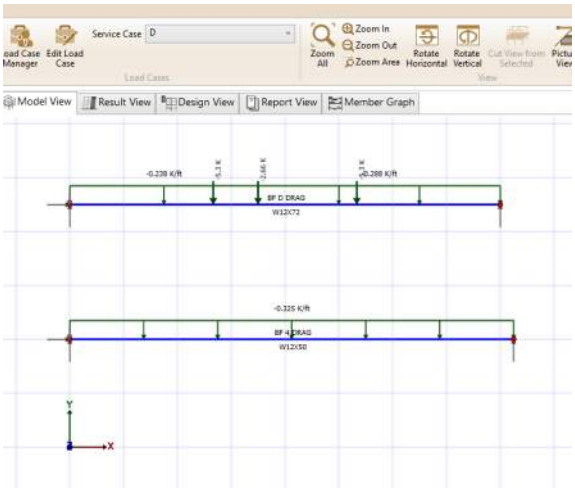
EMBED PLATE W/ REBAR:
(6) #6 BARS X46"

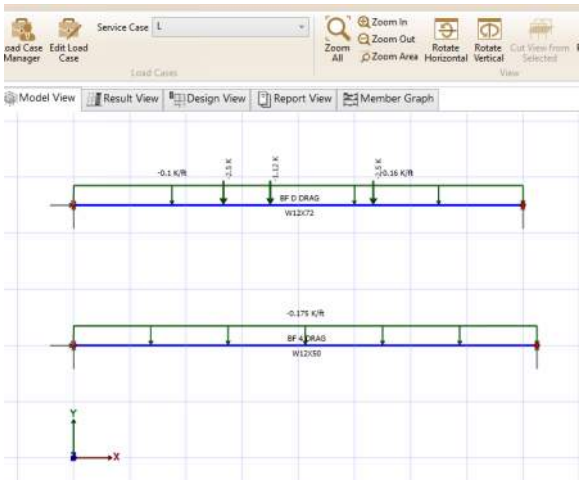
$0.9(60KSI)\left(\frac{\pi(0.75IN)^2}{4}\right) = 23.86K/BAR$ DEVELOP INTO WALL 4KSI $l_d = 37"_{MIN}$

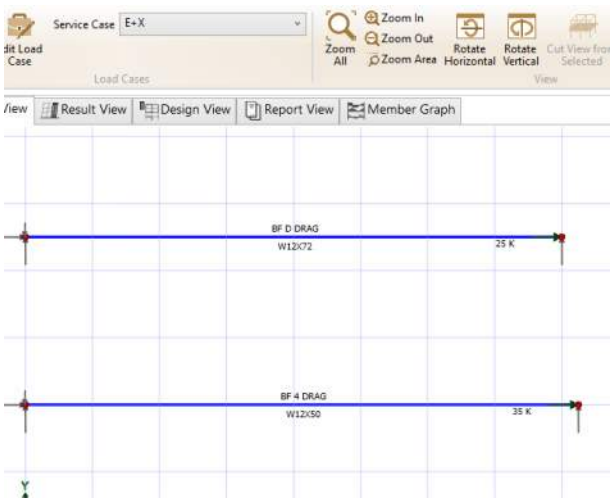
WELD:
$1.392(4) = 5.568 K/IN$
$P = \pi d = \pi(6/8 IN) = 2.36 IN$
$5.568 K/IN \times 2.36 IN = 13.11K$ 1BAR X 6 BARS = 78.7K OK
1/4" FILLET ALL AROUND

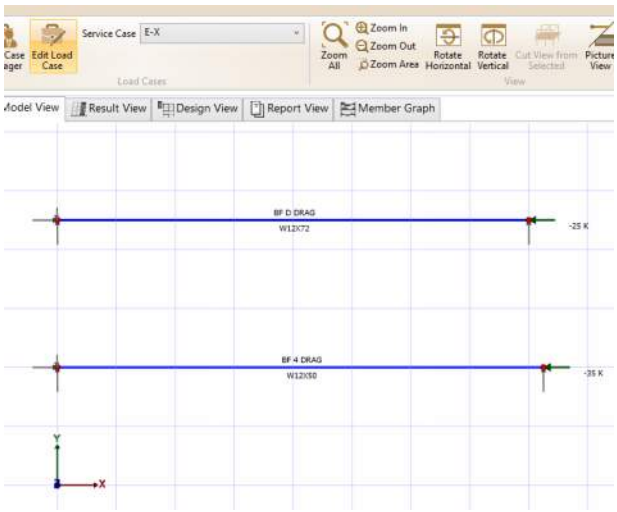
SWENSON SAY FAGET
SEATTLE 2124 Third Ave, Suite 100, Seattle, WA 98121 | O 206.443.6212
TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402 | O 253.284.9470

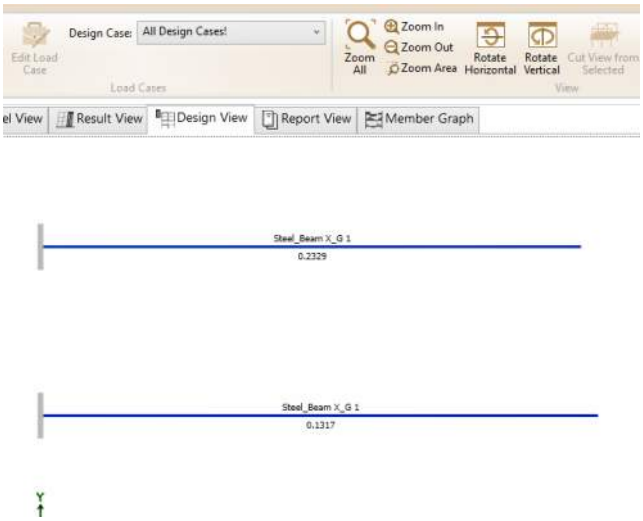
MAIN FLOOR DRAGS - VISUAL ANALYSIS











CONCRETE SHEARWALLS

8480 - Wall Shear Design

0 Piers > 6rt

Pier Design Data						Demand			
Label	Story	Pier ID	Wall Length	Wall Thickness	Concrete Strength	Governing EQ Load Case	Vu, analysis	Vu / √f'c Ac	Area
111 Piers Total								0 Piers Over	
(story-pier)			ft	in	ksi		kips		in^2
Main-P1	Main	P1	22.4	8.0	4.0	EQXNTNYP	55.3	0.41	2148.0
Main-P3	Main	P3	51.4	8.0	4.0	EQXNTNYP	243.6	0.78	4938.0
Main-P5	Main	P5	51.4	8.0	4.0	EQYNTNXP	395.0	1.26	4938.0
Main-PA	Main	PA	22.6	8.0	4.0	EQXNTNYN	21.2	0.15	2165.9
Main-PD-1	Main	PD-1	21.8	8.0	4.0	EQXNTNYP	172.3	1.30	2096.0
Main-PD-2	Main	PD-2	33.2	8.0	4.0	EQXNTNYP	174.0	0.86	3190.0
Main-PB-1	Main	PB-1	8.1	8.0	4.0	EQXPTNYN	34.9	0.71	777.0
Main-PB-2	Main	PB-2	11.8	8.0	4.0	EQYNTNXP	50.3	0.70	1133.0

8480 - Wall Shear Design

8 Piers min governs

Pier Design Data			Calculate Reinforcement Required						Final Design						
Label	Story	Pier ID	ΦVc	ΦVs req'd	rho, n	Min Count	Reinf Req'd	Min Mark	Mark	Vu	ΦVn	Demand/	ΦVn	8Af'c	Demand/
111 Piers Total			phi = 0.6			8	in^2/ft	Req'd	Provided		phi = 0.6	Capacity	phi = 0.6	phi = 0.6	Capacity
(story-pier)			kips	kips			in^2			kips	kips	SSF	kips	kips	COD
Main-P1	Main	P1	163.0	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	55	485	0.12	485	652	0.12
Main-P3	Main	P3	374.8	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	244	1115	0.22	1115	1499	0.22
Main-P5	Main	P5	374.8	20.2	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	395	1115	0.36	1115	1499	0.36
Main-PA	Main	PA	164.4	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	21	489	0.05	489	658	0.05
Main-PD-1	Main	PD-1	159.1	13.2	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	172	473	0.37	473	636	0.37
Main-PD-2	Main	PD-2	242.1	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	174	721	0.25	721	968	0.25
Main-PB-1	Main	PB-1	59.0	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	35	176	0.20	176	236	0.20
Main-PB-2	Main	PB-2	86.0	0.0	0.0025	Min Governs	0.24	#4 @ 18" EF	#4 @ 12" EF	50	256	0.20	256	344	0.20

TABLE: Pier Forces

Story	Pier	Output Case	Location	P	V2	V3	T	M2	M3
				kip	kip	kip	kip-ft	kip-ft	kip-ft
Main	P1	EQXNTNYN	Top	-1.017152338	46.60484908	0.315914483	-5.266532105	-0.037420836	-30.21833297
Main	P1	EQXNTNYN	Bottom	-15.34586529	47.13591522	-0.097810922	-8.212149994	0.077790348	646.4765356
Main	P1	EQXNTNYP	Top	-0.902984283	54.81667838	0.318138901	-6.537732802	-0.028547777	51.82522977
Main	P1	EQXNTNYP	Bottom	-13.08881787	55.33469035	-0.107419677	-9.635134258	0.054607951	794.7051228
Main	P1	EQXNTPYN	Top	-1.017152338	46.60484908	0.315914483	-5.266532105	-0.037420836	-30.21833297
Main	P1	EQXNTPYN	Bottom	-15.34586529	47.13591522	-0.097810922	-8.212149994	0.077790348	646.4765356
Main	P1	EQXNTPYP	Top	-0.902984283	54.81667838	0.318138901	-6.537732802	-0.028547777	51.82522977
Main	P1	EQXNTPYP	Bottom	-13.08881787	55.33469035	-0.107419677	-9.635134258	0.054607951	794.7051228
Main	P1	EQXPTNYN	Top	0.825468675	-54.61299288	-0.31967689	6.268391211	0.024046699	-50.95913996
Main	P1	EQXPTNYN	Bottom	12.33530105	-55.1391444	0.109688126	9.397331733	-0.051525509	-784.1184426
Main	P1	EQXPTNYP	Top	0.939636731	-46.40116358	-0.317452472	4.997190514	0.032919758	31.08442278
Main	P1	EQXPTNYP	Bottom	14.59234848	-46.94036927	0.10007937	7.97434747	-0.074707907	-635.8898554
Main	P1	EQXPTPYN	Top	0.825468675	-54.61299288	-0.31967689	6.268391211	0.024046699	-50.95913996
Main	P1	EQXPTPYN	Bottom	12.33530105	-55.1391444	0.109688126	9.397331733	-0.051525509	-784.1184426
Main	P1	EQXPTPYP	Top	0.939636731	-46.40116358	-0.317452472	4.997190514	0.032919758	31.08442278
Main	P1	EQXPTPYP	Bottom	14.59234848	-46.94036927	0.10007937	7.97434747	-0.074707907	-635.8898554
Main	P1	EQYNTNXN	Top	-0.576357195	0.796084057	0.086396504	0.058765167	-0.030541369	-132.4028483
Main	P1	EQYNTNXN	Bottom	-8.859636426	0.958650765	-0.011548312	-0.535026626	0.061113486	-29.75030838
Main	P1	EQYNTNXP	Top	-0.023570891	-29.56926853	-0.104280908	3.519242162	-0.012101108	-138.6250903
Main	P1	EQYNTNXP	Bottom	-0.555286522	-29.72386712	0.050701402	4.747817893	0.022318729	-458.9288018
Main	P1	EQYNTPXN	Top	-0.576357195	0.796084057	0.086396504	0.058765167	-0.030541369	-132.4028483
Main	P1	EQYNTPXN	Bottom	-8.859636426	0.958650765	-0.011548312	-0.535026626	0.061113486	-29.75030838
Main	P1	EQYNTPXP	Top	-0.023570891	-29.56926853	-0.104280908	3.519242162	-0.012101108	-138.6250903
Main	P1	EQYNTPXP	Bottom	-0.555286522	-29.72386712	0.050701402	4.747817893	0.022318729	-458.9288018
Main	P1	EQYPTNXN	Top	-0.19579701	28.16884839	0.093811232	-4.178570491	-0.000964504	141.0756942
Main	P1	EQYPTNXN	Bottom	-1.336145005	28.28790118	-0.043577498	-5.278307504	-0.016161171	464.3449824
Main	P1	EQYPTNXP	Top	0.356989294	-2.196504193	-0.09686618	-0.718093496	0.017475756	134.8534521
Main	P1	EQYPTNXP	Bottom	6.968204899	-2.394616705	0.018672216	0.004537014	-0.054955929	35.16648893
Main	P1	EQYPTPXN	Top	-0.19579701	28.16884839	0.093811232	-4.178570491	-0.000964504	141.0756942
Main	P1	EQYPTPXN	Bottom	-1.336145005	28.28790118	-0.043577498	-5.278307504	-0.016161171	464.3449824
Main	P1	EQYPTPXP	Top	0.356989294	-2.196504193	-0.09686618	-0.718093496	0.017475756	134.8534521
Main	P1	EQYPTPXP	Bottom	6.968204899	-2.394616705	0.018672216	0.004537014	-0.054955929	35.16648893
Main	P3	EQXNTNYN	Top	24.30789001	-191.4292847	0.129408698	-18.32781349	-0.895781778	621.136629
Main	P3	EQXNTNYN	Bottom	94.86584132	-193.7878506	0.81359616	-16.83888084	0.838283847	479.8169262
Main	P3	EQXNTNYP	Top	30.0910909	-240.618024	-1.081437457	3.858977962	0.13061587	700.9289691
Main	P3	EQXNTNYP	Bottom	126.0930111	-243.5513077	1.20197432	-22.53628322	0.828985631	341.9723835
Main	P3	EQXNTPYN	Top	24.30789001	-191.4292847	0.129408698	-18.32781349	-0.895781778	621.136629
Main	P3	EQXNTPYN	Bottom	94.86584132	-193.7878506	0.81359616	-16.83888084	0.838283847	479.8169262
Main	P3	EQXNTPYP	Top	30.0910909	-240.618024	-1.081437457	3.858977962	0.13061587	700.9289691
Main	P3	EQXNTPYP	Bottom	126.0930111	-243.5513077	1.20197432	-22.53628322	0.828985631	341.9723835
Main	P3	EQXPTNYN	Top	-30.67962549	234.6012393	1.32015852	-6.604316448	-0.357010712	-681.3428361
Main	P3	EQXPTNYN	Bottom	-133.6457318	237.4909858	-1.269302863	22.80492753	-0.742187516	-314.251346
Main	P3	EQXPTNYP	Top	-24.8964246	185.4125	0.109312365	15.58247501	0.669386936	-601.550496
Main	P3	EQXPTNYP	Bottom	-102.4185621	187.7275288	-0.880924703	17.10752516	-0.751485732	-452.0958887
Main	P3	EQXPTPYN	Top	-30.67962549	234.6012393	1.32015852	-6.604316448	-0.357010712	-681.3428361
Main	P3	EQXPTPYN	Bottom	-133.6457318	237.4909858	-1.269302863	22.80492753	-0.742187516	-314.251346
Main	P3	EQXPTPYP	Top	-24.8964246	185.4125	0.109312365	15.58247501	0.669386936	-601.550496
Main	P3	EQXPTPYP	Bottom	-102.4185621	187.7275288	-0.880924703	17.10752516	-0.751485732	-452.0958887
Main	P3	EQYNTNXN	Top	-0.970131577	5.36962776	1.870666919	-41.02731394	-1.929792306	88.35693009
Main	P3	EQYNTNXN	Bottom	-17.57690983	5.345702469	-0.312757476	3.414135297	0.282464217	326.1168943
Main	P3	EQYNTNXP	Top	-17.46638623	133.178785	2.227891865	-37.51026482	-1.768160987	-302.3869094
Main	P3	EQYNTNXP	Bottom	-86.13038177	134.7293534	-0.937627183	15.30727781	-0.191677192	87.89641264
Main	P3	EQYNTPXN	Top	-0.970131577	5.36962776	1.870666919	-41.02731394	-1.929792306	88.35693009
Main	P3	EQYNTPXN	Bottom	-17.57690983	5.345702469	-0.312757476	3.414135297	0.282464217	326.1168943
Main	P3	EQYNTPXP	Top	-17.46638623	133.178785	2.227891865	-37.51026482	-1.768160987	-302.3869094
Main	P3	EQYNTPXP	Bottom	-86.13038177	134.7293534	-0.937627183	15.30727781	-0.191677192	87.89641264
Main	P3	EQYPTNXN	Top	18.30720471	-158.5928366	-2.16548693	32.92865758	1.491533186	354.3313971
Main	P3	EQYPTNXN	Bottom	86.513656	-160.5324877	0.98183639	-15.57720595	0.251470161	-133.3649147

Main	P3	EQYPTNXP	Top	1.810950059	-30.7836794	-1.808261983	36.4457067	1.653164505	-36.41244245
Main	P3	EQYPTNXP	Bottom	17.96018406	-31.14883674	0.356966684	-3.684063442	-0.222671248	-371.5853964
Main	P3	EQYPTPXN	Top	18.30720471	-158.5928366	-2.16548693	32.92865758	1.491533186	354.3313971
Main	P3	EQYPTPXN	Bottom	86.513656	-160.5324877	0.98183639	-15.57720595	0.251470161	-133.3649147
Main	P3	EQYPTPXP	Top	1.810950059	-30.7836794	-1.808261983	36.4457067	1.653164505	-36.41244245
Main	P3	EQYPTPXP	Bottom	17.96018406	-31.14883674	0.356966684	-3.684063442	-0.222671248	-371.5853964
Main	P5	EQXNTNYN	Top	3.426162784	100.0944292	-29.21858836	737.5729754	39.38617154	56.15324476
Main	P5	EQXNTNYN	Bottom	-56.51174748	93.53622813	3.381992522	-73.6755031	0.082080743	-498.1012083
Main	P5	EQXNTNYP	Top	-6.39406816	290.7687257	-54.09546351	1349.59164	69.14208515	-177.732447
Main	P5	EQXNTNYP	Bottom	-129.412354	285.068956	6.734472511	-143.3612884	-0.443337432	-206.6419479
Main	P5	EQXNTPYN	Top	3.426162784	100.0944292	-29.21858836	737.5729754	39.38617154	56.15324476
Main	P5	EQXNTPYN	Bottom	-56.51174748	93.53622813	3.381992522	-73.6755031	0.082080743	-498.1012083
Main	P5	EQXNTPYP	Top	-6.39406816	290.7687257	-54.09546351	1349.59164	69.14208515	-177.732447
Main	P5	EQXNTPYP	Bottom	-129.412354	285.068956	6.734472511	-143.3612884	-0.443337432	-206.6419479
Main	P5	EQXPTNYN	Top	8.784867775	-302.3429886	55.80977626	-1375.633334	-68.94582347	187.8959022
Main	P5	EQXPTNYN	Bottom	143.9382657	-298.0059776	-7.274716137	152.6050385	0.908868094	130.1423871
Main	P5	EQXPTNYP	Top	-1.035363169	-111.6686922	30.93290111	-763.6146697	-39.18990985	-45.98978956
Main	P5	EQXPTNYP	Bottom	71.03765918	-106.4732497	-3.922236148	82.91925326	0.383449919	421.6016475
Main	P5	EQXPTPYN	Top	8.784867775	-302.3429886	55.80977626	-1375.633334	-68.94582347	187.8959022
Main	P5	EQXPTPYN	Bottom	143.9382657	-298.0059776	-7.274716137	152.6050385	0.908868094	130.1423871
Main	P5	EQXPTPYP	Top	-1.035363169	-111.6686922	30.93290111	-763.6146697	-39.18990985	-45.98978956
Main	P5	EQXPTPYP	Bottom	71.03765918	-106.4732497	-3.922236148	82.91925326	0.383449919	421.6016475
Main	P5	EQYNTNXN	Top	17.15898058	-273.7660973	29.01975324	-706.3894572	-33.20566015	412.8178317
Main	P5	EQYNTNXN	Bottom	95.85517346	-277.4881473	-4.121368579	84.11703235	0.801313977	-618.1384846
Main	P5	EQYNTNXP	Top	18.76659207	-394.4973227	54.52826263	-1340.35135	-65.70525865	452.3406289
Main	P5	EQYNTNXP	Bottom	155.9901774	-394.950809	-7.318381177	152.0011948	1.049350182	-429.665406
Main	P5	EQYNTPXN	Top	17.15898058	-273.7660973	29.01975324	-706.3894572	-33.20566015	412.8178317
Main	P5	EQYNTPXN	Bottom	95.85517346	-277.4881473	-4.121368579	84.11703235	0.801313977	-618.1384846
Main	P5	EQYNTPXP	Top	18.76659207	-394.4973227	54.52826263	-1340.35135	-65.70525865	452.3406289
Main	P5	EQYNTPXP	Bottom	155.9901774	-394.950809	-7.318381177	152.0011948	1.049350182	-429.665406
Main	P5	EQYPTNXN	Top	-15.57512257	361.8148908	-53.90316393	1333.672758	65.98071857	-366.8011409
Main	P5	EQYPTNXN	Bottom	-147.1468482	360.954279	7.053564719	-148.1689185	-0.95007994	353.3923834
Main	P5	EQYPTNXP	Top	-13.96751107	241.0836654	-28.39465454	699.7108652	33.48112007	-327.2783437
Main	P5	EQYPTNXP	Bottom	-87.01184421	243.4916173	3.856552121	-80.28475599	-0.702043735	541.865462
Main	P5	EQYPTPXN	Top	-15.57512257	361.8148908	-53.90316393	1333.672758	65.98071857	-366.8011409
Main	P5	EQYPTPXN	Bottom	-147.1468482	360.954279	7.053564719	-148.1689185	-0.95007994	353.3923834
Main	P5	EQYPTPXP	Top	-13.96751107	241.0836654	-28.39465454	699.7108652	33.48112007	-327.2783437
Main	P5	EQYPTPXP	Bottom	-87.01184421	243.4916173	3.856552121	-80.28475599	-0.702043735	541.865462
Main	PA	EQXNTNYN	Top	0.663642487	-21.06686033	-0.150333289	-0.676100731	-0.002495791	13.75923388
Main	PA	EQXNTNYN	Bottom	8.090460956	-21.2166251	0.039272414	0.450650152	-0.012417876	-115.4273461
Main	PA	EQXNTNYP	Top	0.143901663	-18.28140023	-0.080255708	-0.607535575	-0.009815508	13.35882662
Main	PA	EQXNTNYP	Bottom	5.754192527	-18.39856468	0.021485779	0.598777248	0.013282473	-68.8001804
Main	PA	EQXNTPYN	Top	0.663642487	-21.06686033	-0.150333289	-0.676100731	-0.002495791	13.75923388
Main	PA	EQXNTPYN	Bottom	8.090460956	-21.2166251	0.039272414	0.450650152	-0.012417876	-115.4273461
Main	PA	EQXNTPYP	Top	0.143901663	-18.28140023	-0.080255708	-0.607535575	-0.009815508	13.35882662
Main	PA	EQXNTPYP	Bottom	5.754192527	-18.39856468	0.021485779	0.598777248	0.013282473	-68.8001804
Main	PA	EQXPTNYN	Top	-0.081962102	2.015845549	-0.006039566	-0.463397073	-0.009654861	-7.845297889
Main	PA	EQXPTNYN	Bottom	-4.733727925	1.953902032	0.005201126	-0.36867629	-0.01958313	-58.8227186
Main	PA	EQXPTNYP	Top	-0.601702926	4.801305649	0.064038015	-0.394831917	-0.016974579	-8.245705142
Main	PA	EQXPTNYP	Bottom	-7.069996354	4.771962452	-0.012585509	-0.220549193	0.00611722	-12.19555295
Main	PA	EQXPTPYN	Top	-0.081962102	2.015845549	-0.006039566	-0.463397073	-0.009654861	-7.845297889
Main	PA	EQXPTPYN	Bottom	-4.733727925	1.953902032	0.005201126	-0.36867629	-0.01958313	-58.8227186
Main	PA	EQXPTPYP	Top	-0.601702926	4.801305649	0.064038015	-0.394831917	-0.016974579	-8.245705142
Main	PA	EQXPTPYP	Bottom	-7.069996354	4.771962452	-0.012585509	-0.220549193	0.00611722	-12.19555295
Main	PA	EQYNTNXN	Top	1.361810463	-10.47769987	-0.257061661	-0.208166163	0.003165213	5.023530433
Main	PA	EQYNTNXN	Bottom	8.836735704	-10.57453217	0.070098332	-0.091329271	-0.053963289	-104.9332256
Main	PA	EQYNTNXP	Top	1.138129087	-3.552888111	-0.213773544	-0.144355066	0.001017492	-1.457829097
Main	PA	EQYNTNXP	Bottom	4.98947904	-3.623374031	0.059876945	-0.337127204	-0.056112865	-87.95183735
Main	PA	EQYNTPXN	Top	1.361810463	-10.47769987	-0.257061661	-0.208166163	0.003165213	5.023530433
Main	PA	EQYNTPXN	Bottom	8.836735704	-10.57453217	0.070098332	-0.091329271	-0.053963289	-104.9332256
Main	PA	EQYNTPXP	Top	1.138129087	-3.552888111	-0.213773544	-0.144355066	0.001017492	-1.457829097

Main	PA	EQYNTPXP	Bottom	4.98947904	-3.623374031	0.059876945	-0.337127204	-0.056112865	-87.95183735
Main	PA	EQYPTNXN	Top	-0.37065895	-1.192832875	-0.023469725	0.020384356	-0.021233845	3.68883959
Main	PA	EQYPTNXN	Bottom	1.049174276	-1.180997437	0.010809548	0.402427718	0.031704543	50.49065993
Main	PA	EQYPTNXP	Top	-0.594340327	5.731978888	0.019818392	0.084195453	-0.023381566	-2.792519939
Main	PA	EQYPTNXP	Bottom	-2.798082388	5.770160704	0.000588162	0.156629785	0.029554967	67.47204816
Main	PA	EQYPTPXN	Top	-0.37065895	-1.192832875	-0.023469725	0.020384356	-0.021233845	3.68883959
Main	PA	EQYPTPXN	Bottom	1.049174276	-1.180997437	0.010809548	0.402427718	0.031704543	50.49065993
Main	PA	EQYPTPXP	Top	-0.594340327	5.731978888	0.019818392	0.084195453	-0.023381566	-2.792519939
Main	PA	EQYPTPXP	Bottom	-2.798082388	5.770160704	0.000588162	0.156629785	0.029554967	67.47204816
Main	PD-1	EQXNTNYN	Top	-21.31423925	-162.0307889	-1.476962858	-22.02613415	-0.092254	253.8667291
Main	PD-1	EQXNTNYN	Bottom	-71.93018465	-162.6610237	-0.225406986	-3.280001368	-0.93970933	-672.7574588
Main	PD-1	EQXNTNYP	Top	-23.84448905	-172.300066	-1.752796182	-23.5515212	-1.155399004	279.8529321
Main	PD-1	EQXNTNYP	Bottom	-84.48530294	-172.0421039	-0.154143331	-3.934974757	-1.303155422	-689.7164255
Main	PD-1	EQXNTPYN	Top	-21.31423925	-162.0307889	-1.476962858	-22.02613415	-0.092254	253.8667291
Main	PD-1	EQXNTPYN	Bottom	-71.93018465	-162.6610237	-0.225406986	-3.280001368	-0.93970933	-672.7574588
Main	PD-1	EQXNTPYP	Top	-23.84448905	-172.300066	-1.752796182	-23.5515212	-1.155399004	279.8529321
Main	PD-1	EQXNTPYP	Bottom	-84.48530294	-172.0421039	-0.154143331	-3.934974757	-1.303155422	-689.7164255
Main	PD-1	EQXPTNYN	Top	23.09696901	167.1718602	1.684049514	22.87572885	1.237175219	-270.1032922
Main	PD-1	EQXPTNYN	Bottom	82.08814464	166.832337	0.141713019	3.747094284	1.279696027	674.9736971
Main	PD-1	EQXPTNYP	Top	20.56671921	156.9025831	1.408216191	21.3503418	0.174030216	-244.1170891
Main	PD-1	EQXPTNYP	Bottom	69.53302635	157.4512568	0.212976675	3.092120894	0.916249936	658.0147303
Main	PD-1	EQXPTPYN	Top	23.09696901	167.1718602	1.684049514	22.87572885	1.237175219	-270.1032922
Main	PD-1	EQXPTPYN	Bottom	82.08814464	166.832337	0.141713019	3.747094284	1.279696027	674.9736971
Main	PD-1	EQXPTPYP	Top	20.56671921	156.9025831	1.408216191	21.3503418	0.174030216	-244.1170891
Main	PD-1	EQXPTPYP	Bottom	69.53302635	157.4512568	0.212976675	3.092120894	0.916249936	658.0147303
Main	PD-1	EQYNTNXN	Top	-3.15332458	-37.16149602	-0.143612597	-5.23016544	1.664140378	45.30026277
Main	PD-1	EQYNTNXN	Bottom	-4.322018025	-38.70640109	-0.169228246	-0.0826052	0.245519587	-184.1442823
Main	PD-1	EQYNTNXP	Top	10.1700379	61.5992987	0.804691115	8.240393461	2.062969144	-111.8907436
Main	PD-1	EQYNTNXP	Bottom	41.88348076	60.14160712	-0.059092244	2.025523495	0.911341194	220.1750644
Main	PD-1	EQYNTPXN	Top	-3.15332458	-37.16149602	-0.143612597	-5.23016544	1.664140378	45.30026277
Main	PD-1	EQYNTPXN	Bottom	-4.322018025	-38.70640109	-0.169228246	-0.0826052	0.245519587	-184.1442823
Main	PD-1	EQYNTPXP	Top	10.1700379	61.5992987	0.804691115	8.240393461	2.062969144	-111.8907436
Main	PD-1	EQYNTPXP	Bottom	41.88348076	60.14160712	-0.059092244	2.025523495	0.911341194	220.1750644
Main	PD-1	EQYPTNXN	Top	-11.5874906	-71.39241955	-1.063057008	-10.31478895	-1.8796763	131.9209397
Main	PD-1	EQYPTNXN	Bottom	-46.17241233	-69.97666823	0.068317273	-2.265849833	-0.965967385	-240.6741715
Main	PD-1	EQYPTNXP	Top	1.735871874	27.36837518	-0.114753296	3.155769953	-1.480847534	-25.27006669
Main	PD-1	EQYPTNXP	Bottom	0.033086456	28.87133998	0.178453275	-0.157721137	-0.300145778	163.6451753
Main	PD-1	EQYPTPXN	Top	-11.5874906	-71.39241955	-1.063057008	-10.31478895	-1.8796763	131.9209397
Main	PD-1	EQYPTPXN	Bottom	-46.17241233	-69.97666823	0.068317273	-2.265849833	-0.965967385	-240.6741715
Main	PD-1	EQYPTPXP	Top	1.735871874	27.36837518	-0.114753296	3.155769953	-1.480847534	-25.27006669
Main	PD-1	EQYPTPXP	Bottom	0.033086456	28.87133998	0.178453275	-0.157721137	-0.300145778	163.6451753
Main	PD-2	EQXNTNYN	Top	-4.301198593	-115.3429282	-7.541238703	118.8125708	38.93699172	-32.39167076
Main	PD-2	EQXNTNYN	Bottom	57.58782767	-82.52984671	-1.305414038	-21.08194758	1.670128115	11.72094057
Main	PD-2	EQXNTNYP	Top	2.563115395	-173.9732968	-8.419358771	159.7578154	66.1122657	110.8522257
Main	PD-2	EQXNTNYP	Bottom	122.5791416	-113.1040074	-3.186340134	-3.28246099	3.575849564	717.6838293
Main	PD-2	EQXNTPYN	Top	-4.301198593	-115.3429282	-7.541238703	118.8125708	38.93699172	-32.39167076
Main	PD-2	EQXNTPYN	Bottom	57.58782767	-82.52984671	-1.305414038	-21.08194758	1.670128115	11.72094057
Main	PD-2	EQXNTPYP	Top	2.563115395	-173.9732968	-8.419358771	159.7578154	66.1122657	110.8522257
Main	PD-2	EQXNTPYP	Bottom	122.5791416	-113.1040074	-3.186340134	-3.28246099	3.575849564	717.6838293
Main	PD-2	EQXPTNYN	Top	-2.486405758	171.7446109	7.582078715	-151.2258289	-66.65393982	-109.0897806
Main	PD-2	EQXPTNYN	Bottom	-123.2438678	109.259191	3.501522606	-0.39164759	-3.614557599	-765.8861152
Main	PD-2	EQXPTNYP	Top	4.377908231	113.1142423	6.703958647	-110.2805843	-39.47866584	34.15411583
Main	PD-2	EQXPTNYP	Bottom	-58.25255385	78.68503029	1.620596511	17.407839	-1.708836151	-59.9232265
Main	PD-2	EQXPTPYN	Top	-2.486405758	171.7446109	7.582078715	-151.2258289	-66.65393982	-109.0897806
Main	PD-2	EQXPTPYN	Bottom	-123.2438678	109.259191	3.501522606	-0.39164759	-3.614557599	-765.8861152
Main	PD-2	EQXPTPYP	Top	4.377908231	113.1142423	6.703958647	-110.2805843	-39.47866584	34.15411583
Main	PD-2	EQXPTPYP	Bottom	-58.25255385	78.68503029	1.620596511	17.407839	-1.708836151	-59.9232265
Main	PD-2	EQYNTNXN	Top	-13.15122836	58.84237189	-1.284269834	-22.44209333	-29.28604427	-250.9668011
Main	PD-2	EQYNTNXN	Bottom	-85.32926152	26.06113828	2.505213776	-35.34356942	-2.447814488	-1084.850587
Main	PD-2	EQYNTNXP	Top	-12.60679051	144.9686336	3.252725392	-103.4536132	-60.96332373	-273.9762341
Main	PD-2	EQYNTNXP	Bottom	-139.5787702	83.59784958	3.947294769	-29.13647942	-4.033220203	-1318.132704

Main	PD-2	EQYNTPXN	Top	-13.15122836	58.84237189	-1.284269834	-22.44209333	-29.28604427	-250.9668011
Main	PD-2	EQYNTPXN	Bottom	-85.32926152	26.06113828	2.505213776	-35.34356942	-2.447814488	-1084.850587
Main	PD-2	EQYNTPXP	Top	-12.60679051	144.9686336	3.252725392	-103.4536132	-60.96332373	-273.9762341
Main	PD-2	EQYNTPXP	Bottom	-139.5787702	83.59784958	3.947294769	-29.13647942	-4.033220203	-1318.132704
Main	PD-2	EQYPTNXN	Top	9.729818273	-136.5921902	-4.211336728	114.0420553	61.29820232	226.5128536
Main	PD-2	EQYPTNXN	Bottom	131.3084517	-75.85273067	-3.764539877	23.98805254	3.904590339	1268.359042
Main	PD-2	EQYPTNXP	Top	10.27425612	-50.46592846	0.325658498	33.03053545	29.62092286	203.5034207
Main	PD-2	EQYPTNXP	Bottom	77.05894304	-18.31601937	-2.322458883	30.19514254	2.319184625	1035.076925
Main	PD-2	EQYPTPXN	Top	9.729818273	-136.5921902	-4.211336728	114.0420553	61.29820232	226.5128536
Main	PD-2	EQYPTPXN	Bottom	131.3084517	-75.85273067	-3.764539877	23.98805254	3.904590339	1268.359042
Main	PD-2	EQYPTPXP	Top	10.27425612	-50.46592846	0.325658498	33.03053545	29.62092286	203.5034207
Main	PD-2	EQYPTPXP	Bottom	77.05894304	-18.31601937	-2.322458883	30.19514254	2.319184625	1035.076925
Main	PB-1	EQXNTNYN	Top	-2.915661599	12.96676423	-0.507239816	-1.452858322	-0.342776147	-11.73519236
Main	PB-1	EQXNTNYN	Bottom	-16.84311366	13.82193264	0.043485817	-0.211818026	0.048692494	80.89916974
Main	PB-1	EQXNTNYP	Top	-5.999908145	27.66762933	-0.751049081	-2.109232896	-0.630023633	-24.10343644
Main	PB-1	EQXNTNYP	Bottom	-37.29727221	29.2285662	0.082709241	-0.107565719	0.111874126	164.8915469
Main	PB-1	EQXNTPYN	Top	-2.915661599	12.96676423	-0.507239816	-1.452858322	-0.342776147	-11.73519236
Main	PB-1	EQXNTPYN	Bottom	-16.84311366	13.82193264	0.043485817	-0.211818026	0.048692494	80.89916974
Main	PB-1	EQXNTPYP	Top	-5.999908145	27.66762933	-0.751049081	-2.109232896	-0.630023633	-24.10343644
Main	PB-1	EQXNTPYP	Bottom	-37.29727221	29.2285662	0.082709241	-0.107565719	0.111874126	164.8915469
Main	PB-1	EQXPTNYN	Top	7.135103907	-33.18590104	0.731188359	2.15086114	0.636748042	28.6036532
Main	PB-1	EQXPTNYN	Bottom	45.14952664	-34.94290666	-0.079508224	0.156414628	-0.112349764	-195.1618058
Main	PB-1	EQXPTNYP	Top	4.050857361	-18.48503594	0.487379094	1.494486566	0.349500556	16.23540913
Main	PB-1	EQXPTNYP	Bottom	24.69536808	-19.5362731	-0.040284799	0.260666935	-0.049168132	-111.1694286
Main	PB-1	EQXPTPYN	Top	7.135103907	-33.18590104	0.731188359	2.15086114	0.636748042	28.6036532
Main	PB-1	EQXPTPYN	Bottom	45.14952664	-34.94290666	-0.079508224	0.156414628	-0.112349764	-195.1618058
Main	PB-1	EQXPTPYP	Top	4.050857361	-18.48503594	0.487379094	1.494486566	0.349500556	16.23540913
Main	PB-1	EQXPTPYP	Bottom	24.69536808	-19.5362731	-0.040284799	0.260666935	-0.049168132	-111.1694286
Main	PB-1	EQYNTNXN	Top	3.789074831	-18.32498901	0.208635033	0.52413531	0.33469024	15.18210914
Main	PB-1	EQYNTNXN	Bottom	25.84254151	-19.13704594	-0.048758832	-0.234109584	-0.084466832	-102.7153424
Main	PB-1	EQYNTNXP	Top	6.804304483	-32.17078859	0.580163486	1.605251149	0.628547497	27.28376281
Main	PB-1	EQYNTNXP	Bottom	44.4403336	-33.76649773	-0.085657044	-0.123639788	-0.13277951	-185.5336351
Main	PB-1	EQYNTPXN	Top	3.789074831	-18.32498901	0.208635033	0.52413531	0.33469024	15.18210914
Main	PB-1	EQYNTPXN	Bottom	25.84254151	-19.13704594	-0.048758832	-0.234109584	-0.084466832	-102.7153424
Main	PB-1	EQYNTPXP	Top	6.804304483	-32.17078859	0.580163486	1.605251149	0.628547497	27.28376281
Main	PB-1	EQYNTPXP	Bottom	44.4403336	-33.76649773	-0.085657044	-0.123639788	-0.13277951	-185.5336351
Main	PB-1	EQYPTNXN	Top	-6.491746989	30.67789465	-0.604062514	-1.663779935	-0.622801381	-26.04537113
Main	PB-1	EQYPTNXN	Bottom	-42.33798701	32.21839926	0.081985916	0.113398105	0.126138608	177.2592482
Main	PB-1	EQYPTNXP	Top	-3.476517337	16.83209507	-0.232534062	-0.582664097	-0.328944124	-13.94371746
Main	PB-1	EQYPTNXP	Bottom	-23.74019492	17.58894747	0.045087704	0.223867902	0.07782593	94.44095556
Main	PB-1	EQYPTPXN	Top	-6.491746989	30.67789465	-0.604062514	-1.663779935	-0.622801381	-26.04537113
Main	PB-1	EQYPTPXN	Bottom	-42.33798701	32.21839926	0.081985916	0.113398105	0.126138608	177.2592482
Main	PB-1	EQYPTPXP	Top	-3.476517337	16.83209507	-0.232534062	-0.582664097	-0.328944124	-13.94371746
Main	PB-1	EQYPTPXP	Bottom	-23.74019492	17.58894747	0.045087704	0.223867902	0.07782593	94.44095556
Main	PB-2	EQXNTNYN	Top	-0.63654881	9.350008398	-1.248603307	9.228573686	2.076993255	-17.20699114
Main	PB-2	EQXNTNYN	Bottom	-7.06742118	9.190343185	0.024649197	2.426077688	0.063803983	58.9088292
Main	PB-2	EQXNTNYP	Top	0.078943759	32.89499497	-2.161201931	15.43146467	3.585697388	-50.02089041
Main	PB-2	EQXNTNYP	Bottom	-12.47373985	32.50102278	0.129489512	4.305853147	0.207224224	172.5415488
Main	PB-2	EQXNTPYN	Top	-0.63654881	9.350008398	-1.248603307	9.228573686	2.076993255	-17.20699114
Main	PB-2	EQXNTPYN	Bottom	-7.06742118	9.190343185	0.024649197	2.426077688	0.063803983	58.9088292
Main	PB-2	EQXNTPYP	Top	0.078943759	32.89499497	-2.161201931	15.43146467	3.585697388	-50.02089041
Main	PB-2	EQXNTPYP	Bottom	-12.47373985	32.50102278	0.129489512	4.305853147	0.207224224	172.5415488
Main	PB-2	EQXPTNYN	Top	-1.139297085	-49.8857033	2.132856664	-15.48634446	-3.549545342	69.79009077
Main	PB-2	EQXPTNYN	Bottom	10.29078358	-49.76750155	-0.102583836	-4.330278002	-0.212216087	-236.0646697
Main	PB-2	EQXPTNYP	Top	-0.423804516	-26.34071672	1.220258041	-9.28345347	-2.040841209	36.97619151
Main	PB-2	EQXPTNYP	Bottom	4.884464907	-26.45682195	0.002256479	-2.450502543	-0.068795845	-122.4319501
Main	PB-2	EQXPTPYN	Top	-1.139297085	-49.8857033	2.132856664	-15.48634446	-3.549545342	69.79009077
Main	PB-2	EQXPTPYN	Bottom	10.29078358	-49.76750155	-0.102583836	-4.330278002	-0.212216087	-236.0646697
Main	PB-2	EQXPTPYP	Top	-0.423804516	-26.34071672	1.220258041	-9.28345347	-2.040841209	36.97619151
Main	PB-2	EQXPTPYP	Bottom	4.884464907	-26.45682195	0.002256479	-2.450502543	-0.068795845	-122.4319501
Main	PB-2	EQYNTNXN	Top	-1.254047583	-32.53812504	1.007635117	-6.593988369	-1.663175604	44.290242

Main	PB-2	EQYNTNXN	Bottom	6.230301555	-32.21836748	-0.161661748	-2.143167267	-0.207478034	-153.6882085
Main	PB-2	EQYNTNXP	Top	-1.404872065	-50.30883855	2.022073109	-14.00846381	-3.351137183	70.38936657
Main	PB-2	EQYNTNXP	Bottom	11.43776298	-49.9057209	-0.199831658	-4.170073974	-0.290284054	-242.1802581
Main	PB-2	EQYNTPXN	Top	-1.254047583	-32.53812504	1.007635117	-6.593988369	-1.663175604	44.290242
Main	PB-2	EQYNTPXN	Bottom	6.230301555	-32.21836748	-0.161661748	-2.143167267	-0.207478034	-153.6882085
Main	PB-2	EQYNTPXP	Top	-1.404872065	-50.30883855	2.022073109	-14.00846381	-3.351137183	70.38936657
Main	PB-2	EQYNTPXP	Bottom	11.43776298	-49.9057209	-0.199831658	-4.170073974	-0.290284054	-242.1802581
Main	PB-2	EQYPTNXN	Top	1.130927646	45.94516354	-2.034360294	14.08231492	3.365838172	-65.08942223
Main	PB-2	EQYPTNXN	Bottom	-11.79076068	45.48389785	0.187805968	4.122750929	0.270589438	225.0875236
Main	PB-2	EQYPTNXP	Top	0.980103164	28.17445004	-1.019922302	6.667839479	1.677876593	-38.99029765
Main	PB-2	EQYPTNXP	Bottom	-6.583299251	27.79654443	0.149636059	2.095844222	0.187783417	136.5954739
Main	PB-2	EQYPTPXN	Top	1.130927646	45.94516354	-2.034360294	14.08231492	3.365838172	-65.08942223
Main	PB-2	EQYPTPXN	Bottom	-11.79076068	45.48389785	0.187805968	4.122750929	0.270589438	225.0875236
Main	PB-2	EQYPTPXP	Top	0.980103164	28.17445004	-1.019922302	6.667839479	1.677876593	-38.99029765
Main	PB-2	EQYPTPXP	Bottom	-6.583299251	27.79654443	0.149636059	2.095844222	0.187783417	136.5954739

Garage Lateral

STRUCTURAL ENGINEERING
SEATTLE 2124 Third Avenue, Suite 100, Seattle, WA 98121
TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402
CENTRAL WA 414 N Pearl Street, Suite 8, Ellensburg, WA 98926
206.443.6212
ssfengineers.com

Criteria Sheet

Codes

Structural	IBC 2018
Loading	ASCE 7-16
Wood:	NDS 2018
Steel:	AISC 360-16
Concrete:	ACI 318-14
Masonry:	TMS 402/602-16

Project Location

Street & Number	8480 85th Ave SE
City:	Mercer Islan State: WA
ZIP:	98040
Latitude:	47.5249 N
Longitude:	-122.2254 W
Ground Elevation	35 ft

Occupancy Category

Risk Category: II — ASCE 7 Table 1.5-1

Seismic Load Summary:

Analysis Procedure:	Equivalent Lateral Force Procedure		
Lateral System:	Special Reinforced Concrete Shear Walls		
R:	5.00	C_d=	5
Base Shear V =	14 kips	Ω_o=	2
S_S=	1.465	S_1=	0.504
S_{DS}=	1.17	S_{D1}=	0.57
C_s=	0.234	I_E=	1.0

Story Information

# Stories Above Grade (Including Mezzanine Levels)	1

Horizontal and Vertical Irregularities:

Is the building a "Regular Structure"? (No horizontal or vertical irregularities): No

Wind Load Summary:

V= 98 — K_{ZT}= 1.00 — Exposure = C

Dead Loads:

Roof			Main Floor		
Roofing	4.1	psf	Floor Finish	0.0	psf
3/4" Plywood	2.7	psf	4.5" Concrete	65.6	psf
LG Rafters @ 24"oc	2.0	psf	1.5" Metal Deck	2.9	psf
Steel Beams (seismic only)	10.0	psf	Steel Beams (seismic only)	10.0	psf
Ceiling	7.5	psf	Ceiling	4.3	psf
(N) Solar Panels & Misc	6.0	psf	Misc./Mech.	1.5	psf
	32.3	psf		84.3	psf
Use	33.0	psf	Use	85.0	psf

