Structural Calculations For:

8480 Residence

8480 85th Ave SE

Mercer Island, WA 98040

Prepared for: Brandt Design Group

Job #: 01519-2021-09

Date: February 25, 2022

STRUCTURAL ENGINEERING

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Table of Contents

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 =	103 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			Brick Veneer Wall		
Roofing	4.6	psf	Pavers (3cm)	15.6	psf	2x6 Studs @ 16"o.c.	1.5	psf
1/2" Sheathing	1.8	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	psf	Misc./Mech.	2	psf	5/8" GWB	2.8	psf
Ceiling	4.3	psf	Ceiling Finish	2.8	psf	Stone Veneer (1" max)	15.0	psf
(N) Solar Panels & Misc	6.0	psf		25.575		Misc./Mech.	2.0	psf
	38.7	psf	Use	25.0	psf		24.6	psf
Use	40.0	psf				Use	25.0	psf

Main Floor			Typical Interior Wall			Typical Exterior Wall		
Floor Finish	4.0	psf	2x6 Studs@16"oc	1.5	psf	2x6 Studs @ 16"o.c.	1.5	psf
4.5" Concrete	65.6	psf	1/2" plywood	1.8	psf	1/2" plywood	1.8	psf
1.5" Metal Deck	2.69	psf	(2) 5/8" GWB	5.5	psf	Insulation	1.5	psf
Steel Framing	10	psf	ock Wool Insulation	1.3	psf	5/8" GWB	2.8	psf
Ceiling	2.8	psf	Misc./Mech.	1.5	psf	Siding	2.5	psf
Misc./Mech.	1.5	psf		11.6	psf	Misc./Mech.	1.5	psf
	86.6	psf	Use	12.0	psf		11.6	psf
Use	90.0	psf				Use	12.0	psf

Trellis Roof			Concrete Wall			Exterior Glazing		
Rafters @ 24"oc	2.5	psf	8" Concrete	100	psf			
Misc./Mech.	5.0	psf	Brick Veneer Wall	24.6	psf			
	7.5	psf		125	**psf**	Window glass, frames	15	psf
Use	10.0	psf	Use	125.0	psf	Use	15.0	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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8480 Residence		DATE	2/25/2022
Criteria		PROJ. #	01519-2021-09
		DESIGN	HAA/SRW
		SHEET	1

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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	

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	23.5 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
	0.234 Controls	Eq. 12.8-2
C_s	0.535	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	583.7 k	
V = C_SW*	102.6 k	Eq. 12.8-1, Strength Level Base Shear
V = C_{Sasd}W*	71.8 k	Eq. 12.8-1 ASD Base Shear

*75% base shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	1.2
F_v	

$T_a = C_t h_n^x$ Eq. 12.8.7

$S_{MS} = F_a S_S$ Eq. 11.4-1

$S_{M1} = F_v S_1$ Eq. 11.4-2

$S_{DS} = {}^2/_3\, S_{MS}$ Eq. 11.4-3

$S_{D1} = {}^2/_3\, S_{M1}$ Eq. 11.4-4

$C_S = \frac{S_{DS}}{(R/I_e)}$ Eq. 12.8-2

$C_S = \frac{S_{D1}}{T(R/I_e)}$ Eq. 12.8-3

$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)}$ Eq. 12.8-4

$C_S \geq 0.044 S_{DS} I_e$ Eq. 12.8-5

$C_S \geq 0.01$ Eq. 12.8-5

$C_S \geq 0.5 \frac{S_1}{(R/I_e)}$ Eq. 12.8-6

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

$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i}\, w_{px}$ Eq. 12.10-1

$F_{px} \geq 0.2 S_{DS} I_e w_{px}$ Eq. 12.10-2

$F_{px} \leq 0.4 S_{DS} I_e w_{px}$ 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	23.5	161	23.5	3774	0.448	59.7	**59.7**	45.9	37.6	75.3	**45.9**	0.77
Main Floor	11.0	423	11.0	4655	0.552	73.7	**133.4**	74.4	99.2	198.4	**99.2**	1.35
Σ		583.7		8428		**133.4**						

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8480 Residence		DATE	2/25/2022
Seismic Criteria		PROJ. #	01519-2021-09
		DESIGN	HAA/SRW
		SHEET	2

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=	0.92	Table 26.10-1
K_e=	1.00	Table 26.9-1
G=	0.85	26.9.4

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.00	
Roof Type	Hip	
Roof Angle - Transverse Dir	14	degrees
Roof Angle - Long Dir	14	degrees
Ground to top of roof	24.25	ft
Bot of roof to top of roof	3.92	ft
Mean Roof Height, h	22.29	ft
Short Plan Dimension	37.58	ft
Long Plan Dimension	100.42	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade		ft

Velocity Pressure at Mean Roof Height, q_h =	19.3	psf

Transverse Wind Pressures

L/B = 0.37 h/L = 0.59

Pressure Coefficients from Figure 27.3-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.8 / -0.18
Leeward Roof	-0.52

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	8.19	**20.2**
15-20	0.9	18.78	12.77	8.19	**21.0**
20-25	0.94	19.62	13.34	8.19	**21.5**
25-30	0.98	20.45	13.91	8.19	**22.1**
30-40	1.04	21.71	14.76	8.19	**22.9**
41-50	1.09	22.75	15.47	8.19	**23.7**
51-60	1.13	23.59	16.04	8.19	**24.2**
61-70	1.17	24.42	16.61	8.19	**24.8**
71-80	1.21	25.26	17.17	8.19	**25.4**
81-90	1.24	25.88	17.60	8.19	**25.8**
91-100	1.26	26.30	17.88	8.19	**26.1**

Roof Pressures (Unfactored) Strength

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.9	-13.1	-8.6	**8.00**

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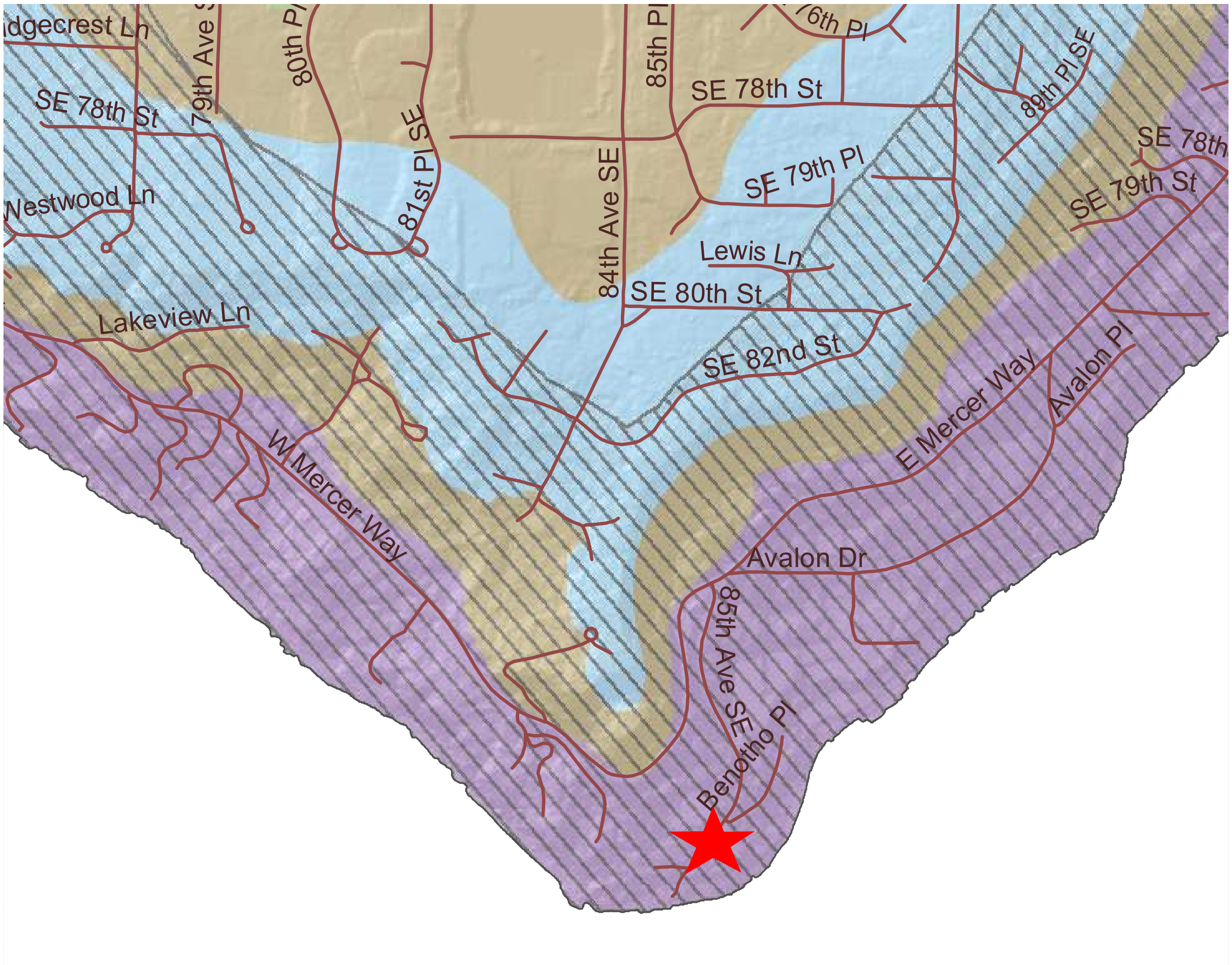
Wind Exposure and Kzt



WIND EXPOSURE CATEGORIES:

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

WIND SPEED-UP (TOPOGRAPHIC EFFECT) - $K_z t$ Factor :

$K_z t$ Factor
$K_z t = 1.0$
$K_z t = 1.3$
$K_z t = 1.6$
$K_z t = 1.9$

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PROJECT	8480 Residence
DATE	2/23/2022
PROJ. #	01519-2021-09
DESIGN	LAN
SHEET	L-4

Seismic Weight

Level	Area 1	A_{TOTAL}	$W_{AREA\,1}$	Area 2	A_{TOTAL}	$W_{AREA\,2}$	Area 3	A_{TOTAL}	$W_{AREA\,3}$	W_{AREA}
	h									
		(ft^2)	(psf)		(ft^2)	(psf)		(ft^2)	(psf)	(k)
Roof	Roof	2962	46	Glass Roof	234	15			0	139.76
Main Floor	Main Floor	2582	102	Trellis Roof	380	10	Deck	885	25	289.29
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			0			0	0.00
			0			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	Exterior Glazing	15	218.0	4.8	Typical Exterior Wal	12	38.6	4.8		0		4.8
Main Floor	Exterior Glazing	15	218.0	4.8	Typical Exterior Wal	12	38.6	4.8	Concrete Wall	125	110.3	5.5
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)	$W_{DIST\ TO\ LVL}$ (k)
Columns	25	26	4.8		0		4.8		0		4.8	20.82	161
Columns	25	26	10.3	ck Veneer W	25	62	10.3	xterior Glazir	15	115	10.3	133.86	423
	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

Wall DL			Area DL			Self Wt (included in ETABS)	
Typical Interior Wall	12	psf	Roof	46	psf		psf
Typical Exterior Wall	12	psf	Main Floor	102	psf		psf
Brick Veneer Wall	25	psf	Trellis Roof	10	psf		psf
Exterior Glazing	15	psf	Deck	25	psf		psf
Concrete Wall	125	psf	Glass Roof	15	psf		psf
Columns	25	plf			psf		psf
Conc+Siding	105	psf			psf		psf
		psf			psf		psf
		psf			psf		psf
		psf			psf		psf
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		psf			psf		psf
		psf			psf		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

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2/23/22
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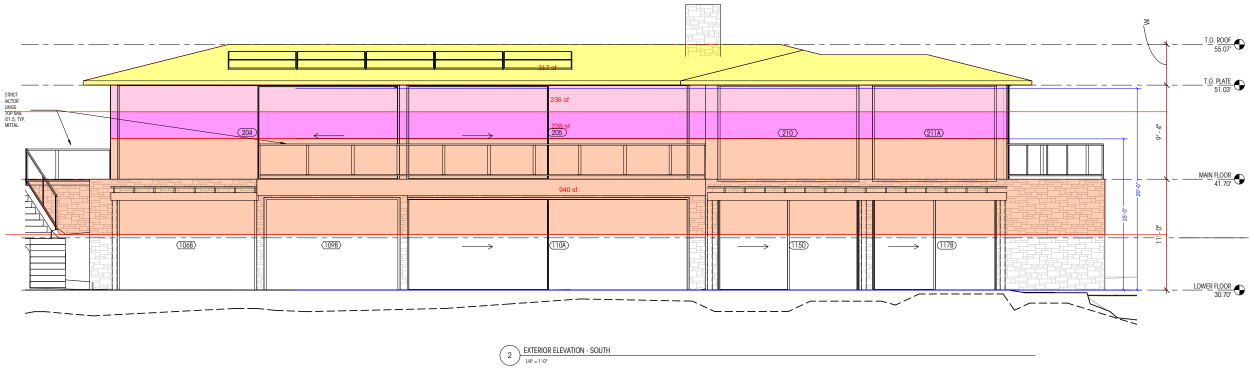
Wind Base Shear Calc - NS Direction



South elevation governs for NS direction

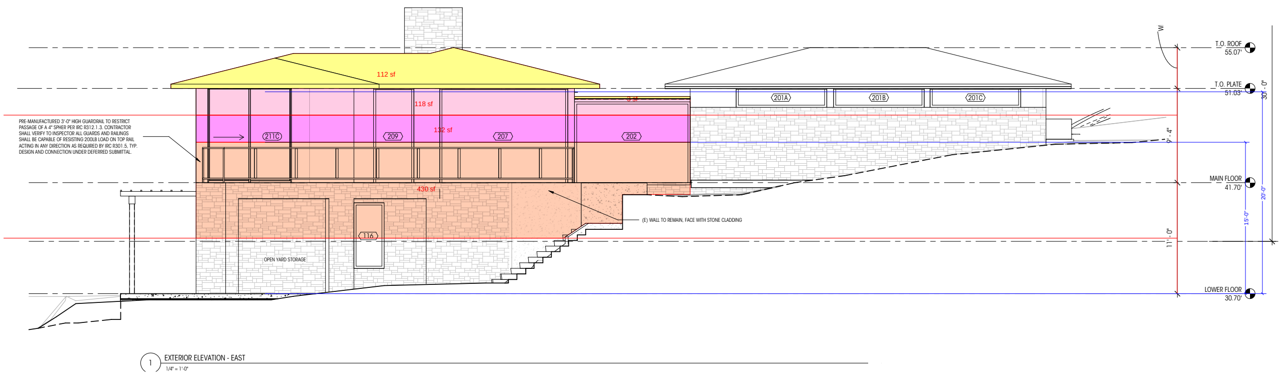
ROOF
roof: 317*8.0 = 2536 lbs
15'-0" < wall < 20'-0": 236*21.1 = 4980 lbs
total: 7516 lbs

FLOOR
15'-0" < wall < 20'-0": 238*21.1 = 5022 lbs
wall < 15'-0": 940*20.2 = 18988 lbs
total: 24010 lbs

TOTAL WIND BASE SHEAR
31.5 kips

seismic governs NS

Wind Base Shear Calc - EW Direction



East elevation governs for EW direction

ROOF
roof: 115*8.0 = 920 lbs
15'-0" < wall < 20'-0": 118*21.1 = 2490 lbs
total: 3410 lbs