Typical Exterior Wall			Concrete Wall		
LG Studs @ 16"o.c.	1.7	psf	8" Concrete	100.0	psf
5/8" ply (1/2"+1/8" fire)	2.3	psf	*with siding:*		
Insulation	1.5	psf	Siding	5.0	psf
5/8" GWB	2.8	psf	Use	105.0	psf
Siding	5.0	psf	*with thin stone veneer (1.5" max):*		
Misc./Mech.	1.5	psf	Veneer (1.5" max)+thinset	21.9	psf
	14.8	psf	Use	122.0	psf
Use	15.0	psf			

Live Loads:

Snow	25	psf	Garage	40	psf
Floor	40	psf	*IBC Table 1607.1*	3000	lbs

Soils: Soils Report Provided? Yes

Allowable Bearing	n/a	psf	Active	40 + 10H/40	pcf (Restrained/Unrestrained)
Sliding, μ	n/a		Seismic Surcharge	9H	psf
Passive	165	pcf (includes FS 1.5)	Traffic Surcharge	40 x 2ft	psf

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Seismic Design

ASCE 7-16 Seismic Analysis Equivalent Lateral Force Procedure

Seismic Force Resisting System Per Table 12.2-1		
	System	Bearing Wall Systems
	Type:	Special Reinforced Concrete Shear Walls

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	D	Per soils report.
Diaphragm Flexibility	Flexible	

S_S	1.465 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.504 g	2% in 50 yr, Latitude & Longitude lookup
R	5.00	
C_d	5.0	
Ω_o	2	
I_e	1.00	Table 1.5-2
h_n	12.0 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.13 sec	
T	0.13 sec	Eq. 12.8-7
T_0	0.10 sec	
T_S	0.49 sec	
T_L	6.00 sec	
F_a	1.20	Table 11.4-1
F_v	1.70	Table 11.4-2
S_{MS}	1.76 g	Eq. 11.4-1
S_{M1}	0.86 g	Eq. 11.4-2
S_{DS}	1.172 g	Eq. 11.4-3
S_{D1}	0.571 g	Eq. 11.4-4
C_s	***0.234 Controls***	Eq. 12.8-2
	0.886	Eq. 12.8-3 need not exceed, I < I_L
	0.010	Eq. 12.8-5 or 12.8-6 minimum
C_s, design	0.234	
Bldg. Weight	61.8 k	
V = C_SW	14.5 k	Eq. 12.8-1, Strength Level Base Shear
V = C_{Sasd}W	10.1 k	Eq. 12.8-1 ASD Base Shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	1.2
F_v	

Section 12.8.1.3 Exceptions

Regular Structure	No
≤ 5 Stories above grade	Yes
T ≤ 0.5s	Yes
ρ = 1.0	No
Not Site Class E or F	Yes
Risk Category I or II	Yes

If all exceptions are met, S_{DS} may be taken as 1, but not less than 0.7*(Calculated S_{DS})

$$T_a = C_t h_n^x \quad \text{Eq. 12.8.7}$$

$$S_{MS} = F_a S_S \quad \text{Eq. 11.4-1}$$
$$S_{M1} = F_v S_1 \quad \text{Eq. 11.4-2}$$
$$S_{DS} = {}^2/_3\, S_{MS} \quad \text{Eq. 11.4-3}$$
$$S_{D1} = {}^2/_3\, S_{M1} \quad \text{Eq. 11.4-4}$$

$$C_S = \frac{S_{DS}}{(R/I_e)} \quad \text{Eq. 12.8-2}$$
$$C_S = \frac{S_{D1}}{T(R/I_e)} \quad \text{Eq. 12.8-3}$$
$$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)} \quad \text{Eq. 12.8-4}$$
$$C_S \geq 0.044 S_{DS} I_e \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.01 \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.5 \frac{S_1}{(R/I_e)} \quad \text{Eq. 12.8-6}$$

$$C_{VX} = w_x h_x^k / \sum_{i=1}^{n} w_x h_i^k \quad \text{Eq. 12.8-12}$$

$$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i}\, w_{px} \quad \text{Eq. 12.10-1}$$
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-2}$$
$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-3}$$

Vertical Distribution Strength ρ= 1.3 k= 1.000

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear Strength			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	γ=F_{px}/F_x
Roof	12.0	62	12.0	742	1.000	18.8	**18.8**	14.5	14.5	29.0	**14.5**	0.77
Σ		61.8		742		**18.8**						

Wind Design - MWFRS

ASCE 7 Chapter 27 - Directional Procedure

Design Method	Strength

Wind Coefficients

Exposure	C	
V=	98	mph
K_d=	0.85	Table 26.6-1
K_h=	0.85	Table 26.10-1
K_e=	1.00	Table 26.9-1
G=	0.85	26.9.4

Transverse Wind Pressures

L/B = 0.45 h/L = 0.52

Pressure Coefficients from Figure 27.3-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.63 / -0.14
Leeward Roof	-0.53

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.00	
Roof Type	Hip	
Roof Slope - Transverse Dir	16.25	degrees
Roof Slope - Long Dir	16.25	degrees
Ground to top of roof	14.08	ft
Bot of roof to top of roof	4.17	ft
Mean Roof Height, h	12.00	ft
Short Plan Dimension	23.25	ft
Long Plan Dimension	51.42	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft

Velocity Pressure at Mean Roof Height, q_h =	17.7	psf

Wall Pressures (Unfactored): Strength

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	17.74	12.06	7.53	**19.6**
15-20	0.9	18.78	12.77	7.53	**20.3**
20-25	0.94	19.62	13.34	7.53	**20.9**
25-30	0.98	20.45	13.91	7.53	**21.4**
30-40	1.04	21.71	14.76	7.53	**22.3**
41-50	1.09	22.75	15.47	7.53	**23.0**
51-60	1.13	23.59	16.04	7.53	**23.6**
61-70	1.17	24.42	16.61	7.53	**24.1**
71-80	1.21	25.26	17.17	7.53	**24.7**
81-90	1.24	25.88	17.60	7.53	**25.1**
91-100	1.26	26.30	17.88	7.53	**25.4**

Roof Pressures (Unfactored) Strength

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.1	-9.6	-7.9	**8.00**

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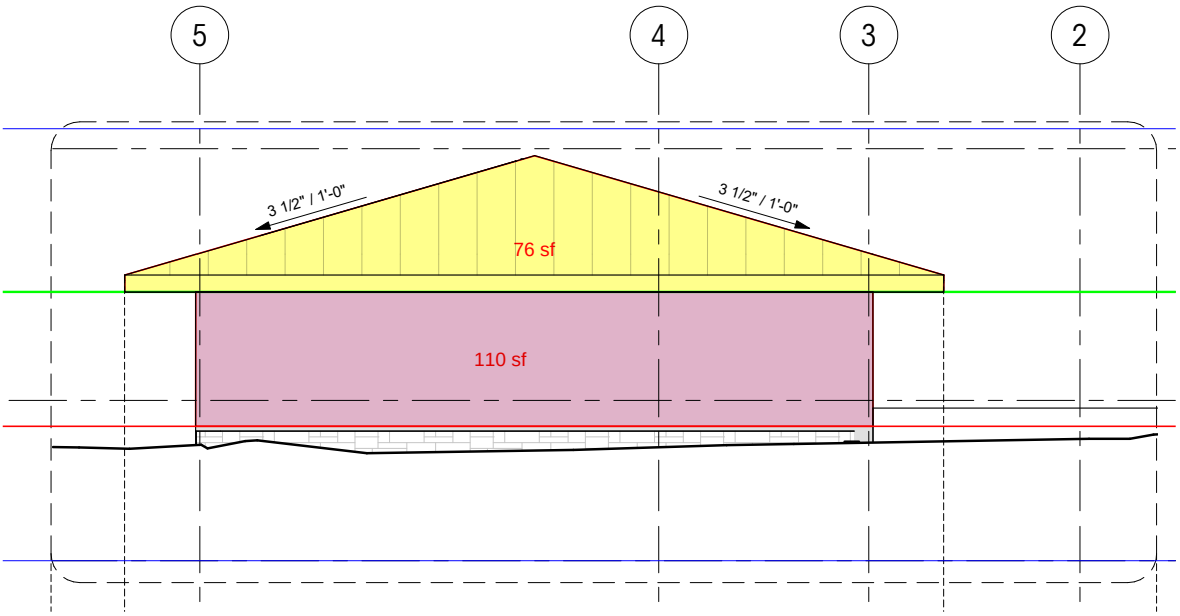
Seismic Weight

Level	Area 1	A_{TOTAL} (ft^2)	$W_{AREA\ 1}$ (psf)	$W_{AREA\ 6}$ (psf)	W_{AREA} (k)
Roof	Roof	1261	33	0	41.61
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00
			0	0	0.00

Level	Cladding 1	$W_{CLAD\ 1}$ (psf)	$L_{CLAD\ 1}$ (ft)	$H_{CLAD\ 1}$ (ft)	Cladding 2	$W_{CLAD\ 2}$ (psf)	$L_{CLAD\ 2}$ (ft)	$H_{CLAD\ 2}$ (ft)	Cladding 3	$W_{CLAD\ 3}$ (psf)	$L_{CLAD\ 3}$ (ft)	$H_{CLAD\ 3}$ (ft)
Roof	Double Siding Wall	30	4.9	5.0	Typical Exterior Wall	15	103.0	5.0	Concrete Wall	100	19.11	5.0
0		0				0		5.0		0		5.6
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0
0		0				0		0.0		0		0.0

Cladding 4	$W_{CLAD\ 4}$ (plf)	$\#Col_{CLAD\ 4}$ (#)	$H_{CLAD\ 4}$ (ft)	Cladding 5	$W_{CLAD\ 5}$ (psf)	$L_{CLAD\ 5}$ (ft)	$H_{CLAD\ 5}$ (ft)	Cladding 6	$W_{CLAD\ 6}$ (psf)	$L_{CLAD\ 6}$ (ft)	$H_{CLAD\ 6}$ (ft)	W_{CLAD} (k)	TOTAL $W_{DIST\ TO\ LVL}$ (k)
Columns	25	8	5.0	Wood Wall (interior)	12	22.58	5.0		0		5.0	20.21	62
	0		10.5		0		5.6		0		5.6	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0
	0		0.0		0		0.0		0		0.0	0.00	0

Wind Base Shear Calc - NS Direction



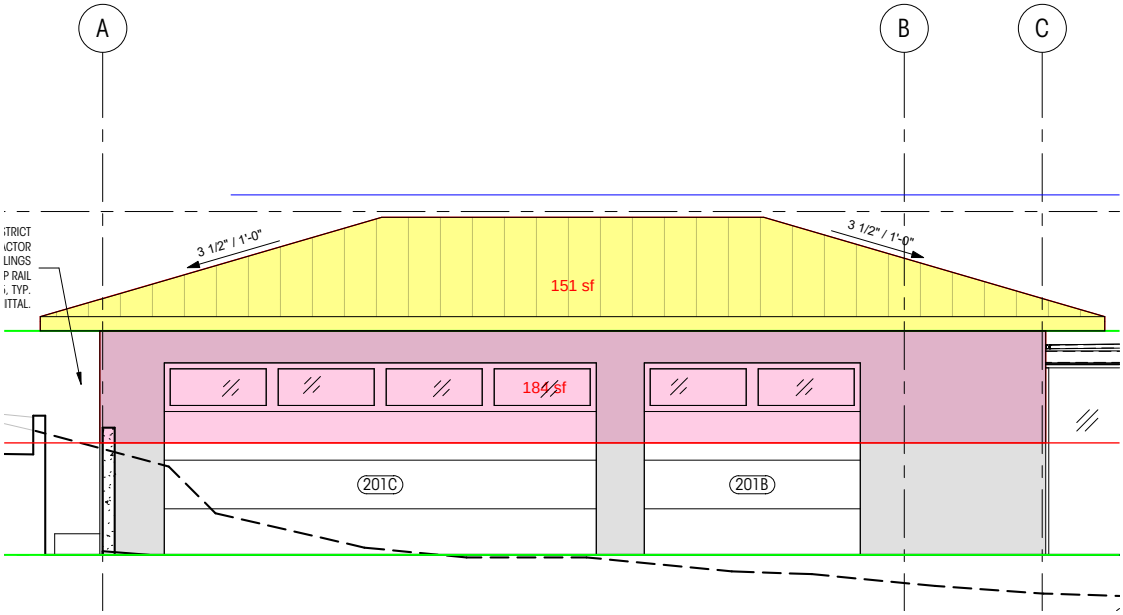
North elevation governs for NS direction

ROOF
roof: 76*8 = 608 lbs
wall: 110*19.6 = 2156 lbs

TOTAL WIND BASE SHEAR
2.76 kips

seismic governs NS

Wind Base Shear Calc - EW Direction



East and West elevations are the same

ROOF
roof: 151*8 = 1208 lbs
wall: 184*19.6 = 3607 lbs

TOTAL WIND BASE SHEAR
4.82 kips

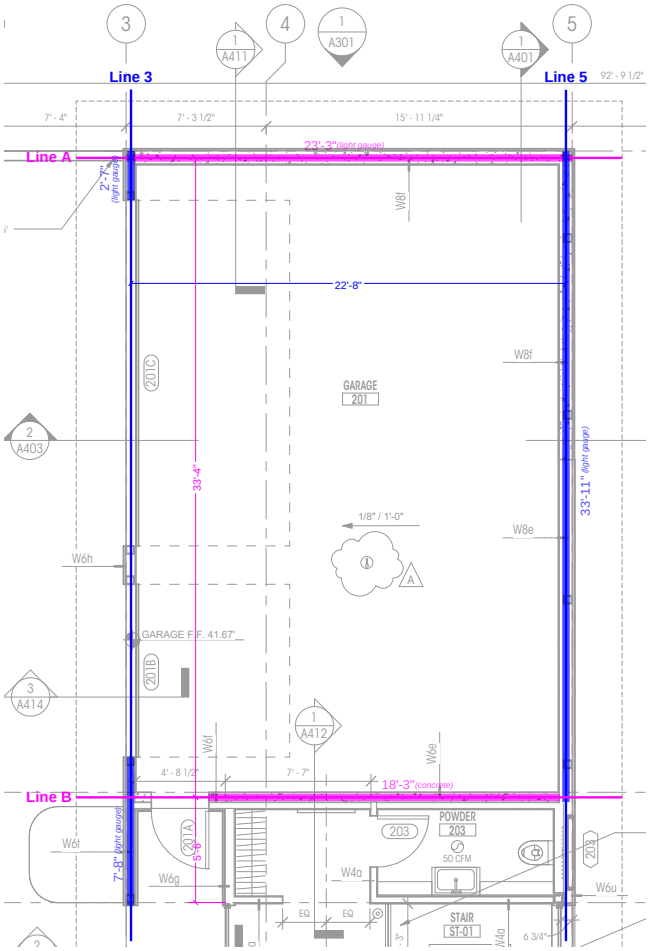
seismic governs EW

Lateral Design - N/S Direction (garage)

ROOF

WIND --- Vx = 2.80 kips
Vx = 1.68 k, ASD
w = 1.68 k / 22.67 ft
w = 74 plf

EQ --- Vx = 18.80 kips
Vx = 13.16 k, ASD
w = 13.16 k / 22.67 ft
w = 581 plf



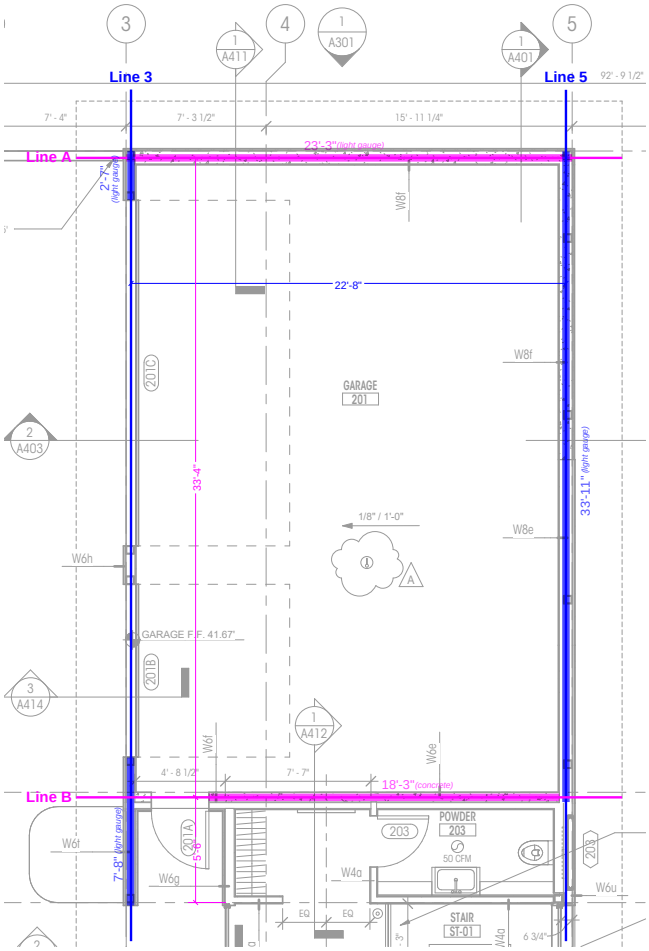
	Line 3	*Line 5*
V (k) W/EQ	0.84 / 6.58	0.84 / 6.58
V cum (k) W/EQ	0.84 / 6.58	0.84 / 6.58
L (ft)	10.25 (9.00 red)	33.92
V (plf) W/EQ	94 / 731	25 / 194
SW type	SW4	SW1
OT (k)	12.84	3.88
OT cum. (k)	12.84	3.88
HD	(2)S/HDU9	S/HDU6

Lateral Design - E/W Direction (garage)

ROOF

WIND --- Vx = 4.82 kips
Vx = 2.89 k, ASD
w = 2.89 k / 38.83 ft
w = 75 plf

EQ --- Vx = 18.80 kips
Vx = 13.16 k, ASD
w = 13.16 k / 38.83 ft
w = 339 plf



	Line B	*Line A*
V (k) W/EQ	1.67 / 7.52	1.25 / 5.65
V cum (k) W/EQ	1.67 / 7.52	1.25 / 5.65
L (ft)	18.25	23.25
V (plf) W/EQ	-	54 / 243
SW type	concrete	SW1
OT (k)	-	4.86
OT cum. (k)	-	4.86
HD	-	S/HDU6

Lateral Design - E/W Direction (garage)

CONCRETE WALL DESIGN

Line B in plane shear design

shear = 1.67 k wind, ASD = 2.79 k, LRFD
= 7.52 k seismic, ASD = 10.75 k, LRFD

L = 18.25 ft
b = 6 in
h = 11.5 ft
h/L = 0.63
α_c = 3
ϕV_c = 0.6*3*sqrt(2500)*18.25*12*6/1000 = 118.3 k, LRFD > 10.75 k

A_c = 18.25*12*6 = 1314 in^2
ρ min = 0.002
min steel = 0.002*12*6 = 0.144 in^2/ft
provide #4 @ 12"oc (0.2 in^2/ft)

moment = 10.75*11.5 = 123.6 kip-ft
lw = 219 in
d = 219-3-3 = 213 in
As = 0.2in^2
a = (0.2*60)/(0.85*2.5*6) = 0.94 in
ϕM_n = 0.65*0.2*60*(213-(0.94/2)) = 1658 k-ft > 123.6 k-ft

Line B axial
trib = (5.25/2)+(6.25/2) = 5.75 ft
DL = 20 psf --> 20*5.75*18.25 = 2.10 k
SL = 25 psf --> 25*5.75*18.25 = 2.63 k
1.2D +1.6L = 6.73 kips

ϕP_n = 0.65*0.8*0.85*2500*18.25*12*6 = 1452 kips > 6.73 kips

Lateral Design - E/W Direction (garage)

CONCRETE WALL DESIGN

Line B out of plane shear
wt = (11.5/2)*4*0.5*150 = 1725 lbs
Fp eq. = 0.4*1.172*1*1725 = 0.81 kips, LRFD
Fp min. = 0.1*3000 = 0.3 kips, LRFD < 0.81
Fp = 0.81 kips

good by inspection with #4 @ 12"oc

Line B out of plane shear anchorage
L_f = 33.33 ft
k_a = 1+(33.33/100) = 1.33 < 2.0
trib = 4.00 ft
wt = 1725
F_p eq. = 0.4*1.172*1*1.33*1725 = 1.08 kips, LRFD
F_p min. = 0.2*1*1.33*1725 = 0.46 kips, LRFD < 1.08
F_p = 1.08 kips
Use = S/LTT20 (capacity = 1.89 kips)

load getting into diaphragm
have 5.83 ft for joist S/LTT20 on
1.08 kips LRFD = 0.75 kips ASD
0.75 k/5.83 = 129 plf OK

Gravity Design

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Main House Gravity

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GARAGE LEVEL BELOW GRADE AREA CALC

WALL SEGMENT	LENGTH	COVERAGE	RESULT
A	23.52'	53%	12.50'
B	34.46'	29%	9.94'
C	23.52'	0%	0'
D	34.46'	0%	0'
TOTALS	115.96'		22.44'

TOTAL BASEMENT GSF = 810.50 SQ.FT.
PORTION OF EXCLUDED BASEMENT FLOOR AREA: (22.44/115.96) X 810.5 = 156.84 SF
NET BASEMENT GFA: (810.5 - 156.84) = **653.66 SF**

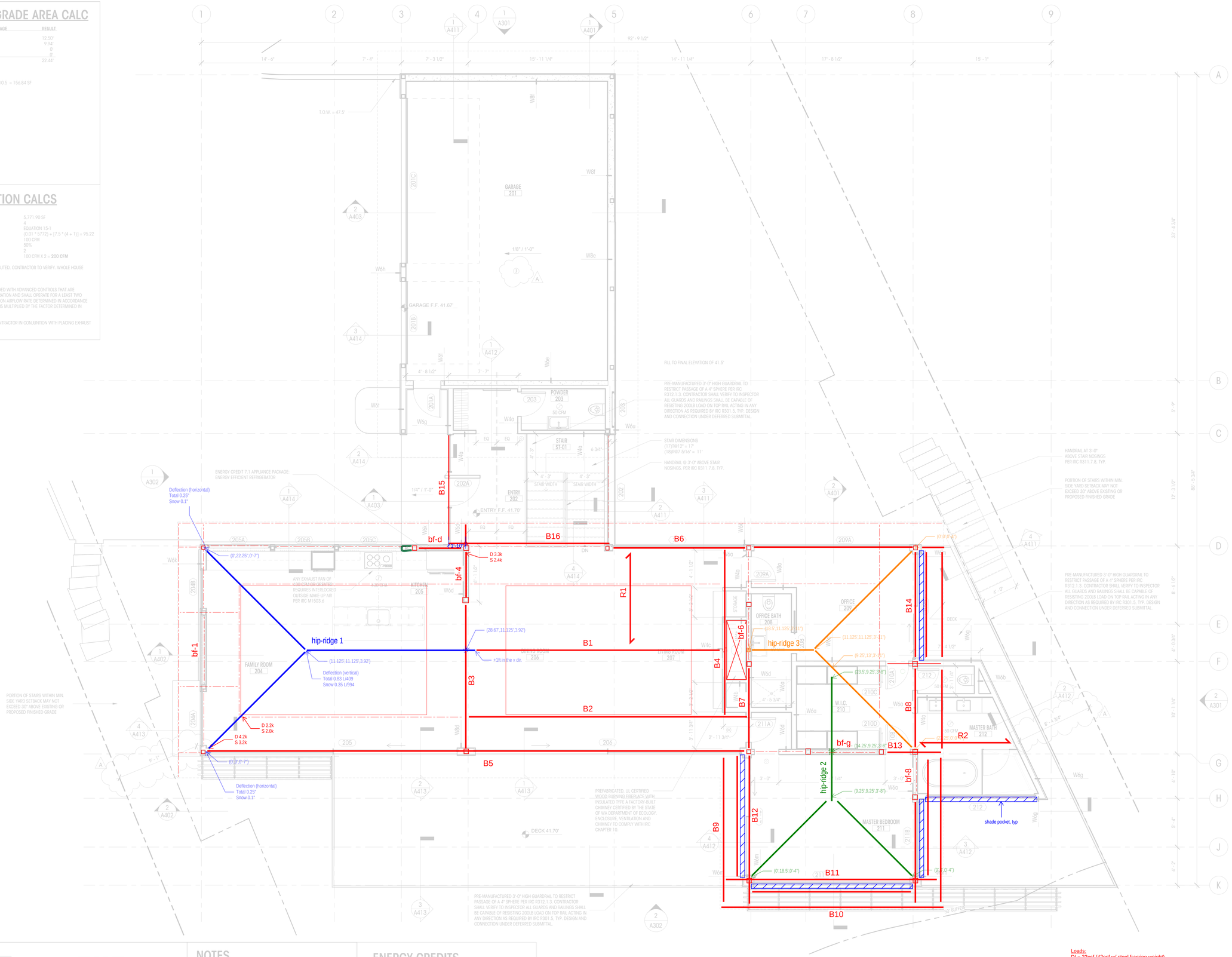
WHOLE HOUSE VENTILATION CALCS

PROPOSED CONDITIONED SF =	5,771.90 SF
NUMBER OF BEDROOMS =	4
AIRFLOW IN CFM REQUIRED FOR CONTINUOUS VENTILATION =	EQUATION 15-1 (0.01 * 5772) + [7.5 * (4 + 1)] = 95.22 100 CFM
RUN TIME PERCENTAGE IN EACH 4 HOUR SEGMENT =	50%
FACTOR =	2
CALCULATION	100 CFM X 2 = **200 CFM**

NOTE: VENTILATION SYSTEM ASSUMED TO BE BALANCED AND DISTRIBUTED, CONTRACTOR TO VERIFY. WHOLE HOUSE VENTILATION TO BE SERVED BY HRV

M1505.4.3.2 INTERMITTENT OFF OPERATION
WHOLE HOUSE MECHANICAL VENTILATION SYSTEMS SHALL BE PROVIDED WITH ADVANCED CONTROLS THAT ARE CONFIGURED TO OPERATE THE SYSTEM WITH INTERMITTENT OFF OPERATION AND SHALL OPERATE FOR A LEAST TWO HOURS IN EACH FOUR-HOUR SEGMENT. THE WHOLE HOUSE VENTILATION AIRFLOW RATE DETERMINED IN ACCORDANCE WITH SECTION M1505.4.3 AS CORRECTED BY SECTION M1505.4.3.1 IS MULTIPLIED BY THE FACTOR DETERMINED IN ACCORDANCE WITH TABLE M1505.4.3(3).

*OUTDOOR AIR INLET DUCT TO BE FIELD LOCATED WITH HVAC SUBCONTRACTOR IN CONJUNTION WITH PLACING EXHAUST DUCTS IN ORDER TO AVOID CONFLICT.



1 MAIN FLOOR PLAN
1/4" = 1'-0"

LEGEND

200A	WINDOW ID
100A	DOOR ID
100A	FINISH ID
ROOM NAME 101	ROOM ID
W4a	ASSEMBLY ID
A	STOREFRONT ID
	NEW WALL
	WALL TO REMAIN
EL= 148.5' (+0'-0") MAIN LEVEL FIN. FLR.	ELEVATION DATUM
0	GRIDLINE
	SMOKE DETECTOR
	SMOKE/CARBON MONOXIDE DETECTOR
	PROPERTY LINE
	SETBACK LINE
	ROOF OVERHANG ABOVE
	BRACE FRAME LOCATION
	FAN - 100 CFM U.N.O.
1	HEAT DETECTOR

NOTES

1. ALL DIMENSIONS AT WALLS TO FACE OF FRAMING OR TO FACE OF CONCRETE, U.N.O.
2. ALL DIMENSIONS AT KITCHEN TO EDGE OF COUNTERTOPS, U.N.O.
3. ALL INTERIOR DOOR SWING-SIDE JAMBS ARE 4" FROM ADJACENT WALL, U.N.O.
4. SEE RCP FOR SMOKE / CARBON MONOXIDE DETECTOR AND EXHAUST FAN LOCATIONS
5. ALL NEW WALLS TYPE W4A UNLESS NOTED OTHERWISE
6. ALL DIMENSIONS ASSOCIATED WITH (E) CONSTRUCTION ARE ASSUMED. CONTRACTOR TO VERIFY ALL DIMS IN FIELD AND CONTACT ARCHITECT WITH ANY DISCREPANCIES PRIOR TO CONSTRUCTION
7. CONTRACTOR TO INSTALL CARBON MONOXIDE ALARMS OUTSIDE OF EACH BEDROOM IN THE IMMEDIATE VICINITY ON EACH FLOOR LEVEL PER IRC SECTION 315.3.
8. CONTRACTOR TO INSTALL SMOKE ALARMS OUTSIDE OF EACH BEDROOM IN THE IMMEDIATE VICINITY ON EACH FLOOR LEVEL PER IRC SECTION 314.4.2.2.
9. FLOOR, CEILING, AND WALL ASSEMBLIES ARE LISTED ON SHEETS A701 & A702.

ENERGY CREDITS

EFFICIENT BUILDING ENVELOPE OPTION 1.3: (0.5)
VERTICAL FENESTRATION U = 0.28, FLOOR R-38, SLAB ON GRADE R-10 PERIMETER AND UNDER ENTIRE SLAB BELOW GRADE SLAB R-10 PERIMETER AND UNDER ENTIRE SLAB

AIR LEAKAGE CONTROL OPTION 2.2: (1.0)
RECOVERY VENTILATOR: MITSUBISHI LOSSNAY, LGHF60ORVX5

HIGH EFFICIENCY HVAC EQUIPMENT OPTION 3.1: (1.0)
HIGH EFFICIENCY HVAC: HYDRONIC BOILER VITODENS. 200-B2HA

HIGH EFFICIENCY HVAC DISTRIBUTION SYSTEM 4.2 (1.0)

RENEWABLE ELECTRIC ENERGY OPTION 6.1: (3.0)
3600 kWh PHOTOVOLTAIC SYSTEM

APPLIANCE PACKAGE OPTION 7.1: (0.5)
ENERGY EFFICIENT APPLIANCE PACKAGE

Loads:
DL= 22psf (42psf w/ steel framing weight)
SL= 25psf

Deflection Limits:
Total L/360 (0.85" Max)
Snow L/480
Glazing 1/8" (Total)

Key Plan - Roof Framing

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Design Group
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Unit 1
Seattle, WA
98121
206.239.0850
brandtdesigninc.com

NOT FOR CONSTRUCTION FOR REFERENCE ONLY

8480 RESIDENCE

8480 85TH AVE SE
MERCER ISLAND, WA 98040

PERMIT SUBMITTAL SET

DATE: 03.11.22
SHEET SIZE: E (30X42)

REVISIONS

NO	DESCRIPTION	DATE
A	PLAN CHECK 1	07.08.22

DRAWN BY: DD
CHECKED BY: KM

MAIN FLOOR PLAN

SCALE: 1/4" = 1'-0"

A212

DEDICATED APPROVAL STAMP SPACE

Gravity Design
Roof Framing

R1

L= 12' f_b= 614 psi
w= 90 plf f_v= 41 psi
R= 0.5 k Δ= 0.18"
M= 1.6 k-ft L/793

2x12@24"oc

R2

L= 12' f_b= 614 psi
w= 90 plf f_v= 41 psi
R= 0.5 k Δ= 0.18"
M= 1.6 k-ft L/793

2x12@24"oc

C1 - TYPICAL COLUMN

h=9.33ft P=24.0 k

P_n/Ω = 162 k
DCR: 24/162 = 0.15

HSS 5X5X1/2

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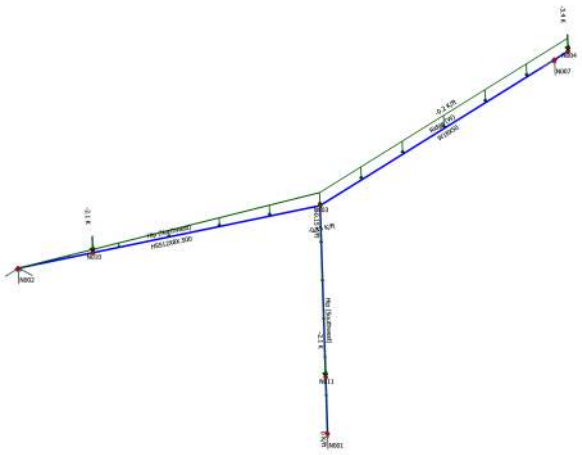
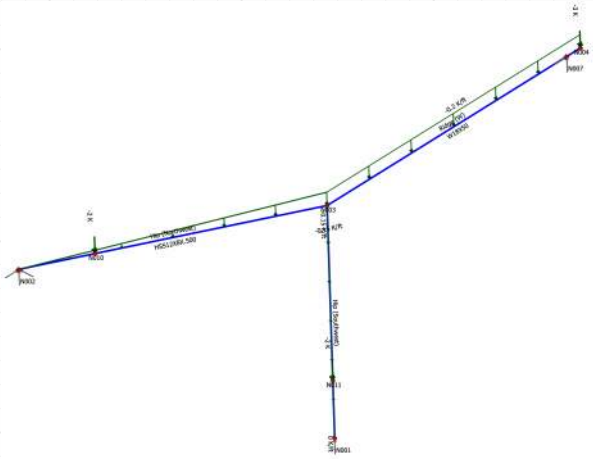
Hip Ridge 1

Dead Loads:

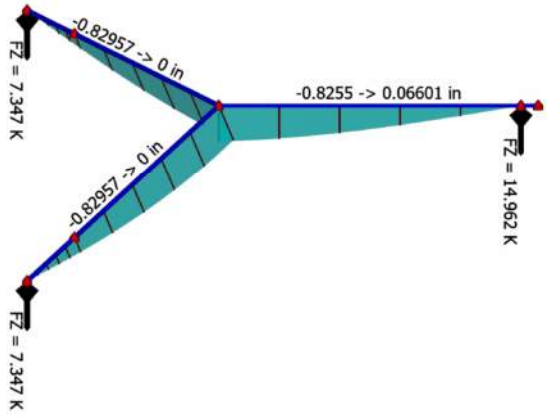


Snow Loads:

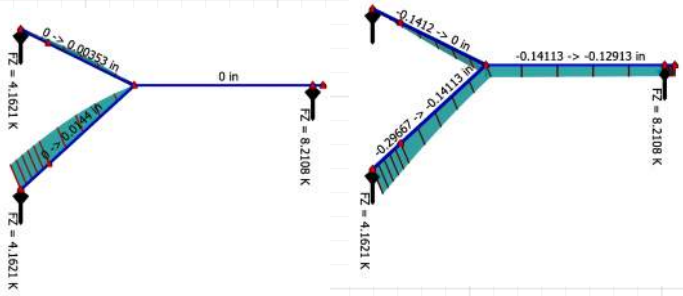


Deflection (D+S)

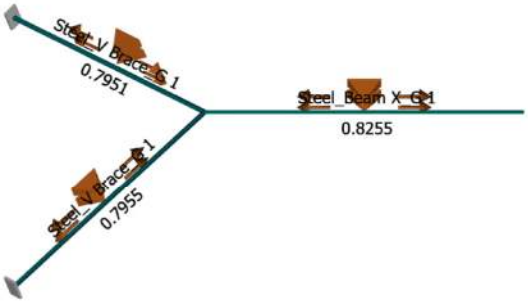
Vertical:



Horizontal:



Unity Values



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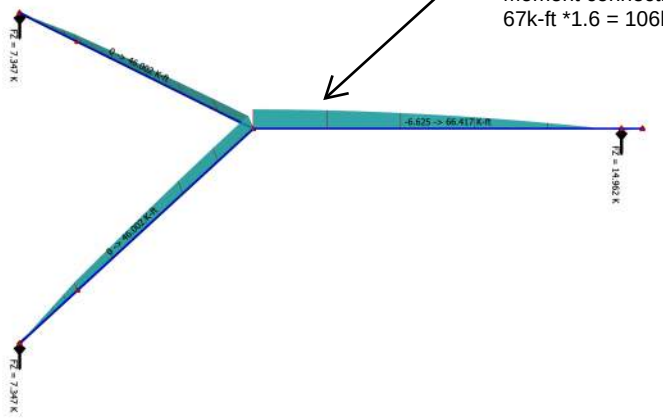
Hip Ridge 1

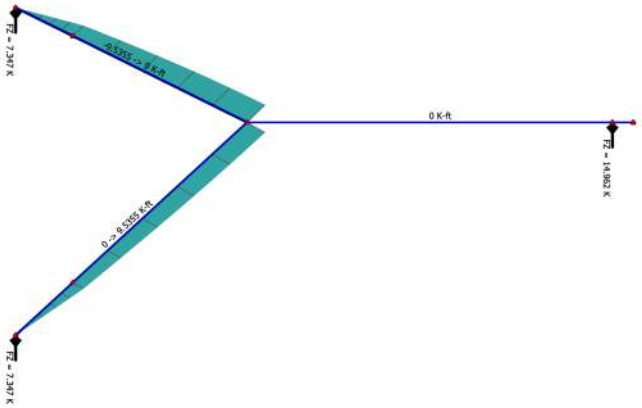
Mz Member Moments (D+S):

Max Moment = 67k-ft [asd]

Moment connection at hip ridge per garage roof calcs control
67k-ft *1.6 = 106k-ft [lrfd] < 139kft at garage roof



My Member Moments (D+S):



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Steel_Beam X_G 1: Results

Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Ridge (W)	W18X50	0.0000	D+S	-0.8255	1.0000	IBC 1604.3.1	**0.8255**	L/Δ = 269.54

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz	Capacity Mz K-ft	Code Reference	Unity Check	Details
Ridge (W)	W18X50	0.0000	3. 1.2D+1.6S+L	91.0683	378.7500	F2-1	**0.2404**	Lb = 0 ft, Cb = 1

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Ridge (W)	W18X50	17.5420	3. 1.2D+1.6S+L	-11.1544	191.7000	G2-1	**0.0582**	Shear Area = 6.39 in^2, Cv = 1, h/tw = 45.228

Steel_V Brace_G 1: Results

Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		

Steel_V Brace_G 1: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_V Brace_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy	Capacity dy in	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	15.1139	D+S	-0.7955	1.0000	IBC 1604.3.1	**0.7955**	L/Δ = 242.59

Steel_V Brace_G 1: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Hip (Northwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	0.3420	1.0000	H1-1b	**0.3420**	KLz = 16.082 ft, KLy = 0 ft, KL(torsion) = 0 ft, Lb = 0 ft, Axial Unity = 0, Mz Unity = 0.26846, My Unity = 0.07358, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1
Hip (Southwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	0.3420	1.0000	H1-1b	**0.3420**	KLz = 16.082 ft, KLy = 0 ft, KL(torsion) = 0 ft, Lb = 0 ft, Axial Unity = 0, Mz Unity = 0.26846, My Unity = 0.07358, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1

Steel_V Brace_G 1: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Hip (Northwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	2.0916	625.6451	E3-2	**0.0033**	KLz = 16.082 ft, KLy = 0 ft, KL(torsion) = 0 ft, Fcr = 40.416 Ksi, Fe (E3-4) = 148.78 Ksi, Kz = 1, Ky = 1, K(torsion) = 1
Hip (Southwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	2.0916	625.6451	E3-2	**0.0033**	KLz = 16.082 ft, KLy = 0 ft, KL(torsion) = 0 ft, Fcr = 40.416 Ksi, Fe (E3-4) = 148.78 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

Steel_V Brace_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Hip (Northwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	63.0730	234.9450	F7-1	**0.2685**	Lb = 0 ft, Cb = 1
Hip (Southwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	63.0730	234.9450	F7-1	**0.2685**	Lb = 0 ft, Cb = 1

Steel_V Brace_G 1: Weak Flexure Check (extreme rows: max)

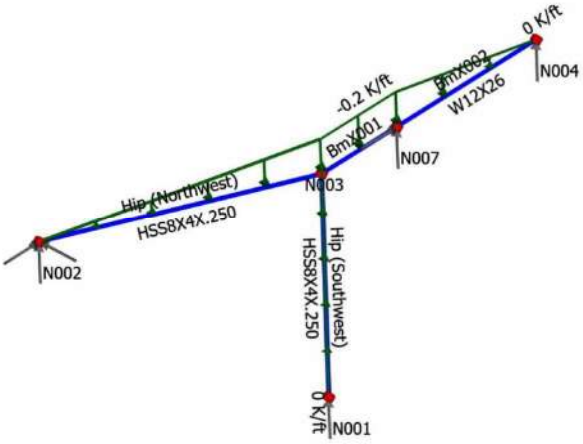
Member	Section	Offset ft	Result Case	Demand My K-ft	Capacity My K-ft	Code Reference	Unity Check	Details
Hip (Northwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	-13.0742	177.6750	F7-1	**0.0736**	
Hip (Southwest)	HSS12X8X.500	15.8403	3. 1.2D+1.6S+L	13.0742	177.6750	F7-1	**0.0736**	

Steel_V Brace_G 1: Strong Shear Check (extreme rows: max)

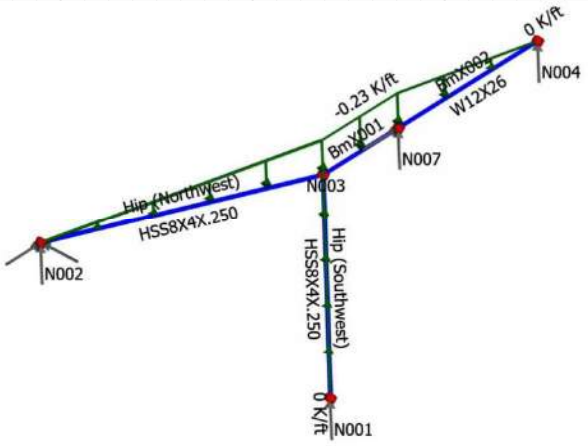
Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Hip (Northwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	9.6658	244.9882	G4-1	**0.0395**	Shear Area = 9.8627 in^2, Cv = 1
Hip (Southwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	9.6658	244.9882	G4-1	**0.0395**	Shear Area = 9.8627 in^2, Cv = 1

Hip Ridge 2

Dead Loads:



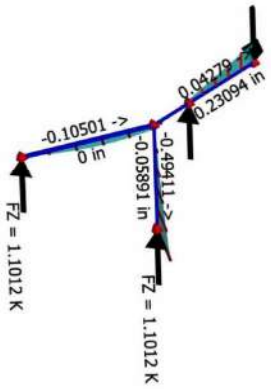
Snow Loads:

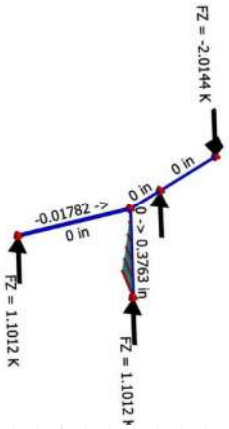


Deflection (D+S)

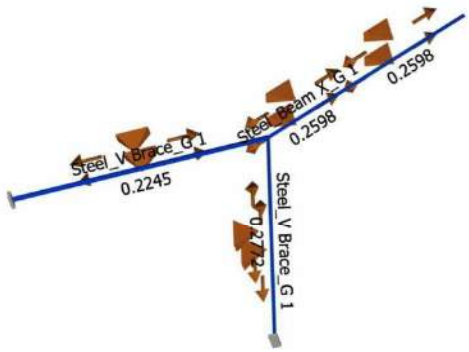
Vertical:

Horizontal:





Unity Values



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Steel_Beam X_G 1: Results

Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
BmX001	W12X26	0.0000	D+S	-0.1635	1.0000	IBC 1604.3.1	**0.1635**	L/Δ = 367.04

Steel_V Brace_G 1: Results

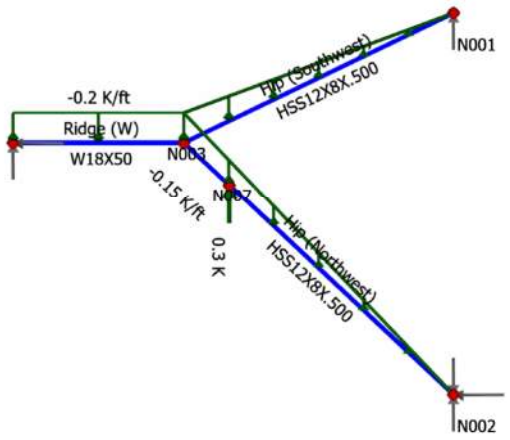
Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_V Brace_G 1: Strong Deflection Check (extreme rows: max)

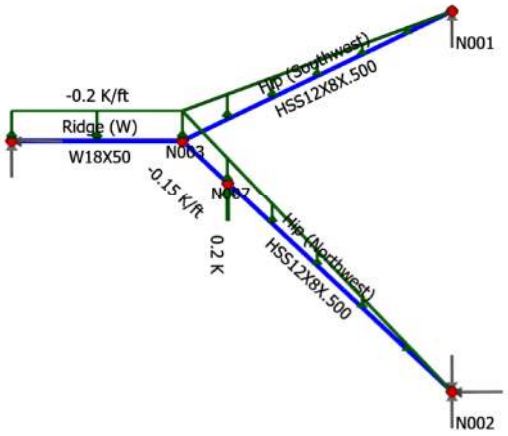
Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Hip (Southwest)	HSS8X4X.250	7.2898	D+S	-0.2772	1.0000	IBC 1604.3.1	**0.2772**	L/Δ = 584.48

Hip Ridge 3

Dead Loads:

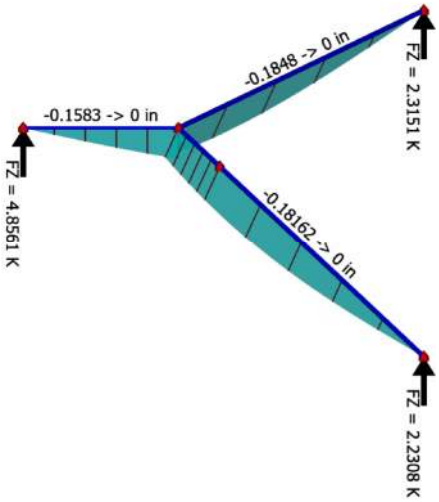


Snow Loads:

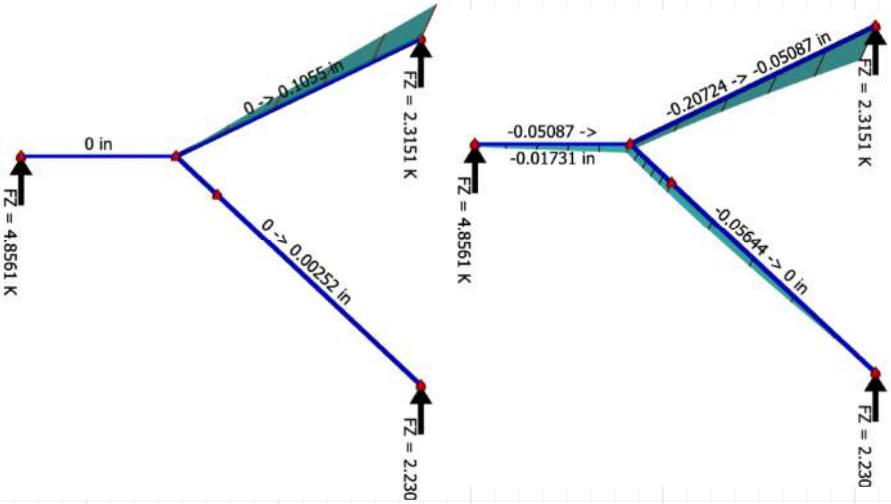


Deflection (D+S)

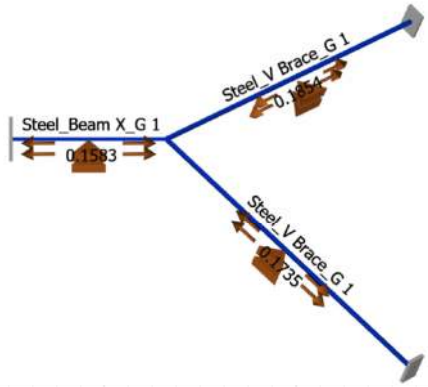
Vertical:



Horizontal:



Unity Values



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Steel_Beam X_G 1: Results

Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Ridge (W)	W18X50	0.0000	D+S	-0.1583	1.0000	IBC 1604.3.1	**0.1583**	L/Δ = 533.79

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Ridge (W)	W18X50	0.0000	3. 1.2D+1.6S+L	31.3547	378.7500	F2-1	**0.0828**	Lb = 0 ft, Cb = 1

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Ridge (W)	W18X50	7.0417	3. 1.2D+1.6S+L	-6.6361	191.7000	G2-1	**0.0346**	Shear Area = 6.39 in^2, Cv = 1, h/tw = 45.228

Steel_V Brace_G 1: Results

Deflections Strong (dy): Total Deflection 'L only': 0.75 in 'W or S only': 0.75 in 'D + L': 1 in Other: 1 in Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		

Steel_V Brace_G 1: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Continuous Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_V Brace_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	11.2953	D+S	-0.1854	1.0000	IBC 1604.3.1	**0.1854**	L/Δ = 1044.3

Steel_V Brace_G 1: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	12.9089	3. 1.2D+1.6S+L	0.1305	1.0000	H1-1b	**0.1305**	KLz = 16.136 ft, KLy = 0 ft, KL(torsion) = 0 ft, Lb = 0 ft, Axial Unity = 0.00001, Mz Unity = 0.1009, My Unity = 0.02963, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1

Steel_V Brace_G 1: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	0.6899	625.1032	E3-2	**0.0011**	KLz = 16.136 ft, KLy = 0 ft, KL(torsion) = 0 ft, Fcr = 40.381 Ksi, Fe (E3-4) = 147.79 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

Steel_V Brace_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	12.9089	3. 1.2D+1.6S+L	23.7059	234.9450	F7-1	**0.1009**	Lb = 0 ft, Cb = 1

Steel_V Brace_G 1: Weak Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand My K-ft	Capacity My K-ft	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	12.9089	3. 1.2D+1.6S+L	5.2653	177.6750	F7-1	**0.0296**	

Steel_V Brace_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	2.9566	244.9882	G4-1	**0.0121**	Shear Area = 9.8627 in^2, Cv = 1

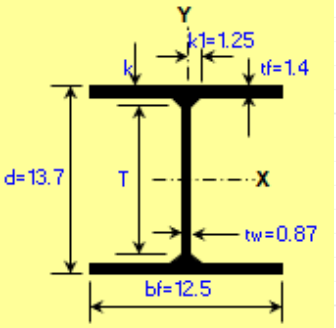
B5 - Custom Header

Summary of design:

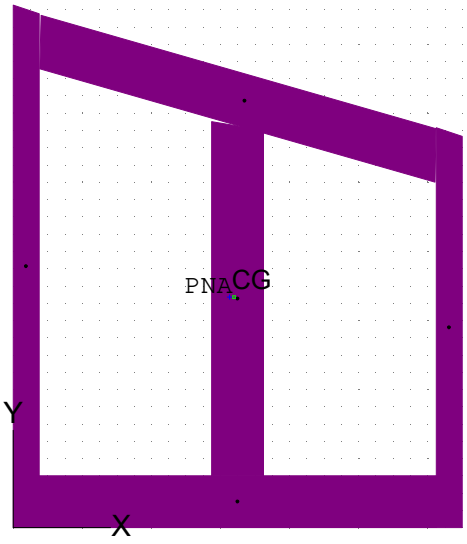
1. Created model in visual analysis to check deflections (maintaining <1/8" deflection over the sliding door using D+S)
2. Use size from VA as minimum section properties required

3. Use shape builder to design a shape that fits in architectural restraints & meets minimum section properties

Note: additional weight of custom shape compared to W12x152 is neglected in VA model design. Deflection checks use D+S conservatively. Deflection due to snow loads and superimposed dead loads will still be less than 1/8" since the structure will already be installed and deflected under self weight before door installation.