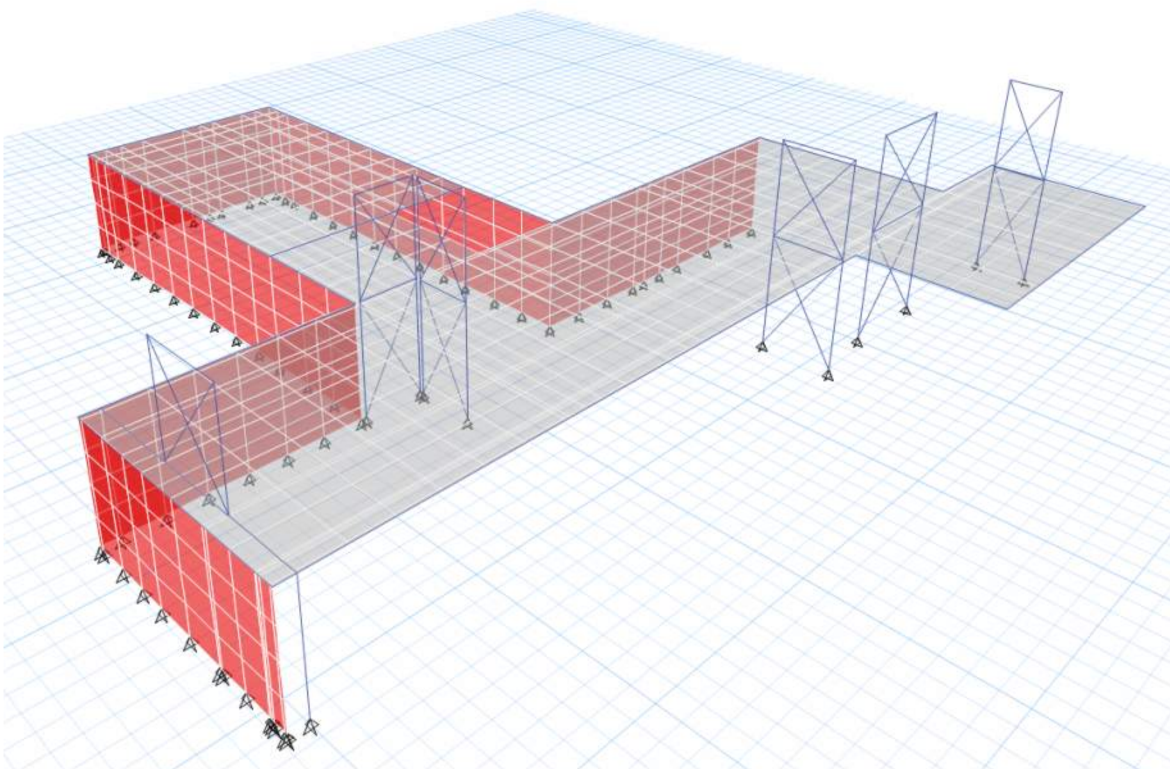
FLOOR
15'-0" < wall < 20'-0": 132*21.1 = 2785 lbs
wall < 15'-0": 430*20.2 = 8686 lbs
total: 11471 lbs

TOTAL WIND BASE SHEAR
14.9 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 transfer beam. 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

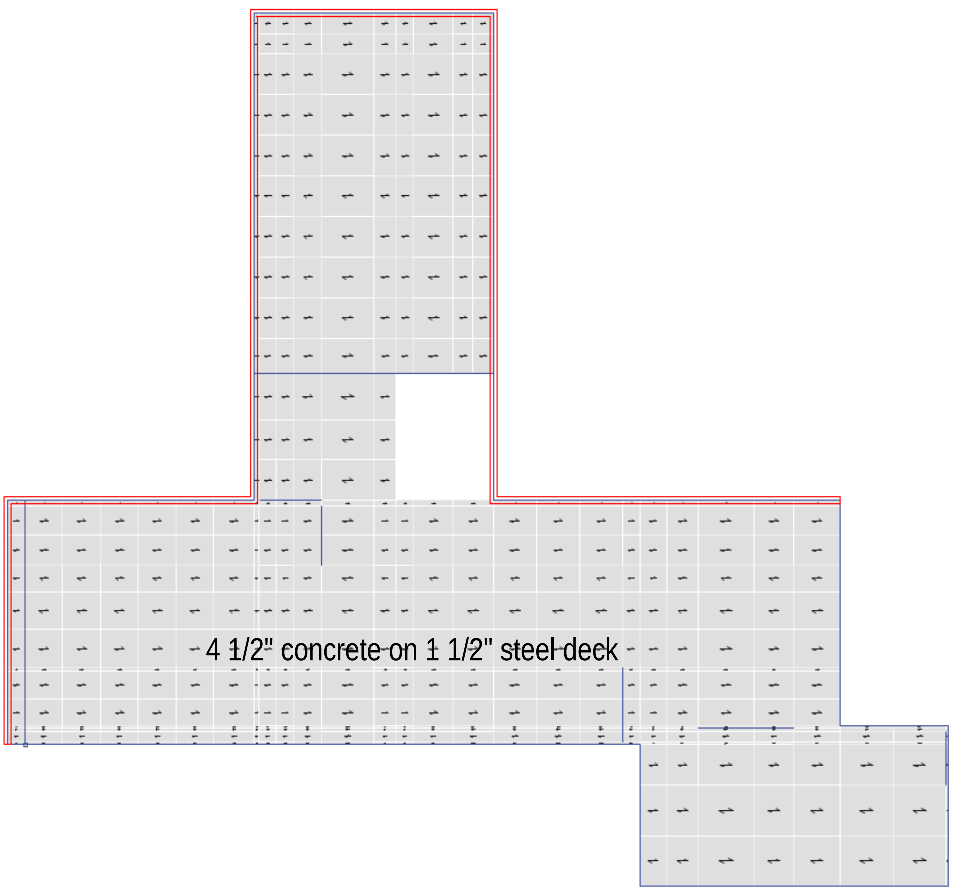
Story Definitions

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	45	1.40	*includes 5 psf for steel columns
MH Main Floor	56.21	1.75	*doesn't include concrete on metal deck, includes 5 psf steel columns
MH Deck	25	0.78	
MH Trellis	10	0.31	
Garage Roof	35	1.09	*includes 5 psf for steel columns
Garage Floor	56.21	1.75	*use same as MH floor (conservative, no finish), doesn't include concrete on metal deck, includes 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	[lb-s^2/ft^2]
South Deck to Gri	585	25	14625	45.5	4.99
South Deck to Gri	585	25	14625	38	5.98
East Deck to Grid	300	25	7500	37.5	6.21
SW Trellis to Gric	116	10	1160	14.5	2.48
SE Trellis to Grid	264	10	2640	38	2.16

ETABS: Additional Seismic Loads

Seismic Load from Garage Shearwalls:

Base Shear 14.75 k (LRFD)

N/S Direction

	Force	total Length	[klf]
West walls	7.375	6.833	1.079
East Walls	7.375	7.08	1.042

E/W Direction

	Force	total Length	[klf]
North Wall	7.375	23.5	0.314
South Wall	7.375	19.67	0.375