	W12X152	
A =	44.7	in.^2
d =	13.7	in.
tw =	0.87	in.
bf =	12.5	in.
tf =	1.4	in.
k(des) =	2	in.
k(det) =	2.3125	in.
k1 =	1.25	in.
T =	9.125	in.
gage =	5.5	in.
wt./ft. =	152	plf.
bf/(2*tf) =	4.46	
h/tw =	11.2	
Ix =	1430	in.^4
Sx =	209	in.^3
rx =	5.66	in.
Zx =	243	in.^3
Iy =	454	in.^4
Sy =	72.8	in.^3
ry =	3.19	in.
Zy =	111	in.^3
rts =	3.66	in.
ho =	12.3	in.
J =	25.8	in.^4
Cw =	17200	in.^6
a =	41.55	in.
Wno =	38.4	in.^2
Sw =	168	in.^4
Qf =	50.1	in.^3
Qw =	121	in.^3



Overall Properties

Name: Shape1	Sy left: 180.1 in^3
Area: 70.13 in^2	Sy right: 167.7 in^3
CG: (6.269,6.670)	rx: 4.875 in
Ix: 1667 in^4	ry: 4.013 in
Iy: 1129 in^4	rp: 6.314 in
Ixy: -162.2 in^4	r1: 4.941 in
Ipolar: 2796 in^4	r2: 3.932 in
I1: 1712 in^4	Zx: 311.1 in^3
I2: 1084 in^4	Zy: 227.7 in^3
Theta: 15.55 deg	PNA: (6.397,6.670)
Sx top: 197.1 in^3	J: 37.09 in^4
Sx bottom: 249.9 in^3	

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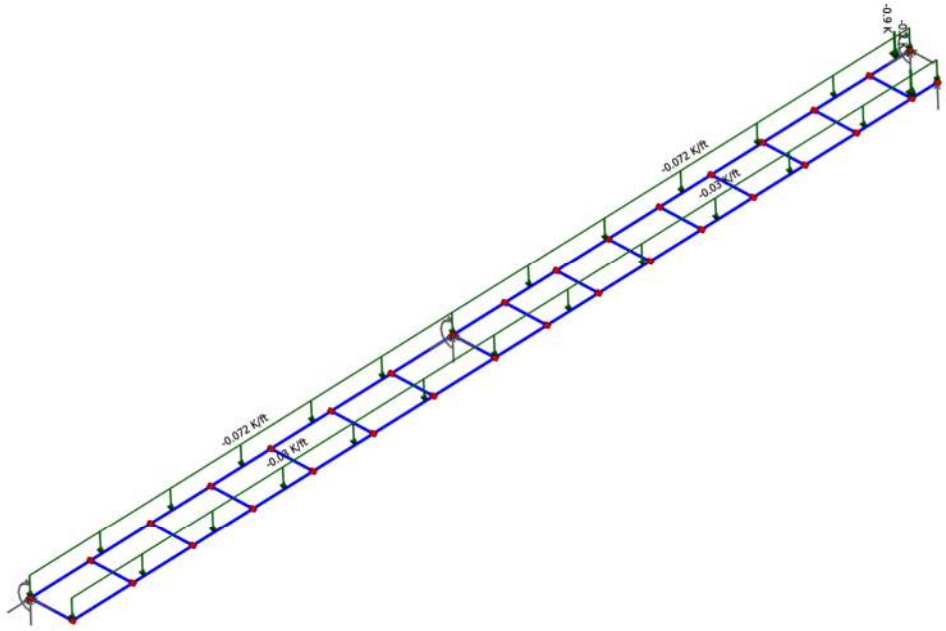
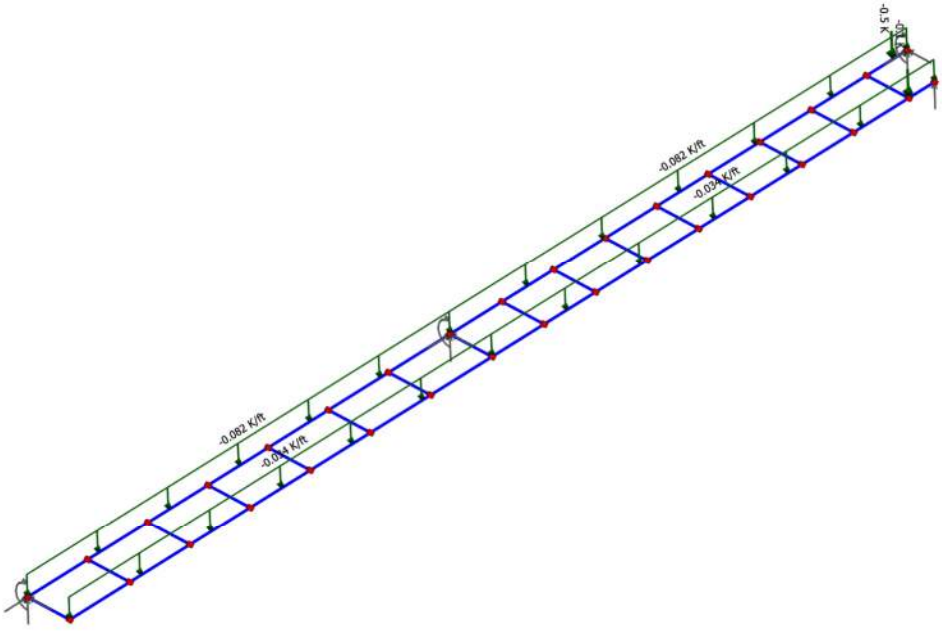
B5 - Custom Header

Dead Loads:



Snow Loads:



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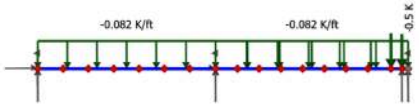
B5 - Custom Header

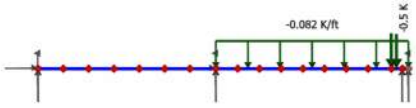
Skip Loading Conditions:

"B5"



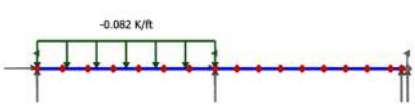
Full D+S

"B5 skip case 1 (snow on east span)"



Full D
+S on east span

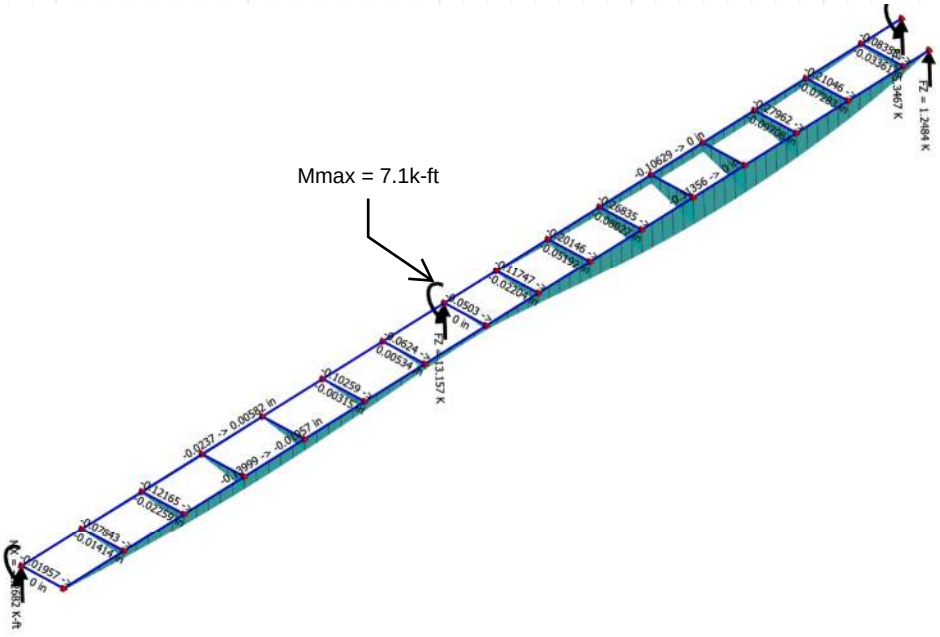
"B5 skip case 2 (snow on east span)"



Full D
+S on west span

Deflection (D+S)

Vertical:



See VA report next page for unity checks

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Steel_Beam X_G 1: Results

Deflections Strong (dy): Total Span Ratio 'L only': 480 'W or S only': 480 'D + L': 360 Other: 240 Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Errors and Warnings

Member	Section	Issues
B5	W12X152	Warping stresses are neglected in this check. To consider warping, use the override for Advanced Torsion.
B5 skip case 1 (snow on east span)	W12X152	Warping stresses are neglected in this check. To consider warping, use the override for Advanced Torsion.
B5 skip case 1 (snow on west span)	W12X152	Warping stresses are neglected in this check. To consider warping, use the override for Advanced Torsion.

Steel_Beam X_G 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
B5 skip case 1 (snow on east span)	W12X152	46.0833	D+S	-0.0984	2.9850	IBC 1604.3.1	**0.0330**	L/Δ = 7279.6

Steel_Beam X_G 1: Torsion Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Tau Ksi	Capacity Tau Ksi	Code Reference	Unity Check	Details
B5	W12X152	28.7848	3. 1.2D +1.6S+L	1.3595	27.0000	H3-8	**0.0504**	Tr = 2.0878 K-ft, Venant Shear = 1.3595 Ksi Warning = Warping stresses are neglected in this check. To consider warping, use the override for Advanced Torsion.

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
B5	W12X152	28.6670	3. 1.2D+1.6S+L	-56.9262	872.3686	F2-2	**0.0653**	Lp = 11.257 ft, Lr = 70.631 ft, Lb = 59.7 ft, Cb = 1.4188

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
B5	W12X152	28.7848	3. 1.2D+1.6S+L	9.1085	357.5700	G2-1	**0.0255**	Shear Area = 11.919 in^2, Cv = 1, h/tw = 11.149

Steel_Beam X_G 3: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 3: Torsion Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Tau Ksi	Capacity Tau Ksi	Code Reference	Unity Check	Details
Fascia 1	HSS4X4X.500	53.3250	3. 1.2D+1.6S+L	1.4459	24.8400	H3-1	**0.0582**	Tr = -1.3535 K-ft, Venant Shear = 1.4459 Ksi

Steel_Beam X_G 3: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Fascia 1	HSS4X4X.500	28.6667	3. 1.2D+1.6S+L	-3.0070	26.5650	F7-1	**0.1132**	Lb = 58.7 ft, Cb = 1.772

Steel_Beam X_G 3: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Fascia 1	HSS4X4X.500	58.7000	3. 1.2D+1.6S+L	-1.6677	60.1786	G4-1	**0.0277**	Shear Area = 2.4227 in^2, Cv = 1

Steel_Beam Y_G 3: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False

Steel_Beam Y_G 3: Results (continued)

Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam Y_G 3: Torsion Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Tau Ksi	Capacity Tau Ksi	Code Reference	Unity Check	Details
OR18	HSS4X4X.500	0.0000	3. 1.2D+1.6S+L	2.1333	24.8400	H3-1	**0.0859**	Tr = 1.997 K-ft, Venant Shear = 2.1333 Ksi

Steel_Beam Y_G 3: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
OR8	HSS4X4X.500	0.0000	3. 1.2D+1.6S+L	-6.3235	26.5650	F7-1	**0.2380**	Lb = 2.75 ft, Cb = 1.5912

Steel_Beam Y_G 3: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
OR8	HSS4X4X.500	0.0000	3. 1.2D+1.6S+L	2.1553	60.1786	G4-1	**0.0358**	Shear Area = 2.4227 in^2, Cv = 1

Beam:	B1 RIDGE				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.193		0.1625	0.356	
w_2	0.050			0.050	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

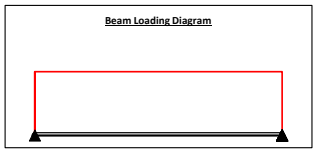
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	28.00	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	5.677	0.00
R_2	5.677	28.00
R_3	0.000	28.00
R_4	0.000	28.00
R_5	0.000	28.00
R_6	0.000	28.00
R_7	0.000	28.00
R_8	0.000	28.00
R_9	0.000	28.00
R_{10}	0.000	28.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	800
S (in^3)	88.9
A (in^2)	6.39

Steel Beam Section	W18x50
F_y, ksi	50
Beam Weight (plf)	50
Axis of Bending	Strong
Unbraced Length (L_b), ft	0
C_b	1
A_w 6.39 in^2	V_u/Ω_v 128 k
S 88.9 in^3	$\phi_v V_n$ 192 k
Z 101 in^3	M_u/Ω_b 252 k-ft
I 800 in^4	$\phi_b M_n$ 379 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	5.68	-5.68	0.00	39.74	-0.242 (↓)	14	L/1388	0	-	L/∞

DCR
M: 0.16
V: 0.04
<L/360? yes

PROJECT
DATE 10/28/2022
PROJ. #
DESIGN haa
SHEET 1

Beam:	B2 CLG BEAM				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.105		0.0375	0.143	
w_2	0.072			0.072	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	3.8		2.7	6.500	28.25
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

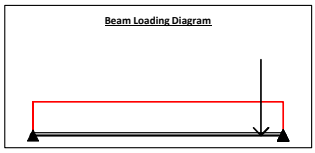
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	31.00	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	3.901	0.00
R_2	9.248	31.00
R_3	0.000	31.00
R_4	0.000	31.00
R_5	0.000	31.00
R_6	0.000	31.00
R_7	0.000	31.00
R_8	0.000	31.00
R_9	0.000	31.00
R_{10}	0.000	31.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	597
S (in^3)	97.4
A (in^2)	5.29

Steel Beam Section	W12x72
F_y, ksi	50
Beam Weight (plf)	72
Axis of Bending	Strong
Unbraced Length (L_b), ft	0
C_b	1
A_w 5.29 in^2	V_u/Ω_v 106 k
S 97.4 in^3	$\phi_v V_n$ 159 k
Z 108 in^3	M_u/Ω_b 269 k-ft
I 597 in^4	$\phi_b M_n$ 405 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	5.88	-11.23	-	50.54	-0.517 (↓)	15.93	L/720	0	-	L/∞

DCR
M: 0.19
V: 0.11
<L/360? yes

PROJECT
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET 2

Beam:	B3 WEST GIRDER				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.123		0.0375	0.161	
w_2	0.090			0.090	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	5.6		4.5	10.100	2.75
P_2	8.2		6.2	14.400	11.13
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

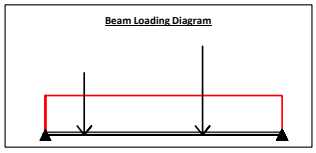
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	16.75	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	15.376	0.00
R_2	13.320	16.75
R_3	0.000	16.75
R_4	0.000	16.75
R_5	0.000	16.75
R_6	0.000	16.75
R_7	0.000	16.75
R_8	0.000	16.75
R_9	0.000	16.75
R_{10}	0.000	16.75

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	999
S (in^3)	143
A (in^2)	6.16

Steel Beam Section	W14x90
F_y, ksi	50
Beam Weight (plf)	90
Axis of Bending	Strong
Unbraced Length (L_b), ft	0
C_b	1
A_w 6.16 in^2	V_u/Ω_v 123 k
S 143 in^3	$\phi_v V_n$ 185 k
Z 157 in^3	M_u/Ω_b 382 k-ft
I 999 in^4	$\phi_b M_n$ 574 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	14.82	-12.76	0.00	68.88	-0.111 (↓)	8.61	L/1811	0	-	L/∞

DCR
M: 0.18
V: 0.12
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET 3

Beam:	B4 EAST GIRDER				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.123		0.0375	0.161	
w_2	0.090			0.090	
w_3	0.136			0.136	7.83
w_4	-0.136			-0.136	14.42
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	2.1		2.0	4.113	3.75
P_2	3.3		3.0	6.275	11.13
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

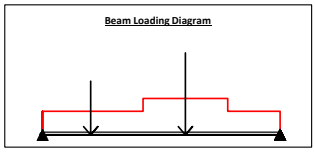
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	18.50	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	8.455	0.00
R_2	7.463	18.50
R_3	0.000	18.50
R_4	0.000	18.50
R_5	0.000	18.50
R_6	0.000	18.50
R_7	0.000	18.50
R_8	0.000	18.50
R_9	0.000	18.50
R_{10}	0.000	18.50

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	999
S (in^3)	143
A (in^2)	6.16

Steel Beam Section	W14x90
F_y, ksi	50
Beam Weight (plf)	90
Axis of Bending	Strong
Unbraced Length (L_b), ft	0
C_b	1
A_w 6.16 in^2	V_u/Ω_v 123 k
S 143 in^3	$\phi_v V_n$ 185 k
Z 157 in^3	M_u/Ω_b 382 k-ft
I 999 in^4	$\phi_b M_n$ 574 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	7.84	-6.85	-	44.73	-0.088 (↓)	9.36	L/2523	0	-	L/∞

DCR
M: 0.12
V: 0.06
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET 4

Multi-Span Beam Analysis

Beam:	B6 flush header					
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.087	0.150	0.0375	0.275	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	5.3	4		9.300	12.83
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

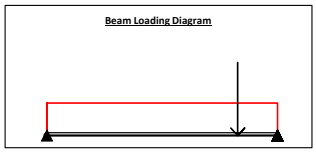
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	15.50
Left End Condition	Pinned
Right End Condition	Pinned
R_1	3.729 / 0.00
R_2	9.825 / 15.50
R_3	0.000 / 15.50
R_4	0.000 / 15.50
R_5	0.000 / 15.50
R_6	0.000 / 15.50
R_7	0.000 / 15.50
R_8	0.000 / 15.50
R_9	0.000 / 15.50
R_{10}	0.000 / 15.50

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	303
S (in^3)	60
A (in^2)	3.74

Steel Beam Section	W10x54		
F_y, ksi	50		
Beam Weight (plf)	54		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	3.74 in^2	V_n/Ω_v	74.7 k
S	60 in^3	$\phi_b V_n$	112 k
Z	66.6 in^3	M_n/Ω_b	166 k-ft
I	303 in^4	$\phi_b M_n$	250 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	4.21	-10.30	-	26.33	-0.121 (↓)	8.37	L/1537	-0.053 (↓)	8.37	L/3509

DCR
M: 0.16
V: 0.14
deflection<1/8"? yes

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Multi-Span Beam Analysis

Beam:	B17 BEAM					
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.059		0.038	0.097	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	6.500		4.700	11.200	5.75
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

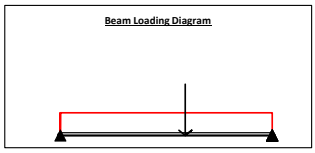
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	9.75
Left End Condition	Pinned
Right End Condition	Pinned
R_1	5.065 / 0.00
R_2	7.076 / 9.75
R_3	0.000 / 9.75
R_4	0.000 / 9.75
R_5	0.000 / 9.75
R_6	0.000 / 9.75
R_7	0.000 / 9.75
R_8	0.000 / 9.75
R_9	0.000 / 9.75
R_{10}	0.000 / 9.75

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	204
S (in^3)	33.4
A (in^2)	2.81

Steel Beam Section	W12x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	2.81 in^2	V_n/Ω_v	56.1 k
S	33.4 in^3	$\phi_b V_n$	84.2 k
Z	37.2 in^3	M_n/Ω_b	92.8 k-ft
I	204 in^4	$\phi_b M_n$	140 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	5.18	-7.19	-	27.78	-0.065 (↓)	5.13	L/1800	0	-	L/∞

DCR
M: 0.30
V: 0.13
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B8 Header					
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.059		0.038	0.097	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft)	t_1	0.010		0.012	0.022	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1				0.000	
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

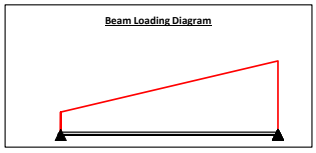
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	10.00
Left End Condition	Pinned
Right End Condition	Pinned
R_1	0.845 / 0.00
R_2	1.267 / 10.00
R_3	0.000 / 10.00
R_4	0.000 / 10.00
R_5	0.000 / 10.00
R_6	0.000 / 10.00
R_7	0.000 / 10.00
R_8	0.000 / 10.00
R_9	0.000 / 10.00
R_{10}	0.000 / 10.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	144
S (in^3)	27.9
A (in^2)	2.68

Steel Beam Section	W10x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	2.68 in^2	V_n/Ω_v	53.6 k
S	27.9 in^3	$\phi_b V_n$	80.3 k
Z	31.3 in^3	M_n/Ω_b	78.1 k-ft
I	144 in^4	$\phi_b M_n$	117 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	2.14	-2.50	0.00	5.80	-0.025 (↓)	5.04	L/4800	0	-	L/∞

DCR
M: 0.07
V: 0.05
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B9 master fascia					
	Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.041		0.022	0.063	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1				0.000	
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

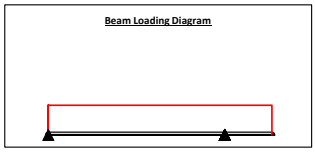
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	14.25
Left End Condition	Pinned
Right End Condition	Pinned
R_1	0.328 / 0.00
R_2	0.566 / 11.25
R_3	0.000 / 11.25
R_4	0.000 / 11.25
R_5	0.000 / 11.25
R_6	0.000 / 11.25
R_7	0.000 / 11.25
R_8	0.000 / 11.25
R_9	0.000 / 11.25
R_{10}	0.000 / 11.25

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	11.9
S (in^3)	5.97
A (in^2)	2.42

Steel Beam Section	HSS4x4x1/2		
F_y, ksi	46		
Beam Weight (plf)	21.63		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	3		
C_b	1		
A_w	2.42 in^2	V_n/Ω_v	40 k
S	5.97 in^3	$\phi_b V_n$	60.1 k
Z	7.7 in^3	M_n/Ω_b	17.7 k-ft
I	11.9 in^4	$\phi_b M_n$	26.6 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	0.33	-0.38	-0.28	0.86	-0.054 (↓)	5.47	L/2495	0	-	L/∞
Right Cantilever	0.19	-	-0.28	-	0.037 (↑)	14.25	L/1942	0	-	L/∞

DCR
M: 0.05
V: 0.01
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B10 master fascia				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.041		0.022	0.063	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

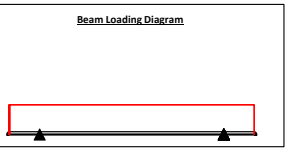
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	24.25	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	0.761	3.00
R_2	0.761	21.25
R_3	0.000	21.25
R_4	0.000	21.25
R_5	0.000	21.25
R_6	0.000	21.25
R_7	0.000	21.25
R_8	0.000	21.25
R_9	0.000	21.25
R_{10}	0.000	21.25

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	11.9
S (in^3)	5.97
A (in^2)	2.42

Steel Beam Section	HSS4x4x1/2		
F_y, ksi	46		
Beam Weight (plf)	21.63		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	3		
C_b	1		
A_w	2.42 in^2	V_y/Ω_v	40 k
S	5.97 in^3	$\phi_v V_n$	60.1 k
Z	7.7 in^3	M_y/Ω_b	17.7 k-ft
I	11.9 in^4	$\phi_b M_n$	26.6 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilver	-	-0.17	-0.26	-	0.18 (↑)	0	L/394	0	-	L/∞
Span 1	0.52	-0.52	-0.26	2.14	-0.362 (↓)	12.13	L/605	0	-	L/∞
Right Cantilever	0.17	-	-0.26	-	0.18 (↑)	24.25	L/394	0	-	L/∞

DCR
M: 0.12
V: 0.01
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B11 master header				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.055		0.038	0.092	0
w_2	-0.055		-0.038	-0.092	3
w_3	0.055		0.038	0.092	21.5
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1	0.011		0.0125	0.024	3
t_2	-0.022		-0.025	-0.047	12.25
t_3	0.011		0.0125	0.024	21.5
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	0.6		0.4	1.000	0
P_2	0.154		0.175	0.329	2.00
P_3	0.154		0.175	0.329	22.50
P_4	0.6		0.4	1.000	24.50
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

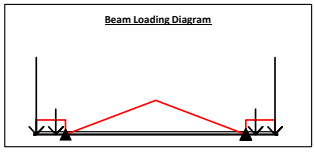
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	24.50	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	2.611	3.00
R_2	2.611	21.50
R_3	0.000	21.50
R_4	0.000	21.50
R_5	0.000	21.50
R_6	0.000	21.50
R_7	0.000	21.50
R_8	0.000	21.50
R_9	0.000	21.50
R_{10}	0.000	21.50

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	11.9
S (in^3)	5.97
A (in^2)	2.42

Steel Beam Section	HSS4x4x1/2		
F_y, ksi	46		
Beam Weight (plf)	21.63		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	3		
C_b	1		
A_w	2.42 in^2	V_y/Ω_v	40 k
S	5.97 in^3	$\phi_v V_n$	60.1 k
Z	7.7 in^3	M_y/Ω_b	17.7 k-ft
I	11.9 in^4	$\phi_b M_n$	26.6 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Left Cantilever	-	-1.65	-3.82	-	-0.044 (↓)	0	L/1630	0	-	L/∞
Span 1	1.01	-1.01	-3.82	2.38	-0.245 (↓)	12.25	L/902	0	-	L/∞
Right Cantilever	1.65	-	-3.82	-	-0.044 (↓)	24.5	L/1630	0	-	L/∞

DCR
M: 0.22
V: 0.04
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B12 Master cantilever				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.055		0.038	0.092	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1	-0.006		-0.004	-0.010	4.75
t_2	0.006		0.004	0.010	14
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	0.204		0.231	0.435	15
P_2	0.53		0.31	0.840	17.00
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

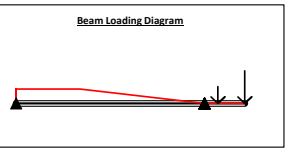
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	17.00	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	0.340	0.00
R_2	1.798	14.00
R_3	0.000	14.00
R_4	0.000	14.00
R_5	0.000	14.00
R_6	0.000	14.00
R_7	0.000	14.00
R_8	0.000	14.00
R_9	0.000	14.00
R_{10}	0.000	14.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	11.9
S (in^3)	5.97
A (in^2)	2.42

Steel Beam Section	HSS4x4x1/2		
F_y, ksi	46		
Beam Weight (plf)	21.63		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	3		
C_b	1		
A_w	2.42 in^2	V_y/Ω_v	40 k
S	5.97 in^3	$\phi_v V_n$	60.1 k
Z	7.7 in^3	M_y/Ω_b	17.7 k-ft
I	11.9 in^4	$\phi_b M_n$	26.6 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	1.09	-0.95	-2.95	2.73	-0.213 (↓)	5.95	L/787	0	-	L/∞
Right Cantilever	1.27	-	-2.95	-	0.008 (↑)	15.33	L/8976	0	-	L/∞

DCR
M: 0.17
V: 0.03
<L/360? yes

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Multi-Span Beam Analysis

Beam:	B13 Cantilever				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.055		0.038	0.092	3.5
w_2	0.022		0.025	0.047	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

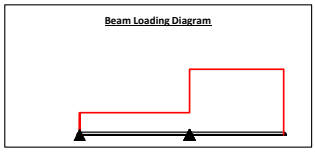
Support Locations and Reactions		
Number of Supports	2	
Total Beam Length	6.50	
Left End Condition	Pinned	
Right End Condition	Pinned	
R_1	-0.097	0.00
R_2	0.679	3.50
R_3	0.000	3.50
R_4	0.000	3.50
R_5	0.000	3.50
R_6	0.000	3.50
R_7	0.000	3.50
R_8	0.000	3.50
R_9	0.000	3.50
R_{10}	0.000	3.50

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	11.9
S (in^3)	5.97
A (in^2)	2.42

Steel Beam Section	HSS4x4x1/2		
F_y, ksi	46		
Beam Weight (plf)	21.63		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	3		
C_b	1		
A_w	2.42 in^2	V_y/Ω_v	40 k
S	5.97 in^3	$\phi_v V_n$	60.1 k
Z	7.7 in^3	M_y/Ω_b	17.7 k-ft
I	11.9 in^4	$\phi_b M_n$	26.6 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	-0.70	-0.87	-2.75	-	0.01 (↑)	2.03	L/4196	0	-	L/∞
Right Cantilever	1.83	-	-2.75	-	-0.078 (↓)	6.5	L/920	0	-	L/∞

DCR
M: 0.16
V: 0.05
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET

Beam:	B14 header				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.059		0.038	0.097	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1	0.010	0.012		0.022	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

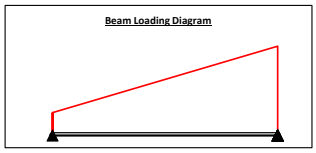
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	12.75
Left End Condition	Pinned
Right End Condition	Pinned
R_1	1.214 / 0.00
R_2	1.814 / 12.75
R_3	0.000 / 12.75
R_4	0.000 / 12.75
R_5	0.000 / 12.75
R_6	0.000 / 12.75
R_7	0.000 / 12.75
R_8	0.000 / 12.75
R_9	0.000 / 12.75
R_{10}	0.000 / 12.75

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	144
S (in^3)	27.9
A (in^2)	2.68

Steel Beam Section	W10x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	14		
C_b	1		
A_w	2.68 in^2	$\phi_v V_n$	53.6 k
S	27.9 in^3	$\phi_v V_n$	80.3 k
Z	31.3 in^3	M_u/Ω_b	51.4 k-ft
I	144 in^4	$\phi_b M_n$	77.3 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	0.77	-1.36	-	3.46	-0.024 (↓)	6.58	L/6375	-0.011 (↓)	6.63	L/13909

DCR
M: 0.07
V: 0.025
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET

Beam:	B15 glass roof				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.049		0.038	0.086	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

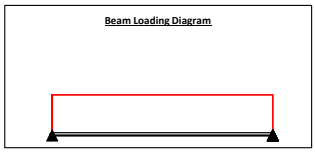
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	12.50
Left End Condition	Pinned
Right End Condition	Pinned
R_1	0.538 / 0.00
R_2	0.538 / 12.50
R_3	0.000 / 12.50
R_4	0.000 / 12.50
R_5	0.000 / 12.50
R_6	0.000 / 12.50
R_7	0.000 / 12.50
R_8	0.000 / 12.50
R_9	0.000 / 12.50
R_{10}	0.000 / 12.50

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	28.5
S (in^3)	7.12
A (in^2)	3.4

Steel Beam Section	HSS8x2x1/4		
F_y, ksi	46		
Beam Weight (plf)	15.62		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	12.5		
C_b	1		
A_w	3.4 in^2	$\phi_v V_n$	56.3 k
S	7.12 in^3	$\phi_v V_n$	84.5 k
Z	9.68 in^3	M_u/Ω_b	22.2 k-ft
I	28.5 in^4	$\phi_b M_n$	33.4 k-ft

Beam Loading Diagram



Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	1.03	-1.03	0.00	3.22	-0.11 (↓)	6.25	L/1364	0	-	L/∞

DCR
M: 0.15
V: 0.02
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET

Beam:	b16 channel ledger				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.058		0.038	0.096	
w_2	0.000		0	0.000	16
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	1.700		1.500	3.200	18
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

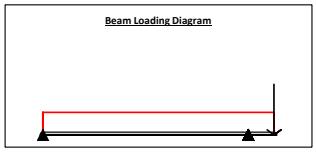
Support Locations and Reactions	
Number of Supports	2
Total Beam Length	18.00
Left End Condition	Pinned
Right End Condition	Pinned
R_1	0.352 / 0.00
R_2	4.567 / 16.00
R_3	0.000 / 16.00
R_4	0.000 / 16.00
R_5	0.000 / 16.00
R_6	0.000 / 16.00
R_7	0.000 / 16.00
R_8	0.000 / 16.00
R_9	0.000 / 16.00
R_{10}	0.000 / 16.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Beam Properties	
E (ksi)	29000
I (in^4)	91.1
S (in^3)	18.2
A (in^2)	5.26

Steel Beam Section	C10x25		
F_y, ksi	36		
Beam Weight (plf)	25		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	14		
C_b	1		
A_w	5.26 in^2	$\phi_v V_n$	68.2 k
S	18.2 in^3	$\phi_v V_n$	102 k
Z	23.1 in^3	M_u/Ω_b	25.3 k-ft
I	91.1 in^4	$\phi_b M_n$	38.1 k-ft

Beam Loading Diagram

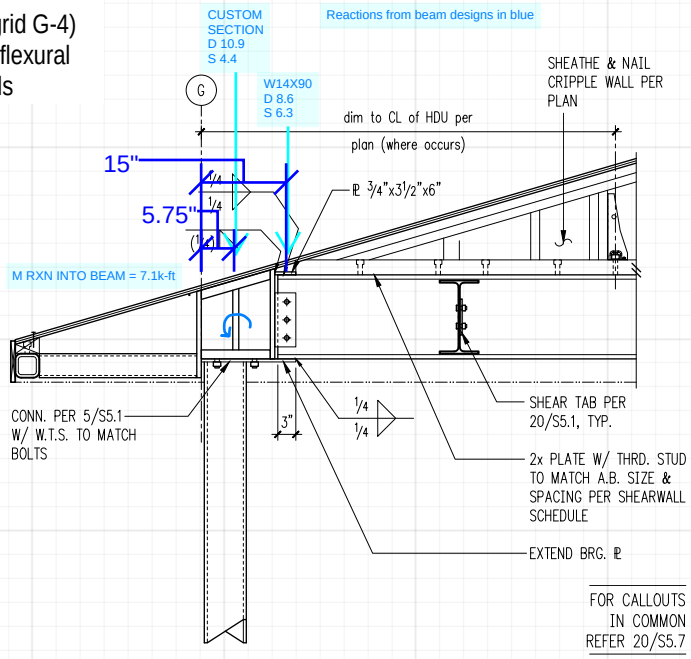


Span	V_{left} (kips)	V_{right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	-0.20	-0.60	-6.45	-	0.056 (↑)	9.58	L/3425	0	-	L/∞
Right Cantilever	3.25	-	-6.45	-	-0.045 (↓)	18	L/1056	0	-	L/∞

DCR
M: 0.25
V: 0.05
<L/360? yes

PROJECT Building
DATE 10/21/2022
PROJ. #
DESIGN haa
SHEET

Detail 17/S5.7
Check post (grid G-4)
for combined flexural
and axial loads



Determine Moment applied at column due to eccentricity of connection:

M (asd) (positive)
= (10.9k+4.4k)*(5.75"-3")+(8.6k+6.3k)*(15"-3")
= 221k-in
=18.4k-ft

P (asd)
= 10.9+4.4+8.6+6.3
= 30.2k

Combined Check on post:

HSS5x5x1/2 Capacities:
Mn/Ω = 32.7k-ft
Pn/Ω (h=10ft) = 172 k

Pr/Pc = 30.2/172=0.18<0.2

therefore the follow shall apply:
Pr/2Pc+(Mr/Mc) </=1.0

30.2/(2*172)+18.4/32.7 = 0.087+0.56
= 0.65<1.0 ok

Decouple moment from reaction at custom beam (Beam 5)

M (asd) (negative)
= 7.1k-ft

decouple into top and bottom flange of W14x90

T=C=7.1k-ft/(14/12) = 6.1k

1/4" fillet weld ea side x 3" long:
Capacity = 3.712k/in*3"*2 = 22.3k

22.3k >> 6.1k OK

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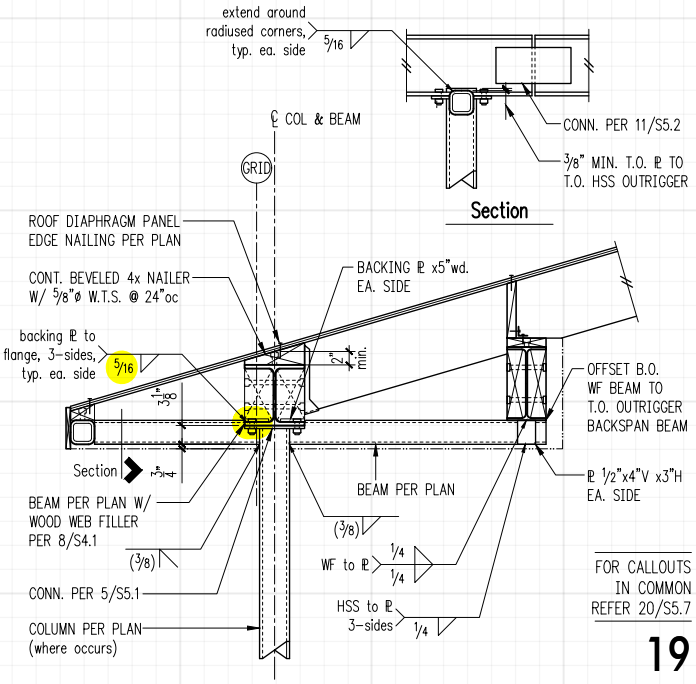
2124 Third Ave, Suite 100, Seattle, WA 98121 | O 206.443.6212
934 Broadway, Suite 100, Tacoma, WA 98402 | O 253.284.9470
ssfengineers.com

Detail 19/S5.7
Outrigger connection design



SIZE WELD FOR OUTRIGGER IN TENSION:
M= 4.5k-ft
M/d=13.5k

surface area of hss4x4x1/2 = 1.19ft^2/ft
perimeter = 1.19ft = 14.28"
corner arc length = (14.28-4*1.75)/4 = 1.82"
assume workable flat plus 2 corners = 1.75+1.82*2 = 5.39 --> use 4" of weld

fillet weld = 5/16" = 4.64k/in * 4" = 18.56k
DCR: 13.5/18.56=0.73

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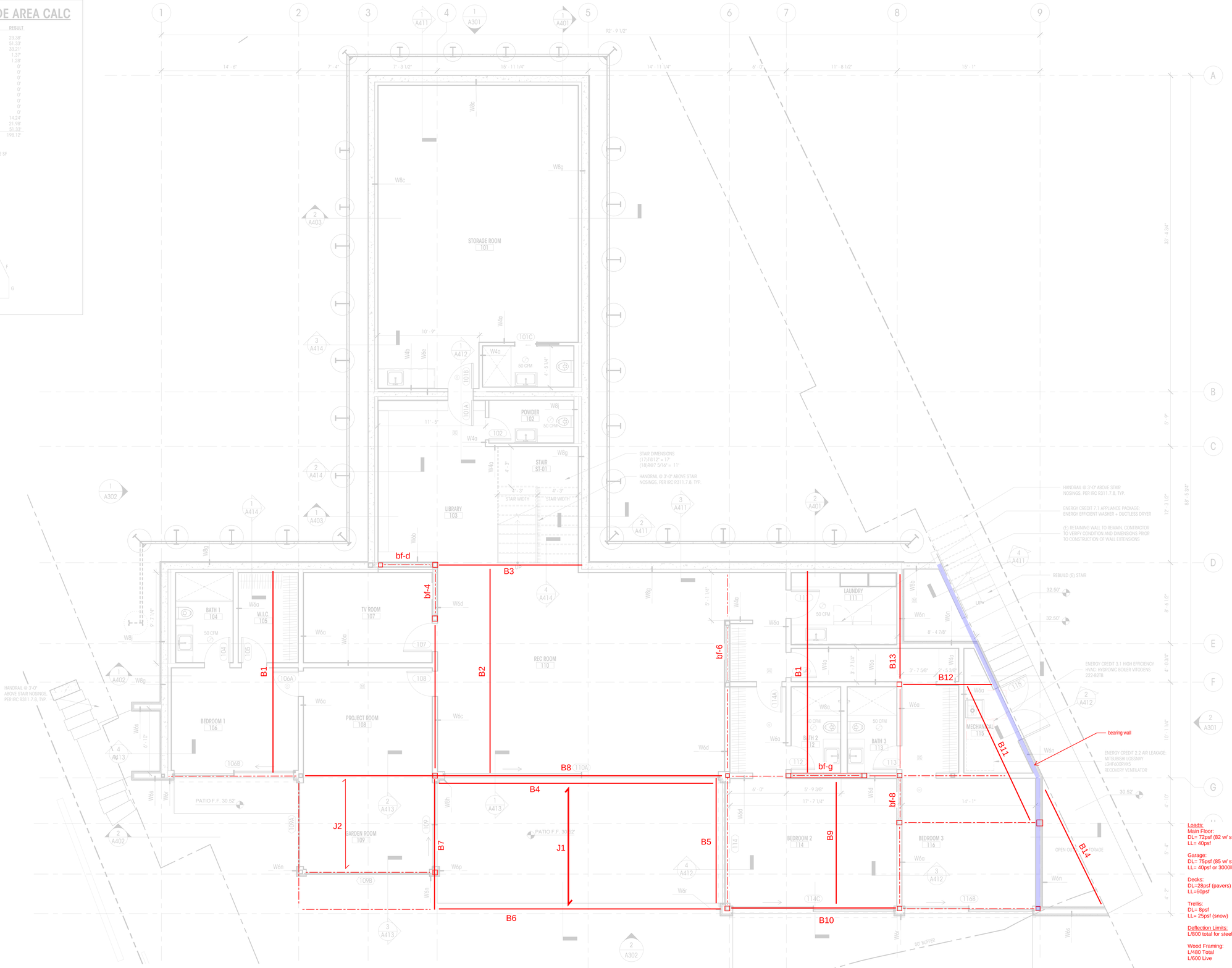
BASEMENT LEVEL BELOW GRADE AREA CALC

WALL SEGMENT	LENGTH	COVERAGE	RESULT
A	23.38'	100%	23.38'
B	51.33'	100%	51.33'
C	33.21'	100%	33.21'
D	8.44'	16%	1.37'
E	7.86'	16%	1.28'
F	15.79'	0%	0'
G	14.73'	0%	0'
H	34.05'	0%	0'
I	14.33'	0%	0'
J	29.64'	0%	0'
K	10.17'	0%	0'
L	15.11'	0%	0'
M	10.17'	0%	0'
N	14.40'	0%	0'
O	23.09'	62%	14.24'
P	21.98'	100%	21.98'
Q	51.33'	100%	51.33'
TOTALS	379.02'		198.12'

TOTAL BASEMENT GSF = 3,821.71 SQ.FT.
PORTION OF EXCLUDED BASEMENT FLOOR AREA: (198.12/379.02) X 3821.71 = 1997.72 SF
NET BASEMENT GFA: (3,821.71 - 1,997.12) = **1823.99 SF**



LEGEND

- 200A WINDOW ID
- 100A DOOR ID
- 100A FINISH ID
- ROOM NAME 101 ROOM ID
- W4a ASSEMBLY ID
- A STOREFRONT ID
- NEW WALL
- WALL TO REMAIN
- EL= 148.5' (+0'-0") MAIN LEVEL FIN. FLR. ELEVATION DATUM
- 0 GRIDLINE
- SMOKE DETECTOR
- SMOKE/CARBON MONOXIDE DETECTOR
- PROPERTY LINE
- SETBACK LINE
- ROOF OVERHANG ABOVE
- BRACE FRAME LOCATION
- FAN - 100 CFM U.N.O.
- HEAT DETECTOR

NOTES

1. ALL DIMENSIONS AT WALLS TO FACE OF FRAMING OR TO FACE OF CONCRETE, U.N.O.
2. ALL DIMENSIONS AT KITCHEN TO EDGE OF COUNTERTOPS, U.N.O.
3. ALL INTERIOR DOOR SWING-SIDE JAMBS ARE 4" FROM ADJACENT WALL, U.N.O.
4. SEE RCP FOR SMOKE / CARBON MONOXIDE DETECTOR AND EXHAUST FAN LOCATIONS
5. ALL NEW WALLS TYPE W4A UNLESS NOTED OTHERWISE
6. ALL DIMENSIONS ASSOCIATED WITH (E) CONSTRUCTION ARE ASSUMED. CONTRACTOR TO VERIFY ALL DIMS IN FIELD AND CONTACT ARCHITECT WITH ANY DISCREPANCIES PRIOR TO CONSTRUCTION
7. CONTRACTOR TO INSTALL CARBON MONOXIDE ALARMS OUTSIDE OF EACH BEDROOM IN THE IMMEDIATE VICINITY ON EACH FLOOR LEVEL PER IRC SECTION 315.3.
8. CONTRACTOR TO INSTALL SMOKE ALARMS OUTSIDE OF EACH BEDROOM IN THE IMMEDIATE VICINITY ON EACH FLOOR LEVEL PER IRC SECTION 314.2.2.
9. FLOOR, CEILING, AND WALL ASSEMBLIES ARE LISTED ON SHEETS A701 & A702.

ENERGY CREDITS

EFFICIENT BUILDING ENVELOPE OPTION 1.3: (0.5)
VERTICAL FENESTRATION U = 0.28, FLOOR R-38, SLAB ON GRADE R-10 PERIMETER AND UNDER ENTIRE SLAB BELOW GRADE SLAB R-10 PERIMETER AND UNDER ENTIRE SLAB

AIR LEAKAGE CONTROL OPTION 2.2: (1.0)
RECOVERY VENTILATOR: MITSUBISHI LOSSNAY, LGHF60DRVX5

HIGH EFFICIENCY HVAC EQUIPMENT OPTION 3.1: (1.0)
HIGH EFFICIENCY HVAC: HYDRONIC BOILER VITODENS, 200-B2HA

HIGH EFFICIENCY HVAC DISTRIBUTION SYSTEM 4.2 (1.0)

RENEWABLE ELECTRIC ENERGY OPTION 6.1: (3.0)
3600 kWh PHOTOVOLTAIC SYSTEM

APPLIANCE PACKAGE OPTION 7.1: (0.5)
ENERGY EFFICIENT APPLIANCE PACKAGE

1 LOWER FLOOR PLAN
1/4" = 1'-0"

138

Loads:
Main Floor:
DL= 72psf (82 w/ steel framing weight)
LL= 40psf

Garage:
DL= 75psf (85 w/ steel framing weight)
LL= 40psf or 3000lbs point load

Decks:
DL=28psf (pavers)
LL=60psf

Trellis:
DL= 8psf
LL= 25psf (snow)

Deflection Limits:
L/800 total for steel/concrete

Wood Framing:
L/480 Total
L/600 Live

Glazing 1/8" (Total)

Key Plan - Floor Framing

Brandt
Design Group
66 Bell Street
Unit 1
Seattle, WA
98121
206.239.0850
brandtdesigninc.com

NOT FOR CONSTRUCTION FOR REFERENCE ONLY

8480 RESIDENCE

8480 85TH AVE SE
MERCER ISLAND, WA 98040

PERMIT SUBMITTAL SET

DATE: 03.11.22
SHEET SIZE: E (30X42)

REVISIONS

NO.	DESCRIPTION	DATE

PLAN

1/4" = 1'-0"

A211

DEDICATED APPROVAL STAMP SPACE

Gravity Design

Floor Framing

J1

L= 14' w= 127 plf Rxn= 0.9 k

11.875" TJI 360 @ 16" oc
See attached Forte Web report

C1 - TYPICAL COLUMN

h=11ft P=80 k

P_n/Ω = 152 k
DCR: 80/152 = 0.53

HSS 5X5X1/2

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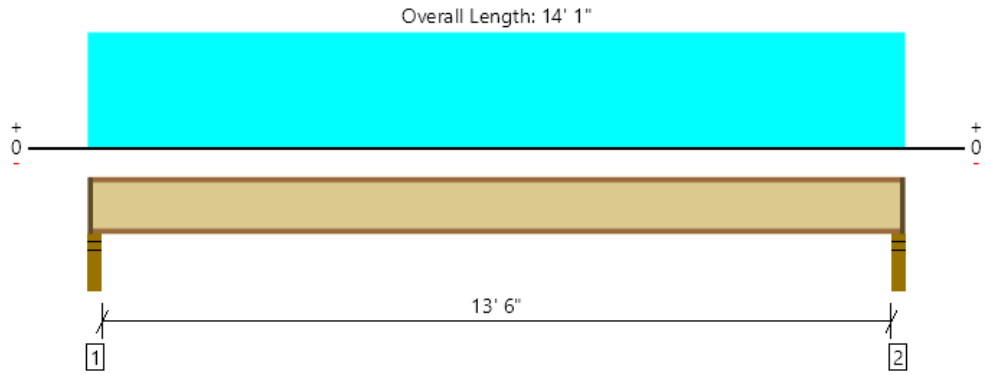
Floor, Deck joists
1 piece(s) 11 7/8" TJI® 360 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	795 @ 2 1/2"	1202 (2.25")	Passed (66%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	774 @ 3 1/2"	1705	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2677 @ 7' 1/2"	6180	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.170 @ 7' 1/2"	0.273	Passed (L/965)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.239 @ 7' 1/2"	0.683	Passed (L/685)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	56	55	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/600) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	244	563	235	1042	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	244	563	235	1042	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 8" o/c	
Bottom Edge (Lu)	13' 11" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 14' 1"	16"	26.0	60.0	25.0	Default Load

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Holly Ashford SSF Engineers (206) 956-3743 hashford@ssfengineers.com	

Weyerhaeuser

2/8/2022 9:29:08 PM UTC
ForteWEB v3.2, Engine: V8.2.0.17, Data: V8.1.0.16
File Name: 8480 Residence

Beam Analysis

Beam:	B1 - TYP BEAM				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.446	0.220		0.666	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions	
Number of Supports	2
Total Beam Length	22.25
Left End Condition	Pinned
Right End Condition	Pinned
R_1	7.409 / 0.00
R_2	7.409 / 22.25
R_3	0.000 / 22.25
R_4	0.000 / 22.25
R_5	0.000 / 22.25
R_6	0.000 / 22.25
R_7	0.000 / 22.25
R_8	0.000 / 22.25
R_9	0.000 / 22.25
R_{10}	0.000 / 22.25

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Stresses @ Input Location	
f_c (psi)	78
f_b (psi)	7686

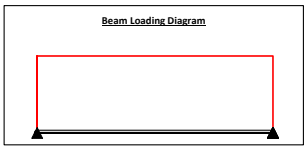
Max/Min Stresses	
f_{c_MAX} (psi)	1643
f_{c_MIN} (psi)	-1643
f_{b_MAX} (psi)	7704
f_{b_MIN} (psi)	0

Demand Output		
Location, ft		10.60
Shear, k	V =	0.35
Moment, k-ft	M =	41.12
Deflection, in	Δ =	-0.32
A/Span		L/827

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	391
S (in^3)	64.2
A (in^2)	4.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x50		
F_y, ksi	50		
Beam Weight (plf)	50		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.51 in^2	$\Phi_v V_n$	90.3 k
S	64.2 in^3	$\Phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\Phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	7.41	-7.41	-	41.21	-0.324 (↓)	11.13	L/824	-0.107 (↓)	11.13	L/2495

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Beam Analysis

Beam:	B2 Typ beam at ceiling cove (not used)				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.473	0.220		0.693	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions	
Number of Supports	2
Total Beam Length	22.25
Left End Condition	Pinned
Right End Condition	Pinned
R_1	7.710 / 0.00
R_2	7.710 / 22.25
R_3	0.000 / 22.25
R_4	0.000 / 22.25
R_5	0.000 / 22.25
R_6	0.000 / 22.25
R_7	0.000 / 22.25
R_8	0.000 / 22.25
R_9	0.000 / 22.25
R_{10}	0.000 / 22.25

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Stresses @ Input Location	
f_c (psi)	1372
f_b (psi)	0

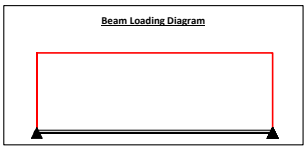
Max/Min Stresses	
f_{c_MAX} (psi)	1372
f_{c_MIN} (psi)	-1372
f_{b_MAX} (psi)	5991
f_{b_MIN} (psi)	0

Demand Output		
Location, ft		0.00
Shear, k	V =	7.71
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
A/Span		L/∞

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	455
S (in^3)	85.9
A (in^2)	5.62
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W10x77		
F_y, ksi	50		
Beam Weight (plf)	77		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	5.62 in^2	$\Phi_v V_n$	112 k
S	85.9 in^3	$\Phi_v V_n$	169 k
Z	97.6 in^3	M_n/Ω_b	244 k-ft
I	455 in^4	$\Phi_b M_n$	366 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	7.71	-7.71	-	42.88	-0.29 (↓)	11.13	L/921	-0.092 (↓)	11.13	L/2902

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Beam Analysis

Beam:	B3 Beam				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.238	0.100		0.338	
w_2	0.288	0.160		0.448	7
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	5.3	2.5		7.800	5.3333333
P_2	5.3	2.5		7.800	10.67
P_3	2.66	1.12		3.780	7.00
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions	
Number of Supports	2
Total Beam Length	16.00
Left End Condition	Pinned
Right End Condition	Pinned
R_1	13.764 / 0.00
R_2	15.056 / 16.00
R_3	0.000 / 16.00
R_4	0.000 / 16.00
R_5	0.000 / 16.00
R_6	0.000 / 16.00
R_7	0.000 / 16.00
R_8	0.000 / 16.00
R_9	0.000 / 16.00
R_{10}	0.000 / 16.00

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Stresses @ Input Location	
f_c (psi)	3135
f_b (psi)	0

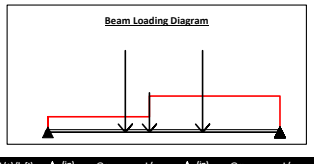
Max/Min Stresses	
f_{c_MAX} (psi)	3135
f_{c_MIN} (psi)	-3430
f_{b_MAX} (psi)	11549
f_{b_MIN} (psi)	0

Demand Output		
Location, ft		0.00
Shear, k	V =	13.76
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
A/Span		L/∞

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	475
S (in^3)	78
A (in^2)	4.39
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x58		
F_y, ksi	50		
Beam Weight (plf)	58		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.39 in^2	$\Phi_v V_n$	87.8 k
S	78 in^3	$\Phi_v V_n$	132 k
Z	86.4 in^3	M_n/Ω_b	216 k-ft
I	475 in^4	$\Phi_b M_n$	324 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	13.76	-15.06	0.00	75.07	-0.247 (↓)	8	L/777	-0.078 (↓)	8.03	L/2462

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Beam Analysis

Beam:	B4 - Deck outer beam				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.266	0.405		0.671	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions	
Number of Supports	2
Total Beam Length	29.75
Left End Condition	Pinned
Right End Condition	Pinned
R_1	9.981 / 0.00
R_2	9.981 / 29.75
R_3	0.000 / 29.75
R_4	0.000 / 29.75
R_5	0.000 / 29.75
R_6	0.000 / 29.75
R_7	0.000 / 29.75
R_8	0.000 / 29.75
R_9	0.000 / 29.75
R_{10}	0.000 / 29.75

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

Stresses @ Input Location	
f_c (psi)	1329
f_b (psi)	0

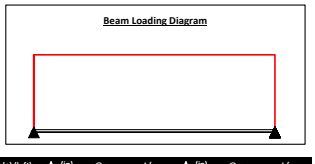
Max/Min Stresses	
f_{c_MAX} (psi)	1329
f_{c_MIN} (psi)	-1329
f_{b_MAX} (psi)	6648
f_{b_MIN} (psi)	0

Demand Output		
Location, ft		0.00
Shear, k	V =	9.98
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
A/Span		L/∞

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	1110
S (in^3)	134
A (in^2)	7.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W16x77		
F_y, ksi	50		
Beam Weight (plf)	77		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	30		
C_b	1		
A_w	7.51 in^2	$\Phi_v V_n$	150 k
S	134 in^3	$\Phi_v V_n$	225 k
Z	150 in^3	M_n/Ω_b	212 k-ft
I	1110 in^4	$\Phi_b M_n$	318 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	9.98	-9.98	-	74.23	-0.367 (↓)	14.88	L/973	-0.222 (↓)	14.88	L/1608

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L/800 = 0.44 (DIFFERENCE = 0.128)

1/4" CAMBER TO GET CLOSER TO SLIDER LIMIT

Beam Analysis

Beam:	B5 - Shade pocket beam				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.082	0.120		0.202	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	4.2	6		10.200	13.75
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		14.50
Left End Condition		Pinned
Right End Condition		Pinned
R_1	1.992	0.00
R_2	11.137	14.50
R_3	0.000	14.50
R_4	0.000	14.50
R_5	0.000	14.50
R_6	0.000	14.50
R_7	0.000	14.50
R_8	0.000	14.50
R_9	0.000	14.50
R_{10}	0.000	14.50

Demand Output		
Location, ft		0.00
Shear, k	V =	1.99
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

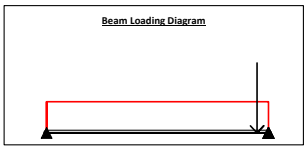
Stresses @ Input Location	
f_c (psi)	709
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	709
f_{c_MIN} (psi)	-3963
f_{b_MAX} (psi)	3529
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	204
S (in^3)	33.4
A (in^2)	2.81
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	14		
C_b	1		
A_w	2.81 in^2	$\Phi_v V_n$	56.1 k
S	33.4 in^3	$\Phi_b V_n$	84.2 k
Z	37.2 in^3	M_n/Ω_b	61.5 k-ft
I	204 in^4	$\Phi_b M_n$	92.5 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{TL} (in)	@ x =	L/
Span 1	1.99	-11.14	0.00	9.82	-0.064 (↓)	7.74	L/2719	-0.038 (↓)	7.74	L/4579

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Beam Analysis

Beam:	B6 - outer deck beam				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.334	0.405		0.739	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	0.9	1.2		2.100	30
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		31.00
Left End Condition		Pinned
Right End Condition		Pinned
R_1	11.522	0.00
R_2	13.487	31.00
R_3	0.000	31.00
R_4	0.000	31.00
R_5	0.000	31.00
R_6	0.000	31.00
R_7	0.000	31.00
R_8	0.000	31.00
R_9	0.000	31.00
R_{10}	0.000	31.00

Demand Output		
Location, ft		0.00
Shear, k	V =	11.52
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

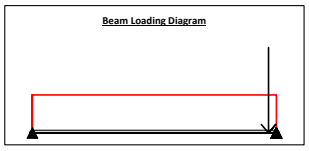
Stresses @ Input Location	
f_c (psi)	1534
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	1534
f_{c_MIN} (psi)	-1796
f_{b_MAX} (psi)	8044
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	1110
S (in^3)	134
A (in^2)	7.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W16x77		
F_y, ksi	50		
Beam Weight (plf)	77		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	7.51 in^2	$\Phi_v V_n$	150 k
S	134 in^3	$\Phi_b V_n$	225 k
Z	150 in^3	M_n/Ω_b	374 k-ft
I	1110 in^4	$\Phi_b M_n$	563 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{TL} (in)	@ x =	L/
Span 1	11.52	-13.49	-	89.83	-0.484 (↓)	15.5	L/769	-0.265 (↓)	15.56	L/1404

PROJECT Building
L/800 = 0.465 ; DIFF = 0.2
CAMBER 1/4"
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SHEET

Beam Analysis

Beam:	B7 - CANTILEVERED DECK BEAM				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.100	0.080		0.180	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1	2.5	0.0		2.460	1
P_2	1.1	0.9		2.002	13.75
P_3	5.5	6.3		11.800	13.75
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		13.75
Left End Condition		Pinned
Right End Condition		Pinned
R_1	-7.419	0.00
R_2	26.161	8.00
R_3	0.000	8.00
R_4	0.000	8.00
R_5	0.000	8.00
R_6	0.000	8.00
R_7	0.000	8.00
R_8	0.000	8.00
R_9	0.000	8.00
R_{10}	0.000	8.00

Demand Output		
Location, ft		0.00
Shear, k	V =	-7.42
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

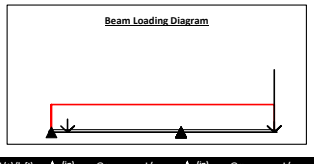
Stresses @ Input Location	
f_c (psi)	-1152
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	2304
f_{c_MIN} (psi)	-1758
f_{b_MAX} (psi)	0
f_{b_MIN} (psi)	-8445

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	954
S (in^3)	117
A (in^2)	6.44
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W16x67		
F_y, ksi	50		
Beam Weight (plf)	67		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	6.44 in^2	$\Phi_v V_n$	129 k
S	117 in^3	$\Phi_b V_n$	193 k
Z	130 in^3	M_n/Ω_b	324 k-ft
I	954 in^4	$\Phi_b M_n$	488 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{TL} (in)	@ x =	L/
Span 1	-7.42	-11.32	-82.34	-	0.02 (↑)	4.68	L/4785	0.011 (↑)	4.65	L/8700
Right Cantilever	14.84	-	-82.34	-	-0.132 (↓)	13.75	L/1046	-0.07 (↓)	13.75	L/1984

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Beam Analysis

Beam:	B9 - TYP BEAM				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.386	0.200		0.586	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft/ft) t_1				0.000	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		14.00
Left End Condition		Pinned
Right End Condition		Pinned
R_1	4.102	0.00
R_2	4.102	14.00
R_3	0.000	14.00
R_4	0.000	14.00
R_5	0.000	14.00
R_6	0.000	14.00
R_7	0.000	14.00
R_8	0.000	14.00
R_9	0.000	14.00
R_{10}	0.000	14.00

Demand Output		
Location, ft		0.00
Shear, k	V =	4.10
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

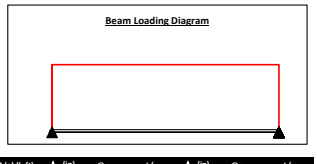
Stresses @ Input Location	
f_c (psi)	1460
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	1460
f_{c_MIN} (psi)	-1460
f_{b_MAX} (psi)	5158
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	204
S (in^3)	33.4
A (in^2)	2.81
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	2.81 in^2	$\Phi_v V_n$	56.1 k
S	33.4 in^3	$\Phi_b V_n$	84.2 k
Z	37.2 in^3	M_n/Ω_b	92.8 k-ft
I	204 in^4	$\Phi_b M_n$	140 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{TL} (in)	@ x =	L/
Span 1	4.10	-4.10	-	14.36	-0.086 (↓)	7	L/1953	-0.029 (↓)	7	L/5793

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Beam Analysis

Beam: B10 - SLIDER HEADER

Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.270	0.038		0.307	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	2.7	1.4		4.100	4.625
	P_2	2.7	1.4		4.100	9.25
	P_3	2.7	1.4		4.100	13.88
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		18.25
Left End Condition		Pinned
Right End Condition		Pinned
R_1	8.871	0.00
R_2	9.040	18.25
R_3	0.000	18.25
R_4	0.000	18.25
R_5	0.000	18.25
R_6	0.000	18.25
R_7	0.000	18.25
R_8	0.000	18.25
R_9	0.000	18.25
R_{10}	0.000	18.25

Demand Output		
Location, ft		0.00
Shear, k	V =	8.87
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

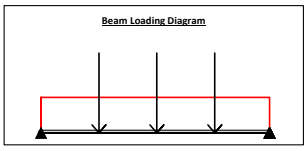
Stresses @ Input Location	
f_c (psi)	1967
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	1967
f_{c_MIN} (psi)	-2004
f_{b_MAX} (psi)	9330
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	391
S (in^3)	64.2
A (in^2)	4.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x50		
F_y, ksi	50		
Beam Weight (plf)	50		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	4.51 in^2	Φ_v/Ω_v	90.3 k
S	64.2 in^3	$\Phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	179 k-ft
I	391 in^4	$\Phi_b M_n$	270 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	8.87	-9.04	-	49.91	-0.254 (↓)	9.13	L/862	-0.072 (↓)	9.13	L/3042

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Beam Analysis

Beam: B11 - MASTER BATH BEAM

Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.180	0.150		0.150	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	2.7	1.4		1.400	5.3333333
	P_2	2.7	1.4		1.400	10.67
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		16.00
Left End Condition		Pinned
Right End Condition		Pinned
R_1	2.600	0.00
R_2	2.600	16.00
R_3	0.000	16.00
R_4	0.000	16.00
R_5	0.000	16.00
R_6	0.000	16.00
R_7	0.000	16.00
R_8	0.000	16.00
R_9	0.000	16.00
R_{10}	0.000	16.00

Demand Output		
Location, ft		0.00
Shear, k	V =	2.60
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	0.00
Live	1.00
Snow	0.00

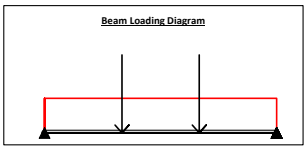
Stresses @ Input Location	
f_c (psi)	813
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	2106
f_{c_MIN} (psi)	-2106
f_{b_MAX} (psi)	10081
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	238
S (in^3)	38.6
A (in^2)	3.2
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x30		
F_y, ksi	50		
Beam Weight (plf)	30		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	3.2 in^2	Φ_v/Ω_v	64 k
S	38.6 in^3	$\Phi_v V_n$	95.9 k
Z	43.1 in^3	M_n/Ω_b	108 k-ft
I	238 in^4	$\Phi_b M_n$	162 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	6.74	-6.74	0.00	32.43	-0.22 (↓)	8	L/873	-0.083 (↓)	8	L/2313

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Beam Analysis

Beam: B12 - MASTER BATH BEAM

Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.383	0.180		0.563	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1	4.1	2.6		6.700	7
	P_2				0.000	
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		10.00
Left End Condition		Pinned
Right End Condition		Pinned
R_1	4.826	0.00
R_2	7.506	10.00
R_3	0.000	10.00
R_4	0.000	10.00
R_5	0.000	10.00
R_6	0.000	10.00
R_7	0.000	10.00
R_8	0.000	10.00
R_9	0.000	10.00
R_{10}	0.000	10.00

Demand Output		
Location, ft		0.00
Shear, k	V =	4.83
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

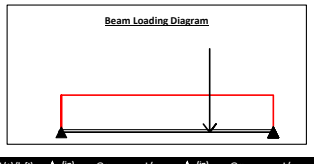
Stresses @ Input Location	
f_c (psi)	1718
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	1718
f_{c_MIN} (psi)	-2671
f_{b_MAX} (psi)	7180
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	204
S (in^3)	33.4
A (in^2)	2.81
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	2.81 in^2	Φ_v/Ω_v	56.1 k
S	33.4 in^3	$\Phi_v V_n$	84.2 k
Z	37.2 in^3	M_n/Ω_b	92.8 k-ft
I	204 in^4	$\Phi_b M_n$	140 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	4.83	-7.51	-	19.98	-0.054 (↓)	5.3	L/2222	-0.019 (↓)	5.34	L/6316

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Beam Analysis

Beam: B13 - BEAM

Load		Dead	Live	Snow	Factored	Location
Distributed (k/ft)	w_1	0.346	0.400		0.746	
	w_2				0.000	
	w_3				0.000	
	w_4				0.000	
	w_5				0.000	
	w_6				0.000	
	w_7				0.000	
	w_8				0.000	
	w_9				0.000	
	w_{10}				0.000	
Trapezoidal (k/ft/ft)	t_1				0.000	
	t_2				0.000	
	t_3				0.000	
	t_4				0.000	
	t_5				0.000	
	t_6				0.000	
Point (k)	P_1					
	P_2					
	P_3				0.000	
	P_4				0.000	
	P_5				0.000	
	P_6				0.000	
	P_7				0.000	
	P_8				0.000	
	P_9				0.000	
	P_{10}				0.000	

Support Locations and Reactions		
Number of Supports		2
Total Beam Length		12.50
Left End Condition		Pinned
Right End Condition		Pinned
R_1	4.663	0.00
R_2	4.663	12.50
R_3	0.000	12.50
R_4	0.000	12.50
R_5	0.000	12.50
R_6	0.000	12.50
R_7	0.000	12.50
R_8	0.000	12.50
R_9	0.000	12.50
R_{10}	0.000	12.50

Demand Output		
Location, ft		0.00
Shear, k	V =	4.66
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
Δ/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

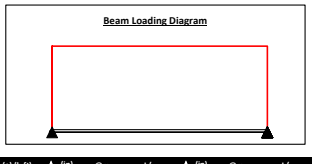
Stresses @ Input Location	
f_c (psi)	1659
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	1659
f_{c_MIN} (psi)	-1659
f_{b_MAX} (psi)	5235
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	29000
b (in)	5.5
d (in)	7.5
I (in^4)	204
S (in^3)	33.4
A (in^2)	2.81
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	W12x26		
F_y, ksi	50		
Beam Weight (plf)	26		
Axis of Bending	Strong		
Unbraced Length (L_b), ft	0		
C_b	1		
A_w	2.81 in^2	Φ_v/Ω_v	56.1 k
S	33.4 in^3	$\Phi_v V_n$	84.2 k
Z	37.2 in^3	M_n/Ω_b	92.8 k-ft
I	204 in^4	$\Phi_b M_n$	140 k-ft

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_{LL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	4.66	-4.66	-	14.57	-0.069 (↓)	6.25	L/2174	-0.037 (↓)	6.25	L/4054

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Beam Analysis

Beam:	B14 - WOOD BEAM				
Load	Dead	Live	Snow	Factored	Location
Distributed (k/ft) w_1	0.068			0.068	
w_2				0.000	
w_3				0.000	
w_4				0.000	
w_5				0.000	
w_6				0.000	
w_7				0.000	
w_8				0.000	
w_9				0.000	
w_{10}				0.000	
Trapezoidal (k/ft) t_1	0.00525	0.0075		0.013	
t_2				0.000	
t_3				0.000	
t_4				0.000	
t_5				0.000	
t_6				0.000	
Point (k) P_1				0.000	
P_2				0.000	
P_3				0.000	
P_4				0.000	
P_5				0.000	
P_6				0.000	
P_7				0.000	
P_8				0.000	
P_9				0.000	
P_{10}				0.000	

Support Locations and Reactions	
Number of Supports	2
Total Beam Length	16.00
Left End Condition	Pinned
Right End Condition	Pinned
R_1	1.088 @ 0.00
R_2	1.632 @ 16.00
R_3	0.000 @ 16.00
R_4	0.000 @ 16.00
R_5	0.000 @ 16.00
R_6	0.000 @ 16.00
R_7	0.000 @ 16.00
R_8	0.000 @ 16.00
R_9	0.000 @ 16.00
R_{10}	0.000 @ 16.00

Demand Output		
Location, ft		0.00
Shear, k	V =	1.09
Moment, k-ft	M =	0.00
Deflection, in	Δ =	0.00
L/Span		L/∞

Load Factors	
Dead	1.00
Live	1.00
Snow	1.00

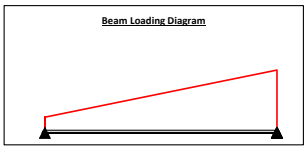
Stresses @ Input Location	
f_c (psi)	39
f_b (psi)	0

Max/Min Stresses	
f_{c_MAX} (psi)	39
f_{c_MIN} (psi)	-59
f_{b_MAX} (psi)	801
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	1700
b (in)	3.5
d (in)	11.875
I (in^4)	488.4135
S (in^3)	82.25911
A (in^2)	41.5625
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section	NONE

Beam Loading Diagram



Span	V_{Left} (kips)	V_{Right} (kips)	M (-) (k-ft)	M (+) (k-ft)	Δ_L (in)	@ x =	L/	Δ_L (in)	@ x =	L/
Span 1	1.09	-1.63	-	5.49	-0.331 (↓)	8.19	L/580	-0.117 (↓)	8.32	L/1641

PROJECT Building

DATE 10/19/2022

PROJ. #

DESIGN ENG

SHEET

PLB™-36 FormLok® Composite Steel Deck-Slab (LRFD)

with 4.5 in. 150 pcf 2500 psi NWC

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	6'-0"	7'-0"	7'-1"
20	7'-2"	8'-3"	8'-5"
18	8'-2"	9'-8"	10'-0"
16	8'-9"	10'-9"	10'-10"

6ft spacing typical, multispan ok

Maximum Unshored Span based on:

Uniform Construction Load	20.00	psf	Minimum End Bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum Interior Bearing	5.50	in.
Concrete Ponding Allowance	2.00	psf	Maximum Deflection L/	180	≤ 0.75 in.
Concrete Volume	1.09	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ $in.^3/ft$	Se- $in.^3/ft$	Id+ $in.^4/ft$	Id- $in.^4/ft$	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Typical Loading:
DL 105psf
LL 40psf

Wf=1.2*105+1.6*40=190psf
OK

Superimposed Design Load, ϕWn, / Deflection at L/360 ,psf [1]

Gage	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	1886/3767	1290/1929	879/1116	631/702	470/470	360/330	281/241	222/181	178/139
20	1886/4036	1497/2066	1040/1195	749/753	560/504	431/354	338/258	270/194	218/149
18	1885/4515	1496/2312	1238/1338	971/842	730/564	565/396	447/289	359/217	293/167
16	1884/4942	1496/2530	1237/1464	1052/922	890/617	691/433	549/316	443/237	363/183

Notes: [1] For high loads, commonly in excess of 325 psf, dynamic or impact loading, and long term concrete creep should be considered. Contact Verco for further assistance.

Composite Steel Deck-Slab Properties

Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft
22	46.1	3.74	7.30	5.52	4.21	3.88
20	46.5	4.25	7.57	5.91	4.93	3.88
18	47.1	5.14	8.08	6.61	6.30	3.88
16	47.7	5.92	8.55	7.24	7.58	3.88

Min. Temperature & Shrinkage

Gage	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	0.028	15
20	0.028	15
18	0.028	15
16	0.028	15

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Tables generated using calculator V3.2 based on ANSI/SDI C-2017 in accordance with 2018 IBC Section 2210.

Date: 2/25/2022

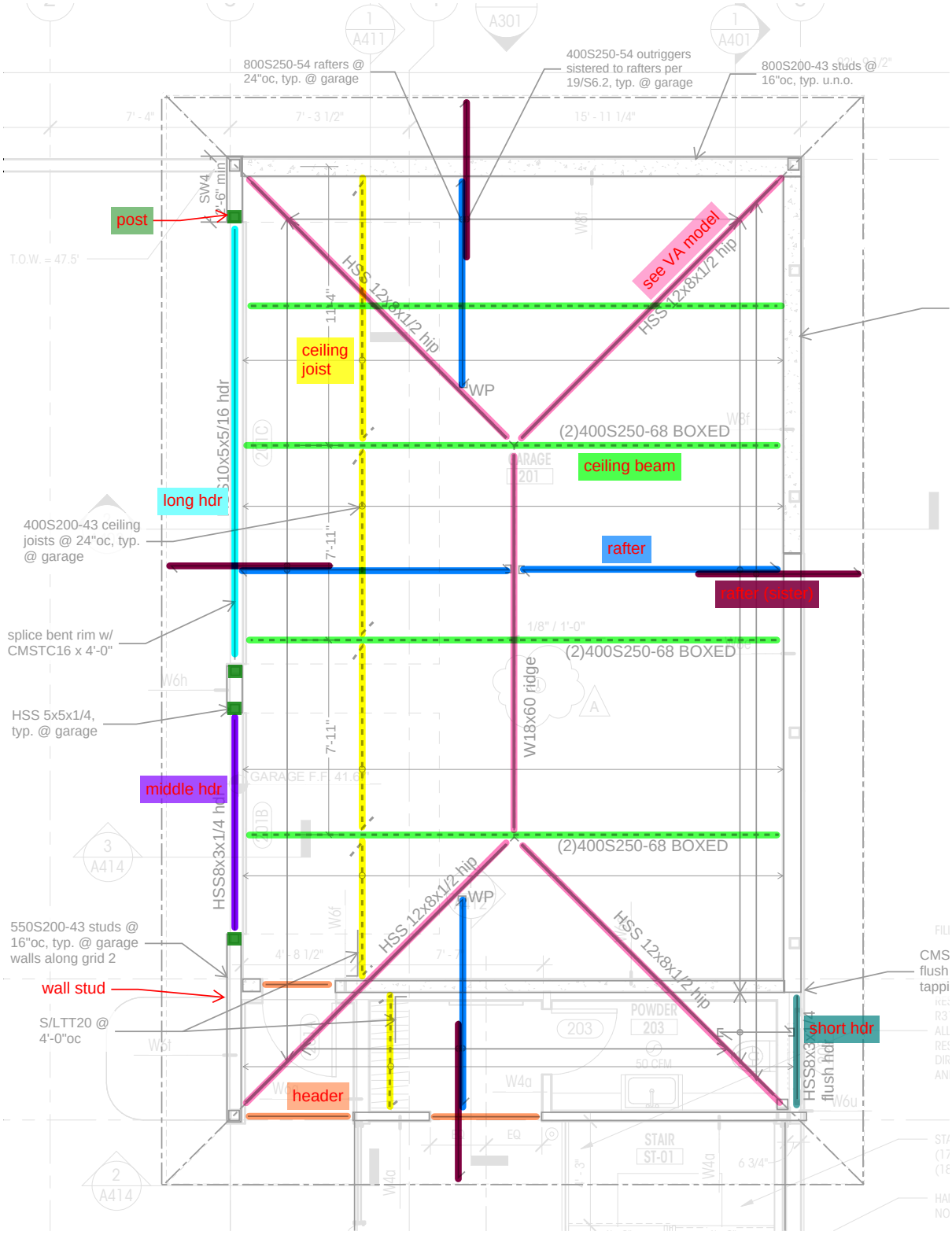
Garage Gravity

STRUCTURAL ENGINEERING

SEATTLE 2124 Third Avenue, Suite 100, Seattle, WA 98121
TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402
CENTRAL WA 414 N Pearl Street, Suite 8, Ellensburg, WA 98926

206.443.6212
ssfengineers.com

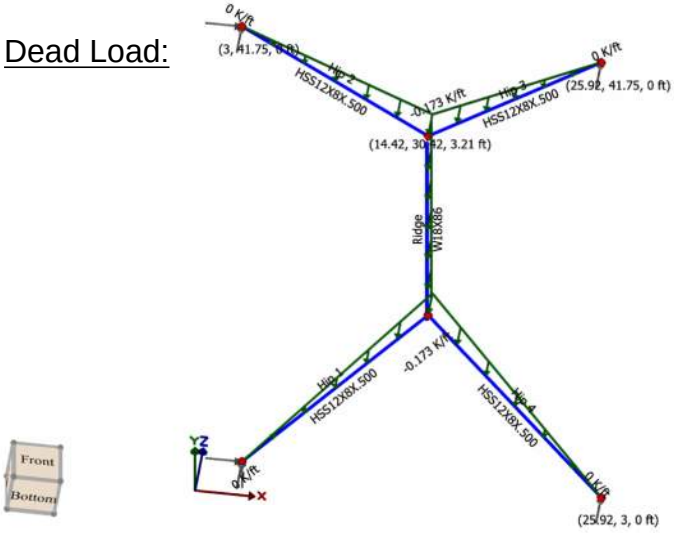
Garage Roof Key Plan



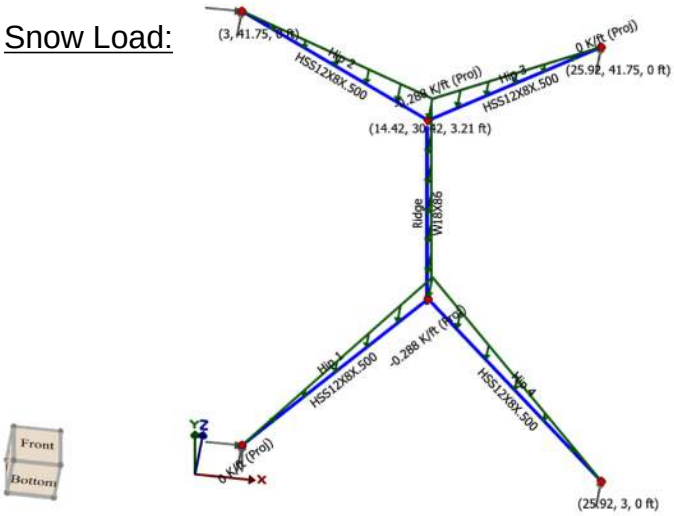
STRUCTURAL ENGINEERING

8480 Residence	10/10/2022
PROJECT	DATE
147	01519-2021-09
	PROJ. #
	LAN
	DESIGN
	SHEET

Dead Load:



Snow Load:



Members

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
Hip 1	N001	N002	HSS12X8X.500	16.4662	0.9652	Bracing
Hip 2	N004	N005	HSS12X8X.500	16.4039	0.9616	Bracing
Hip 3	N004	N006	HSS12X8X.500	16.4597	0.9648	Bracing
Hip 4	N002	N003	HSS12X8X.500	16.5218	0.9685	Bracing
Ridge	N002	N004	W18X86	16.0000	1.3796	Beam

Result Cases

Name	ID	Design Checks	Result Type
1. 1.4D	1	Strength (LRFD)	Static
2. 1.2D+1.6L+0.5Lr	2	Strength (LRFD)	Static
2. 1.2D+1.6L+0.5S	3	Strength (LRFD)	Static
3. 1.2D+1.6S+L	4	Strength (LRFD)	Static
5. 0.9D+W	5	Strength (LRFD)	Static
6. 1.2D+E+L+0.2S	6	Strength (LRFD)	Static
D+L	7	Deflections	Static
D+S	8	Deflections	Static
Snow	9	Deflections	Static

Member Displacements

(extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
Hip 2	0.0000 (5)	**1.9553 (4)**	-0.7625 (4)	-0.0044 (5)
Hip 3	**-1.9329 (4)**	0.0239 (4)	-0.7041 (4)	0.0203 (4)
Hip 4	-1.9404 (4)	0.0225 (4)	**-0.7853 (4)**	**-0.0075 (5)**
Ridge	**-2.1916 (4)**	**-0.2744 (5)**	**0.0057 (5)**	**0.0433 (4)**

Member Forces

(extreme rows: max and min)

Member	Fx Min K	Fx Max K	Vy K	Vz K	Torsion K-ft	My Min K-ft	My Max K-ft	Mz Min K-ft	Mz Max K-ft
Hip 1	-1.2869 (4)	-0.0611 (5)	6.3549 (4)	**1.2389 (4)**	0.0000 (4)	0.0000 (4)	**16.4053 (4)**	0.0000 (9)	84.1536 (4)
Hip 2	**-1.2946 (4)**	**-0.0622 (5)**	**6.3691 (4)**	1.2365 (4)	0.0000 (4)	**-16.3367 (4)**	0.0000 (4)	**-84.1478 (4)**	0.0000 (8)
Hip 3	-1.2839 (4)	-0.0607 (5)	**-6.3410 (4)**	1.2184 (4)	0.0000 (4)	**-16.1183 (4)**	0.0000 (5)	0.0000 (9)	**83.8892 (4)**
Hip 4	**-1.2762 (4)**	**-0.0596 (5)**	-6.3269 (4)	**-1.2207 (4)**	0.0000 (4)	0.0000 (8)	**16.1860 (4)**	0.0000 (9)	83.8928 (4)
Ridge	0.0000 (9)	0.0000 (8)	-4.5510 (4)	0.0000 (4)	**0.0000 (4)**	0.0000 (4)	0.0000 (4)	**16.6311 (5)**	**138.6549 (4)**

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K
N002	3. 1.2D+1.6S+L	0.0422	0.5267	**-2.0263**	0.0000	0.0000	0.0000
N002	5. 0.9D+W	**0.0057**	0.0716	-0.2754	0.0000	0.0000	0.0000
N003	3. 1.2D+1.6S+L	0.0836	**-0.0006**	0.0000	0.0000	0.0000	6.5071
N003	5. 0.9D+W	0.0113	-0.0001	0.0000	0.0000	0.0000	**0.7216**
N004	5. 0.9D+W	0.0059	0.0716	**-0.2744**	0.0000	0.0000	0.0000
N005	3. 1.2D+1.6S+L	0.0000	**1.0544**	0.0000	0.0000	0.0000	**6.5546**
N006	3. 1.2D+1.6S+L	**0.0889**	1.0519	0.0000	0.0000	0.0000	6.5219

Member Unity Checks

Member	Section	Unity Check	Status	Result Case	Code Reference	Type	Design Group
Hip 1	HSS12X8X.500	0.4509	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1

Member Unity Checks (continued)

Member	Section	Unity Check	Status	Result Case	Code Reference	Type	Design Group
Hip 2	HSS12X8X.500	0.4505	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Hip 3	HSS12X8X.500	0.4482	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Hip 4	HSS12X8X.500	0.4486	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Ridge	W18X86	0.2251	Pass	3. 1.2D+1.6S+L	F2-2	Strong Flexure Check	Steel_Beam_G 1

Steel_V Brace_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_V Brace_G 1: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Hip 1	HSS12X8X.500	16.4662	3. 1.2D+1.6S+L	0.4509	1.0000	H1-1b	**0.4509**	KLz = 16.466 ft, KLy = 16.466 ft, KL(torsion) = 16.466 ft, Lb = 16.466 ft, Axial Unity = 0.0008, Mz Unity = 0.35818, My Unity = 0.09233, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.4962

Steel_V Brace_G 1: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Hip 2	HSS12X8X.500	14.7636	3. 1.2D+1.6S+L	1.2946	553.5327	E3-2	**0.0023**	KLz = 16.404 ft, KLy = 16.404 ft, KL(torsion) = 16.404 ft, Fcr = 35.758 Ksi, Fe (E3-4) = 76.442 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

Steel_V Brace_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Hip 1	HSS12X8X.500	16.4662	3. 1.2D+1.6S+L	84.1536	234.9450	F7-1	**0.3582**	Lb = 16.466 ft, Cb = 1.4962

Steel_V Brace_G 1: Weak Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand My K-ft	Capacity My K-ft	Code Reference	Unity Check	Details
Hip 1	HSS12X8X.500	16.4662	3. 1.2D+1.6S+L	16.4053	177.6750	F7-1	**0.0923**	

Steel_V Brace_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Hip 2	HSS12X8X.500	14.7636	3. 1.2D+1.6S+L	6.3691	244.9882	G4-1	**0.0260**	Shear Area = 9.8627 in^2, Cv = 1

Steel_Beam_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

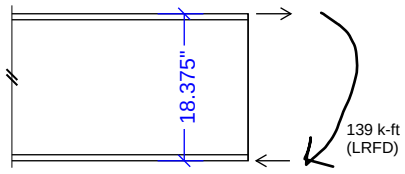
Steel_Beam_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Ridge	W18X86	8.0000	3. 1.2D+1.6S+L	138.6549	616.0552	F2-2	**0.2251**	Lp = 9.2898 ft, Lr = 28.511 ft, Lb = 16 ft, Cb = 1.0159

Steel_Beam_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Ridge	W18X86	16.0000	3. 1.2D+1.6S+L	-4.5510	264.9600	G2-1	**0.0172**	Shear Area = 8.832 in^2, Cv = 1, h/tw = 33.458

Ridge to Hip Connection - Garage Controls



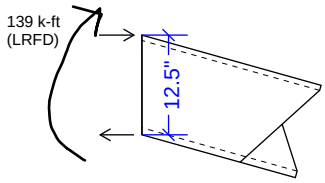
FOR MOMENT: *top + bot welds*
139 k-ft / 1.53 ft = 90.8 kips

3/8" bevel weld capacity:
0.8*0.6*70*11*0.375 = 138.6 kips
OK

FOR SHEAR: *side vertical welds*
10 kips

3/8" bevel weld capacity:
0.75*0.6*70*11*0.375 = 130.0 kips

OK



FOR MOMENT: *top + bot welds*
139 k-ft / 1.04 ft = 133.4 kips

3/8" bevel weld capacity:
0.8*0.6*70*11*0.375 = 138.6 kips
OK

FOR SHEAR: *side vertical welds*
10 kips

3/8" bevel weld capacity:
0.75*0.6*70*11*0.375 = 130.0 kips
OK

Beam Analysis

Beam:		short hdr					
Load		Dead	Snow	Other	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.125**	**0.166**			0.290	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1					0.000	
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**5.00**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	0.726	**0.00**
R_2	0.726	**5.00**
R_3	0.000	**5.00**
R_4	0.000	**5.00**
R_5	0.000	**5.00**
R_6	0.000	**5.00**
R_7	0.000	**5.00**
R_8	0.000	**5.00**
R_9	0.000	**5.00**
R_{10}	0.000	**5.00**

Demand Output		
Location, ft		**2.50**
Shear, k	V	0.00
Moment, k-ft	M =	0.91
Deflection, in	D =	0.00
Δ/Span		L/12999

Load Factors	
Dead	**1.00**
Snow	**1.00**
Other	**1.00**
Seismic	**1.00**

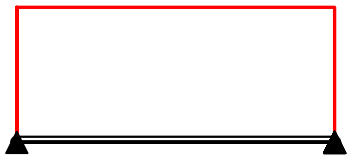
Stresses @ Input Location	
f_v (psi)	0
f_b (psi)	1249

Max/Min Stresses	
f_{v_MAX} (psi)	247
f_{v_MIN} (psi)	-247
f_{b_MAX} (psi)	1248
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	30.5
S (in^3)	8.72
A (in^2)	2.94
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		HSS7x4x1/4	
F_y, ksi		46	
Beam Weight (plf)		17.32	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		5.00	
C_b		1	
A_w	2.94 in^2	V_n/Ω_v	48.6 k
S	8.72 in^3	$\phi_v V_n$	72.9 k
Z	10.8 in^3	M_n/Ω_b	24.8 k-ft
I	30.5 in^4	$\phi_b M_n$	37.3 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	0.726	-0.726	-	0.907	-0.005 (↓)	2.5	L/12000	-0.003 (↓)	2.5	L/20000

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Beam Analysis

Beam:		middle hdr					
Load		Dead	Snow	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.175**	**0.250**			0.425	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1	**0.65**				0.650	**4.08**
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**9.00**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	2.269	**0.00**
R_2	2.209	**9.00**
R_3	0.000	**9.00**
R_4	0.000	**9.00**
R_5	0.000	**9.00**
R_6	0.000	**9.00**
R_7	0.000	**9.00**
R_8	0.000	**9.00**
R_9	0.000	**9.00**
R_{10}	0.000	**9.00**