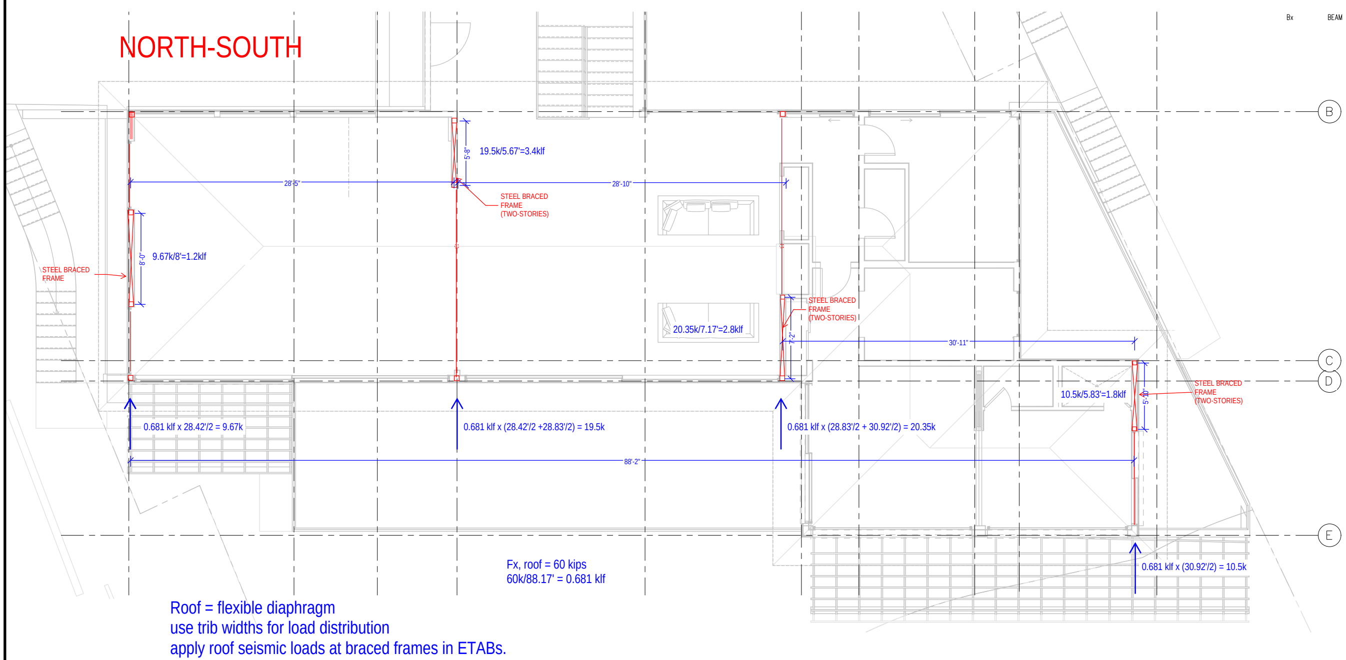
Loads from Wood portions getting dragged into House

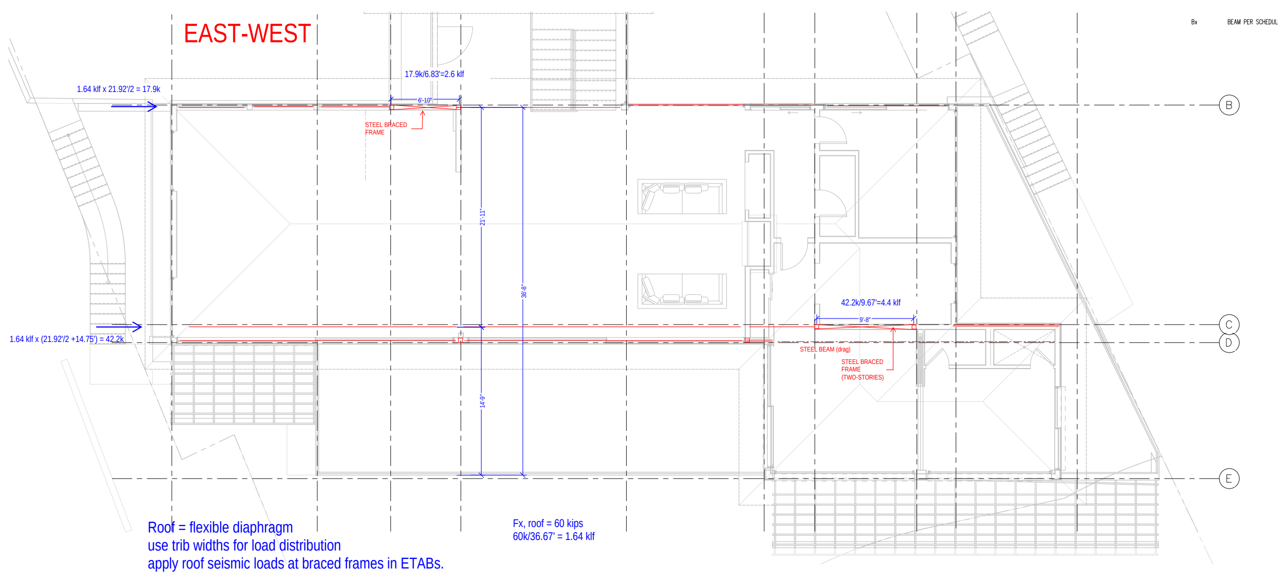
Main Floor Weigl 480 k

Main Floor Shear 71.1 k

	Area [ft2]	[psf]	Weight [k]	% weight	Force [[k]	Length	[klf]	
SW Trellis:	116	10	1.16	0.002	0.17	14.5	0.01	along grid E between 1 & 1.7
SE Trellis	264	10	2.64	0.006	2.64	38	0.07	along grid F between 4 & 6
South Deck	585	25	14.625	0.030	2.17	45.5	0.05	along grid E between 1.7 & 4
East Deck	300	25	7.5	0.016	1.11	37.5	0.03	

ETABS: ADDED SEISMIC LOADS FROM ROOF





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8480 Residence
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2/23/22
DATE
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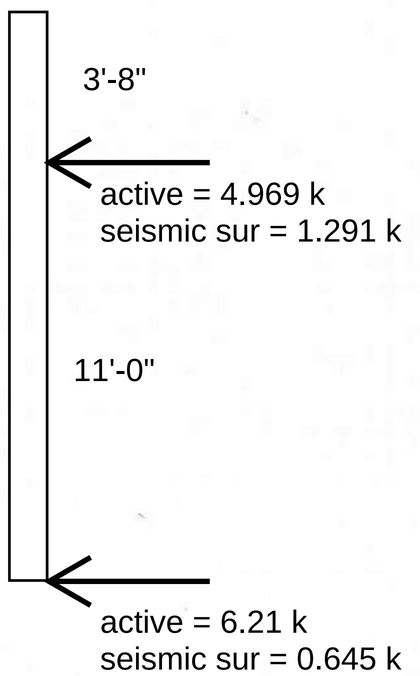
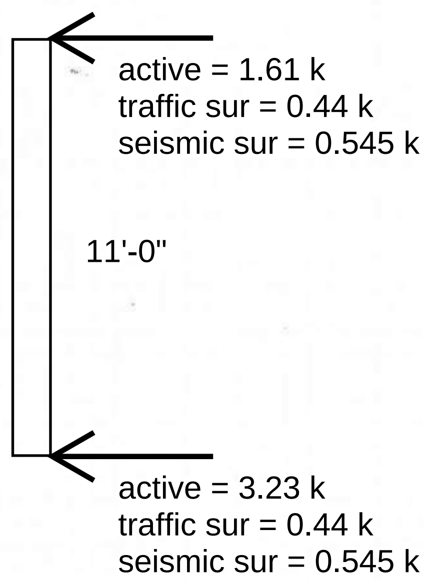
SHEET