Demand Output		
Location, ft		**4.50**
Shear, k	V	-0.29
Moment, k-ft	M =	5.63
Deflection, in	D =	-0.09
Δ/Span		L/1200

Load Factors	
Dead	**1.00**
Snow	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

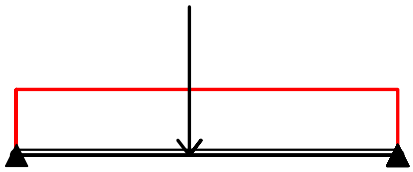
Stresses @ Input Location	
f_v (psi)	-100
f_b (psi)	7752

Max/Min Stresses	
f_{v_MAX} (psi)	772
f_{v_MIN} (psi)	-752
f_{b_MAX} (psi)	7872
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	30.5
S (in^3)	8.72
A (in^2)	2.94
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		HSS7x4x1/4	
F_y, ksi		46	
Beam Weight (plf)		17.32	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		9.00	
C_b		1	
A_w	2.94 in^2	V_n/Ω_v	48.6 k
S	8.72 in^3	$\phi_v V_n$	72.9 k
Z	10.8 in^3	M_n/Ω_b	24.8 k-ft
I	30.5 in^4	$\phi_b M_n$	37.3 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	2.27	-2.21	0	5.72	-0.09 (↓)	4.5	L/1200	-0.042 (↓)	4.5	L/2571

Beam Analysis

Beam:		long hdr					
Load		Dead	Snow	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.186**	**0.250**			0.436	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1	**0.730**				0.730	**1.08**
	P_2	**0.630**				0.630	**9.00**
	P_3	**0.520**				0.520	**14.67**
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**18.08**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	5.039	**0.00**
R_2	4.717	**18.08**
R_3	0.000	**18.08**
R_4	0.000	**18.08**
R_5	0.000	**18.08**
R_6	0.000	**18.08**
R_7	0.000	**18.08**
R_8	0.000	**18.08**
R_9	0.000	**18.08**
R_{10}	0.000	**18.08**

Demand Output		
Location, ft		**4.50**
Shear, k	V	2.35
Moment, k-ft	M =	15.77
Deflection, in	D =	-0.34
Δ/Span		L/632

Load Factors	
Dead	**1.00**
Snow	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

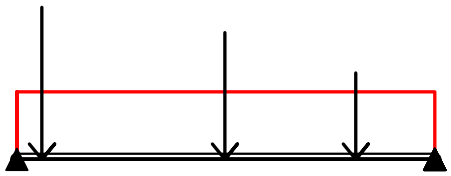
Stresses @ Input Location	
f_v (psi)	442
f_b (psi)	10515

Max/Min Stresses	
f_{v_MAX} (psi)	949
f_{v_MIN} (psi)	-889
f_{b_MAX} (psi)	14600
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	90.1
S (in^3)	18
A (in^2)	5.31
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		HSS10x4x5/16	
F_y, ksi		46	
Beam Weight (plf)		27.59	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		19.00	
C_b		1	
A_w	5.31 in^2	V_n/Ω_v	88 k
S	18 in^3	$\phi_v V_n$	132 k
Z	23.1 in^3	M_n/Ω_b	53 k-ft
I	90.1 in^4	$\phi_b M_n$	79.7 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	5.04	-4.72	0	21.9	-0.486 (↓)	9.1	L/447	-0.23 (↓)	9	L/943

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STRUCTURAL ENGINEERING

PROJECT Project

DATE 10/10/2022
PROJ. #
DESIGN ENG
SHEET

Beam Analysis

Beam:		header					
Load		Dead	Snow	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.120**	**0.166**			0.286	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1					0.000	
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Steel Beam Section	NONE

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**4.75**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	0.679	**0.00**
R_2	0.679	**4.75**
R_3	0.000	**4.75**
R_4	0.000	**4.75**
R_5	0.000	**4.75**
R_6	0.000	**4.75**
R_7	0.000	**4.75**
R_8	0.000	**4.75**
R_9	0.000	**4.75**
R_{10}	0.000	**4.75**

Demand Output		
Location, ft		**2.25**
Shear, k	V	0.04
Moment, k-ft	M =	0.80
Deflection, in	D =	-0.03
Δ/Span		L/2162

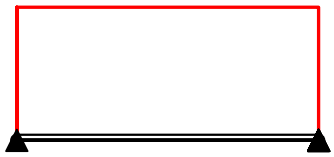
Load Factors	
Dead	**1.00**
Snow	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

Stresses @ Input Location	
f_v (psi)	2
f_b (psi)	367

Max/Min Stresses	
f_{v_MAX} (psi)	47
f_{v_MIN} (psi)	-47
f_{b_MAX} (psi)	368
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**1300**
b (in)	**3**
d (in)	**7.25**
I (in^4)	95.27
S (in^3)	26.281
A (in^2)	21.75
I (Override)	
S (Override)	
A (Override)	

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	0.679	-0.679	-	0.807	-0.026 (↓)	2.4	L/2192	-0.015 (↓)	2.4	L/3800

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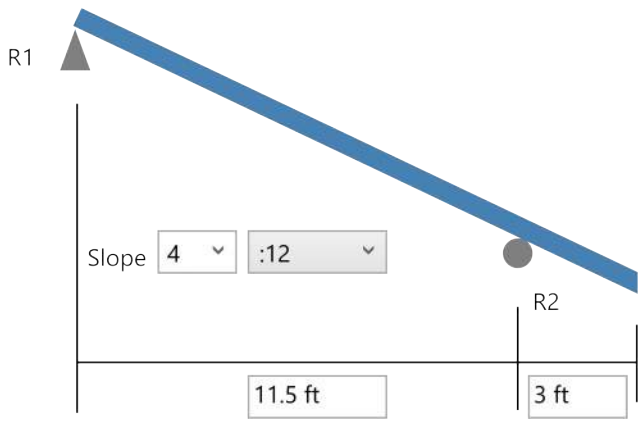
STRUCTURAL ENGINEERING

PROJECT Project

DATE 10/10/2022
PROJ. #
DESIGN ENG
SHEET

Project Name: garage LG v2
Model: Rafter
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/10/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.16



Section : 800S250-54 (50 ksi) @ 24" o.c. Single C Stud (punched)
Maxo = 3804.9(ft-lb) **Va =** 2091.3 **I =** 7.378 in^4

Bracing and Distortional Buckling Parameters

	Span	Overhang
Flexural Bracing	None	None
Axial Bracing	NA	None
Distortional Buckling, K-phi	0 lb-in/in	0 lb-in/in
Distortional Buckling Bracing, Lm	None	None

Load Cases

	Span (psf)	Overhang (psf)
Dead Load	15	15
Live Load	0.01	0.01
Snow Load	25	25
Inward Wind Load	10.2	10.2
Outward Wind Load	-21.2	-28.4

Load Combinations

LC Number	Dead	Live	Snow	Inward Wind	Outward Wind
1	1	0	1	0	0
2	1	0	0	1	0
3	1	0	0.75	0.75	0
4	0.6	0	0	0	1
5	0	0	0	0	0

6	0	0	0	0	0

Reactions

	Vertical				Horizontal			
	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.
R1	437.39	1	-71.41	4	98.60	2	-219.33	4
R2	787.27	3	-311.47	4	0.00	1	0.00	1

Rafter Flexural and Deflection

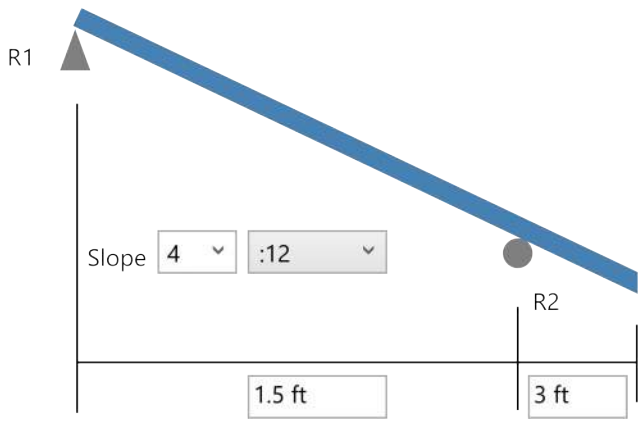
Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
1237	1833	0.67	3	L/1004	3	L/779	3

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	437.8	3	1.00	574.6	1005.5	0.40	3	NO
R2	746.9	3	6.00	1741.4	2873.4	0.30	3	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	438	3	1.000	0.21	0.00	0.21	3	N/A	N/A
R2	502	3	1.000	0.24	0.10	0.26	3	N/A	N/A



Section : 800S250-54 (50 ksi) @ 24" o.c. Single C Stud (punched)
Maxo = 3804.9(ft-lb) **Va =** 2091.3 **I =** 7.378 in^4

Bracing and Distortional Buckling Parameters

	Span	Overhang
Flexural Bracing	None	None
Axial Bracing	NA	None
Distortional Buckling, K-phi	0 lb-in/in	0 lb-in/in
Distortional Buckling Bracing, Lm	None	None

Load Cases

	Span (psf)	Overhang (psf)
Dead Load	15	15
Live Load	0.01	0.01
Snow Load	25	25
Inward Wind Load	10.2	10.2
Outward Wind Load	-21.2	-28.4

Load Combinations

LC Number	Dead	Live	Snow	Inward Wind	Outward Wind
1	1	0	1	0	0
2	1	0	0	1	0
3	1	0	0.75	0.75	0
4	0.6	0	0	0	1
5	0	0	0	0	0

6	0	0	0	0	0

Reactions

	Vertical				Horizontal			
	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.
R1	137.31	4	-201.43	3	30.60	2	-78.00	4
R2	581.33	3	-285.93	4	0.00	1	0.00	1

Rafter Flexural and Deflection

Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
388	3261	0.12	3	L/42638	3	L/6071	3

Rafter Bending and Web Crippling

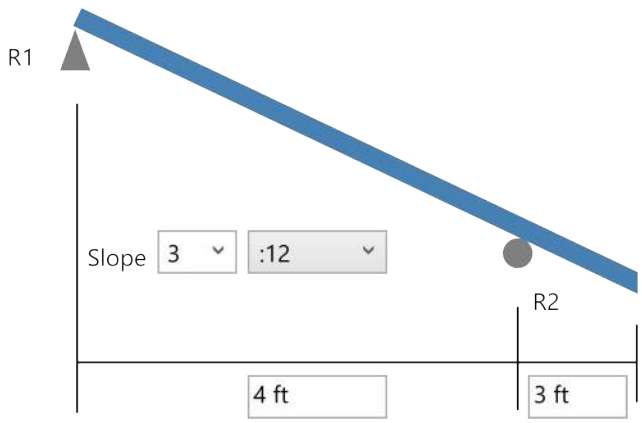
Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	-183.8	3	1.00	574.6	1005.5	0.17	3	NO
R2	551.5	3	6.00	1741.4	2873.4	0.24	3	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	184	3	1.000	0.09	0.00	0.09	3	N/A	N/A
R2	306	3	1.000	0.15	0.10	0.18	3	N/A	N/A

Project Name: garage LG v2
Model: Rafter (sister)
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/10/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.16



Section : 800S250-54 (50 ksi) @ 24" o.c. Single C Stud (punched)
Maxo = 3804.9(ft-lb) **Va =** 2091.3 **I =** 7.378 in^4

Bracing and Distortional Buckling Parameters

	Span	Overhang
Flexural Bracing	None	None
Axial Bracing	NA	None
Distortional Buckling, K-phi	0 lb-in/in	0 lb-in/in
Distortional Buckling Bracing, Lm	None	None

Load Cases

	Span (psf)	Overhang (psf)
Dead Load	15	15
Live Load	0.01	0.01
Snow Load	25	25
Inward Wind Load	10.2	10.2
Outward Wind Load	-21.2	-28.4

Load Combinations

LC Number	Dead	Live	Snow	Inward Wind	Outward Wind
1	1	0	1	0	0
2	1	0	0	1	0
3	1	0	0.75	0.75	0
4	0.6	0	0	0	1
5	0	0	0	0	0

6	0	0	0	0	0

Reactions

	Vertical				Horizontal			
	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.	Max Rxn (lb)	Load Comb.	Min Rxn (lb)	Load Comb.
R1	70.81	1	0.00	5	35.70	2	-85.00	4
R2	518.66	3	-225.40	4	0.00	1	0.00	1

Rafter Flexural and Deflection

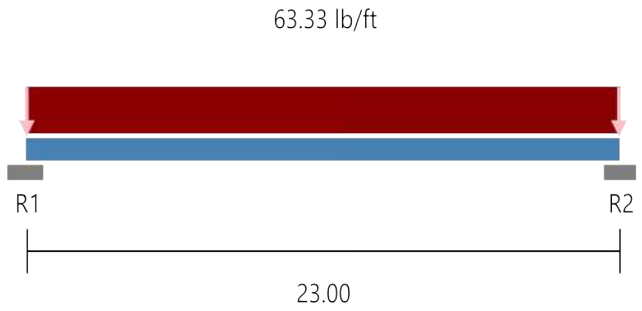
Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
381	3261	0.12	3	L/43915	3	L/5162	3

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	71.9	3	1.00	574.6	1005.5	0.07	3	NO
R2	503.2	3	6.00	1741.4	2873.4	0.22	3	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	72	3	1.000	0.03	0.00	0.03	3	N/A	N/A
R2	257	3	1.000	0.12	0.10	0.16	3	N/A	N/A



Reactions

Support	Reactions (lb)
R2	728.30
R1	728.30

Shear and Web Crippling Checks

Bending and Shear (Unstiffened): 14.4% Stressed @R2

Bending and Shear (Stiffened): NA

Web Stiffeners Required?: No

Section: (2) 550S250-68 (50 ksi) Boxed C Stud (punched)
Maxo = 6151.6 ft-lb **Va =** 10700.6 lb **I =** 7.73 in^4

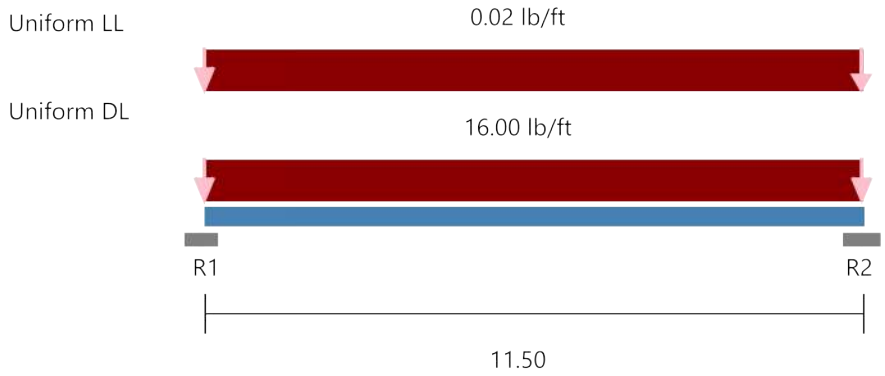
Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection

	Mmax (ft-lb)	Mmax/ Maxo	Mpos (ft-lb)	Bracing (in)	Ma-Brc (ft-lb)	Mpos/ Ma-Brc	Deflection (in)	Deflection Ratio
Span	4187.7	0.681	4187.7	None	6151.6	0.681	1.749	L/158

Project Name: garage LG v2
Model: Ceiling Joist
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/10/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.16



Section : 400S162-43 (33 ksi) @ 24 in" o.c. Single C Stud (punched)
Maxo = 686.0 ft-lb **Va =** 1739.1 lb **I =** 0.892 in^4

Deflection Limits: Total Load - 360 Live Load - 360

Load Comb:
1. DL + LL All spans
2. DL + LL Even spans
3. DL + LL Odd spans
4. LL All spans
5. LL Even spans
6. LL Odd spans

Joist Flexural and Deflection

	Mmax (ft-lb)	K-phi (lb-in/in)	Lm (in)	Ma-dist (ft-lb)	Mmax/ Ma min	Load Comb.	TL Defl	Load Comb.	LL Defl	Load Comb.
Span	265	0.0	138.0	712.0	0.386	1	L/576	1	L/4613 35	4

Joist Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Required
R1	92.1	1	1.00	273.6	478.8	0.18	1	NO
R2	92.1	1	1.00	273.6	478.8	0.18	1	NO

Joist Bending and Shear

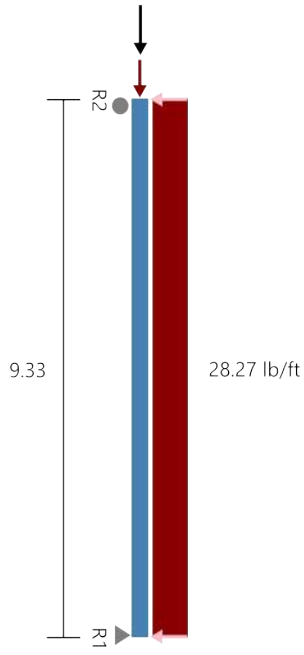
Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffened	Load Comb.	Intr. Stiffened	Load Comb.
R1	92.1	1	1.000	0.05	0.00	0.05	1	N/A	N/A
R2	92.1	1	1.000	0.05	0.00	0.05	1	N/A	N/A

Joist Reaction and Connections

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	92.1	SSC2.25 Min (3#10) & (2) #10 to Carrying (20/33) (Side	40.94 %	55.83 %

R2	0.0	92.1	SSC2.25 Min (3#10) & (2) #10 to Attached) Carrying (20/33) (Side Attached)	40.94 %	55.83 %

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



Section : 550S200-43 (33 ksi) @ 16" o.c. Single C Stud (punched)
Maxo = 1277.9 ft-lb **Va =** 1550.0 lb **I =** 2.19 in^4

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

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	None, None	None, 112.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	131.9	--Shear Connection w/ clip--				NO
R1	131.9	--Stud/Track Design, Ref Connectors--				NO

Gravity Load

Type	Load (lb)
Uniform	20.00plf
P1y	547lb @ 9.33ft

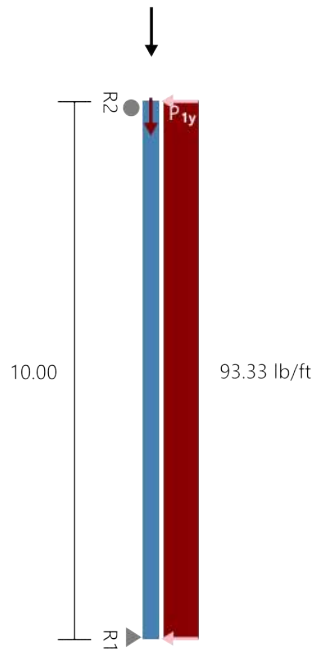
	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	733.6(c)	1977.0(c)	37%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	131.9	1198.9	11%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	307.6	1163.7	26%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	307.6	876.6	35%	
	Shear/Moment	0.24	1.00	24%	Shear 0.0, Moment 307.6
	Axial/Moment	0.68	1.00	68%	Axial 646.6(c), Moment 306.2
	Deflection Span, in	0.052	--meets L/2143--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	131.9	0.0	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	21.62 %	11.83 %
R1	131.9	733.6	550T125-33 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	31.77 %	59.88 %

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

Project Name: garage LG v2
Model: Post - 6" (2 studs box)
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/10/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.16



Section : (2) 550S200-54 (50 ksi) @ 16" o.c. Boxed C Stud (punched)
Maxo = 4497.4 ft-lb **Va =** 6186.9 lb **I =** 5.41 in^4

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

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	None, None	None, N/A	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	466.7	--Shear Connection w/ clip--				NO
R1	466.7	--Stud/Track Design, Ref Connectors--				NO

Gravity Load

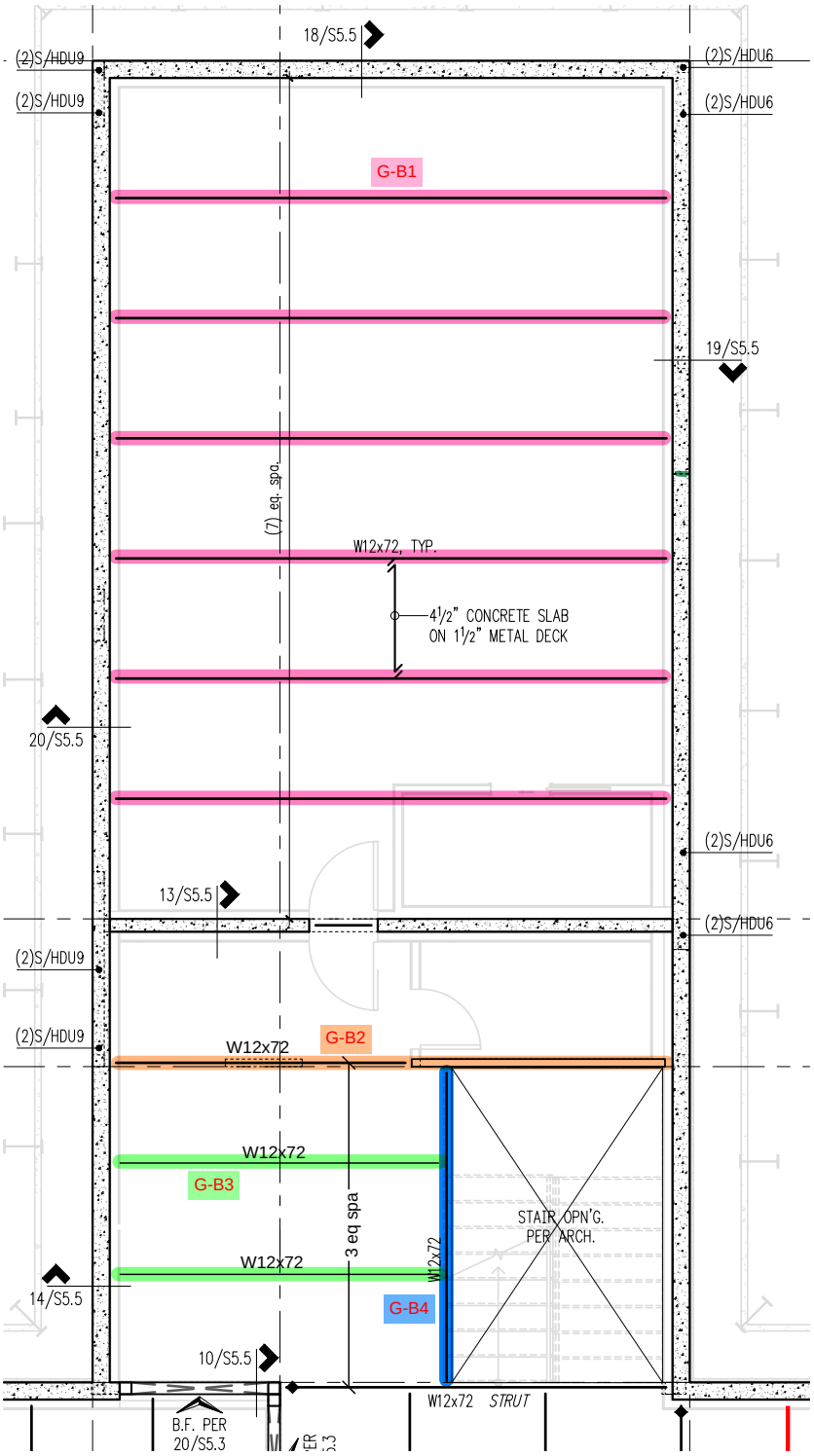
Type	Load (lb)
Uniform	6.67plf
P1y	7000lb @ 9.33ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	7066.7(c)	14232.8(c)	50%	KΦ=0.00 lb-in/in
	Max. Shear, lbs	466.7	3762	12%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	1166.7	4497.4	26%	
	Moment Stability, ft-lbs	1166.7	4497.4	26%	
	Shear/Moment	0.26	1.00	26%	Shear 0.0, Moment 1166.7
	Axial/Moment	0.79	1.00	79%	Axial 7033.5(c), Moment 1166.6
	Deflection Span, in	0.092	--meets L/1304--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	466.7	0.0	SCB45.5(3) & (2) #12-24 SST X or XL to A36 Steel	47.14 %	41.85 %
R1	466.7	7066.7	550T125-33 (33) & (3) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	67.83 %	70.64 %

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

Garage Floor Key Plan



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8480 Residence	10/10/2022
PROJECT	DATE
168	01519-2021-09
	PROJ. #
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Beam Analysis

Beam:		G-B1					
Load		Dead	Live	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.406**	**0.190**			0.596	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1					0.000	
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**22.00**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	6.559	**0.00**
R_2	6.559	**22.00**
R_3	0.000	**22.00**
R_4	0.000	**22.00**
R_5	0.000	**22.00**
R_6	0.000	**22.00**
R_7	0.000	**22.00**
R_8	0.000	**22.00**
R_9	0.000	**22.00**
R_{10}	0.000	**22.00**

Demand Output		
Location, ft		**10.00**
Shear, k	V	0.60
Moment, k-ft	M =	35.78
Deflection, in	D =	-0.27
Δ/Span		L/962

Load Factors	
Dead	**1.00**
Live	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

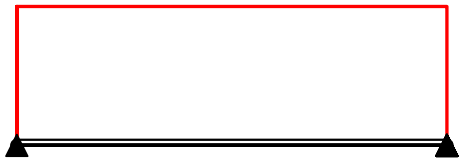
Stresses @ Input Location	
f_v (psi)	132
f_b (psi)	6687

Max/Min Stresses	
f_{v_MAX} (psi)	1455
f_{v_MIN} (psi)	-1455
f_{b_MAX} (psi)	6748
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	391
S (in^3)	64.2
A (in^2)	4.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		22.00	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	119 k-ft
I	391 in^4	$\phi_b M_n$	179 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	6.56	-6.56	-	36.1	-0.277 (↓)	11	L/953	-0.088 (↓)	11	L/3000

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PROJECT 8480 Residence

DATE *10/7/2022*
PROJ. # 01519-2021-09
DESIGN LAN
SHEET

Beam Analysis

Beam:		GB-2					
Load		Dead	Live	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.422**	**0.190**			0.612	**0.00**
	w_2	**0.433**	**0.155**			0.588	**4.58**
	w_3	**-0.433**	**-0.155**			-0.588	**7.58**
	w_4	**0.433**	**0.155**			0.588	**12.17**
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1	**5.326**	**3.854**			9.180	**4.58**
	P_2	**0.326**	**0.354**			0.680	**7.58**
	P_3	**0.326**	**0.354**			0.680	**12.17**
	P_4	**2.660**	**1.120**			3.780	**13.25**
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**22.00**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	18.817	**0.00**
R_2	16.504	**22.00**
R_3	0.000	**22.00**
R_4	0.000	**22.00**
R_5	0.000	**22.00**
R_6	0.000	**22.00**
R_7	0.000	**22.00**
R_8	0.000	**22.00**
R_9	0.000	**22.00**
R_{10}	0.000	**22.00**

Demand Output		
Location, ft		**6.00**
Shear, k	V	5.13
Moment, k-ft	M =	88.30
Deflection, in	D =	-0.40
Δ/Span		L/660

Load Factors	
Dead	**1.00**
Live	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

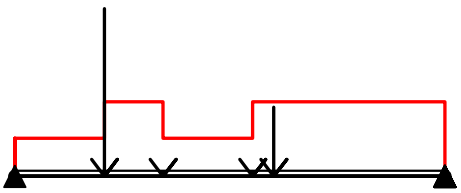
Stresses @ Input Location	
f_v (psi)	970
f_b (psi)	10878

Max/Min Stresses	
f_{v_MAX} (psi)	3554
f_{v_MIN} (psi)	-3119
f_{b_MAX} (psi)	12320
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	597
S (in^3)	97.4
A (in^2)	5.29
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x72	
F_y, ksi		50	
Beam Weight (plf)		72	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		22.00	
C_b		1	
A_w	5.29 in^2	V_n/Ω_v	106 k
S	97.4 in^3	$\phi_v V_n$	159 k
Z	108 in^3	M_n/Ω_b	228 k-ft
I	597 in^4	$\phi_b M_n$	342 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	18.8	-16.5	-	100	-0.516 (↓)	10.9	L/512	-0.173 (↓)	10.7	L/1526

Beam Analysis

Beam:		GB-3					
Load		Dead	Live	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.356**	**0.170**			0.526	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1					0.000	
	P_2					0.000	
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**13.17**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	3.463	**0.00**
R_2	3.463	**13.17**
R_3	0.000	**13.17**
R_4	0.000	**13.17**
R_5	0.000	**13.17**
R_6	0.000	**13.17**
R_7	0.000	**13.17**
R_8	0.000	**13.17**
R_9	0.000	**13.17**
R_{10}	0.000	**13.17**

Demand Output		
Location, ft		**10.00**
Shear, k	V	-1.80
Moment, k-ft	M =	8.33
Deflection, in	D =	-0.02
Δ/Span		L/7286

Load Factors	
Dead	**1.00**
Live	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

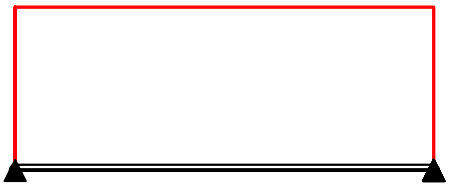
Stresses @ Input Location	
f_v (psi)	-398
f_b (psi)	1557

Max/Min Stresses	
f_{v_MAX} (psi)	767
f_{v_MIN} (psi)	-767
f_{b_MAX} (psi)	2131
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**8.75**
d (in)	**21**
I (in^4)	391
S (in^3)	64.2
A (in^2)	4.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		13.17	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	155 k-ft
I	391 in^4	$\phi_b M_n$	232 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	3.46	-3.46	-	11.4	-0.031 (↓)	6.6	L/5097	-0.01 (↓)	6.6	L/15800

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PROJECT	8480 Residence
DATE	*10/7/2022*
PROJ. #	01519-2021-09
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Beam Analysis

Beam:		GB-4					
Load		Dead	Live	Roof Live	Seismic	Factored	Location
Distributed (k/ft)	w_1	**0.050**				0.000	
	w_2					0.000	
	w_3					0.000	
	w_4					0.000	
	w_5					0.000	
	w_6					0.000	
	w_7					0.000	
	w_8					0.000	
	w_9					0.000	
	w_{10}					0.000	
Trapezoidal (k/ft/ft)	t_1					0.000	
	t_2					0.000	
	t_3					0.000	
	t_4					0.000	
	t_5					0.000	
	t_6					0.000	
Point (k)	P_1	**2.34**	**1.12**			1.120	**4.22**
	P_2	**2.34**	**1.12**			1.120	**8.44**
	P_3					0.000	
	P_4					0.000	
	P_5					0.000	
	P_6					0.000	
	P_7					0.000	
	P_8					0.000	
	P_9					0.000	
	P_{10}					0.000	

Support Locations and Reactions		
# of Supports		2
Total Beam Length		**12.67**
Left End Condition		**Pinned**
Right End Condition		**Pinned**
R_1	1.120	**0.00**
R_2	1.120	**12.67**
R_3	0.000	**12.67**
R_4	0.000	**12.67**
R_5	0.000	**12.67**
R_6	0.000	**12.67**
R_7	0.000	**12.67**
R_8	0.000	**12.67**
R_9	0.000	**12.67**
R_{10}	0.000	**12.67**

Demand Output		
Location, ft		**10.00**
Shear, k	V	-1.12
Moment, k-ft	M =	2.99
Deflection, in	D =	-0.01
Δ/Span		L/20067

Load Factors	
Dead	**0.00**
Live	**1.00**
Roof Live	**1.00**
Seismic	**1.00**

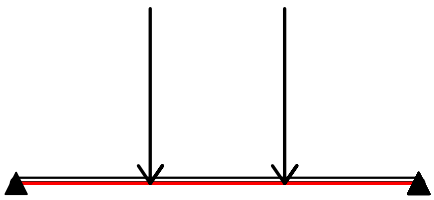
Stresses @ Input Location	
f_v (psi)	-248
f_b (psi)	558

Max/Min Stresses	
f_{v_MAX} (psi)	248
f_{v_MIN} (psi)	-248
f_{b_MAX} (psi)	884
f_{b_MIN} (psi)	0

Beam Properties	
E (ksi)	**29000**
b (in)	**3.5**
d (in)	**12**
I (in^4)	391
S (in^3)	64.2
A (in^2)	4.51
I (Override)	
S (Override)	
A (Override)	

Steel Beam Section		W12x50	
F_y, ksi		50	
Beam Weight (plf)		50	
Axis of Bending		Strong	
Unbr. Length (L_b), ft		12.67	
C_b		1	
A_w	4.51 in^2	V_n/Ω_v	90.3 k
S	64.2 in^3	$\phi_v V_n$	135 k
Z	71.9 in^3	M_n/Ω_b	157 k-ft
I	391 in^4	$\phi_b M_n$	235 k-ft

Beam Loading Diagram



Span	V_L (kips)	V_R (kips)	M(-) (k-ft)	M(+) (k-ft)	Δ_{TL} (in)	@ x =	L/	Δ_{LL} (in)	@ x =	L/
Span 1	1.12	-1.12	0	4.73	-0.041 (↓)	6.3	L/3707	-0.012 (↓)	6.3	L/12667

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PROJECT 8480 Residence

DATE *10/7/2022*
PROJ. # 01519-2021-09
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PLB™-36 FormLok® Composite Steel Deck-Slab (LRFD) - Distributed Load

with 6 in. 150 pcf 4000 psi NWC

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	5'-4"	6'-2"	6'-3"
20	6'-3"	7'-2"	7'-4"
18	7'-3"	8'-6"	8'-9"
16	7'-10"	9'-5"	9'-8"

6ft typ. spacing, OK

Maximum Unshored Span based on:

Uniform Construction Load	20.00	psf	Minimum End Bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum Interior Bearing	5.50	in.
Concrete Ponding Allowance	3.00	psf	Maximum Deflection L/	180	≤ 0.75 in.
Concrete Volume	1.55	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ $in.^3/ft$	Se- $in.^3/ft$	Id+ $in.^4/ft$	Id- $in.^4/ft$	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Garage Loading:
DL - 75 psf
LL - 40 psf
Factored - 1.2(75)+1.6(40) = 154 psf, OK

Superimposed Design Load, ϕWn, / Deflection at L/360 ,psf [1]

Gage	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	3265/1036	2062/5305	1408/3070	1014/1933	758/1295	582/909	457/663	364/498	293/383
20	3623/1099	2448/5630	1676/3258	1210/2051	908/1374	701/965	553/703	443/528	360/407
18	3622/1215	2882/6223	2187/3601	1586/2267	1195/1519	928/1067	736/777	595/584	487/450
16	3621/1321	2881/6764	2387/3914	1946/2465	1471/1651	1145/1159	912/845	740/635	609/489

Notes: [1] For high loads, commonly in excess of 325 psf, dynamic or impact loading, and long term concrete creep should be considered. Contact Verco for further assistance.

Composite Steel Deck-Slab Properties

Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft
22	64.8	8.46	21.89	15.18	6.69	7.36
20	65.2	9.72	22.48	16.10	7.89	7.40
18	65.8	12.00	23.60	17.80	10.20	7.40
16	66.4	14.04	24.66	19.35	12.41	7.40

Min. Temperature & Shrinkage

Gage	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	0.041	15
20	0.041	15
18	0.041	15
16	0.041	15

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Tables generated using calculator V3.3 based on ANSI/SDI C-2017 in accordance with 2018 IBC Section 2210.

Date: 9/23/2022

PLB-36 (20GA) w/ 4.5" topping - Concentrated Load

look at a 3000 lb concentrated load over area of 4.5"x4.5" for the garage slab

clear span - 6 ft max
slab total thickness - 6 in
w slab = 65.6 psf
w deck = 2.9 psf
w superimposed = 6.5 psf

composite properties -
ϕMno = 7.89 kip-ft
ϕVno = 7.40 kip

b2 = b3 = 4.5 in
t_t = 0 in
t_c = 4.5 in
h = 6 in
bm = b2 + 2t_c + 2t_t = 4.5+(2*4.5)+(2*0) = 13.5 in

MOMENT CHECK
put the pt load in the center of the span
x = L/2 = 6/2 = 3 ft = 36 in
be = bm+2(1-(x/L))x = 13.5 + 2(1-36/72)36 = 49.5 in
be < 8.9(tc/h) = 8.9(4.5/6)(12) = 80.1 in
be = 49.5 in

M concentrated = PL/4 per foot of width
P = 1.6*3000 = 4800 lbs
M = (4800*6/4)*(12/49.5) = 1746 lb-ft = 1.75 kip-ft
M distributed = wL^2/8
w=1.2*(65.6+2.9+6.5) = 90 psf
M = 90*(6^2)/8 = 405 lb-ft = 0.41 kip-ft
total M = 1.75 + 0.41 = 2.16 kip-ft < ϕMno = 7.89 kip-ft **OK**

SHEAR CHECK
put the pt load one slab depth away from the support
x = h = 6 in
bve = bm+(1-x/l)x = 13.5 + (1-6/72)6 = 19 in

V concentrated = 4800(12/19) = 3032 lbs
V distributed = 90(6/2) = 270 lbs
total V = 3032 + 270 = 3302 lbs = 3.30 kips < ϕVno = 7.40 kip **OK**

WEAK AXIS CHECK
M weak axis = Pbe/16W
W = L/2+b3 = 36+4.5 = 40.5 in < L=72 in
M = (4800*49.5*12)/(15*40.5) = 4693 lb-in = 4.69 kip-in

Use #4 @ 12"oc --> As = 0.2 in^2
#4 located at mid height of the 4.5" topping --> d=6-1.5-(4.5/2) = 2.25 in
a = (0.2*60)/(0.85*12*4) = 0.294 in
ϕ = 0.85
ϕMn = 0.85*0.2*60000*(2.25-(0.294/2)) = 21451 lb-in = 21.45 kip-in > 4.69 kip-in **OK**

DEFLECTION CHECK
defl. = PL^3/48EI
P = 3000(12/49.5) = 728 lbs = 0.728 kips
L = 6 ft = 72 in
E = 29000 ksi
Ic = 9.72 in^4/ft
defl. = (0.728*(72^3))/(48*29000*9.72) = 0.02 in

L/360 = 72/360 = 0.2 in > 0.02 in **OK**

Misc Gravity

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CENTRAL WA 414 N Pearl Street, Suite 8, Ellensburg, WA 98926
206.443.6212
ssfengineers.com

Base Plate Design - 7x5x3/8 HEAVY

DL = 59.9 kips
LL = 19.1 kips
SL = 10.7 kips
Pu = 108.1 kips

$$A_{1(req)} = \frac{P_u}{\phi 0.85 f_c'}$$ ← f'c = 4000 psi

A1 req = 48.9 in^2

Column = HSS7x5x3/8
N = column width + 1 = 5+1 = 6"
B = column width + 6 = 7+6 = 13"
A1 = 78 in^2 > 48.9 in^2 (OK)

A1=A2

$$P_u \le \phi P_p = \phi 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$$

ϕPp = 0.65*0.85*4*78*sqrt(78/78)
= 172.4 kips > 108.1 kips (OK)

$$m = \frac{N - 0.95d}{2}$$ ← d = 5 in

m = 0.625

$$n = \frac{B - 0.8b_f}{2}$$ ← bf = 7 in

n = 3.70

$$X = \left\{ \frac{4db_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi P_p}$$

X = 0.610

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1.0$$

λ = 0.962 < 1.0 so use 0.96

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$

λn' = 1.43

l = max(m,n,λn') = 3.70 in

$$t_{min} = l \sqrt{\frac{2P_u}{\phi F_y BN}}$$

tmin = 0.986 in --> use 1.00 in

TYPICAL HSS7x5x3/8 LOADS ARE CONSISTENT WITH TYPICAL HSS5x5x1/2 SO USE BASE PLATE FOR TYPICAL HSS5x5x1/2 FOR ALL TYPICAL COLUMNS

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Base Plate Design - HSS 5x5x1/2 HEAVY

DL = 14.9 kips
LL = 17.5 kips
SL = 0.0 kips
Pu = 45.9 kips

$$A_{1(req)} = \frac{P_u}{\phi 0.85 f_c'}$$ ← f'c = 4000 psi

A1 req = 20.8 in^2

Column = HSS5x5x1/2
N = column width + 1 = 5+1 = 6"
B = column width + 6 = 5+6 = 11"
A1 = 66 in^2 > 20.8 in^2 (OK)

A1=A2

$$P_u \le \phi P_p = \phi 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$$

ϕPp = 0.65*0.85*4*66*sqrt(66/66)
= 145.9 kips > 45.9 kips (OK)

$$m = \frac{N - 0.95d}{2}$$ ← d = 5 in

m = 0.625

$$n = \frac{B - 0.8b_f}{2}$$ ← bf = 5 in

n = 3.50

$$X = \left\{ \frac{4db_f}{(d + b_f)^2} \right\} \frac{P_u}{\phi P_p}$$

X = 0.315

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1.0$$

λ = 0.614 < 1.0 so use 0.614

$$\lambda n' = \lambda \frac{\sqrt{db_f}}{4}$$

λn' = 0.768

l = max(m,n,λn') = 3.70 in

$$t_{min} = l \sqrt{\frac{2P_u}{\phi F_y BN}}$$

tmin = 0.578 in --> use 0.625 in

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Base Plate Design - HSS 5x5x1/2 TYPICAL

Column = HSS5x5x1/2
B = column width + 1 = 6"
N = column width + 6 = 11"

$$m = \frac{N - 0.95d}{2}$$ ← d = 5 in

m = 3.125

$$n = \frac{B - 0.8b_f}{2}$$ ← bf = 5 in

n = 1.00

l = max(m,n) = 3.125 in
*assume λn' doesn't govern

$$t_{min} = l\sqrt{\frac{2P_u}{\phi F_y BN}}$$

tmin = 0.5in --> Pu = 32.94 k

Verify 1/2" base plate works for Pu max of 32.90 kips

$$A_{1(req)} = \frac{P_u}{\phi 0.85 f_c'}$$

A1 req = 14.9 in^2

Column = HSS5x5x1/2
B = column width + 1 = 6"
N = column width + 6 = 11"
A1 = 66 in^2 > 14.9in^2 (OK)

A1=A2

$$P_u \le \phi P_p = \phi 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$$

ϕPp = 0.65*0.85*4*66*sqrt(66/66)
= 145.9 kips > 32.9 kips (OK)

$$m = \frac{N - 0.95d}{2}$$ ← d = 5 in

m = 3.125

$$n = \frac{B - 0.8b_f}{2}$$ ← bf = 5 in

n = 1.00

$$X = \left\{\frac{4db_f}{(d + b_f)^2}\right\}\frac{P_u}{\phi P_p}$$

X = 0.226

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1.0$$

λ = 0.51 < 1.0 (OK)

$$\lambda n' = \lambda\frac{\sqrt{db_f}}{4}$$

λn' = 0.64

l = max(m,n,λn') = 3.125 in

$$t_{min} = l\sqrt{\frac{2P_u}{\phi F_y BN}}$$

tmin = 0.50 in (OK)

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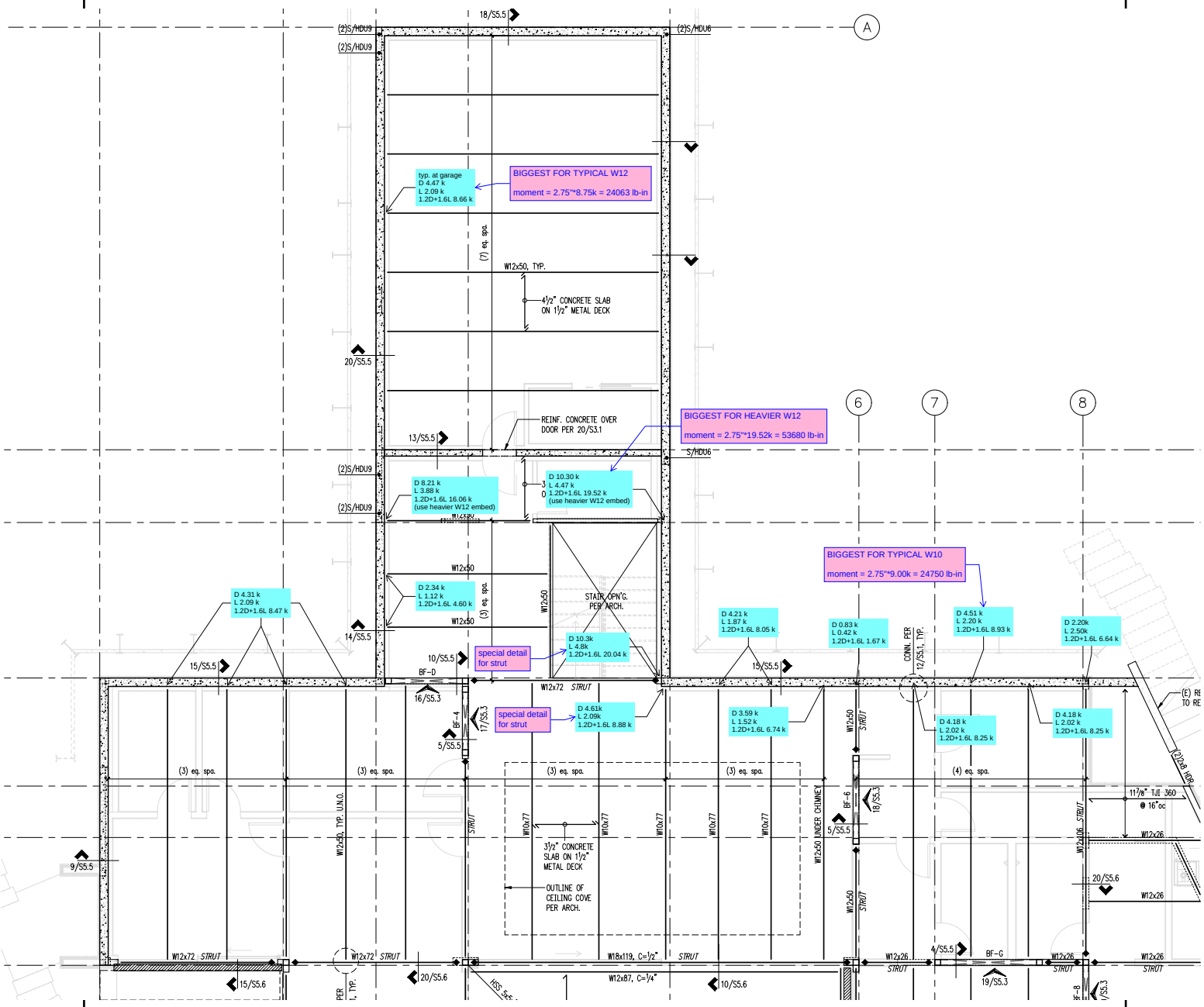
Gravity Key Plan - Embed Loads



STRUCTURAL ENGINEERING

8480 Residence	10/07/2022
PROJECT	DATE
180	01519-2021-09
	PROJ. #
	LAN
	DESIGN
	SHEET

Company: SSF - 8480 Residence (01519-2021-09)
Address:
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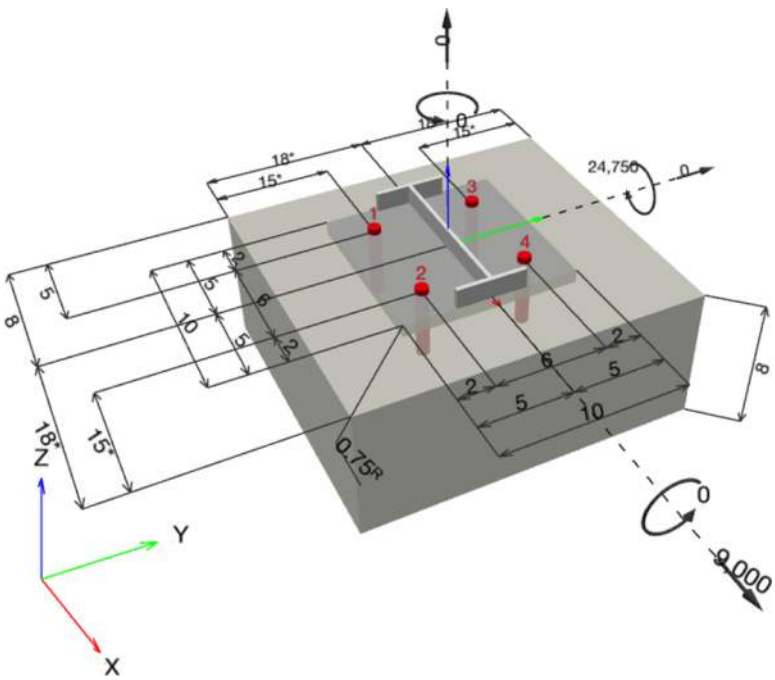
Specifier's comments:

1 Input data

Anchor type and diameter:	**AWS D1.1 GR. B 3/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 3.974 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.750 in.
Anchor plate[R]:	l_x x l_y x t = 10.000 in. x 10.000 in. x 0.750 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Reinforcement:	tension: not present, shear: not present;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

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



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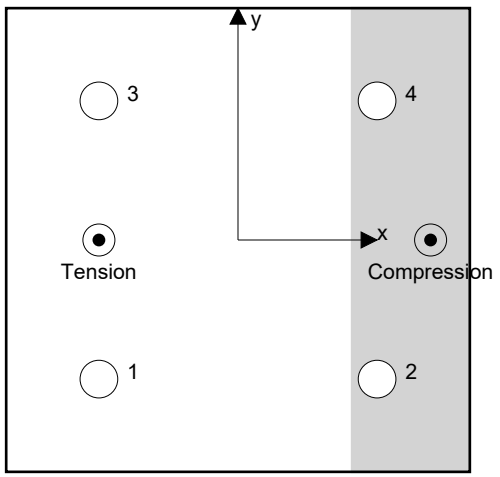
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 9,000; V_y = 0; M_x = 0; M_y = 24,750; M_z = 0;	no	86

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,730	2,250	2,250	0
2	0	2,250	2,250	0
3	1,730	2,250	2,250	0
4	0	2,250	2,250	0

max. concrete compressive strain: 0.06 [‰]
max. concrete compressive stress: 273 [psi]
resulting tension force in (x/y)=(-3.000/0.000): 3,460 [lb]
resulting compression force in (x/y)=(4.154/0.000): 3,460 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	1,730	21,547	9	OK
Pullout Strength*	1,730	17,584	10	OK
Concrete Breakout Failure**	3,460	11,071	32	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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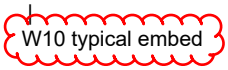

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3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

N_{sa} [lb]
28,730

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
28,730	0.750	21,547	1,730

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.79	1.000	4,000

Calculations

N_p [lb]
25,120

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
25,120	0.700	17,584	1,730

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3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.974	0.000	0.000	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
196.44	142.13	1.000	1.000	0.952	1.000	12,025

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
15,816	0.700	11,071	3,460

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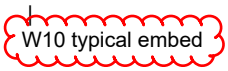

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2,250	18,674	13	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	9,000	34,263	27	OK
Concrete edge failure in direction x+**	9,000	11,042	82	OK

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

4.1 Steel Strength

$V_{sa} = A_{se,V} f_{uta}$ ACI 318-19 Eq. (17.7.1.2a)
$\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

V_{sa} [lb]
28,730

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
28,730	0.650	18,674	2,250

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4.2 Pryout Strength

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

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

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.974	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
303.98	142.13	1.000	1.000	0.952	1.000	12,025

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
48,947	0.700	34,263	9,000

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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

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

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
10.000	15.000	0.000	1.000	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
3.974	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
288.00	450.00	1.000	1.000	1.369	18,000

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
15,774	0.700	11,042	9,000

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.312	0.815	5/3	86	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

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

Fastening meets the design criteria!

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7 Installation data

Profile: W shape (AISC), W10X15; (L x W x T x FT) = 9.990 in. x 4.000 in. x 0.230 in. x 0.270 in.

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 0.750 in.

Recommended plate thickness: not calculated

Anchor type and diameter: AWS D1.1 GR. B 3/4

Item number: not available

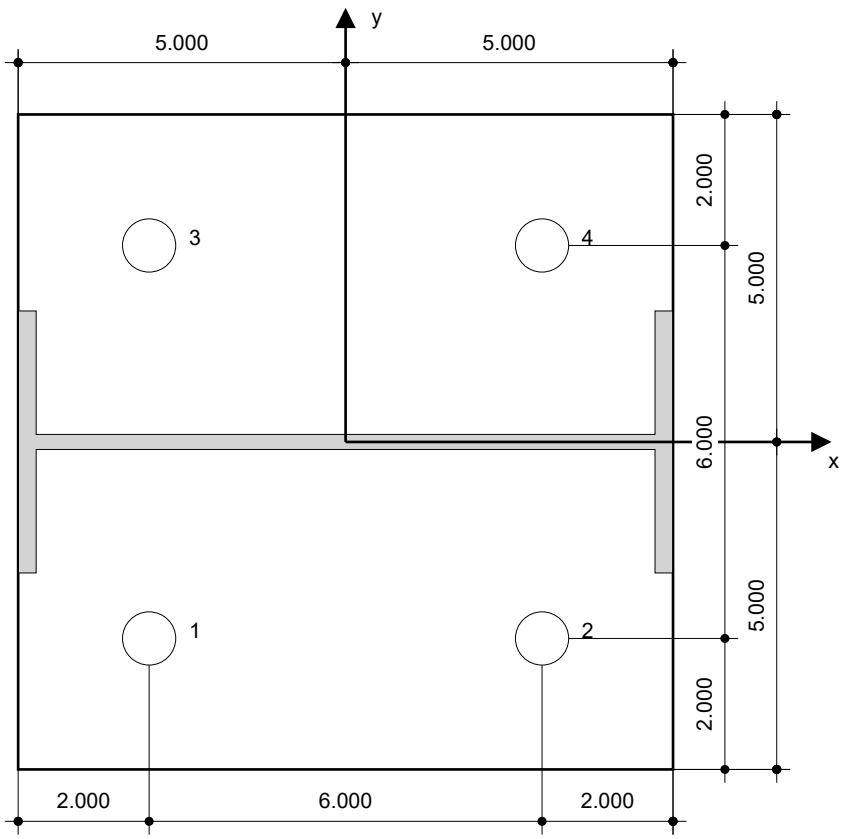
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 3.974 in.

Minimum thickness of the base material: 4.849 in.

Hilti AWS welded headed stud anchor with 3.974 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-3.000	5.000	21.000	15.000	21.000
2	3.000	-3.000	11.000	15.000	15.000	21.000
3	-3.000	3.000	5.000	21.000	21.000	15.000
4	3.000	3.000	11.000	15.000	21.000	15.000

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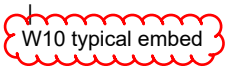

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8 Remarks; Your Cooperation Duties

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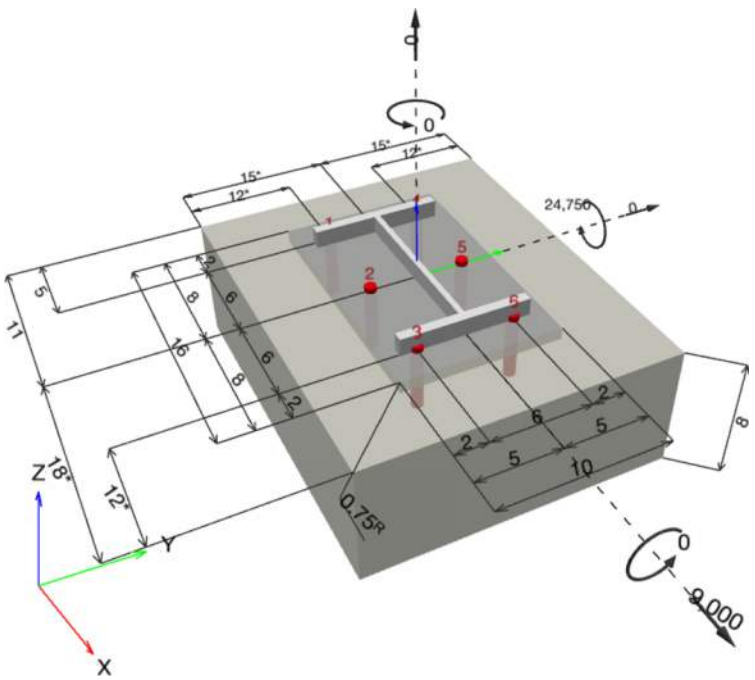
Specifier's comments:

1 Input data

Anchor type and diameter:	**AWS D1.1 GR. B 3/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 3.974 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.750 in.
Anchor plate[R]:	l_x x l_y x t = 16.000 in. x 10.000 in. x 0.750 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W12X50; (L x W x T x FT) = 12.200 in. x 8.080 in. x 0.370 in. x 0.640 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

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



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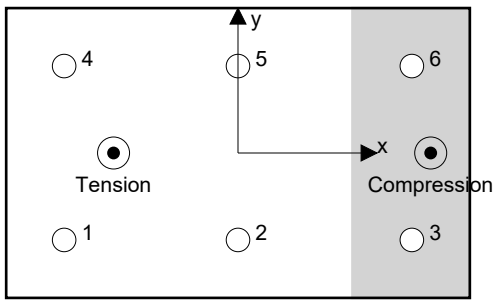
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 9,000; V_y = 0; M_x = 0; M_y = 24,750; M_z = 0;	no	98

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	809	1,500	1,500	0
2	321	1,500	1,500	0
3	0	1,500	1,500	0
4	809	1,500	1,500	0
5	321	1,500	1,500	0
6	0	1,500	1,500	0



max. concrete compressive strain: 0.03 [‰]
max. concrete compressive stress: 112 [psi]
resulting tension force in (x/y)=(-4.295/0.000): 2,261 [lb]
resulting compression force in (x/y)=(6.651/0.000): 2,261 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	809	21,547	4	OK
Pullout Strength*	809	17,584	5	OK
Concrete Breakout Failure**	2,261	14,075	17	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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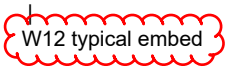

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3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

N_{sa} [lb]
28,730

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
28,730	0.750	21,547	809

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.79	1.000	4,000

Calculations

N_p [lb]
25,120

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
25,120	0.700	17,584	809

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3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.974	1.295	0.000	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
303.98	142.13	0.822	1.000	0.952	1.000	12,025

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
20,107	0.700	14,075	2,261

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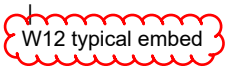

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,500	18,674	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	9,000	46,383	20	OK
Concrete edge failure in direction x+**	9,000	9,202	98	OK

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

4.1 Steel Strength

$V_{sa} = A_{se,V} f_{uta}$ ACI 318-19 Eq. (17.7.1.2a)
$\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

V_{sa} [lb]
28,730

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
28,730	0.650	18,674	1,500

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4.2 Pryout Strength

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

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

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.974	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
411.51	142.13	1.000	1.000	0.952	1.000	12,025

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
66,262	0.700	46,383	9,000

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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

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

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
8.000	12.000	0.000	1.000	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
3.974	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
240.00	288.00	1.000	1.000	1.225	12,880

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
13,145	0.700	9,202	9,000

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.161	0.978	1.000	95	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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6 Warnings

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

Fastening meets the design criteria!

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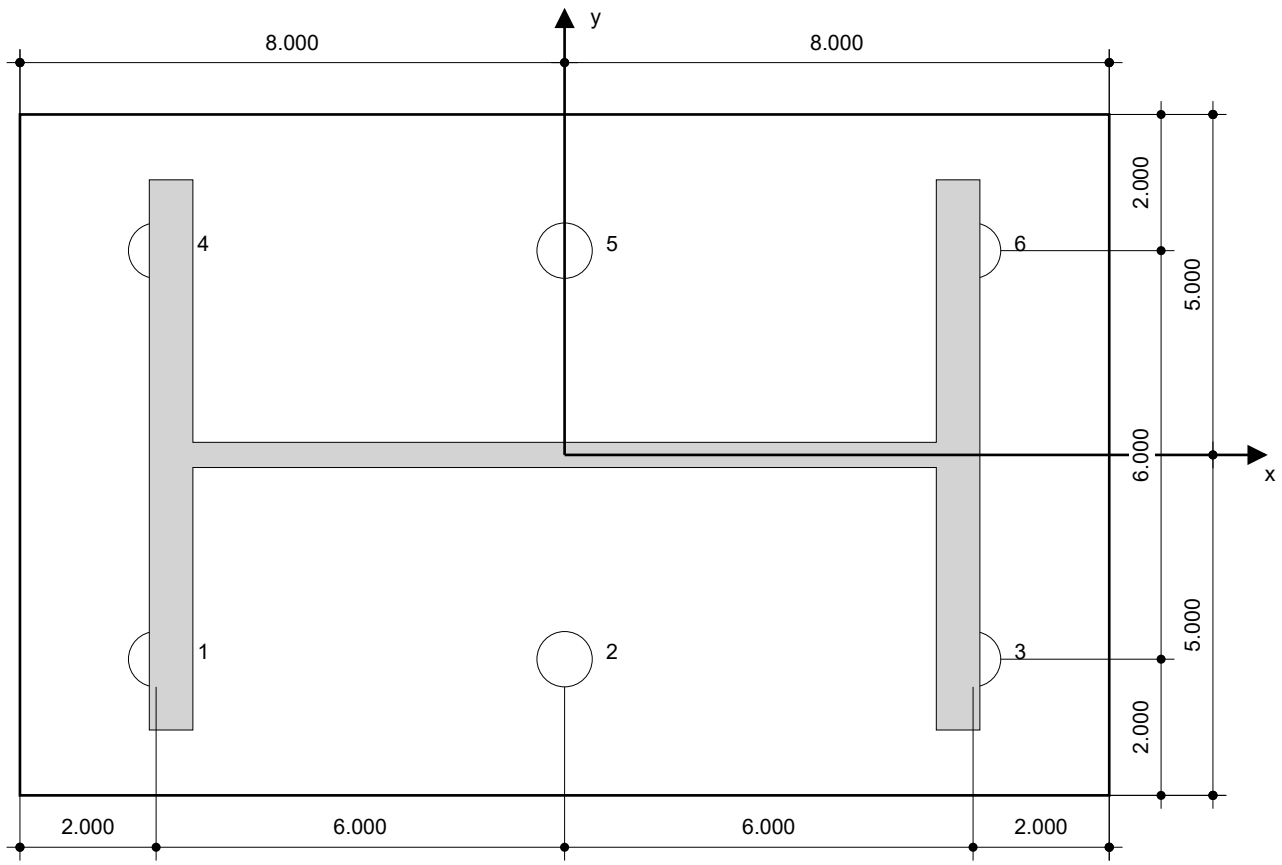
7 Installation data

Profile: W shape (AISC), W12X50; (L x W x T x FT) = 12.200 in. x 8.080 in. x 0.370 in. x 0.640 in.
Hole diameter in the fixture: d_f = 0.812 in.
Plate thickness (input): 0.750 in.
Recommended plate thickness: not calculated

Anchor type and diameter: AWS D1.1 GR. B 3/4
Item number: not available

Maximum installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 3.974 in.
Minimum thickness of the base material: 4.849 in.

Hilti AWS welded headed stud anchor with 3.974 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.000	-3.000	5.000	24.000	12.000	18.000
2	0.000	-3.000	11.000	18.000	12.000	18.000
3	6.000	-3.000	17.000	12.000	12.000	18.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	-6.000	3.000	5.000	24.000	18.000	12.000
5	0.000	3.000	11.000	18.000	18.000	12.000
6	6.000	3.000	17.000	12.000	18.000	12.000

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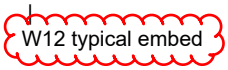

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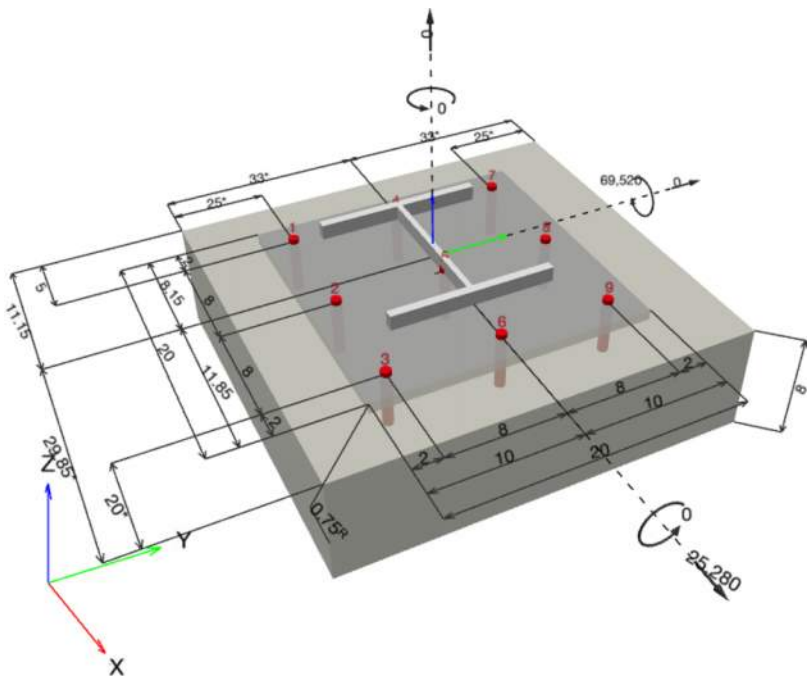
Specifier's comments:

1 Input data

Anchor type and diameter:	**AWS D1.1 GR. B 3/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 3.974 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.750 in.
Anchor plate[R]:	l_x x l_y x t = 20.000 in. x 20.000 in. x 0.750 in.; (Recommended plate thickness: not calculated)
Profile:	W shape (AISC), W12X72; (L x W x T x FT) = 12.300 in. x 12.000 in. x 0.430 in. x 0.670 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 8.000 in.
Reinforcement:	tension: not present, shear: present; edge reinforcement: > No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

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



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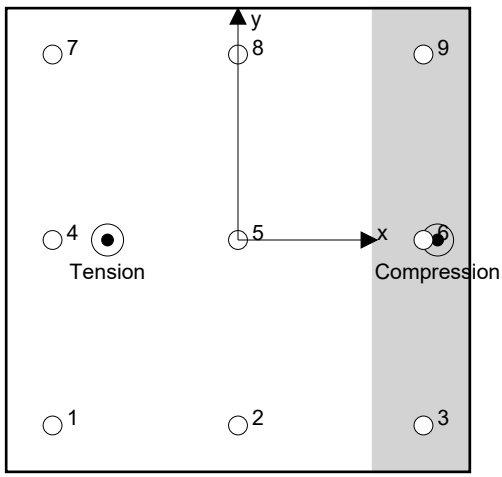
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 0; V_x = 25,280; V_y = 0; M_x = 0; M_y = 69,520; M_z = 0;	no	99

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,145	2,809	2,809	0
2	483	2,809	2,809	0
3	0	2,809	2,809	0
4	1,145	2,809	2,809	0
5	483	2,809	2,809	0
6	0	2,809	2,809	0
7	1,145	2,809	2,809	0
8	483	2,809	2,809	0
9	0	2,809	2,809	0

max. concrete compressive strain: 0.03 [‰]
max. concrete compressive stress: 117 [psi]
resulting tension force in (x/y)=(-5.627/0.000): 4,883 [lb]
resulting compression force in (x/y)=(8.610/0.000): 4,883 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	1,145	21,547	6	OK
Pullout Strength*	1,145	17,584	7	OK
Concrete Breakout Failure**	4,883	23,438	21	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

N_{sa} [lb]
28,730

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
28,730	0.750	21,547	1,145

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.79	1.000	4,000

Calculations

N_p [lb]
25,120

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
25,120	0.700	17,584	1,145

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3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
3.974	1.627	0.000	5.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
529.43	142.13	0.786	1.000	0.952	1.000	12,025

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
33,483	0.700	23,438	4,883

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	2,809	18,674	16	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	25,280	84,853	30	OK
Concrete edge failure in direction x+**	25,280	26,028	98	OK

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

4.1 Steel Strength

$V_{sa} = A_{se,V} f_{uta}$ ACI 318-19 Eq. (17.7.1.2a)
$\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.44	65,000

Calculations

V_{sa} [lb]
28,730

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
28,730	0.650	18,674	2,809

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4.2 Pryout Strength

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

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

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

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

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

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

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

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

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	3.974	0.000	0.000	5.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
752.81	142.13	1.000	1.000	0.952	1.000	12,025

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
121,218	0.700	84,853	25,280

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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

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

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
16.667	25.000	0.000	1.200	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
3.974	1.000	0.750	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
528.00	1,250.00	1.000	1.000	1.768	38,730

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
34,704	0.750	26,028	25,280

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.208	0.971	1.000	99	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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6 Warnings

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

Fastening meets the design criteria!

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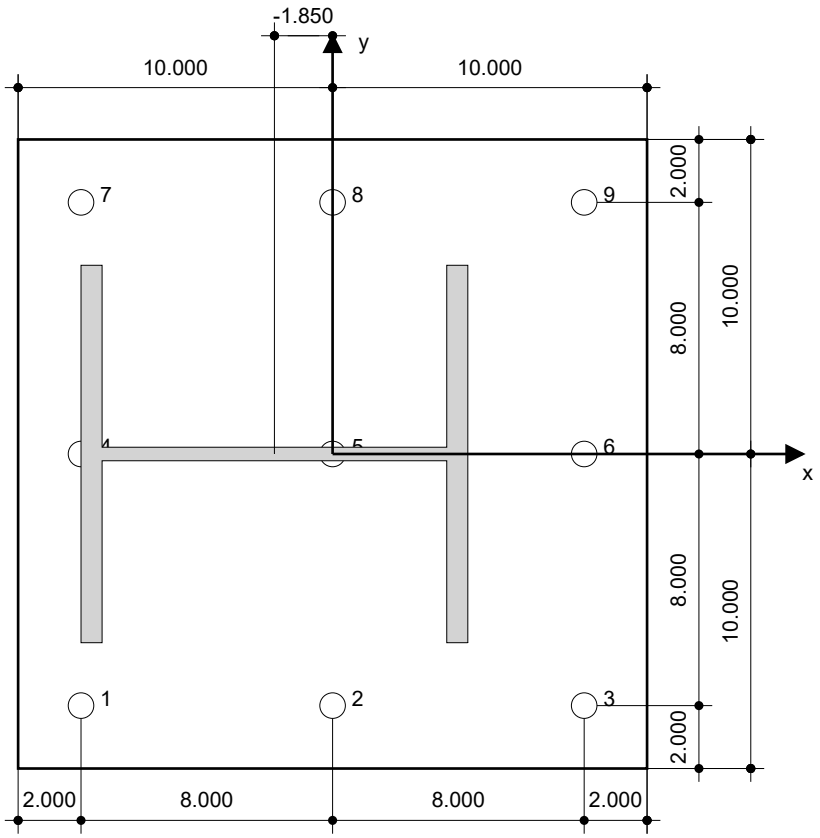
7 Installation data

Profile: W shape (AISC), W12X72; (L x W x T x FT) = 12.300 in. x 12.000 in. x 0.430 in. x 0.670 in.
Hole diameter in the fixture: d_f = 0.812 in.
Plate thickness (input): 0.750 in.
Recommended plate thickness: not calculated

Anchor type and diameter: AWS D1.1 GR. B 3/4
Item number: not available

Maximum installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 3.974 in.
Minimum thickness of the base material: 4.849 in.

Hilti AWS welded headed stud anchor with 3.974 in embedment, 3/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-8.000	-8.000	5.000	36.000	25.000	41.000
2	-0.000	-8.000	13.000	28.000	25.000	41.000
3	8.000	-8.000	21.000	20.000	25.000	41.000
4	-8.000	0.000	5.000	36.000	33.000	33.000
5	-0.000	0.000	13.000	28.000	33.000	33.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
6	8.000	0.000	21.000	20.000	33.000	33.000
7	-8.000	8.000	5.000	36.000	41.000	25.000
8	-0.000	8.000	13.000	28.000	41.000	25.000
9	8.000	8.000	21.000	20.000	41.000	25.000

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8 Remarks; Your Cooperation Duties

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Foundation Design

STRUCTURAL ENGINEERING

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TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402
CENTRAL WA 414 N Pearl Street, Suite 8, Ellensburg, WA 98926

206.443.6212
ssfengineers.com

GARAGE Basement Walls - North and East (typ walls)

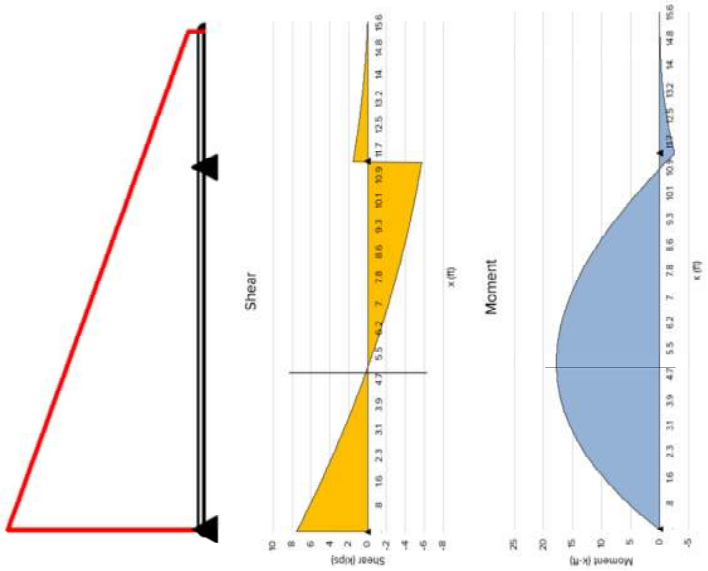
Soil info:

backspan H =	11.33	ft
cantilever H =	4.25	ft
total H =	15.58	ft
f'c =	4000	psi
fy =	60	ksi
active pressure =	55	pcf
restrain pressure =	10H	psf
seismic surcharge =	9H	psf

look at load over 1'-0" width

1.6H + 1.0E + 1.0L

factored active =	1621	plf (max)
factored sur =	140	plf



Vu max =	7.52	kips
Mu pos max =	17.80	kip-ft
Mu neg max =	2.60	kip-ft

Wall Info:

Wall t =	8.00	in
Cover =	1.50	in (outer face rebar)
	1.00	in (inner face rebar)
Rebar db =	0.50	in (#4 bar, outer face rebar)
	0.88	in (#7 bar, inner face rebar)
d =	6.25	in (outer face rebar)
	6.56	in (inner face rebar)
bw =	12.00	in

Gravity Loading:

roof DL =	23	psf
roof SL =	25	psf
roof trib =	9.1	ft
floor DL =	110	psf
floor LL =	40	psf
floor trib =	3.0	ft
wall above DL =	15	psf
wall above H =	5.5	ft
brick wall DL =	122	psf
brick wall H =	4.8	ft
wall DL =	100	psf
wall H =	11.3	ft
total DL =	2336	plf
total SL =	229	plf
total LL =	120	plf
factored TL =	2968	plf (1.2D+1.0L+0.2S)
trib =	1.0	ft
Nu =	2968	lbs
	2.97	kips
Ag =	96	in^2

Out of Plane Shear Check:

ϕVc =	7.59	kips	SHEAR OK

Min Reinforcement:

In plane Vu < 0.5ϕVn

vertical reinf

max spacing =	18.0	in
ρL =	0.0012	for #5 for less bars
min reqd =	0.17	in^2 per foot
		so #4 @ 18" works (double curtain)

horizontal reinf

max spacing =	18.0	in	
ρL =	0.0020	for #5 for less bars	
min reqd =	0.22	in^2 per foot	
		so #4 @ 18" works (double curtain)	

Flexural Check (positive moment):

Reinf use =	#7 @ 9"oc		
spacing =	9.0	in	
As =	0.60	in^2 (#6 bar)	
As/ft =	0.80	in^2	
bw =	12.0	in	
db =	6.56	in	
a =	1.18	in	
ϕMn =	258.1	kip-in	
	21.51	kip-ft	FLEXURE OK

Flexural Check (negative moment):

Reinf use =	#4 @ 18"oc		
spacing =	18.0	in	
As =	0.20	in^2 (#6 bar)	
As/ft =	0.13	in^2	
bw =	12.0	in	
db =	6.25	in	
a =	0.20	in	
ϕMn =	44.3	kip-in	
	3.69	kip-ft	FLEXURE OK

Axial Check:

ecc =	0.63	in	
Use Simplified eq?	YES		
Lc =	187.0	in	
h =	8.0	in	
k =	0.8		
bw =	12.0	in	
Ag =	96	in^2	
ϕPn =	90	kips	AXIAL OK

GARAGE Basement Walls - North and West at Driveway

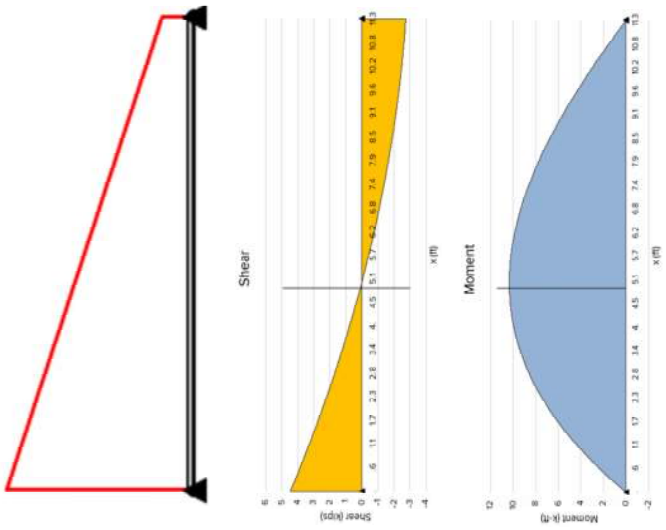
Soil info:

backspan H =	11.33	ft
cantilever H =	0.00	ft
total H =	11.33	ft
f'c =	4000	psi
fy =	60	ksi
active pressure =	40	pcf
restrain pressure =	10H	psf
seismic surcharge =	9H	psf
traffic surcharge =	2*40	psf

look at load over 1'-0" width

1.6H + 1.0E + 1.0L

factored active =	907	plf (max)
factored seis sur =	102	plf
factored traffic sur =	80	plf
factored tot sur =	182	plf



Vu max =	4.46	kips
Mu pos max =	10.30	kip-ft

Wall Info:

Wall t =	8.00	in
Cover =	1.50	in (outer face rebar)
	1.00	in (inner face rebar)
Rebar db =	0.50	in (#4 bar, outer face rebar)
	0.88	in (#7 bar, inner face rebar)
d =	6.25	in (outer face rebar)
	6.56	in (inner face rebar)
bw =	12.00	in

Gravity Loading:

roof DL =	23	psf
roof SL =	25	psf
roof trib =	9.1	ft
floor DL =	76	psf
floor LL =	40	psf
floor trib =	1.5	ft
wall above DL =	15	psf
wall above H =	10.3	ft
wall DL =	100	psf
wall H =	11.3	ft
total DL =	1611	plf
total SL =	229	plf
total LL =	60	plf
factored TL =	2039	plf (1.2D+1.0L+0.2S)
trib =	1.0	ft
Nu =	2039	lbs
	2.04	kips
Ag =	96	in^2

Out of Plane Shear Check:

ϕVc =	7.55	kips	SHEAR OK

Min Reinforcement:

In plane Vu < 0.5ϕVn

vertical reinf

max spacing =	18.0	in
ρL =	0.0012	for #5 for less bars
min reqd =	0.17	in^2 per foot
		so #4 @ 18" works (double curtain)

horizontal reinf

max spacing =	18.0	in	
ρL =	0.0020	for #5 for less bars	
min reqd =	0.22	in^2 per foot	
		so #4 @ 18" works (double curtain)	

Flexural Check (positive moment):

Reinf use =	#7 @ 9"oc		
spacing =	9.0	in	
As =	0.60	in^2 (#6 bar)	
As/ft =	0.80	in^2	
bw =	12.0	in	
db =	6.56	in	
a =	1.18	in	
ϕMn =	258.1	kip-in	
	21.51	kip-ft	FLEXURE OK

Axial Check:

ecc =	0.63	in	
Use Simplified eq?	YES		
Lc =	136.0	in	
h =	8.0	in	
k =	0.8		
bw =	12.0	in	
Ag =	96	in^2	
ϕPn =	112	kips	AXIAL OK

Slab on Grade Design

SOG Info

h =	6.0	in
b =	12.0	in
bar spacing =	12.0	in
size of bar =	4	
area of bar =	0.2	in^2
bar dia =	0.5	in
cover =	2.0	in
d =	3.8	in
fy =	60.0	ksi
f'c =	4.0	ksi

Loads

SW =	75	psf
DL =	10	psf
tot DL =	85	psf
LL =	40	psf, house
	60	psf, patio

Min Reinf

As min =	0.13	in^2 per ft
		so #4 @ 12"oc is ok

Max Span

a =	0.29	in
ϕMn =	28.1	kip-in
	2.3	kip-ft
ϕVn =	4.3	kips

at interior

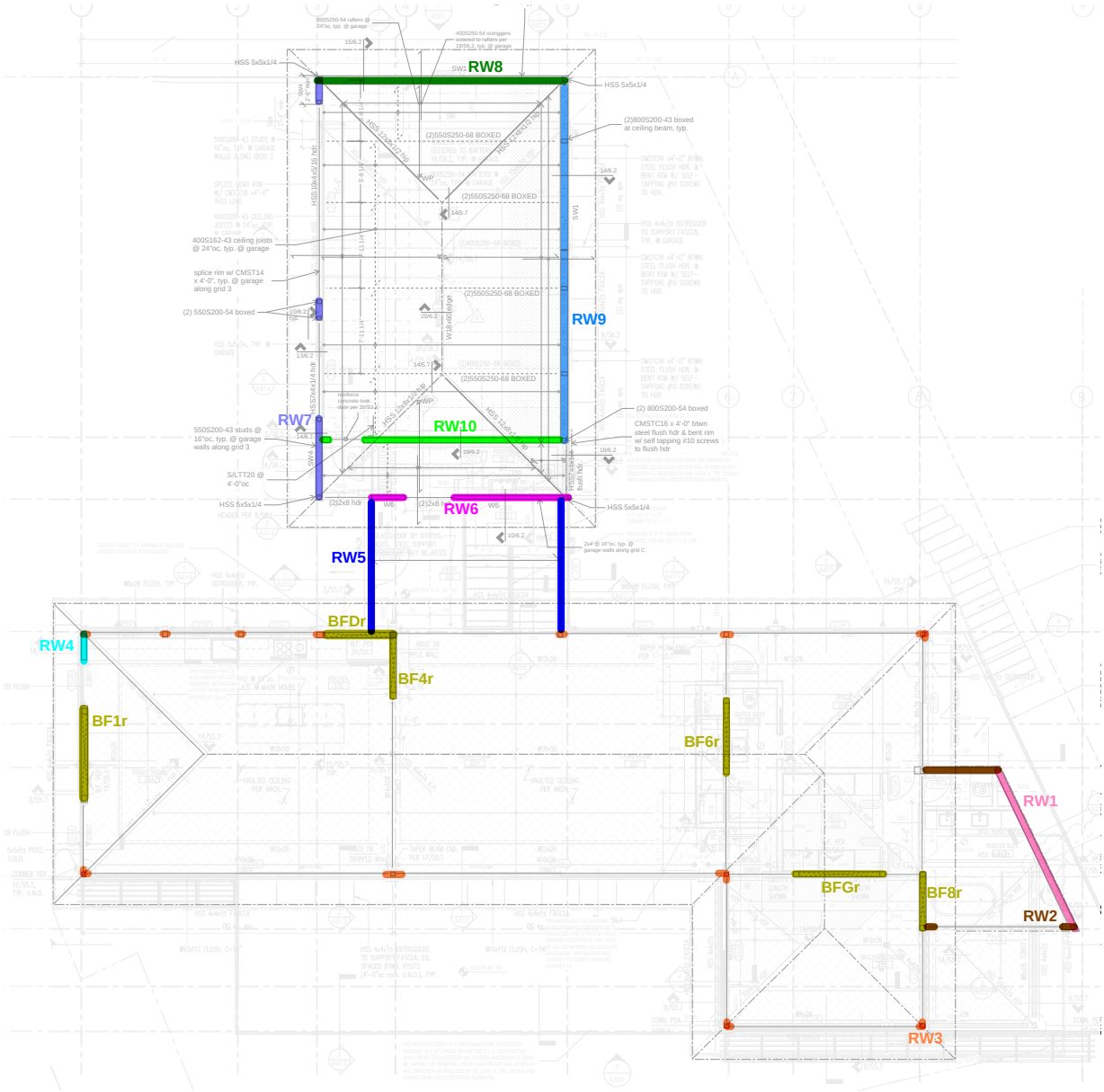
TL =	0.166	klf
L from M =	10.6	ft
L from V =	51.4	ft

at exterior

TL =	0.20	klf
L from M =	9.7	ft
L from V =	43.1	ft

Pile Load Key Plans

ROOF - WALL LOADS



NOTE: RW3 WAS INCLUDED IN THE POINT LOAD INSTEAD OF AS A DISTRIBUTED LOAD

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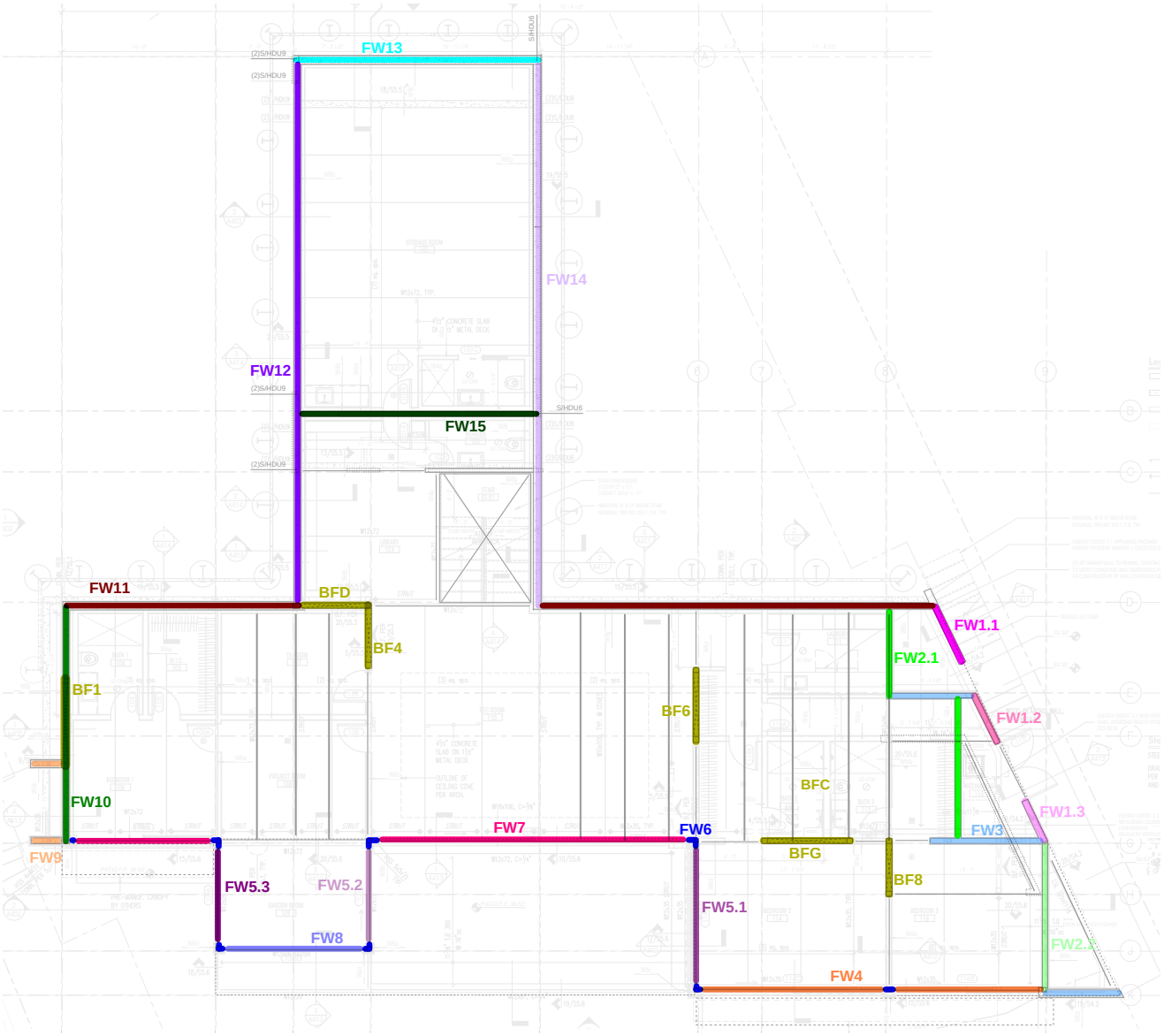
Pile Load Key Plans

FLOOR - WALL LOADS



NOTE: FW6 WAS INCLUDED IN THE POINT LOAD INSTEAD OF AS A DISTRIBUTED LOAD

SSF STRUCTURAL ENGINEERING

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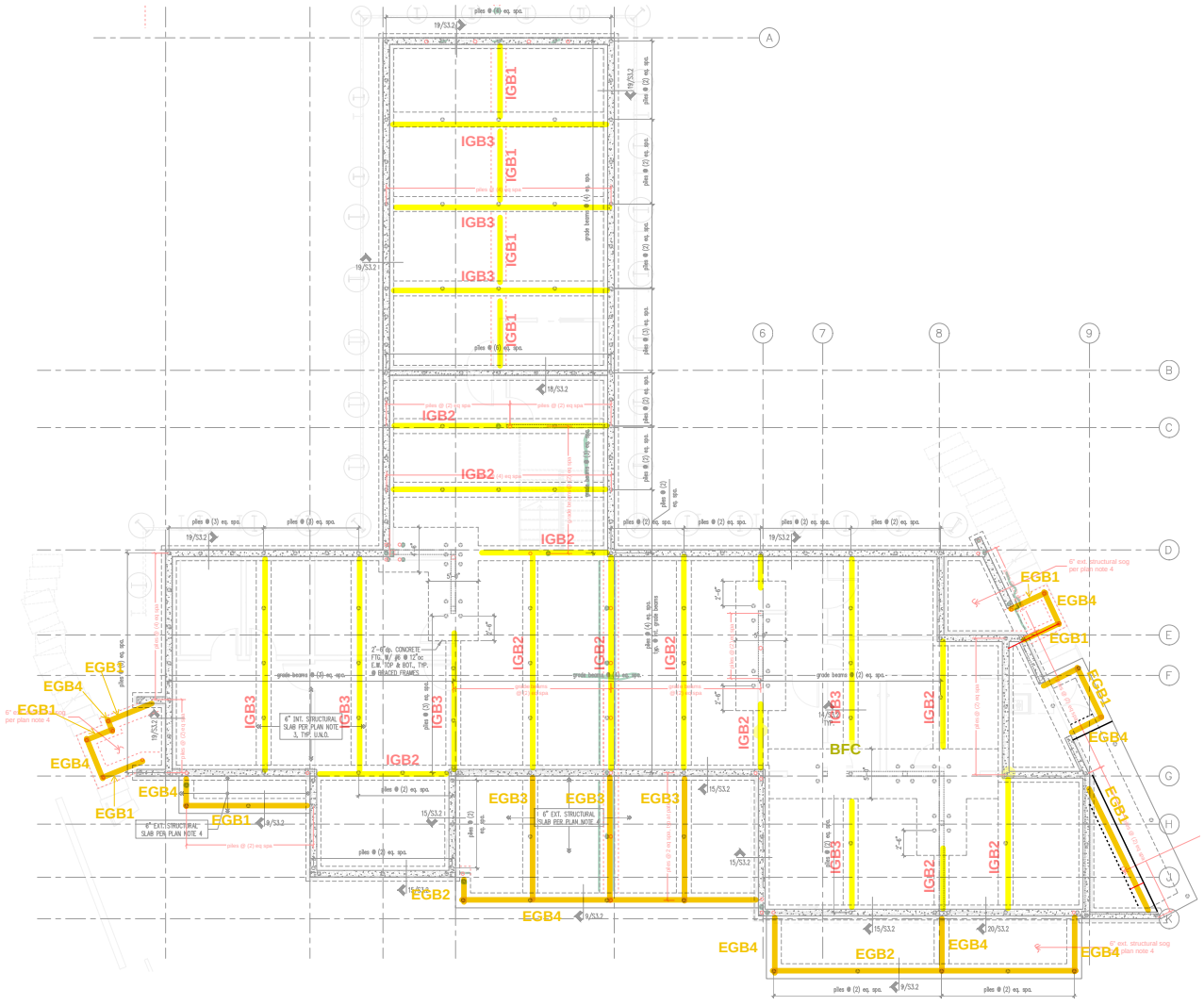
	DESIGN
	SHEET

Pile Load Key Plans

FOUNDATION - WALL LOADS



STRUCTURAL ENGINEERING

PROJECT	8480 Residence
	221
DATE	10/10/2022
PROJ. #	01519-2021-09
DESIGN	LAN
SHEET	

Loads for Pile Design

Ftg Name	Ftg W (ft)	Ftg D (ft)	DL (plf)	Stem Wall W (ft)	Stem Wall H (ft)	DL (plf)	Total Ftg DL (plf)
20/S3.2	1.50	1.00	225	0.67	2.71	271	496
19/S3.2	1.50	1.00	225	0.67	1.88	188	413
18/S3.2	1.50	1.00	225	0.67	1.88	188	413
15/S3.2	1.50	1.00	225	0.67	2.71	271	496
14/S3.2	1.75	2.50	656	0.00	0.00	0	656
13/S3.2	1.75	2.50	656	0.00	0.00	0	656
10/S3.2	1.50	1.00	338	0.83	2.71	282	620
9/S3.2	1.75	2.50	656	0.00	0.00	0	656
8/S3.2	1.25	2.00	375	0.00	0.00	0	375
BFD	5.00	2.50	1875	0.00	0.00	0	1875
BF4	4.00	2.50	1500	0.00	0.00	0	1500
BF6	4.00	2.50	1500	0.00	0.00	0	1500
BFG	5.00	2.50	1875	0.00	0.00	0	1875
BF8	5.00	2.50	1875	0.00	0.00	0	1875

***weights in lbs*

**stem wall height increased by 25% from min height to be conservative

Loads for Pile Design Cont.