Basement Wall Reactions

NORTH AND EAST WALLS



NORTH AND WEST WALLS
AT GARAGE ENTRANCE



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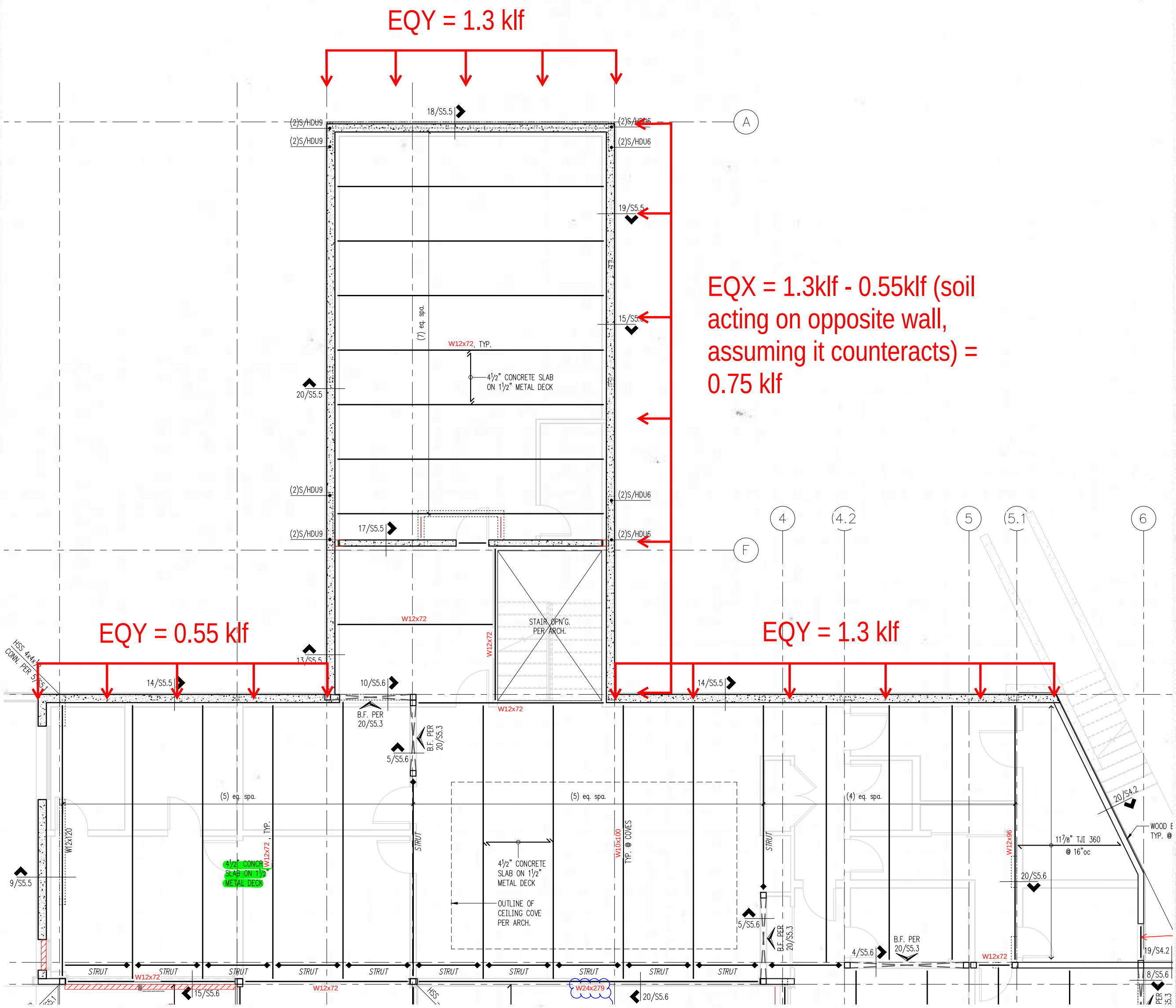
Basement Wall Reactions



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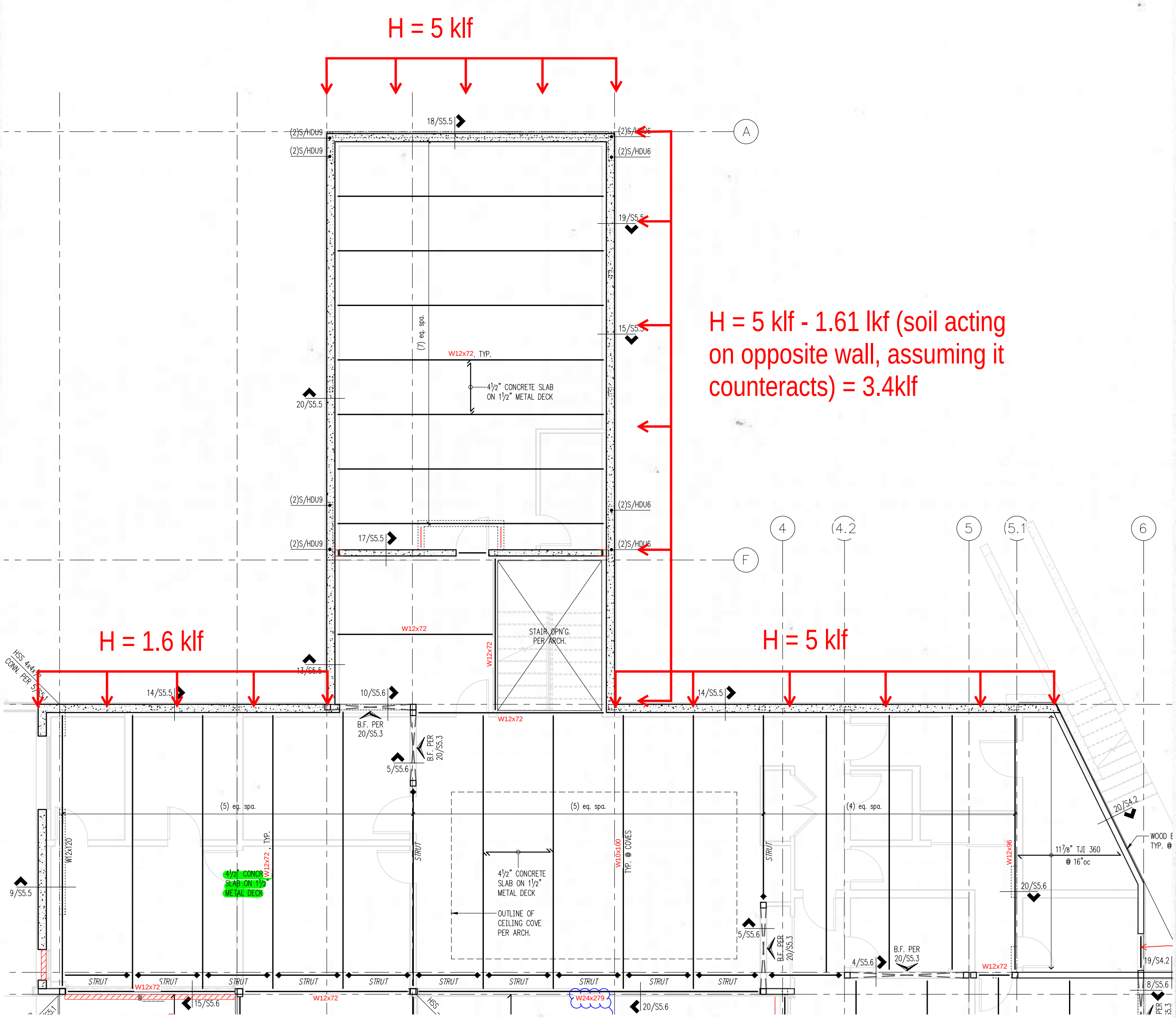
SSF STRUCTURAL ENGINEERING

PROJECT	8480 Wu Residence
DATE	02/23/2021
PROJ. #	01519-2021-09
DESIGN	
SHEET	L-13

Basement Wall Reactions



STRUCTURAL ENGINEERING

8480 Wu Residence

PROJECT

DATE 02/23/2021

PROJ. # 01519-2021-09

DESIGN

SHEET L-14

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TABLE: Story Forces

Story	Output Case	Case Type	Step Type	Step Number	Step Label	Location	P	VX	VY	T	MX	MY
							kip	kip	kip	kip-ft	kip-ft	kip-ft
Upper	EQXNTNYN	Combination				Top	0	55.825	17.291	-425.453	0	0
Upper	EQXNTNYN	Combination				Bottom	0.379	55.389	17.22	-415.398	-77.5794	250.789
Upper	EQXNTNYP	Combination				Top	0	55.825	-17.31	-1938.44	0	0
Upper	EQXNTNYP	Combination				Bottom	0.577	55.374	-17.08	-1913.77	116.8294	248.1155
Upper	EQXNTPYN	Combination				Top	0	55.825	17.291	-425.453	0	0
Upper	EQXNTPYN	Combination				Bottom	0.379	55.389	17.22	-415.398	-77.5794	250.789
Upper	EQXNTPYP	Combination				Top	0	55.825	-17.31	-1938.44	0	0
Upper	EQXNTPYP	Combination				Bottom	0.577	55.374	-17.08	-1913.77	116.8294	248.1155
Upper	EQXPTNYN	Combination				Top	0	-55.83	17.291	1938.432	0	0
Upper	EQXPTNYN	Combination				Bottom	-0.547	-55.37	17.076	1914.863	-115.307	-248.247
Upper	EQXPTNYP	Combination				Top	0	-55.83	-17.31	425.4477	0	0
Upper	EQXPTNYP	Combination				Bottom	-0.349	-55.38	-17.23	416.4875	79.1018	-250.921
Upper	EQXPTPYN	Combination				Top	0	-55.83	17.291	1938.432	0	0
Upper	EQXPTPYN	Combination				Bottom	-0.547	-55.37	17.076	1914.863	-115.307	-248.247
Upper	EQXPTPYP	Combination				Top	0	-55.83	-17.31	425.4477	0	0
Upper	EQXPTPYP	Combination				Bottom	-0.349	-55.38	-17.23	416.4875	79.1018	-250.921
Upper	EQYNTNXN	Combination				Top	0	16.747	57.636	2167.049	0	0
Upper	EQYNTNXN	Combination				Bottom	-0.208	16.646	57.185	2149.779	-318.347	80.7855
Upper	EQYNTNXP	Combination				Top	0	-16.75	57.636	2876.214	0	0
Upper	EQYNTNXP	Combination				Bottom	-0.486	-16.58	57.142	2848.857	-329.665	-68.9253
Upper	EQYNTPXN	Combination				Top	0	16.747	57.636	2167.049	0	0
Upper	EQYNTPXN	Combination				Bottom	-0.208	16.646	57.185	2149.779	-318.347	80.7855
Upper	EQYNTPXP	Combination				Top	0	-16.75	57.636	2876.214	0	0
Upper	EQYNTPXP	Combination				Bottom	-0.486	-16.58	57.142	2848.857	-329.665	-68.9253
Upper	EQYPTNXN	Combination				Top	0	16.747	-57.71	-2876.23	0	0
Upper	EQYPTNXN	Combination				Bottom	0.452	16.599	-57.16	-2844.81	329.6821	71.874
Upper	EQYPTNXP	Combination				Top	0	-16.75	-57.71	-2167.07	0	0
Upper	EQYPTNXP	Combination				Bottom	0.174	-16.63	-57.2	-2145.73	318.3638	-77.8369
Upper	EQYPTPXN	Combination				Top	0	16.747	-57.71	-2876.23	0	0
Upper	EQYPTPXN	Combination				Bottom	0.452	16.599	-57.16	-2844.81	329.6821	71.874
Upper	EQYPTPXP	Combination				Top	0	-16.75	-57.71	-2167.07	0	0
Upper	EQYPTPXP	Combination				Bottom	0.174	-16.63	-57.2	-2145.73	318.3638	-77.8369
Main	EQXNTNYN	Combination				Top	0.314	188.24	74.007	-4687.39	69.6175	-6.3154
Main	EQXNTNYN	Combination				Bottom	0.314	188.24	74.007	-4687.39	-744.457	2064.27
Main	EQXNTNYP	Combination				Top	0.472	187.58	-48.4	-10333.3	113.2332	12.9513
Main	EQXNTNYP	Combination				Bottom	0.472	187.58	-48.4	-10333.3	645.6808	2076.366
Main	EQXNTPYN	Combination				Top	0.314	188.24	74.007	-4687.39	69.6175	-6.3154
Main	EQXNTPYN	Combination				Bottom	0.314	188.24	74.007	-4687.39	-744.457	2064.27
Main	EQXNTPYP	Combination				Top	0.472	187.58	-48.4	-10333.3	113.2332	12.9513
Main	EQXNTPYP	Combination				Bottom	0.472	187.58	-48.4	-10333.3	645.6808	2076.366
Main	EQXPTNYN	Combination				Top	-0.447	-152.6	74.132	9364.757	-96.5442	-21.7966
Main	EQXPTNYN	Combination				Bottom	-0.447	-152.6	74.132	9364.757	-911.992	-1700.62
Main	EQXPTNYP	Combination				Top	-0.29	-153.3	-48.28	3718.867	-52.9285	-2.5298
Main	EQXPTNYP	Combination				Bottom	-0.29	-153.3	-48.28	3718.867	478.1464	-1688.52
Main	EQXPTPYN	Combination				Top	-0.447	-152.6	74.132	9364.757	-96.5442	-21.7966
Main	EQXPTPYN	Combination				Bottom	-0.447	-152.6	74.132	9364.757	-911.992	-1700.62
Main	EQXPTPYP	Combination				Top	-0.29	-153.3	-48.28	3718.867	-52.9285	-2.5298
Main	EQXPTPYP	Combination				Bottom	-0.29	-153.3	-48.28	3718.867	478.1464	-1688.52
Main	EQYNTNXN	Combination				Top	-0.161	57.453	246.88	8881.916	-19.2394	-40.7439
Main	EQYNTNXN	Combination				Bottom	-0.161	57.453	246.88	8881.916	-2734.88	591.2341
Main	EQYNTNXP	Combination				Top	-0.39	-44.8	246.91	13097.56	-69.0879	-45.3882
Main	EQYNTNXP	Combination				Bottom	-0.39	-44.8	246.91	13097.56	-2785.14	-538.232
Main	EQYNTPXN	Combination				Top	-0.161	57.453	246.88	8881.916	-19.2394	-40.7439
Main	EQYNTPXN	Combination				Bottom	-0.161	57.453	246.88	8881.916	-2734.88	591.2341
Main	EQYNTPXP	Combination				Top	-0.39	-44.8	246.91	13097.56	-69.0879	-45.3882
Main	EQYNTPXP	Combination				Bottom	-0.39	-44.8	246.91	13097.56	-2785.14	-538.232
Main	EQYPTNXN	Combination				Top	0.363	55.279	-161.2	-9937.72	126.1463	23.4787
Main	EQYPTNXN	Combination				Bottom	0.363	55.279	-161.2	-9937.72	1898.915	631.5531
Main	EQYPTNXP	Combination				Top	0.134	-46.98	-161.1	-5722.07	76.2978	18.8343
Main	EQYPTNXP	Combination				Bottom	0.134	-46.98	-161.1	-5722.07	1848.655	-497.913
Main	EQYPTPXN	Combination				Top	0.363	55.279	-161.2	-9937.72	126.1463	23.4787
Main	EQYPTPXN	Combination				Bottom	0.363	55.279	-161.2	-9937.72	1898.915	631.5531
Main	EQYPTPXP	Combination				Top	0.134	-46.98	-161.1	-5722.07	76.2978	18.8343
Main	EQYPTPXP	Combination				Bottom	0.134	-46.98	-161.1	-5722.07	1848.655	-497.913

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	0.77	0.66
Main to Base	11	2.64	0.32	0.54