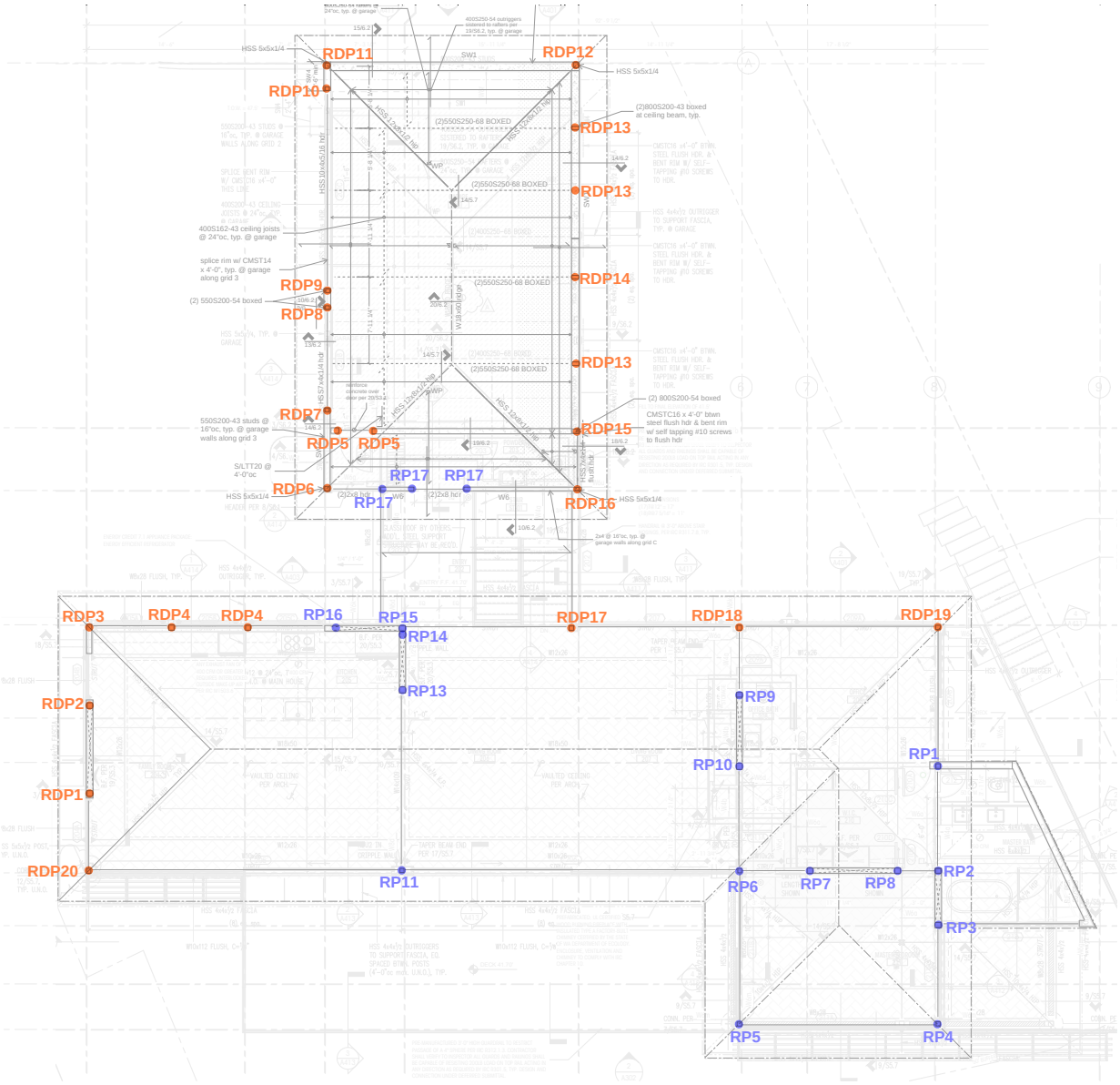
Wall Name	Max Trib (ft)	DL (psf)	Wall H (ft)	Wall Wt (psf)	DL (plf)	SL/LL (psf)	SL/LL (plf)	Ftg Name	Ftg DL (plf)	SOG L (ft)	SOG SW DL (plf)	SOG DL (psf)	DL (plf)	SOG tot DL (plf)	SL/LL (psf)	SL/LL (plf)
RW1	7.0	22	9.3	15	293	25	175	-	0	0.0	0	0	0	0	0	0
RW2	1.0	22	9.3	15	161	25	25	-	0	0.0	0	0	0	0	0	0
RW3- included in pt load	*0.0*	*22*	*10.0*	*15*	*150*	*25*	*0*	*-*	*0*	*0.0*	*0*	*0*	*0*	*0*	*0*	*0*
RW4- included in pt load	*0.0*	*22*	*10.0*	*15*	*150*	*25*	*0*	*-*	*0*	*0.0*	*0*	*0*	*0*	*0*	*0*	*0*
RW5	0.0	30	8.5	15	128	25	0	-	0	0.0	0	0	0	0	0	0
RW6	6.6	23	10.3	12	304	25	189	-	0	0.0	0	0	0	0	0	0
RW7	9.1	23	10.3	15	364	25	229	-	0	0.0	0	0	0	0	0	0
RW8	9.1	23	4.75/5.25	122/15	869	25	229	-	0	0.0	0	0	0	0	0	0
RW9	9.1	23	4.75/5.25	122/15	869	25	229	-	0	0.0	0	0	0	0	0	0
RW10	5.6	23	11.5	105	1337	25	141	-	0	0.0	0	0	0	0	0	0
FW1.1	3.5	28	11.3	100	1231	60	210	19/S3.2	413	3.6	272	10	36	308	40	145
FW1.2	5.0	28	11.3	34	525	60	300	20/S3.2	496	2.0	150	10	20	170	40	80
FW1.3	1.5	28	11.3	34	427	60	90	20/S3.2	496	6.0	450	10	60	510	40	240
FW2.1	0.0	28	11.3	34	385	60	0	20/S3.2	496	4.0	300	10	40	340	40	160
FW2.2	10.5	28	11.3	34	679	60	630	20/S3.2	496	3.5	263	10	35	298	40	140
FW3	1.0	28	11.3	34	447	60	60	20/S3.2	496	1.0	75	10	10	85	40	40
FW4	0.0	72	11.3	15	170	40	0	15/S3.2	496	5.5	413	10	55	468	60	330
FW5.1	0.0	72	11.3	15	170	40	0	15/S3.2	496	8.3	619	10	83	701	40/60	405
FW5.2	0.0	72	11.3	15	170	40	0	15/S3.2	496	5.0	375	10	50	425	60	300
FW5.3	0.0	72	11.3	15	170	40	0	15/S3.2	496	2.0	150	10	20	170	60	120
FW6- included in pt load	*0.0*	*28*	*11.3*	*34*	*385*	*60*	*0*	*-*	*0*	*0.0*	*0*	*10*	*0*	*0*	*60*	*0*
FW7	0.0	72	11.3	15	170	40	0	15/S3.2	496	2.0	150	10	20	170	60	120

Loads for Pile Design Cont.

Wall Name	Max Trib (ft)	DL (psf)	Wall H (ft)	Wall Wt (psf)	DL (plf)	SL/LL (psf)	SL/LL (plf)	Ftg Name	Ftg DL (plf)	SOG L (ft)	SOG SW DL (plf)	SOG DL (psf)	DL (plf)	SOG tot DL (plf)	SL/LL (psf)	SL/LL (plf)
FW8	0.0	72	11.3	15	170	40	0	15/S3.2	496	5.1	384	10	51	436	40	205
FW9	3.6	72	11.3	62	964	25	91	20/S3.2	496	3.6	272	10	36	308	40	145
FW10	2.4	72	11.3	122	1554	40	95	19/S3.2	413	5.8	431	10	58	489	40	230
FW11	1.0	72	11.3	122	1455	40	40	19/S3.2	413	1.0	75	10	10	85	40	40
FW12	1.0	75	11.3	105	1265	40	40	19/S3.2	413	1.0	75	10	10	85	40	40
FW13	2.4	75	11.3	105	1368	40	95	19/S3.2	413	4.3	319	10	43	361	40	170
FW14	1.0	75	11.3	122	1458	40	40	19/S3.2	413	1.0	75	10	10	85	40	40
FW15	5.1	75	11.3	105	1574	40	205	18/S3.2	413	6.9	519	10	69	588	40	277
BFD	0.0	0	11.3	12	136	0	0	BFD	1875	4.2	312	10	42	354	40	167
BF4	0.0	0	11.3	12	136	0	0	BF4	1500	8.8	656	10	88	744	40	350
BF6	0.0	0	11.3	12	136	0	0	BF6	1500	8.3	625	10	83	708	40	333
BFG	0.0	0	11.3	12	136	0	0	BFG	1875	2.0	150	10	20	170	40	80
BF8	0.0	0	11.3	12	136	0	0	BF8	1875	7.4	553	10	74	627	40	295
IGB1	0.0	0	0.0	0	0	0	0	8/S3.2	375	2.0	150	10	20	170	40	80
IGB2	0.0	0	0.0	0	0	0	0	14/S3.2	656	8.0	600	10	80	680	40	320
IGB3	0.0	0	0.0	0	0	0	0	14/S3.2	656	9.5	713	10	95	808	40	380
EGB1	0.0	0	0.0	0	0	0	0	9/S3.2	656	3.0	228	10	30	259	60	182
EGB2	0.0	0	0.0	0	0	0	0	9/S3.2	656	4.5	338	10	45	383	60	270
EGB3	0.0	0	0.0	0	0	0	0	14/S3.2	656	8.0	600	10	80	680	60	480
EGB4	0.0	0	0.0	0	0	0	0	9/S3.2	656	1.0	75	10	10	85	60	60

Pile Load Key Plans

ROOF - POINT LOADS

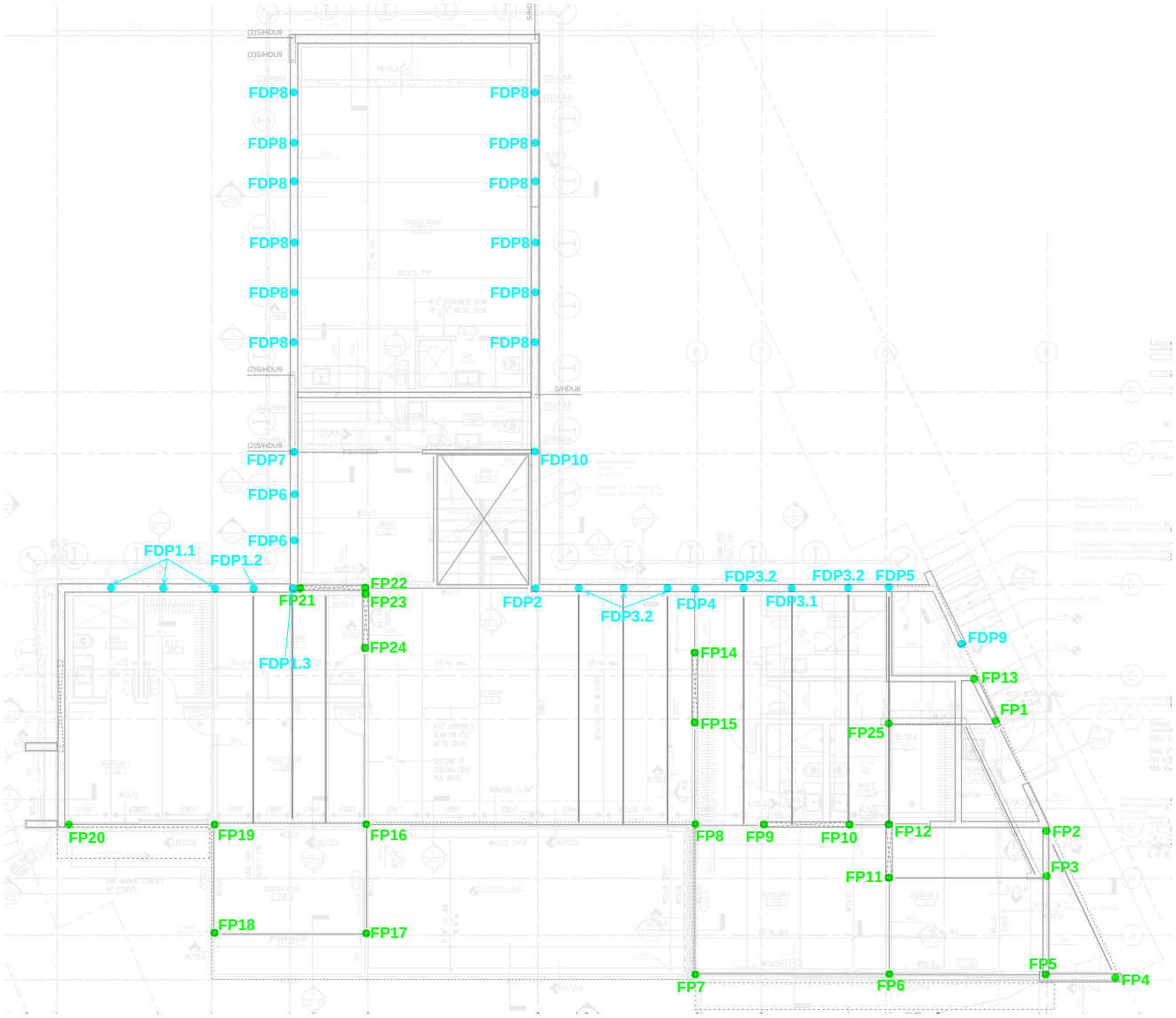


STRUCTURAL ENGINEERING

8480 Residence	10/10/2022
PROJECT	DATE
225	01519-2021-09
	PROJ. #
	LAN
	DESIGN
	SHEET

Pile Load Key Plans

FLOOR - POINT LOADS



STRUCTURAL ENGINEERING

8480 Residence	10/10/2022
PROJECT	DATE
226	01519-2021-09
	PROJ. #
	LAN
	DESIGN
	SHEET

Loads for Pile Design

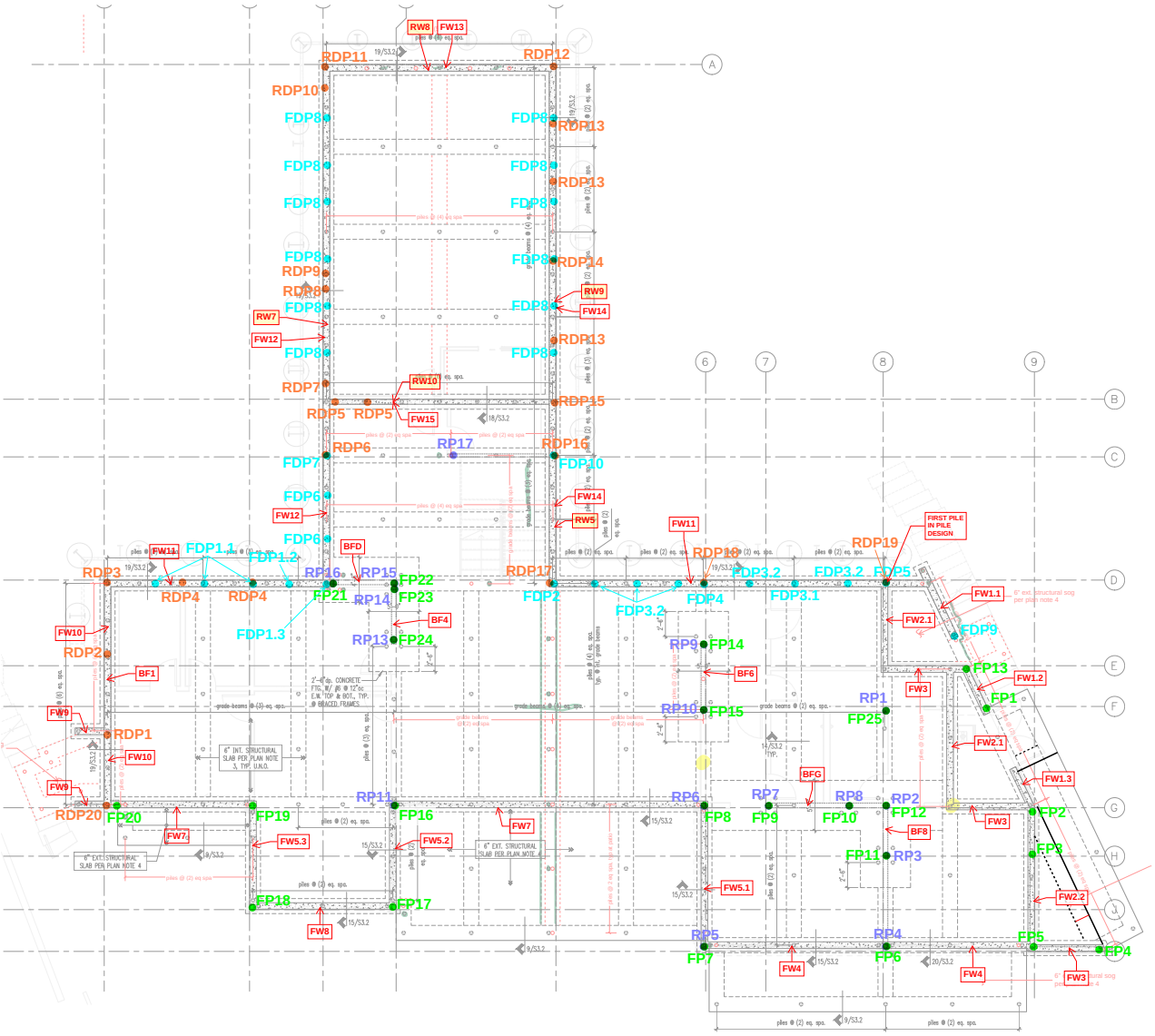
Pt Name	DL (lbs)	SL/LL (lbs)	TL (lbs)	Dist Over (ft)	DL (plf)	SL/LL (plf)
RP1	3200	3300	6500	-	-	-
RP2	3500	2800	6300	-	-	-
RP3	3500	3400	6900	-	-	-
RP4	3525	2900	6425	-	-	-
RP5	3525	2900	6425	-	-	-
RP6	10438	6900	17338	-	-	-
RP7	4300	3900	8200	-	-	-
RP8	1900	1500	3400	-	-	-
RP9	1600	1200	2800	-	-	-
RP10	5900	4100	10000	-	-	-
RP11	19800	10700	30500	-	-	-
RP13	11600	8700	20300	-	-	-
RP14	2200	1600	3800	-	-	-
RP15	9600	7300	16900	-	-	-
RP16	3488	2800	6288	-	-	-
RP17	330	350	680	-	-	-
RDP1	3313	2200	5513	12.8	258	171
RDP2	3313	2200	5513	12.8	258	171
RDP3	7525	5200	12725	12.8	586	405
RDP4	2875	2000	4875	12.8	224	156
RDP5	120	80	200	12.8	9	6
RDP6	1970	3070	5040	12.8	153	239
RDP7	1090	1180	2270	12.8	85	92
RDP8	1060	1150	2210	12.8	83	90
RDP9	2420	2620	5040	12.8	188	204
RDP10	2260	2460	4720	12.8	176	191
RDP11	1600	2650	4250	12.8	125	206
RDP12	1600	2650	4250	12.8	125	206
RDP13	650	0	650	12.8	51	0
RDP14	730	0	730	12.8	57	0
RDP15	350	380	730	12.8	27	30
RDP16	2930	4160	7090	12.8	228	324
RDP17	6850	6300	13150	12.8	533	490
RDP18	9075	6800	15875	12.8	706	529
RDP19	6700	4200	10900	12.8	522	327
RDP20	8600	5700	14300	6.4	1339	887

Loads for Pile Design Cont.

Pt Name	DL (lbs)	SL/LL (lbs)	TL (lbs)	Dist Over (ft)	DL (plf)	SL/LL (plf)
FP1	4100	2700	6800	-	-	-
FP2	770	320	1090	-	-	-
FP3	5500	3700	9200	-	-	-
FP4	990	640	1630	-	-	-
FP5	1900	1600	3500	-	-	-
FP6	11150	6300	17450	-	-	-
FP7	16635	10900	27535	-	-	-
FP8	30199	17100	47299	-	-	-
FP9	12900	6500	19400	-	-	-
FP10	11400	5600	17000	-	-	-
FP11	4800	4800	9600	-	-	-
FP12	6600	2800	9400	-	-	-
FP13	210	450	660	-	-	-
FP14	3800	1950	5750	-	-	-
FP15	3800	1950	5750	-	-	-
FP16	41331	19100	60431	-	-	-
FP17	15671	17500	33171	-	-	-
FP18	5671	3300	8971	-	-	-
FP19	14267	10600	24867	-	-	-
FP20	7585	2800	10385	-	-	-
FP21	1300	640	1940	-	-	-
FP22	1510	6900	8410	-	-	-
FP23	600	240	840	-	-	-
FP24	5300	2800	8100	-	-	-
FP25	7100	5400	12500	-	-	-
FDP1.1	4310	2090	6400	9	479	232
FDP1.2	3520	1650	5170	9	391	183
FDP1.3	3520	1650	5170	9	391	183
FDP2	14910	6890	21800	10	1491	689
FDP3.1	4510	2200	6710	11	410	200
FDP3.2	4180	2020	6200	11	380	184
FDP4	830	420	1250	11	75	38
FDP5	2200	2500	4700	11	200	227
FDP6	2340	1120	3460	11	213	102
FDP7	12200	6650	18850	11	1109	605
FDP8	4470	2090	6560	11	406	190
FDP9	210	450	660	11	19	41
FDP10	11300	5190	16490	11	1027	472

Pile Load Key Plans

ALL LOADS AT BASE



Pile Design - Distributed Load (Wall + Dist. Pt)

Pile Dia. (in)	Cap (tons)	ASD (lbs)
4	10	20000

Cumulative Wall Name	Wall DL (plf)	Wall SL/LL (plf)	Ftg DL (plf)	SOG DL (plf)	SOG LL (plf)	Pt Dist-1 Name	Pt Dist-1 TL (plf)	Pt Dist-2 Name	Pt Dist-2 TL (plf)	Pt Dist-3 Name	Pt Dist-3 TL (plf)	Total Load ASD (plf)
FW1.1	1231	210	413	308	145	-	0	-	0	-	0	2307
FW1.1+FDP9	1231	210	413	308	145	FDP9	60	-	0	-	0	2367
FW1.2	525	300	496	170	80	-	0	-	0	-	0	1571
FW1.3	427	90	496	510	240	-	0	-	0	-	0	1763
FW2.1	385	0	496	340	160	-	0	-	0	-	0	1381
FW2.2	679	630	496	298	140	-	0	-	0	-	0	2243
FW3	447	60	496	85	40	-	0	-	0	-	0	1127
FW4	170	0	496	468	330	-	0	-	0	-	0	1463
FW5.1	170	0	496	701	405	-	0	-	0	-	0	1772
FW5.2	170	0	496	425	300	-	0	-	0	-	0	1391
FW5.3	170	0	496	170	120	-	0	-	0	-	0	956
FW7	170	0	496	170	120	-	0	-	0	-	0	956
FW8	170	0	496	436	205	-	0	-	0	-	0	1306
FW9	964	91	496	308	145	-	0	-	0	-	0	2003
FW10	1554	95	413	489	230	-	0	-	0	-	0	2780
FW10+RDP1+RDP20	1554	95	413	489	230	RDP1	429	RDP20	2226	-	0	5435
FW10+RDP1	1554	95	413	489	230	RDP1	429	-	0	-	0	3209
FW10+RDP1+RDP2	1554	95	413	489	230	RDP1	429	RDP2	429	-	0	3638
FW10+RDP2+RDP3	1554	95	413	489	230	RDP2	429	RDP3	991	-	0	4200
FW11	1455	40	413	85	40	-	0	-	0	-	0	2032
FW11+RDP3+RDP4+(2)FDP1.1	1455	40	413	85	40	RDP3	991	RDP4	379	(2) FDP1.1	1422	4824

Cumulative Wall Name	Wall DL (plf)	Wall SL/LL (plf)	Ftg DL (plf)	SOG DL (plf)	SOG LL (plf)	Pt Dist-1 Name	Pt Dist-1 TL (plf)	Pt Dist-2 Name	Pt Dist-2 TL (plf)	Pt Dist-3 Name	Pt Dist-3 TL (plf)	Total Load (plf)
FW11+RDP4+FDP1.2+(2)FDP1.1	1455	40	413	85	40	RDP4	379	FDP1.2	574	(2) FDP1.1	1422	4408
FW11+RDP4+FDP1.2+FDP1.3	1455	40	413	85	40	RDP4	379	FDP1.2	574	FDP1.3	574	3561
FW11+RDP17+FDP2+FDP3.2	1455	40	413	85	40	RDP17	1024	FDP2	2180	FDP3.2	564	5799
FW11+RDP18+(3)FDP3.2+FDP4	1455	40	413	85	40	RDP18	1236	FDP4	114	(3) FDP3.2	1691	5073
FW11+FDP3.1+(2)FDP3.2	1455	40	413	85	40	FDP3.1	610	FDP3.2	564	FDP3.2	564	3769
FW11+RDP19+FDP3.1+FDP3.2+FDP5	1455	40	413	85	40	RDP19	849	FDP5	427	FDP3.1+ FDP3.2	1174	4482
FW11+RDP19+FDP5	1455	40	413	85	40	RDP19	849	FDP5	427	-	0	3308
FW12	1265	40	413	85	40	-	0	-	0	-	0	1842
FW12+FDP6	1265	40	413	85	40	FDP6	315	-	0	-	0	2157
FW12+(2)FDP6+FDP1.3	1265	40	413	85	40	FDP6	315	FDP6	315	FDP1.3	574	3046
FW12+RDP6+FDP6+FDP7	1265	40	413	85	40	RDP6	392	FDP6	315	FDP7	1714	4263
RDP5+RDP6	0	0	-	-	-	RDP5	16	RDP6	392	-	0	-
RW7+FW12+RDP5+RDP6+RDP7+FDP7	1629	269	413	85	40	RDP5+ RDP6	408	RDP7	177	FDP7	1714	4733
RW7+FW12+RDP5+RDP7+FDP8	1629	269	413	85	40	RDP5	16	RDP7	177	FDP8	596	3224
RW7+FW12+RDP7+RDP8+(2)FDP8	1629	269	413	85	40	RDP7	177	RDP8	172	(2) FDP8	1193	3977
RW7+FW12+RDP8+RDP9+(2)FDP8	1629	269	413	85	40	RDP8	172	RDP9	392	(2) FDP8	1193	4192
FW12+(3)FDP8	1265	40	413	85	40	FDP8	596	FDP8	596	FDP8	596	3632

Cumulative Wall Name	Wall DL (plf)	Wall SL/LL (plf)	Ftg DL (plf)	SOG DL (plf)	SOG LL (plf)	Pt Dist-1 Name	Pt Dist-1 TL (plf)	Pt Dist-2 Name	Pt Dist-2 TL (plf)	Pt Dist-3 Name	Pt Dist-2 TL (plf)	Total Load (plf)
RW7+FW12+RDP10+RDP11+(2)FDP8	1629	269	413	85	40	RDP10	367	RDP11	331	(2) FDP8	1193	4326
RW8+FW13	2237	324	413	361	170	-	0	-	0	-	0	3504
RW8+FW13+RDP11	2237	324	413	361	170	RDP11	331	-	0	-	0	3835
RW8+FW13+RDP12	2237	324	413	361	170	RDP12	331	-	0	-	0	3835
RW9+FW14	2326	269	413	85	40	-	0	-	0	-	0	3132
RW9+FW14+RDP12+RDP13+FDP8	2326	269	413	85	40	RDP12	331	RDP13	51	FDP8	596	4110
RW9+FW14+(2)RDP13+(2)FDP8	2326	269	413	85	40	(2) RDP13	101	(2) FDP8	1193	-	0	4426
RW9+FW14+RDP13+RDP14+(3)FDP8	2326	269	413	85	40	RDP13	51	RDP14	57	(3) FDP8	1789	5029
RW9+FW14+RDP14+(3)FDP8	2326	269	413	85	40	RDP14	57	(3) FDP8	1789	-	0	4978
RW9+FW14+RDP13+RDP15+FDP8	2326	269	413	85	40	RDP13	51	RDP15	57	FDP8	596	3836
RW9+FW14+RDP13+RDP15+(2)FDP8	2326	269	413	85	40	RDP13	51	RDP15	57	(2) FDP8	1193	4432
FW14+RDP15+RDP16+FDP10	1458	40	413	85	40	RDP15	57	RDP16	552	FDP10	1499	4143
RW5+FW14+RDP16+FDP10	1585	40	413	85	40	RDP16	552	FDP10	1499	-	0	4214
RW5+FW14+RDP17+FDP2	1585	40	413	85	40	RDP17	1024	FDP2	2180	-	0	5366
RW10+FW15	2911	346	413	588	277	-	0	-	0	-	0	4534
RW10+FW15+(2)RDP5	2911	346	413	588	277	RDP5	16	RDP5	16	-	0	4565

Cumulative Wall Name	Wall DL (plf)	Wall SL/LL (plf)	Ftg DL (plf)	SOG DL (plf)	SOG LL (plf)	Pt Dist-1 Name	Pt Dist-1 TL (plf)	Pt Dist-2 Name	Pt Dist-2 TL (plf)	Pt Dist-3 Name	Pt Dist-2 TL (plf)	Total Load (plf)
BFD	136	0	1875	354	167	-	0	-	0	-	0	2532
BF4	136	0	1500	744	350	-	0	-	0	-	0	2730
BF6	136	0	1500	708	333	-	0	-	0	-	0	2678
BFG	136	0	1875	170	80	-	0	-	0	-	0	2261
BF8	136	0	1875	627	295	-	0	-	0	-	0	2933
IGB1	0	0	375	170	80	-	0	-	0	-	0	625
IGB2	0	0	656	680	320	-	0	-	0	-	0	1656
IGB3	0	0	656	808	380	-	0	-	0	-	0	1844
EGB1	0	0	656	259	182	-	0	-	0	-	0	1097
EGB2	0	0	656	383	270	-	0	-	0	-	0	1309
EGB3	0	0	656	680	480	-	0	-	0	-	0	1816
EGB4	0	0	656	85	60	-	0	-	0	-	0	801

Pile Design - Intersection and Point Loads

Pile Dia. (in)	Cap (tons)	ASD (lbs)
4	10	20000

Intersection Name	Pt-1 Name	Pt Load-1 (lbs)	Pt-2 Name	Pt Load-2 (lbs)	GB-1 Wall Name	GB-1 Wall TL (plf)	GB-1 Length (ft)	GB-1 Rxn (lbs)	GB-2 Wall Name	GB-2 Wall TL (plf)	GB-2 Length (ft)	GB-2 Rxn (lbs)	GB-3 Wall Name	GB-3 Wall TL (plf)	GB-3 Length (ft)	GB-3 Rxn (lbs)	GB-4 Wall Name	GB-4 Wall TL (plf)	GB-4 Length (ft)	GB-4 Rxn (lbs)	Total Pt Load (lbs)	No. of 4" Piles
[FW11+RDP19+FDP3.1+FDP3.2+FDP5]+[FW11+RDP19+FDP5]+FW2.1 int.	-	0	-	0	FW11+RDP19+FDP3.1+FDP3.2+FDP5	4482	3.0	6785	FW11+RDP19+FDP5	3308	4.3	7167	FW2.1	1381	4.2	2906		0		0	16858	1
FW2.1+FW3+IGB2 int.	-	0	-	0	FW2.1	1381	4.2	2906	FW3	1127	6.6	3711	IGB2	1656	4.2	3451		0		0	10068	1
FW2.1+FW3+FW3 int.	-	0	-	0	FW2.1	1381	4.6	3146	FW3	1127	6.6	3711	FW3	1127	1.7	940		0		0	7797	1
FW3+(FW1.1+FDP9)+FW1.2+EGB1 int. + FP13+FP1 (East patio)	FP13	660	FP1	971	FW3	1127	1.7	940	FW1.1+FDP9	2367	3.8	4438	EGB1	1097	4.3	2332	FW1.2	1571	5.3	4124	13465	1
FW1.2+FW1.3+EGB4 int. + FP1	FP1	5829	-	0	FW1.2	1571	5.3	4124	FW1.3	1763	5.0	4408	EGB4	801	4.0	1603		0		0	15963	1
FW1.3+FW1.3 int.	-	0	-	0	FW1.3	1763	5.0	4408	FW1.3	1763	5.0	4408		0		0		0		0	8816	1
FW2.1+FW3+IGB2+BFG int.	-	0	-	0	FW2.1	1381	4.8	3280	FW3	1127	8.1	4557	IGB2	1656	7.0	5762	BFG	2261	3.3	3674	17273	1
FW1.3+FW2.2+FW3+EGB1 int. + FP2	FP2	1090	-	0	FW1.3	2	5.0	6	FW2.2	2243	4.92	5513	FW3	1127	8.1	4557	EGB1	1097	7.7	4229	15395	1
FW2.2+FW2.2 int. + FP2+FP3	FP3	9200	FP2	1090	FW2.2	2243	4.92	5513	FW2.2	2243	3.11	3489		0		0		0		0	19292	1
FW2.2+FW3+FW4 int. + FP5	FP5	3500	-	0	FW2.2	2243	4.67	5233	FW3	1127	7.3	4134	FW4	1463	1.3	976		0		0	13842	1
FW3+EGB1 int. + FP4 (East patio)	FP4	1630	-	0	FW3	1127	7.3	4134	EGB1	1097	7.7	4229		0		0		0		0	9993	1
EGB1+EGB1 int. (East patio)	-	0	-	0	EGB1	1097	7.7	4229	EGB1	1097	7.7	4229		0		0		0		0	8458	1
EGB4+FW4+FW4 int. (SE patio)	-	0	-	0	EGB4	801	9.0	3606	FW4	1463	1.3	976	FW4	1463	6.9	5061		0		0	9642	1
EGB4+EGB2 int. (SE patio)	-	0	-	0	EGB4	801	9.0	3606	EGB2	1309	6.7	4390		0		0		0		0	7995	1
EGB4+EGB2+EGB2 int. (SE patio)	-	0	-	0	EGB4	801	9.0	3606	EGB2	1309	6.7	4390	EGB2	1309	8.5	5535		0		0	13530	1
FW4+FW4+IGB2 int.	-	0	-	0	FW4	1463	6.5	4756	FW4	1463	6.9	5061	IGB2	1656	7.0	5762		0		0	15579	1
FW4+FW4+IGB2+EGB4 int. + RP4+FP6	RP4	6425	FP6	17450	FW4	1463	6.5	4756	FW4	1463	9.1	6646	EGB4	801	5.8	2337	BF8	2933	9.0	13198	50811	3
FW4+FW4+IGB3 int.	-	0	-	0	FW4	1463	9.1	6646	FW4	1463	2.6	1890	IGB3	1844	7.0	6453		0		0	14989	1
FW4+FW4 int. + 1/2(RP5+FP7)	RP5	3212	FP7	13767	FW4	1463	2.6	1890	FW4	956	1.2	558		0		0		0		0	19428	1
FW5.1+FW5.1 int. + 1/2(RP5+FP7)	RP5	3206	FP7	13740	FW5.1	1772	2.6	2259	FW5.1	1319	1.2	769		0		0		0		0	19975	1
FW5.1+FW5.1 int.	-	0	-	0	FW5.1	1772	2.6	2259	FW5.1	1772	2.6	2259		0		0		0		0	4519	1
FW5.1+FW7+IGB2+BFG int. + RP6+FP8	RP6	17338	FP8	47299	FW5.1	1772	2.6	2259	FW7	956	7.6	3624	BF6	2678	3.2	4240	BFG	2261	3.2	3580	78340	4
FW7+FW7+EGB3+IGB2 int.	-	0	-	0	FW7	956	7.6	3624	FW7	956	7.6	3624	EGB3	1816	6.4	5827	IGB2	1656	5.5	4555	17630	1
FW7+FW5.2+IGB2+BF4 int. + RP11+FP16	RP11	27825	FP16	55656	FW7	926	7.9	3665	FW5.2	1316	3.4	2230	IGB2	1576	4.8	3744		0		0	94619	5
FW5.2+FW5.2 int.	-	0	-	0	FW5.2	1391	3.4	2357	FW5.2	1391	3.4	2357		0		0		0		0	4713	1
FW5.2+FW8+EGB4 int. + FP17	FP17	33171	-	0	FW5.2	1391	3.4	2357	FW8	1306	4.7	3048	EGB4	801	1.3	534		0		0	39110	2
FW5.3+FW8 int. + FP18	FP18	8971	-	0	FW5.3	956	3.4	1633	FW8	1306	4.7	3048		0		0		0		0	13652	1

Cumulative Pt Name	Pt-1 Name	Pt Load-1 (lbs)	Pt-2 Name	Pt Load-2 (lbs)	GB-1 Wall Name	GB-1 Wall TL (plf)	GB-1 Length (ft)	GB-1 Rxn (lbs)	GB-2 Wall Name	GB-2 Wall TL (plf)	GB-2 Length (ft)	GB-2 Rxn (lbs)	GB-3 Wall Name	GB-3 Wall TL (plf)	GB-3 Length (ft)	GB-3 Rxn (lbs)	GB-4 Wall Name	GB-4 Wall TL (plf)	GB-4 Length (ft)	GB-4 Rxn (lbs)	Total Pt Load (lbs)	No. of 4" Piles
FW5.3+FW5.3+EGB1 int.	-	0	-	0	FW5.3	956	3.4	1633	FW5.3	956	3.3	1593	EGB1	1097	6.4	3498		0		0	6724	1
FW5.3+FW7+IGB2 int. + FP19	FP19	24867	-	0	FW5.3	956	3.3	1593	FW7	956	5.0	2390	IGB2	1656	4.6	3796		0		0	32645	2
FW7+FW7+IGB3 int.	-	0	-	0	FW7	956	5.0	2390	FW7	956	7.8	3704	IGB3	1844	5.5	5070		0		0	11164	1
FW7+FW7+EGB4 int. + 1/2(FP2)	FP20	6750	-	0	FW7	956	7.8	3704	FW7	956	1.8	836	EGB4	801	3.3	1335		0		0	12626	1
FW7+[FW10+RDP1+RDP20]+FW9 int. + 1/2(FP2)	FP20	5193	-	0	FW7	956	1.8	836	FW10+RDP1+RDP20	5435	2.5	6719	FW9	2003	3.0	3005		0		0	15753	1
EGB3+EGB3 int. (South patio)	-	0	-	0	EGB3	1816	6.4	5827	EGB3	1816	6.4	5827		0		0		0		0	11654	1
EGB3+EGB4 int. (South patio)	-	0	-	0	EGB3	1816	6.4	5827	EGB4	801	7.92	3172		0		0		0		0	8999	1
EGB1+EGB1 int. (SW patio)	-	0	-	0	EGB1	1097	6.4	3498	EGB1	1097	6.4	3498		0		0		0		0	6995	1
[FW10+RDP1+RDP20]+[FW10+RDP1+RDP20] int.	-	0	-	0	FW10+RDP1+RDP20	5435	2.5	6719	FW10+RDP1+RDP20	5435	2.5	6719		0		0		0		0	13438	1
[FW10+RDP1+RDP20]+FW9+[FW10+RDP2+RDP3] int. + 0.5EGB1	-	0	-	0	FW10+RDP1+RDP20	5435	2.5	6719	FW9	2003	3.0	3005	FW10+RDP2+RDP3	4200	3.7	7743	EGB1	1097		0	17467	1
[FW10+RDP2+RDP3]+[FW10+RDP2+RDP3] int.	-	0	-	0	FW10+RDP2+RDP3	4200	3.7	7743	FW10+RDP2+RDP3	4200	3.7	7743		0		0		0		0	15486	1
[FW10+RDP2+RDP3]+[FW11+RDP3+RDP4+(2)FDP1.1] int.	-	0	-	0	FW10+RDP2+RDP3	4200	3.7	7743	FW11+RDP3+RDP4+(2)FDP1.1	4824	3.2	7706		0		0		0		0	15449	1
[FW11+RDP3+RDP4+(2)FDP1.1]+[FW11+RDP4+FDP1.2+(2)FDP1.1]+IGB3 int.	-	0	-	0	FW11+RDP3+RDP4+(2)FDP1.1	4824	3.2	7706	FW11+RDP4+FDP1.2+(2)FDP1.1	4408	3.2	7041	IGB3	1844	5.5	5070		0		0	19817	1
[FW11+RDP4+FDP1.2+(2)FDP1.1]+[FW11+RDP4+FDP1.2+(2)FDP1.1] int.	-	0	-	0	FW11+RDP4+FDP1.2+(2)FDP1.1	4408	3.2	7041	FW11+RDP4+FDP1.2+(2)FDP1.1	4408	3.2	7041		0		0		0		0	14082	1
[FW11+RDP4+FDP1.2+(2)FDP1.1]+[FW11+RDP4+FDP1.2+FDP1.3]+IGB3 int.	-	0	-	0	FW11+RDP4+FDP1.2+(2)FDP1.1	4408	3.2	7041	FW11+RDP4+FDP1.2+FDP1.3	3561	3.5	6231	IGB3	1844	5.5	5070		0		0	18342	1
[FW11+RDP4+FDP1.2+FDP1.3]+BFD+[FW12+(2)FDP6+FDP1.3] int. + RP16+FP21	RP16	6288	FP21	1940	FW11+RDP4+FDP1.2+FDP1.3	3561	3.5	6231	BFD	2532	6.1	7701	FW12+(2)FDP6+FDP1.3	3046	3.2	4823		0		0	26982	2
[FW12+(2)FDP6+FDP1.3]+[FW12+(2)FDP6+FDP1.3] int.	-	0	-	0	FW12+(2)FDP6+FDP1.3	3046	3.2	4823	FW12+(2)FDP6+FDP1.3	3046	3.2	4823		0		0		0		0	9646	1
[FW12+(2)FDP6+FDP1.3]+[FW12+RDP6+FDP6+FDP7]+IGB2 int.	-	0	-	0	FW12+(2)FDP6+FDP1.3	3046	3.2	4823	FW12+RDP6+FDP6+FDP7	4263	3.2	6750	IGB2	1656	5.6	4675		0		0	16248	1
[FW12+RDP6+FDP6+FDP7]+[FW12+RDP6+FDP6+FDP7] int.	-	0	-	0	FW12+RDP6+FDP6+FDP7	4263	3.2	6750	FW12+RDP6+FDP6+FDP7	4263	3.2	6750		0		0		0		0	13500	1

Cumulative Pt Name	Pt-1 Name	Pt Load-1 (lbs)	Pt-2 Name	Pt Load-2 (lbs)	GB-1 Wall Name	GB-1 Wall TL (plf)	GB-1 Length (ft)	GB-1 Rxn (lbs)	GB-2 Wall Name	GB-2 Wall TL (plf)	GB-2 Length (ft)	GB-2 Rxn (lbs)	GB-3 Wall Name	GB-3 Wall TL (plf)	GB-3 Length (ft)	GB-3 Rxn (lbs)	GB-4 Wall Name	GB-4 Wall TL (plf)	GB-4 Length (ft)	GB-4 Rxn (lbs)	Total Pt Load (lbs)	No. of 4" Piles
[FW12+RDP6+FDP6+FDP7]+[RW7+FW12+RDP5+RDP6+RDP7+FDP7]+IGB2 int.	-	0	-	0	FW12+RDP6+FDP6+FDP7	4263	3.2	6750	RW7+FW12+RDP5+RDP6+RDP7+FDP7	4733	2.7	6311	IGB2	1656	5.6	4675		0		0	17736	1
[RW7+FW12+RDP5+RDP6+RDP7+FDP7]+[RW7+FW12+RDP5+RDP6+RDP7+FDP7] int.	-	0	-	0	RW7+FW12+RDP5+RDP6+RDP7+FDP7	4733	2.7	6311	RW7+FW12+RDP5+RDP6+RDP7+FDP7	4733	2.7	6311		0		0		0		0	12622	1
[RW7+FW12+RDP5+RDP6+RDP7+FDP7]+[RW7+FW12+RDP5+RDP7+FDP8]+[RW10+FW15+(2)RDP5] int.	-	0	-	0	RW7+FW12+RDP5+RDP6+RDP7+FDP7	4733	2.7	6311	RW7+FW12+RDP5+RDP7+FDP8	3224	2.8	4477	RW10+FW15+(2)RDP5	4565	3.8	8591		0		0	19380	1
[RW7+FW12+RDP5+RDP7+FDP8]+[RW7+FW12+RDP7+RDP8+(2)FDP8] int.	-	0	-	0	RW7+FW12+RDP5+RDP7+FDP8	3224	2.8	4477	RW7+FW12+RDP7+RDP8+(2)FDP8	3977	2.8	5523		0		0		0		0	10000	1
[RW7+FW12+RDP7+RDP8+(2)FDP8]+[RW7+FW12+RDP8+RDP9+(2)FDP8]+IGB3 int.	-	0	-	0	RW7+FW12+RDP7+RDP8+(2)FDP8	3977	4.2	8284	RW7+FW12+RDP8+RDP9+(2)FDP8	4192	2.8	5822	IGB3	1844	5.6	5205		0		0	19312	1
[RW7+FW12+RDP8+RDP9+(2)FDP8]+[RW7+FW12+RDP8+RDP9+(2)FDP8] int.	-	0	-	0	RW7+FW12+RDP8+RDP9+(2)FDP8	4192	2.8	5822	RW7+FW12+RDP8+RDP9+(2)FDP8	4192	2.8	5822		0		0		0		0	11645	1
[RW7+FW12+RDP8+RDP9+(2)FDP8]+[FW12+(3)FDP8]+IGB3 int.	-	0	-	0	RW7+FW12+RDP8+RDP9+(2)FDP8	4192	2.8	5822	FW12+(3)FDP8	3632	2.8	5044	IGB3	1844	5.6	5205		0		0	16071	1
[FW12+(3)FDP8]+[FW12+(3)FDP8] int.	-	0	-	0	FW12+(3)FDP8	3632	2.8	5044	FW12+(3)FDP8	3632	2.8	5044		0		0		0		0	10088	1
[FW12+(3)FDP8]+[RW7+FW12+RDP10+RDP11+(2)FDP8]+IGB3 int.	-	0	-	0	FW12+(3)FDP8	3632	2.8	5044	RW7+FW12+RDP10+RDP11+(2)FDP8	4326	2.8	6008	IGB3	1844	5.6	5205		0		0	16257	1
[RW7+FW12+RDP10+RDP11+(2)FDP8]+[RW8+FW13+RDP11] int.	-	0	-	0	RW7+FW12+RDP10+RDP11+(2)FDP8	4326	2.8	6008	RW8+FW13+RDP11	3835	4.5	8660		0		0		0		0	14669	1
[RW8+FW13+RDP11]+[RW8+FW13+RDP11]+IGB1 int.	-	0	-	0	RW8+FW13+RDP11	3835	4.5	8660	RW8+FW13+RDP11	3835	4.5	8660	IGB1	625	8.3	1302		0		0	18622	1
[RW8+FW13+RDP12]+[RW9+FW14+RDP12+RDP13+FDP8] int.	-	0	-	0	RW8+FW13+RDP12	3835	4.5	8660	RW9+FW14+RDP12+RDP13+FDP8	4110	2.8	5708		0		0		0		0	14368	1
[RW9+FW14+RDP12+RDP13+FDP8]+[RW9+FW14+(2)RDP13+(2)FDP8] int.	-	0	-	0	RW9+FW14+RDP12+RDP13+FDP8	4110	2.8	5708	RW9+FW14+(2)RDP13+(2)FDP8	4426	2.8	6147		0		0		0		0	11856	1
[RW9+FW14+(2)RDP13+(2)FDP8]+[RW9+FW14+(2)RDP13+(2)FDP8]+IGB3 int.	-	0	-	0	RW9+FW14+(2)RDP13+(2)FDP8	4426	2.8	6147	RW9+FW14+(2)RDP13+(2)FDP8	4426	2.8	6147	IGB3	1844	5.6	5205		0		0	17500	1
[RW9+FW14+(2)RDP13+(2)FDP8]+[RW9+FW14+RDP13+RDP14+(3)FDP8] int.	-	0	-	0	RW9+FW14+(2)RDP13+(2)FDP8	4426	2.8	6147	RW9+FW14+RDP13+RDP14+(3)FDP8	5029	2.8	6984		0		0		0		0	13132	1