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	0.77	0.66

Upper-Main

Unique Node	Ux [in]	Drift, X	Uy [in]	Drift, Y
22	0.00	0.05	0.08	0.40
35	0.01		0.00	
23	0.03	0.09	0.08	0.40
36	0.02		0.00	
240	0.11	0.51	0.01	0.03
472	0.01		0.00	
58	0.11	0.49	0.02	0.05
56	0.01		0.01	
60	0.02	0.06	0.14	0.66
59	0.01		0.01	
62	0.03	0.06	0.14	0.66
61	0.01		0.01	
67	0.08	0.25	0.17	0.57
55	0.03		0.06	
66	0.08	0.20	0.17	0.57
39	0.04		0.05	
32	0.20	0.77	0.13	0.29
45	0.04		0.07	
31	0.20	0.77	0.16	0.35
44	0.04		0.08	
34	0.09	0.20	0.22	0.57
47	0.05		0.11	
33	0.10	0.22	0.22	0.57
46	0.05		0.11	
	0.00	0.00	0.00	0.00
	0.00		0.00	
	0.00	0.00	0.00	0.00
	0.00		0.00	
	0.00	0.00	0.00	0.00
	0.00		0.00	
	0.00	0.00	0.00	0.00
	0.00		0.00	
	0.00	0.00	0.00	0.00
	0.00		0.00	

Cd	5
I	1
ρ	1.3
% for drift	0.02

	Max Drift, X	Max Drift, Y
Main-Bas	0.32	0.54

GR-BASE

Unique Node	Ux [in]	Drift, X	Uy [in]	Drift, Y
408	0.02	0.07	0.00	0.00
410	0.00		0.00	
3	0.00	0.01	0.00	0.01
88	0.00		0.00	
4	0.01	0.03	0.00	0.01
2	0.00		0.00	
308	0.01	0.03	0.00	0.00
269	0.00		0.00	
5	0.00	0.01	0.00	0.01
82	0.00		0.00	
7	0.00	0.01	0.00	0.01
90	0.00		0.00	
8	0.02	0.08	0.03	0.11
93	0.00		0.00	
557	0.00	0.01	0.09	0.36
583	0.00		0.00	
168	0.02	0.09	0.01	0.04
	0.00		0.00	
615	0.07	0.26	0.06	0.22
	0.00		0.00	
411	0.04	0.16	0.06	0.22
	0.00		0.00	
35	0.01	0.04	0.00	0.00
	0.00		0.00	
36	0.02	0.06	0.00	0.00
	0.00		0.00	
578	0.02	0.07	0.00	0.00
	0.00		0.00	
472	0.01	0.03	0.00	0.01
699	0.00		0.00	
56	0.01	0.06	0.01	0.04
701	0.00		0.00	
59	0.01	0.04	0.01	0.04
69	0.00		0.00	
61	0.01	0.07	0.01	0.04
68	0.00		0.00	
55	0.03	0.15	0.06	0.28
76	0.00		0.00	
39	0.04	0.20	0.05	0.27
75	0.00		0.00	
45	0.04	0.22	0.07	0.34
224	0.00		0.00	
44	0.04	0.22	0.08	0.42
225	0.00		0.00	
47	0.05	0.23	0.11	0.54
16	0.00		0.00	
46	0.05	0.26	0.11	0.54
15	0.00		0.00	
12	0.06	0.32	0.11	0.54
	0.00		0.00	
578	0.02	0.09	0.00	0.00
505	0.00		0.00	
	0.00	0.00	0.00	0.00
	0.00		0.00	

TORSIONAL AMPLIFICATION

	Ux	Unique Nc	Uy	Unique Node Name
Upper	0.197631538	32	0.220373	34
Main	0.920820469	1	0.330346	1

Upper	Unique Node Name	Ux	Avg	Ux Max	Ax
	23	0.03	0.06	0.20	6.53
	33	0.10			
	34	0.09	0.06	0.20	7.22
	22	0.04			

Unique Node Name	Uy	Avg	Uy Max	Ax
22	0.08	0.08	0.22	5.31
23	0.08			
66	0.17	0.17	0.22	1.19
67	0.17			
33	0.22	0.22	0.22	0.69
34	0.22			

Main	Unique Node Name	Ux	Avg	Ux Max	Ax
	408	0.02	0.01	0.02	2.06
	3	0.00			
	4	0.01	0.00	0.01	1.83
	5	0.00			
	7	0.00	0.01	0.02	2.27
	8	0.02			
	557	0.00	0.02	0.04	2.48
	555	0.04			

Unique Node Name	Uy	Avg	Uy Max	Ax
3	0.00	0.05	0.09	2.67
557	0.09			
408	0.00	0.05	0.11	2.73
596	0.11			
5	0.00	0.00	0.00	0.71
7	0.00			

Use Ax=3

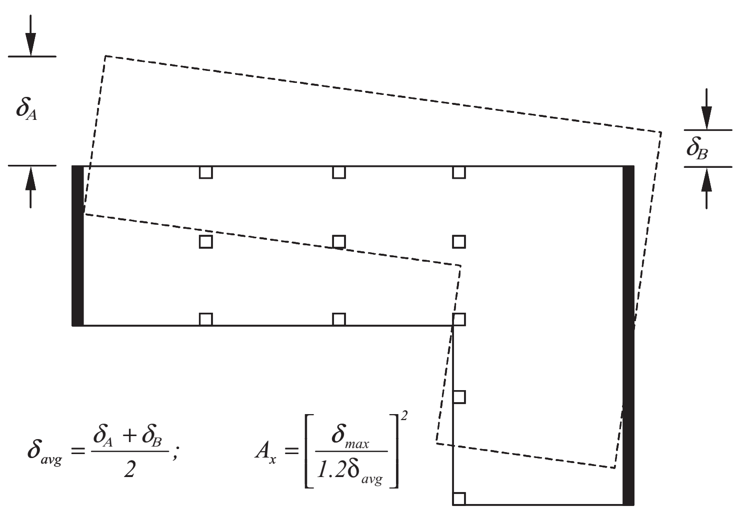


FIGURE 12.8-1 Torsional Amplification Factor, A_x

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

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 from RAM Steel. 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 gravity loads from RAM Steel. 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						
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-2.2-U	2	9.33	COL2.2-1U	COL2.2-2U	U-BM2.2
3	3	BF-2.2-M	2	11.00	COL2.2-1M	COL2.2-2M	M-BM2.2
4	4	BF-4-U	2	9.33	COL4-1U	COL4-2U	U-BM4
5	5	BF-4-M	2	11.00	COL4-1M	COL4-2M	M-BM4
6	6	BF-6-U	2	9.33	COL6-1U	COL6-2U	U-BM6
7	7	BF-6-M	2	11.00	COL6-1M	COL6-2M	M-BM6
8	8	BF-B-U	2	9.33	COLB-1U	COLB-2U	U-BMB
9	9	BF-B-M	2	11.00	COLB-1M	COLB-2M	M-BMB
10	10	BF-C-U	2	9.33	COLC-1U	COLC-2U	U-BMC
11	11	BF-C-M	2	11	COLC-1M	COLC-2M	M-BMC

GRAVITY LOADS - COLUMNS

			DL [k]	Self Wt DL [k	LL [k]	Total DL [k]	Total LL [k]	1.2D+1.6L
COL2.2-1U	1U	COL2.2	4.3	0.33	4.3	4.63	4.3	12.43
COL2.2-1M	1M	COL2.2	17.8	0.39	10.8	18.19	10.8	39.10
COL2.2-2U	2U	COL2.2	11.1	0.33	8	11.43	8	26.51
COL2.2-2M	2M	COL2.2	17.2	0.39	10.7	17.59	10.7	38.22
COL4-1U	1U	COL4	8.9	0.33	7.2	9.23	7.2	22.59
COL4-1M	1M	COL4	15.4	0.39	11	15.79	11	36.54
COL4-2U	2U	COL4	5.6	0.33	3.4	5.93	3.4	12.55
COL4-2M	2M	COL4	26.8	0.39	18.6	27.19	18.6	62.38
COL6-1U	1U	COL6	3.3	0.33	3.8	3.63	3.8	10.43
COL6-1M	1M	COL6	4.5	0.39	5.2	4.89	5.2	14.18
COL6-2U	2U	COL6	1.9	0.33	2.4	2.23	2.4	6.51
COL6-2M	2M	COL6	4.9	0.39	5.2	5.29	5.2	14.66
COLB-1U	1U	COLB	1.6	0.33	1.7	1.93	1.7	5.03
COLB-1M	1M	COLB	8.9	0.39	4.9	9.29	4.9	18.98
COLB-2U	2U	COLB	3	0.33	3.4	3.33	3.4	9.43
COLB-2M	2M	COLB	3.9	0.39	3.8	4.29	3.8	11.22
COLC-1U	1U	COLC	1.8	0.33	2.3	2.13	2.3	6.23
COLC-1M	1M	COLC	15.2	0.39	7.8	15.59	7.8	31.18
COLC-2U	2U	COLC	1.8	0.33	2.3	2.13	2.3	6.23
COLC-2M	2M	COLC	11.4	0.39	6.3	11.79	6.3	24.22
COL1-1	1	COL1	1.3	0.33	1.7	1.63	1.7	4.67
COL1-2	2	COL1	1.3	0.33	1.7	1.63	1.7	4.67