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

236

10/10/2022
DATE

01519-2021-09
PROJ. #

LAN
DESIGN

SHEET

Cumulative Pt Name	Pt-1 Name	Pt Load-1 (lbs)	Pt-2 Name	Pt Load-2 (lbs)	GB-1 Wall Name	GB-1 Wall TL (plf)	GB-1 Length (ft)	GB-1 Rxn (lbs)	GB-2 Wall Name	GB-2 Wall TL (plf)	GB-2 Length (ft)	GB-2 Rxn (lbs)	GB-3 Wall Name	GB-3 Wall TL (plf)	GB-3 Length (ft)	GB-3 Rxn (lbs)	GB-4 Wall Name	GB-4 Wall TL (plf)	GB-4 Length (ft)	GB-4 Rxn (lbs)	Total Pt Load (lbs)	No. of 4" Piles
[RW9+FW14+RDP13+RDP14+(3)FDP8]+[RW9+FW14+RDP14+(3)FDP8]+IGB3 int.	-	0	-	0	RW9+FW14+RDP13+RDP14+(3)FDP8	5029	2.8	6984	RW9+FW14+RDP14+(3)FDP8	4978	2.8	6914	IGB3	1844	5.6	5205		0		0	19103	1
[RW9+FW14+RDP13+RDP14+(3)FDP8]+[RW9+FW14+RDP13+RDP14+(3)FDP8] int.	-	0	-	0	RW9+FW14+RDP13+RDP14+(3)FDP8	5029	2.8	6984	RW9+FW14+RDP13+RDP14+(3)FDP8	5029	2.8	6984		0		0		0		0	13969	1
[RW9+FW14+RDP13+RDP14+(3)FDP8]+[RW9+FW14+RDP13+RDP15+(2)FDP8]+IGB3 int.	-	0	-	0	RW9+FW14+RDP13+RDP14+(3)FDP8	5029	2.8	6984	RW9+FW14+RDP13+RDP15+(2)FDP8	4432	2.8	6156	IGB3	1844	5.6	5205		0		0	18345	1
[RW9+FW14+RDP13+RDP15+(2)FDP8]+[RW9+FW14+RDP13+RDP15+(2)FDP8] int.	-	0	-	0	RW9+FW14+RDP13+RDP15+(2)FDP8	4432	2.8	6156	RW9+FW14+RDP13+RDP15+(2)FDP8	4432	2.8	6156		0		0		0		0	12312	1
[RW9+FW14+RDP13+RDP15+FDP8]+[FW14+RDP15+RDP16+FDP10]+[RW10+FW15] int.	-	0	-	0	RW9+FW14+RDP13+RDP15+FDP8	3836	2.8	5328	FW14+RDP15+RDP16+FDP10	4143	2.7	5524	RW10+FW15	4534	3.8	8533		0		0	19384	1
[FW14+RDP15+RDP16+FDP10]+[FW14+RDP15+RDP16+FDP10] int.	-	0	-	0	FW14+RDP15+RDP16+FDP10	4143	2.7	5524	FW14+RDP15+RDP16+FDP10	4143	2.7	5524		0		0		0		0	11048	1
[FW14+RDP15+RDP16+FDP10]+[RW5+FW14+RDP16+FDP10]+IGB2 int.	-	0	-	0	FW14+RDP15+RDP16+FDP10	4143	2.7	5524	RW5+FW14+RDP16+FDP10	4214	3.2	6672	IGB2	1656	5.6	4675		0		0	16871	1
[RW5+FW14+RDP16+FDP10]+[RW5+FW14+RDP16+FDP10] int.	-	0	-	0	RW5+FW14+RDP16+FDP10	4214	3.2	6672	RW5+FW14+RDP16+FDP10	4214	3.2	6672		0		0		0		0	13343	1
[RW5+FW14+RDP16+FDP10]+[RW5+FW14+RDP17+FDP2]+IGB2 int.	-	0	-	0	RW5+FW14+RDP16+FDP10	4214	3.2	6672	RW5+FW14+RDP17+FDP2	5366	3.2	8497	IGB2	1656	5.6	4675		0		0	19844	1
[RW5+FW14+RDP17+FDP2]+[RW5+FW14+RDP17+FDP2] int.	-	0	-	0	RW5+FW14+RDP17+FDP2	5366	3.2	8497	RW5+FW14+RDP17+FDP2	5366	3.2	8497		0		0		0		0	16993	1
[RW5+FW14+RDP17+FDP2]+[FW11+RDP17+FDP2+FDP3.2] int.	-	0	-	0	RW5+FW14+RDP17+FDP2	5366	3.2	8497	FW11+RDP17+FDP2+FDP3.2	5799	3.8	10874		0		0		0		0	19371	1
[FW11+RDP17+FDP2+FDP3]+[FW11+RDP17+FDP2+FDP3] int.	-	0	-	0	FW11+RDP17+FDP2+FDP3.2	5799	3.8	10874	FW11+RDP17+FDP2+FDP3.2	5799	2.5	7249		0		0		0		0	18123	1
[FW11+RDP17+FDP2+FDP3.2]+[FW11+RDP18+(3)FDP3.2+FDP4]+IGB2 int.	-	0	-	0	FW11+RDP17+FDP2+FDP3.2	5799	2.5	7249	FW11+RDP18+(3)FDP3.2+FDP4	5073	2.5	6341	IGB2	1656	5.5	4555		0		0	18145	1
[FW11+RDP18+(3)FDP3.2+FDP4]+[FW11+RDP18+(3)FDP3.2+FDP4]+BF6 int.	-	0	-	0	FW11+RDP18+(3)FDP3.2+FDP4	5073	2.5	6341	FW11+RDP18+(3)FDP3.2+FDP4	5073	3.0	7679	BF6	2678	3.0	4072		0		0	18092	1
[FW11+RDP18+(3)FDP3.2+FDP4]+[FW11+FDP3.1+(2)FDP3.2] int.	-	0	-	0	FW11+RDP18+(3)FDP3.2+FDP4	5073	3.0	7679	FW11+FDP3.1+(2)FDP3.2	3769	3.0	5707		0		0		0		0	13386	1
[FW11+FDP3.1+(2)FDP3.2]+[FW11+RDP19+FDP3.1+FDP3.2+FDP5]+IGB3 int.	-	0	-	0	FW11+FDP3.1+(2)FDP3.2	3769	3.0	5707	FW11+RDP19+FDP3.1+FDP3.2+FDP5	4482	3.0	6785	IGB3	1844	5.5	5070		0		0	17561	1

Cumulative Pt Name	Pt-1 Name	Pt Load-1 (lbs)	Pt-2 Name	Pt Load-2 (lbs)	GB-1 Wall Name	GB-1 Wall TL (plf)	GB-1 Length (ft)	GB-1 Rxn (lbs)	GB-2 Wall Name	GB-2 Wall TL (plf)	GB-2 Length (ft)	GB-2 Rxn (lbs)	GB-3 Wall Name	GB-3 Wall TL (plf)	GB-3 Length (ft)	GB-3 Rxn (lbs)	GB-4 Wall Name	GB-4 Wall TL (plf)	GB-4 Length (ft)	GB-4 Rxn (lbs)	Total Pt Load (lbs)	No. of 4" Piles
[FW11+RDP19+FDP3.1+FDP3.2+FDP5]+[FW11+RDP19+FDP3.1+FDP3.2+FDP5] int.	-	0	-	0	FW11+RDP19+FDP3.1+FDP3.2+FDP5	4482	3.0	6785	FW11+RDP19+FDP3.1+FDP3.2+FDP5	4482	3.0	6785		0		0		0		0	13569	1
[RW10+FW15+(2)RDP5]+[RW10+FW15+(2)RDP5] int.	-	0	-	0	RW10+FW15+(2)RDP5	4565	3.8	8591	RW10+FW15+(2)RDP5	4565	3.8	8591		0		0		0		0	17183	1
BFD+BFD+IGB2+BF4 int. + RP14+RP15+FP22+FP23	RP14+RP15	20700	FP22+FP23	9250	BFD	2532	6.1	7701	BFD	2532	2.0	5064	IGB2	1656	2.0	3243	BF4	2730	5.6	7621	53579	3
BF4+BF4 int. + RP13+FP24	RP13	20300	FP24	8100	BF4	2730	5.6	7621	BF4	2730	3.5	4749		0		0		0		0	40769	3
BF4+BF4	-	0	-	0	BF4	2730	3.5	4749	BF4	2730	3.5	4749		0		0		0		0	9497	1
BF6+BF6 int. + RP9+FP14	RP9	2800	FP14	5750	BF6	2678	6.6	8814	BF6	2678	6.1	8145		0		0		0		0	25509	2
BF6+BF6 int. + RP10+FP15	RP10	10000	FP15	5750	BF6	2678	6.6	8814	BF6	2678	3.2	4240		0		0		0		0	28804	2
BFG+BFG int. + RP7+FP9	RP7	8200	FP9	19400	BFG	2261	3.2	3580	BFG	2261	2.6	2920		0		0		0		0	34100	2
BFG+BFG+(2)IGB3 int.	-	0	-	0	BFG	2261	2.6	2920	BFG	2261	5.5	6218	IGB3	1844	1.0	1844	IGB3	1844	3.0	2804	13786	1
BFG+BFG int. + RP8+FP10	RP8	3400	FP10	17000	BFG	2261	5.5	6218	BFG	2261	3.7	4145		0		0		0		0	30763	2
BFG+BFG+BF8+IGB2 int. + RP2+FP12	RP2	6300	FP12	9400	BFG	2261	3.7	4145	BFG	2261	6.5	7348	BF8	2933	5.0	7332	IGB2	1656	0.9	1518	36044	2
BF8+BF8 int. + RP3+FP11	RP3	6900	FP11	9600	BF8	2933	5.0	7332	BF8	2933	9.0	13198		0	-4.5	0		0		0	37030	2
IGB3+IGB3 int.	-	0	-	0	IGB3	1844	7.0	6453	IGB3	1844	7.0	6453		0		0		0		0	12906	1
IGB2+IGB2 int. + FP25	-	0	FP25	12500	IGB2	1656	4.2	3451	IGB2	1656	4.7	3934		0		0		0		0	19884	1
IGB3+IGB3+IGB1 int.	-	0	-	0	IGB3	1844	5.6	5205	IGB3	1844	5.6	5205	IGB1	625	8.3	2604		0		0	13014	1

Pile Design - Intersection and Point Loads With Seismic

Pile Dia. (in)	Cap (tons)	ASD (lbs)
4	10	20000

FOR BF PT DISTRIBUTED (FOR BF-1)

Pt Name	DL (lbs)	SL/LL (lbs)	EQ (lbs)	Dist Over (ft)	DL (plf)	SL/LL (plf)	EQ (plf)	1.2DL+1.6(SL/LL) (plf)	Governing Load w/ EQ* (plf)	Governing Load (plf)
RDP1	3313	2200	8.5	12.8	258	171	0.66	583	417	583
RDP2	3313	2200	8.5	12.8	258	171	0.66	583	417	583

^^ since gravity governs for these, don't need to check piles for load combos with seismic

Intersection Name	BF Name/ Side	Pt Loads Dead (kips)	Pt Loads Live (kips)	GB Rxn Dead (kips)	GB Rxn Live (kips)	Total Dead (kips)	Total Live (kips)	Seismic (kips)	Governing Load w/ EQ* (kips)	No. of 4" Piles
[FW11+RDP4+(2)FDP1.2]+ BFD+[FW12+(2)FDP6] int. + RP16+FP21	BF-D, W	9.6	6.9	16.5	2.3	26.0	9.2	40.5	58.7	3
BFD+BFD+IGB2+BF4 int. + RP14+RP15+FP22+FP23	BF-D, E/ BF-4, N	13.9	16.0	21.2	2.4	35.1	18.5	72	91.3	5
BF4+BF4 int. + RP13+FP24	BF-4, S	16.9	11.5	14.9	2.2	31.8	13.7	31.5	62.4	4
BF6+BF6 int. + RP9+FP14	BF-6, N	5.4	3.2	22.0	3.1	27.4	6.3	33.8	55.5	3
BF6+BF6 int. + RP10+FP15	BF-6, S	9.7	6.1	15.1	2.2	24.8	8.2	33.8	52.6	3
BFG+BFG+BF8+IGB2 int. + RP2+FP12	BF-8, N	10.1	5.6	18.9	1.4	29.0	7.0	40.3	62.0	4
BF8+BF8 int. + RP3+FP11	BF-8, S	8.3	8.2	30.3	3.4	38.6	11.6	40.3	73.2	4
BFG+BFG int. + RP7+FP9	BF-G, W	17.2	10.4	6.3	0.2	23.5	10.6	67.3	74.4	4
BFG+BFG int. + RP8+FP10	BF-G, E	13.3	7.1	10.0	0.4	23.3	7.5	67.3	74.2	4

* 2 load cases were considered:
Load case #1- 1.1172D+0.525E+0.75L
Load case #2- 1.1641D+0.7E

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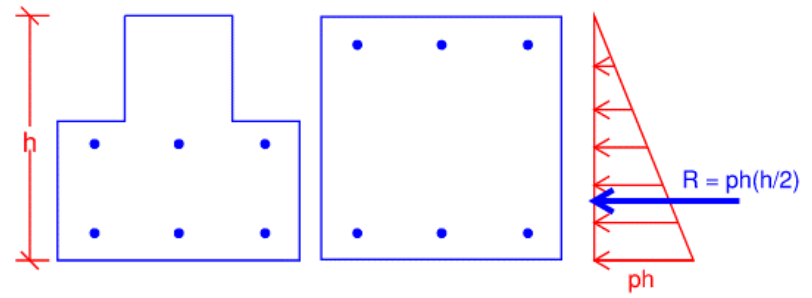
Lateral Loads for Sliding Check

SF =	1.2	w/ seismic
SF =	1.5	
passive press =	250	pcf, LRFD



garage is good by inspection for sliding since surrounded by concrete walls on all sides

main house lateral loads

base shear =	122.3	kips, ASD

rxns from the basement wall restrained at the SOG:

N rxn 1 is for the North wall at the garage entrance

soil rxn =	2.1	klf
traffic rxn =	0.5	klf
earthquake =	0.6	klf (LRFD)
N rxn 1 =	2.8	klf, includes active/traffic/seismic (ASD - 1.0H+0.75L+0.525E)
N rxn 1 =	2.6	klf, includes active/traffic (ASD - 1.0H+1.0L)

N rxn 2 is for the North walls, typ.

soil rxn =	4.3	klf
traffic rxn =	0.0	klf
earthquake =	0.7	klf (LRFD)
N rxn 2 =	4.6	klf, includes active/traffic/seismic (ASD - 1.0H+0.75L+0.525E)
N rxn 2 =	4.3	klf, includes active/traffic (ASD - 1.0H+1.0L)

Global Sliding Check - Pushing South (w/ Seismic)

pushing south will govern over pushing north (and will govern over pushing east/ west)

L w/ N rxn 1 =	21.8	ft	
L w/ N rxn 2 =	37.5	ft	
Tot sliding =	357	kips, ASD	
Tot GB L =	318	ft	
Sliding =	1120	plf	
Pressure =	208	pcf	
min h =	2.32	ft	<< use 2'-6"

Global Sliding Check - Pushing South (w/OUT Seismic)

L w/ N rxn 1 =	21.8	ft	
L w/ N rxn 2 =	37.5	ft	
Tot sliding =	217	kips, ASD	
Tot GB L =	318	ft	
Sliding =	681	plf	
Pressure =	167	pcf	
min h =	2.02	ft	<< 2'-6" OK

Grade Beam Design - 18"w x 12"dp w/ stem wall

Grade Beam Info

h =	12.0	in	
b =	18.0	in	
f'c =	4.0	ksi	
fy =	60.0	ksi	
# of bars =	3	top and bottom	
size of bar =	5		
area of bar =	0.31	in^2	
bar dia =	0.625	in	
cover =	3.0	in	
d =	8.7	in	

Pile Cap Punching Shear Check

pile capacity =	20	kips (Vu)	
pile cap L =	6.0	in	
pile cap W =	6.0	in	
c1 =	14.7	in	
c2 =	14.7	in	
b0 =	58.75	in	
ϕVn =	97	kips	OK

Moment and Shear Check

a =	0.91	in
ϕMn =	299	kip-in
	24.9	kip-ft
ϕVc =	14.8	kips

shear reinf:

min spacing =	6.0	in, ACI 18.13.3.2
Vu > 0.5ϕVc?	YES	so use ACI Table 9.6.3.3
min Av/s =	0.180	in^2/ft
use =	#3 @ 6"oc	(2 legs)
Av =	0.220	in^2
s =	6.0	in
have =	0.440	in^2/ft OK
ϕVs =	14.3	kips
ϕVn =	29.2	kips

Local Out of Plane Check

passive press =	250	pcf, LRFD	
depth =	2.50	ft	
w max =	781	plf	
max L =	20.0	ft	
Mu =	28.4	kip-ft, cont. span moment	
Vu =	15.6	kips, cont. span shear	
h =	18.0	in	
b =	12.0	in	
# of bars =	2		
size of bar =	5		
area of bar =	0.31	in^2	
bar dia =	0.625	in	
cover =	3.0	in	
d =	14.7	in	
a =	0.91	in	
ϕMn =	344	kip-in	
	28.7	kip-ft	OK
ϕVn =	16.7	kips	OK

Grade Beam Design - 18"w x 30"dp (no stem wall)

Grade Beam Info

h =	30.0	in	
b =	18.0	in	
f'c =	4.0	ksi	
fy =	60.0	ksi	
# of bars =	3	top and bottom	
size of bar =	5		
area of bar =	0.31	in^2	
bar dia =	0.625	in	
cover =	3.0	in	
d =	26.7	in	

Pile Cap Punching Shear Check

pile capacity =	20	kips (Vu)	
pile cap L =	6.0	in	
pile cap W =	6.0	in	
c1 =	32.7	in	
c2 =	32.7	in	
b0 =	130.75	in	
ϕVn =	662	kips	OK

Moment and Shear Check

a =	0.91	in
ϕMn =	951	kip-in
	79.3	kip-ft
ϕVc =	45.6	kips

shear reinf:

min spacing =	9.0	in, ACI 18.13.3.2
Vu > 0.5ϕVc?	NO	use some anyways so use ACI Table 9.6.3.3
min Av/s =	0.180	in^2/ft
use =	#3 @ 9"oc	(2 legs)
Av =	0.220	in^2
s =	9.0	in
have =	0.293	in^2/ft OK
ϕVs =	29.4	kips
ϕVn =	74.9	kips

Local Out of Plane Check

passive press =	250	pcf, LRFD	
depth =	2.50	ft	
w max =	781	plf	
max L =	24.0	ft	
Mu =	40.9	kip-ft, cont. span moment	
Vu =	18.8	kips, cont. span shear	
h =	18.0	in	
b =	30.0	in	
# of bars =	3		
size of bar =	5		
area of bar =	0.31	in^2	
bar dia =	0.625	in	
cover =	3.0	in	
d =	14.7	in	
a =	0.55	in	
ϕMn =	523	kip-in	
	43.6	kip-ft	OK
ϕVn =	41.8	kips	OK

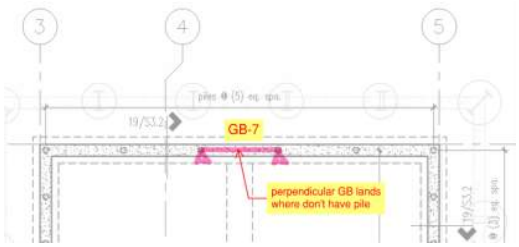
Grade Design - Distributed Load (Wall + Dist. Pt) with Point Load - LRFD

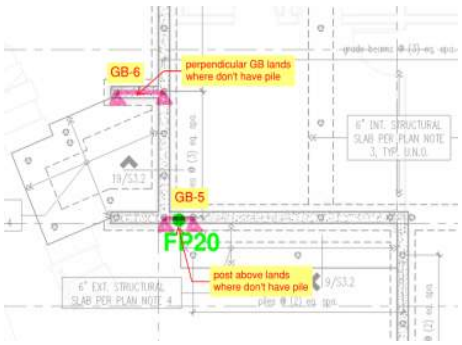
Ftg Name	Ftg Type	Moment Capacity (k-ft)	Conc. Shear Capacity (k)	Comb. Shear Capacity (k)
20/S3.2	18"w x 12"dp	24.88	14.84	29.17
19/S3.2	18"w x 12"dp	24.88	14.84	29.17
18/S3.2	18"w x 12"dp	24.88	14.84	29.17
15/S3.2	18"w x 12"dp	24.88	14.84	29.17
14/S3.2	18"w x 30"dp	79.29	45.57	74.93
13/S3.2	18"w x 30"dp	79.29	45.57	74.93
9/S3.2	18"w x 30"dp	79.29	45.57	74.93
BFD	18"w x 30"dp	79.29	45.57	74.93
BF4	18"w x 30"dp	79.29	45.57	74.93
BF6	18"w x 30"dp	79.29	45.57	74.93
BFG	18"w x 30"dp	79.29	45.57	74.93
BF8	18"w x 30"dp	79.29	45.57	74.93

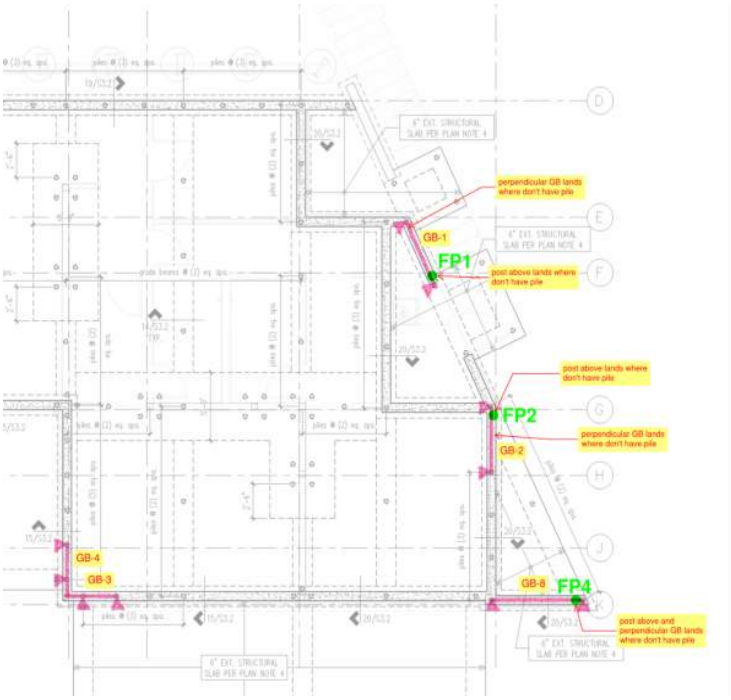
GRADE BEAMS WITH POINT LOAD

Design Name	Length (ft)	Total Fact Load (plf)	Total Fact Point Load-1 (lbs)	Point Load-1 Loc. from L (ft)	Total Fact Point Load-2 (lbs)	Point Load-2 Loc. from L (ft)	Ftg Name	Max Moment (k-ft)	Moment OK?	Max Shear (k)	Conc. Shear OK?	Vu > 0.5ϕVc?	Comb. Shear OK?
GB-1	5.25	1442	2954	0.33	9240	4.50	20/S3.2	9.37	OK	11.90	OK	YES	OK
GB-2	4.92	2363	6076	0.67	0	0.00	20/S3.2	9.31	OK	11.10	OK	YES	OK
GB-3*	8.92	1095	0	0.00	0	0.00	15/S3.2	7.04	OK	17.25	NG!!	YES	OK
GB-4***	4.44	1693	0	0.00	0	0.00	15/S3.2	6.58	OK	15.36	NG!!	YES	OK
GB-5	1.75	600	13582	0.92	0	0.00	15/S3.2	6.16	OK	7.64	OK	YES	OK
GB-6	2.92	1903	3533	1.33	0	0.00	20/S3.2	4.56	OK	4.69	OK	NO	OK
GB-7	4.52	3907	3258	2.26	0	0.00	19/S3.2	13.60	OK	10.50	OK	YES	OK
GB-8	7.33	798	6852	6.58	0	0.00	20/S3.2	8.24	OK	9.08	OK	YES	OK

* for GB-3, total factored load of 1095 only occurs on backspan (total factored load on cantilever = 10349 plf)
*** for GB-4, total factored load of 1693 only occurs on backspan (total factored load on cantilever = 8485 plf)







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Grade Design - Distributed Load (Wall + Dist. Pt) - LRFD

Ftg Name	Ftg Type	Moment Capacity (k-ft)	Conc. Shear Capacity (k)	Comb. Shear Capacity (k)
20/S3.2	18"w x 12"dp	24.88	14.84	29.17
19/S3.2	18"w x 12"dp	24.88	14.84	29.17
18/S3.2	18"w x 12"dp	24.88	14.84	29.17
15/S3.2	18"w x 12"dp	24.88	14.84	29.17
14/S3.2	18"w x 30"dp	79.29	45.57	74.93
13/S3.2	18"w x 30"dp	79.29	45.57	74.93
9/S3.2	18"w x 30"dp	79.29	45.57	74.93
BFD	18"w x 30"dp	79.29	45.57	74.93
BF4	18"w x 30"dp	79.29	45.57	74.93
BF6	18"w x 30"dp	79.29	45.57	74.93
BFG	18"w x 30"dp	79.29	45.57	74.93
BF8	18"w x 30"dp	79.29	45.57	74.93

GRADE BEAMS WITH DISTRIBUTED LOAD

Intersection Name	Length (ft)	Total Fact Load (plf)	Ftg Name	Moment (k-ft)	Moment OK?	Shear (k)	Conc. Shear OK?	Vu > 0.5ϕVc?	Comb. Shear OK?
BF4	5.58	1616	BF4	6.30	OK	10.31	OK	NO	OK
BF6	6.58	1547	BF6	8.38	OK	11.64	OK	NO	OK
BF8	5.00	1387	BF8	4.34	OK	7.93	OK	NO	OK
BFD	6.08	855	BFD	3.95	OK	5.94	OK	NO	OK
BFG	6.50	495	BFG	2.62	OK	3.68	OK	NO	OK
EGB1	12.75	602	9/S3.2	12.24	OK	8.78	OK	NO	OK
EGB2	8.46	891	9/S3.2	7.97	OK	8.61	OK	NO	OK
EGB3	6.42	1584	14/S3.2	8.15	OK	11.62	OK	NO	OK
EGB4	7.92	198	9/S3.2	1.55	OK	1.79	OK	NO	OK
FW1.1+FDP9	3.75	2504	19/S3.2	4.40	OK	10.73	OK	YES	OK
FW1.2	5.25	1442	20/S3.2	4.97	OK	8.66	OK	YES	OK
FW1.3	5.00	1653	20/S3.2	5.16	OK	9.45	OK	YES	OK
FW10+RDP1+RDP20	2.47	6581	19/S3.2	5.03	OK	18.60	NG!!	YES	OK
FW10+RDP2+RDP3	3.69	4905	19/S3.2	8.34	OK	20.67	NG!!	YES	OK
FW11+FDP3.1+(2)FDP3.2	3.03	4287	19/S3.2	4.91	OK	14.84	NG!!	YES	OK
FW11+RDP17+FDP2+FDP3.2	3.75	6606	19/S3.2	11.61	OK	28.31	NG!!	YES	OK
FW11+RDP18+(3)FDP3.2+FDP4	3.03	6071	19/S3.2	6.96	OK	21.01	NG!!	YES	OK
FW11+RDP19+FDP3.1+FDP3.2+FDP5	3.03	5290	19/S3.2	6.06	OK	18.31	NG!!	YES	OK
FW11+RDP19+FDP5	4.33	3728	19/S3.2	8.75	OK	18.47	NG!!	YES	OK
FW11+RDP3+RDP4+(2)FDP1.1	3.19	5736	19/S3.2		OK	20.94	NG!!	YES	OK
FW11+RDP4+(2)FDP1.2	3.50	4019	19/S3.2	6.15	OK	16.08	NG!!	YES	OK

Intersection Name	Length (ft)	Total Fact Load (plf)	Ftg Name	Moment (k-ft)	Moment OK?	Shear (k)	Conc. Shear OK?	Vu > 0.5ϕVc?	Comb. Shear OK?
FW11+RDP4+FDP1.2+(2)FDP 1.1	3.19	5148	19/S3.2	6.57	OK	18.80	NG!!	YES	OK
FW12+(2)FDP6+FDP1.3	3.17	3347	19/S3.2	4.20	OK	12.11	OK	YES	OK
FW12+(3)FDP8	2.78	4123	19/S3.2	3.98	OK	13.09	OK	YES	OK
FW12+RDP6+FDP6+FDP7	3.17	5031	19/S3.2	6.31	OK	18.21	NG!!	YES	OK
FW14+RDP15+RDP16+FDP1	2.67	4839	19/S3.2	4.30	OK	14.75	OK	YES	OK
FW2.1	4.56	1126	20/S3.2	2.92	OK	5.87	OK	NO	OK
FW2.2	4.67	2404	20/S3.2	6.54	OK	12.82	OK	YES	OK
FW3	8.08	798	20/S3.2	6.52	OK	7.37	OK	NO	OK
FW4	9.08	1293	15/S3.2	13.34	OK	13.42	OK	YES	OK
FW5.1	4.64	1693	15/S3.2	4.56	OK	8.98	OK	YES	OK
FW5.2	3.39	1194	15/S3.2	1.71	OK	4.62	OK	NO	OK
FW5.3	3.42	600	15/S3.2	0.88	OK	2.34	OK	NO	OK
FW7	7.92	600	15/S3.2	4.70	OK	5.43	OK	NO	OK
FW8	4.67	1055	15/S3.2	2.87	OK	5.63	OK	NO	OK
FW9	3.00	1903	20/S3.2	2.14	OK	6.53	OK	NO	OK
IGB2	9.00	1328	14/S3.2	13.45	OK	13.66	OK	NO	OK
IGB3	7.00	1577	14/S3.2	9.66	OK	12.62	OK	NO	OK
RW10+FW15	3.76	5195	18/S3.2	9.20	OK	22.35	NG!!	YES	OK
RW10+FW15+(2)RDP5	3.76	5237	18/S3.2	9.27	OK	22.53	NG!!	YES	OK
RW5+FW14+RDP16+FDP10	3.17	4912	19/S3.2	6.16	OK	17.78	NG!!	YES	OK
RW5+FW14+RDP17+FDP2	3.17	6448	19/S3.2	8.08	OK	23.34	NG!!	YES	OK
RW7+FW12+RDP10+RDP11+(2)FDP8	2.78	5131	19/S3.2	4.95	OK	16.29	NG!!	YES	OK
RW7+FW12+RDP5+RDP6+R	2.67	5685	19/S3.2	5.05	OK	17.33	NG!!	YES	OK
RW7+FW12+RDP5+RDP7+FDP8	2.78	3612	19/S3.2	3.48	OK	11.47	OK	YES	OK
RW7+FW12+RDP7+RDP8+(2)FDP8	4.17	4625	19/S3.2	10.04	OK	22.03	NG!!	YES	OK
RW7+FW12+RDP8+RDP9+(2)FDP8	2.78	4928	19/S3.2	4.75	OK	15.65	NG!!	YES	OK
RW8+FW13+RDP11	4.52	4387	19/S3.2	11.19	OK	22.65	NG!!	YES	OK
RW8+FW13+RDP12	4.52	4387	19/S3.2	11.19	OK	22.65	NG!!	YES	OK
RW9+FW14+(2)RDP13+(2)FD	2.78	5092	19/S3.2	4.91	OK	16.17	NG!!	YES	OK
RW9+FW14+RDP12+RDP13+FDP8	2.78	4719	19/S3.2	4.55	OK	14.98	NG!!	YES	OK
RW9+FW14+RDP13+RDP14+	2.78	5891	19/S3.2	5.68	OK	18.70	NG!!	YES	OK
RW9+FW14+RDP13+RDP15+	2.78	5111	19/S3.2	4.93	OK	16.23	NG!!	YES	OK
RW9+FW14+RDP13+RDP15+	2.78	4319	19/S3.2	4.17	OK	13.71	OK	YES	OK
RW9+FW14+RDP14+(3)FDP8	2.78	5830	19/S3.2	5.62	OK	18.51	NG!!	YES	OK

Pile Cap Design

The pile cap is located below point "FW7+FW5.2+IGB2+BF4 int. + RP11+FP16"

total dead load (excluding pile wt) = 69.462 kips
total live load = 20.843 kips
total snow load = 10.700 kips
1.2D+1.6L+0.5S = 108.161 kips << governs
1.2D+1.0L+1.6S = 107.425 kips
load to 1 pile = 21.632 kips LRFD

Assume pile cap is 2'-6" deep with the top of the pile 4" above the bottom of the pile cap and reinforcement is 3" above the bottom of the pile cap -- d = 27"

f'c = 4000 psi
fy = 60 ksi

ONE-WAY SHEAR

3 piles fall within the tributary area
Vu = 21.632*3 = 64.9 kips

bw = 42 in

so vc = 2*sqrt(4000)*42*27 = 143 kips
fvc = 0.75*143 = 107 kips > 64.9 kips

TWO-WAY SHEAR

all 5 piles fall within the tributary area
Vu = 108.2 kips

b = 7/5 = 1.4 --> 2+(4/1.4) = 4.86
as = 40
b0 = 2*(13.5+7+13.5)+ 2*(13.5+5+13.5)
b0 = 68+64 = 132 in
as*d/b0 = 40*27/132 = 8.18

so vc = 4*sqrt(4000)*132*27 = 901 kips
fvc = 0.75*901 = 676 kips > 108.2 kips

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Pile Cap Design

FLEXURE

span = 4 ft
pt load (108.2 kips) at 2 ft
moment = 108.2 k-ft

Try (5)#5 ea way, top and bottom --> As = 1.55 in^2

a = 1.55*60/0.85*4*3.5*12 = 0.65 in
Mn = 1.55*60*(27-(0.65/2)) = 206.7 kip-ft
ϕMn = 0.9*206.7 = 186 kip-ft > 108.2 kip-ft

MIN REINF

0.0018*4*12*27 = 2.33 in^2
have 1.55 in^2 top and bottom = 3.10 in^2 > 2.33 in^2

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Site Retaining Walls

Project Name/Number : 8480 wu driveway wall
Title **Retaining Wall Schedule**
Dsgnr: **LAN** Date: 24 APR 2017
Description....
4'-6" Ret Wall w/ Key (passive includes 1.5 SF)

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

Cantilevered Retaining Wall

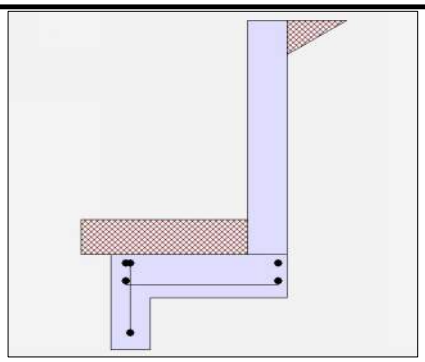
Code: IBC 2018,ACI 318-14,TMS 402-16

Criteria

Retained Height	=	4.50 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	8.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	130.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.000
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	1.78	OK
Sliding	=	1.15	Ratio < 1.5!
Total Bearing Load	=	927 lbs	
...resultant ecc.	=	7.79 in	
Soil Pressure @ Toe	=	726 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,017 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	8.5 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	94.9 psi	

Sliding Calcs

Lateral Sliding Force	=	568.9 lbs	
less 100% Passive Force	= -	656.3 lbs	
less 100% Friction Force	= -	0.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	197.1 lbs	NG

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2018,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

		Bottom
		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Design Method	=	LRFD
Thickness	=	8.00
Rebar Size	=	# 5
Rebar Spacing	=	12.00
Rebar Placed at	=	Edge
Design Data		
fb/FB + fa/Fa	=	**0.116**
Total Force @ Section		
Service Level	lbs =	
Strength Level	lbs =	648.0
Moment....Actual		
Service Level	ft-# =	
Strength Level	ft-# =	972.0
Moment.....Allowable	=	8,312.6
Shear.....Actual		
Service Level	psi =	
Strength Level	psi =	8.7
Shear.....Allowable	psi =	94.9
Anet (Masonry)	in2 =	
Rebar Depth 'd'	in =	6.19
Masonry Data		
f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Wall Weight	psf =	100.0
Short Term Factor	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	Medium Weight
Masonry Design Method	=	ASD
Concrete Data		
f'c	psi =	4,000.0
Fy	psi =	60,000.0

Site Retaining Walls

Project Name/Number : 8480 wu driveway wall
Title **Retaining Wall Schedule**
Dsgnr: **LAN** Date: 24 APR 2017
Description....
4'-6" Ret Wall w/ Key (passive includes 1.5 SF)

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0368 in2/ft		
(4/3) * As :	0.0491 in2/ft	Min Stem T&S Reinf Area 0.864 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.31 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	1.3411 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	2.33 ft
Heel Width	=	0.67
Total Footing Width	=	3.00
Footing Thickness	=	10.00 in
Key Width	=	8.00 in
Key Depth	=	12.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,017	0	psf
Mu' : Upward	=	23,041	0	ft-#
Mu' : Downward	=	8,274	0	ft-#
Mu: Design	=	1,231	0	ft-#
Actual 1-Way Shear	=	8.53	0.03	psi
Allow 1-Way Shear	=	94.87	50.60	psi

Toe Reinforcing = # 5 @ 12.00 in
Heel Reinforcing = None Spec'd
Key Reinforcing = # 4 @ 12.00 in
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 11.11 in, #5@ 17.22 in, #6@ 24.44 in, #7@ 33.33 in, #8@ 43.88 in, #9@ 5
Heel: phiMn = phi'5'lambda'sqrt(fc)'Sm
Key: #4@ 13.88 in, #5@ 18 in, #6@ 18 in, #7@ 18 in, #8@ 18 i

Min footing T&S reinf Area 0.65 in2
Min footing T&S reinf Area per foot 0.22 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 11.11 in	#4@ 22.22 in
#5@ 17.22 in	#5@ 34.44 in
#6@ 24.44 in	#6@ 48.89 in

Site Retaining Walls

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

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Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Summary of Overturning & Resisting Forces & Moments

Item	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#	.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	568.9	1.78	1,011.4			
HL Act Pres (be water tbl)						
Hydrostatic Force						
Buoyant Force =						
Surcharge over Heel =						
Surcharge Over Toe =						
Adjacent Footing Load =						
Added Lateral Load =						
Load @ Stem Above Soil =						
=						
Soil Over HL (ab. water tbl)				2.0	3.00	5.8
Soil Over HL (bel. water tbl)					3.00	5.8
Watre Table						
Sloped Soil Over Heel =						
Surcharge Over Heel =						
Adjacent Footing Load =						
Axial Dead Load on Stem =						
* Axial Live Load on Stem =						
Soil Over Toe =					1.17	
Surcharge Over Toe =						
Stem Weight(s) =				450.0	2.66	1,198.5
Earth @ Stem Transitions =						
Footing Weight =				375.0	1.50	562.5
Key Weight =				100.0	0.33	33.3
Vert. Component =						
Total =	568.9	**O.T.M.** =	1,011.4	927.0 lbs	**R.M.=**	1,800.2

Resisting/Overturning Ratio = **1.78**
Vertical Loads used for Soil Pressure = 927.0 lbs

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

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.030	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Site Retaining Walls

Project Name/Number : 8480 wu driveway wall
Title **Retaining Wall Schedule**
Dsgnr: **DMR** Date: 24 APR 2017
Description....
4'-6" Ret Wall w/ Key w/ Seismic (passive includes 1.5 SF)

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

Cantilevered Retaining Wall

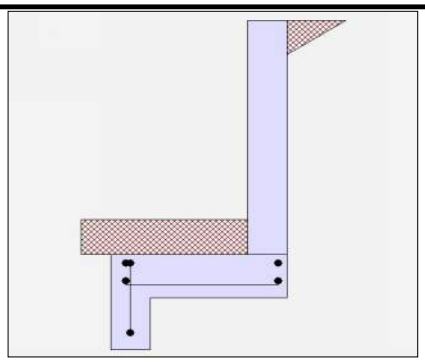
Code: IBC 2018,ACI 318-14,TMS 402-16

Criteria

Retained Height	=	4.50 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	8.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,667.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	130.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Earth Pressure Seismic Load

Method : Uniform
Multiplier Used = 9.000
(Multiplier used on soil density)

Uniform Seismic Force = 48.000
Total Seismic Force = 256.000

Design Summary

Wall Stability Ratios

Overturning	=	1.20	Ratio < 1.5!
Sliding	=	1.31	Ratio < 1.5!
Total Bearing Load	=	925 lbs	
...resultant ecc.	=	14.04 in	
Soil Pressure @ Toe	=	1,876 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,667 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	2,626 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	10.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	75.0 psi	

Sliding Calcs

Lateral Sliding Force	=	748.1 lbs	
less 100% Passive Force	= -	656.3 lbs	
less 100% Friction Force	= -	323.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	142.3 lbs	NG

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2018,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

	Bottom
	Stem OK
Design Height Above Ftg ft =	0.00
Wall Material Above "Ht" =	Concrete
Design Method =	LRFD
Thickness =	8.00
Rebar Size =	# 5
Rebar Spacing =	12.00
Rebar Placed at =	Edge
Design Data	
fb/FB + fa/Fa =	**0.179**
Total Force @ Section	
Service Level lbs =	
Strength Level lbs =	864.0
Moment....Actual	
Service Level ft-# =	
Strength Level ft-# =	1,458.0
Moment.....Allowable =	8,121.3
Shear.....Actual	
Service Level psi =	
Strength Level psi =	11.6
Shear.....Allowable psi =	75.0
Anet (Masonry) in2 =	
Rebar Depth 'd' in =	6.19
Masonry Data	
f'm psi =	
Fs psi =	
Solid Grouting =	
Modular Ratio 'n' =	
Wall Weight psf =	100.0
Short Term Factor =	
Equiv. Solid Thick. =	
Masonry Block Type =	Medium Weight
Masonry Design Method =	ASD
Concrete Data	
f'c psi =	2,500.0
Fy psi =	60,000.0

Site Retaining Walls

Project Name/Number : 8480 wu driveway wall
Title **Retaining Wall Schedule**
Dsgnr: **DMR** Date: 24 APR 2017
Description....
4'-6" Ret Wall w/ Key w/ Seismic (passive includes 1.5 SF)

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0552 in2/ft		
(4/3) * As :	0.0736 in2/ft	Min Stem T&S Reinf Area 0.864 in2	
200bd/fy : 200(12)(6.1875)/60000 :	0.2475 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft	
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in	#4@ 25.00 in
Provided Area :	0.31 in2/ft	#5@ 19.38 in	#5@ 38.75 in
Maximum Area :	0.8382 in2/ft	#6@ 27.50 in	#6@ 55.00 in

Footing Data

Toe Width	=	2.33 ft
Heel Width	=	0.67
Total Footing Width	=	3.00
Footing Thickness	=	10.00 in
Key Width	=	8.00 in
Key Depth	=	12.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	2,626	0 psf
Mu' : Upward	=	31,087	0 ft-#
Mu' : Downward	=	8,274	0 ft-#
Mu: Design	=	1,901	0 ft-#
Actual 1-Way Shear	=	9.98	0.03 psi
Allow 1-Way Shear	=	75.00	40.00 psi

Toe Reinforcing = # 5 @ 12.00 in
Heel Reinforcing = None Spec'd
Key Reinforcing = # 4 @ 12.00 in
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 11.11 in, #5@ 17.22 in, #6@ 24.44 in, #7@ 33.33 in, #8@ 43.88 in, #9@ 5
Heel: phiMn = phi'5'lambda'sqrt(fc)'Sm
Key: #4@ 13.88 in, #5@ 18 in, #6@ 18 in, #7@ 18 in, #8@ 18 i

Min footing T&S reinf Area 0.65 in2
Min footing T&S reinf Area per foot 0.22 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 11.11 in	#4@ 22.22 in
#5@ 17.22 in	#5@ 34.44 in
#6@ 24.44 in	#6@ 48.89 in

Site Retaining Walls

Project Name/Number : 8480 wu driveway wall
Title **Retaining Wall Schedule**
Dsgnr: **DMR** Date: 24 APR 2017
Description....
4'-6" Ret Wall w/ Key w/ Seismic (passive includes 1.5 SF)

This Wall in File: K:\2021\01519-2021-09 8480 Residence\Engineering\Gravity\retaining and basement w

Cantilevered Retaining Wall

Code: IBC 2018,ACI 318-14,TMS 402-16

Summary of Overturning & Resisting Forces & Moments

Item	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#	.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)	568.9	1.78	1,011.4			
HL Act Pres (be water tbl)						
Hydrostatic Force						
Buoyant Force =						
Surcharge over Heel =						
Surcharge Over Toe =						
Adjacent Footing Load =						
Added Lateral Load =						
Load @ Stem Above Soil =						
Seismic Earth Load =	179.2	2.67	477.9			
=						
Total =	748.1	**O.T.M.** =	1,489.2			
Soil Over HL (ab. water tbl)						
Soil Over HL (bel. water tbl)						
Watre Table						
Sloped Soil Over Heel =						
Surcharge Over Heel =						
Adjacent Footing Load =						
Axial Dead Load on Stem =						
* Axial Live Load on Stem =						
Soil Over Toe =					1.17	
Surcharge Over Toe =						
Stem Weight(s) =				450.0	2.66	1,198.5
Earth @ Stem Transitions =						
Footing Weight =				374.6	1.50	561.3
Key Weight =				100.0	0.33	33.3
Vert. Component =						
Total =				924.6 lbs	**R.M.=**	1,793.1

Resisting/Overturning Ratio = **1.20**
Vertical Loads used for Soil Pressure = 924.6 lbs

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

If seismic is included, the OTM and sliding ratios may be 1.1 per section 1807.2.3 of IBC.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.078	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.