GRAVITY LOADS - BEAMS

	kips		kip-ft	
Beam #	Vu	phiVn	Mu	phiMn
U-BM1	1.092	153.9	5	263
U-BM2.2	6.504	153.9	20	263
U-BM4	6.96	153.9	20	263
U-BM6	0.756	153.9	5	263
U-BMB	3.48	153.9	10	263
U-BMC	3.2	153.9	10	263

	kips		kip-ft	
Beam #	Vu	phiVn	Mu	phiMn
M-BM2.2	2.92	135	5	269.6
M-BM4	7.24	135	15	269.6
M-BM6	2.528	135	5	269.6
M-BMB	1.608	135	5	269.6
M-BMC	17.92	158.7	40	405

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

<u>Brace Design</u>

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	12.0	6.0	7.1		51	84	0.14
BF-2.2-U	HSS3X.250	10.9	5.5	18.4		59	84	0.31
BF-4-U	HSS3X.250	11.7	5.9	16.4		59	84	0.28
BF-6-U	HSS3X.250	10.7	5.3	9.6		59	84	0.16
BF-B-U	HSS3X.250	11.1	5.5	14.7		59	84	0.25
BF-C-U	HSS3X.250	13.1	6.5	28.7		51	84	0.56
BF-C-M	HSS3X.250	14.3	7.2	7.0		44	84	0.16
BF-B-M	HSS3X.250	12.5	6.3	4.2		51	84	0.08
BF-6-M	HSS3X.250	12.2	6.1	14.5		51	84	0.28
BF-2.2-M	HSS3X.250	12.4	6.2	7.3		51	84	0.14
BF-4-M	HSS3X.250	13.1	6.6	6.1		51	84	0.12

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	6.01	1.4	2.03	131	1	0.982	73	100	53	39	90	27	51
	0																

	Summary of Expected Strengths																			
	Tensile Strength			Compressive Strength			Post-Buckling Strength				Column Length		Ram Steel Col ID		Loads					
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	101	82	90	70	57	27	21	17	BF-1	9.33	10	COL1-1	COL1-2	70	101	1.6	1.6	1.7	1.7

	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	2	5	105	71	105	71	HSS6x6x1/2	1.4	9.91	13.23	OK	354.67	438.3	✓ 0.29	✓ 0.16

STRUCTURAL ENGINEERING

8480 Residence
PROJECT
Lateral Design

2/23/22
DATE
PROJ. #
SRW
DESIGN
L-25
SHEET

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	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	7.59	2.12	7.9	8	U-BM1	139	1	5	W8X67	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	19.7	4.4	7	OK	11	0.14	51.3	OK	763.1	263	154	✓ 0.18	✓ 0.02	✓ 0.01	✓ 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-2.2-U	2	HSS3X.250	5.46	1.4	2.03	131	1	0.982	67	100	64	42	98	29	59
3	3	BF-2.2-M	2	HSS3X.250	6.19	1.4	2.03	131	1	0.982	76	100	50	38	87	26	63
4	4	BF-4-U	2	HSS3X.250	5.87	1.4	2.03	131	1	0.982	72	100	56	40	92	28	53
5	5	BF-4-M	2	HSS3X.250	6.55	1.4	2.03	131	1	0.982	80	100	45	35	81	24	57
6	6	BF-6-U	2	HSS3X.250	5.34	1.4	2.03	131	1	0.982	65	100	67	43	100	30	61
7	7	BF-6-M	2	HSS3X.250	6.08	1.4	2.03	131	1	0.982	74	100	52	38	89	27	65
8	8	BF-B-U	2	HSS3X.250	5.54	1.4	2.03	131	1	0.982	68	100	63	42	97	29	57
9	9	BF-B-M	2	HSS3X.250	6.26	1.4	2.03	131	1	0.982	76	100	49	37	86	26	62
10	10	BF-C-U	2	HSS3X.250	6.54	1.4	2.03	131	1	0.982	80	100	45	35	82	24	45
11	11	BF-C-M	2	HSS3X.250	7.16	1.4	2.03	131	1	0.982	87	100	37	31	72	22	50
	0																

	Summary of Expected Strengths																			
	Tensile Strength			Compressive Strength			Post-Buckling Strength				Column Length		Ram Steel Col ID		Loads					
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 RAM Steel are at foundation.					
BF-2.2-U	131	112	68	98	84	51	29	25	15	BF-2.2-U	9.33	10	COL2.2-1U	COL2.2-2U			4.6	11.4	4.3	8.0
BF-2.2-M	131	116	60	87	77	40	26	23	12	BF-2.2-M	11.00	11	COL2.2-1M	COL2.2-2M	273	312	18.2	17.6	10.8	10.7
BF-4-U	131	104	79	92	73	56	28	22	17	BF-4-U	9.33	10	COL4-1U	COL4-2U			9.2	5.9	7.2	3.4
BF-4-M	131	110	71	81	68	44	24	21	13	BF-4-M	11.00	11	COL4-1M	COL4-2M	245	287	15.8	27.2	11.0	18.6
BF-6-U	131	114	63	100	87	48	30	26	15	BF-6-U	9.33	10	COL6-1U	COL6-2U			3.6	2.2	3.8	2.4
BF-6-M	131	118	56	89	80	38	27	24	11	BF-6-M	11.00	11	COL6-1M	COL6-2M	282	320	4.9	5.3	5.2	5.2
BF-B-U	131	110	70	97	82	52	29	24	16	BF-B-U	9.33	10	COLB-1U	COLB-2U			1.9	3.3	1.7	3.4
BF-B-M	131	115	62	86	76	41	26	23	12	BF-B-M	11.00	11	COLB-1M	COLB-2M	267	307	9.3	4.3	4.9	3.8
BF-C-U	131	93	92	82	58	57	24	17	17	BF-C-U	9.33	10	COLC-1U	COLC-2U			2.1	2.1	2.3	2.3
BF-C-M	131	100	84	72	56	46	22	17	14	BF-C-M	11.00	11	COLC-1M	COLC-2M	207	252	15.6	11.8	7.8	6.3

	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-2.2-U							HSS6X6X5/8	1.1	7.33	14.93	OK				
BF-2.2-M	25	39	343	261	343	261	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✔ 0.85	✔ 0.50
BF-4-U							HSS6X6X5/8	1.1	7.33	14.93	OK				
BF-4-M	38	62	335	235	335	235	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✔ 0.83	✔ 0.45
BF-6-U							HSS6X6X5/8	1.1	7.33	14.93	OK				
BF-6-M	7	15	330	279	330	279	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✔ 0.82	✔ 0.53
BF-B-U							HSS6X6X5/8	1.1	7.33	14.93	OK				
BF-B-M	13	19	322	264	322	264	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✔ 0.80	✔ 0.50
BF-C-U							HSS6X6X5/8	1.1	7.33	14.93	OK				
BF-C-M	22	31	278	199	278	199	HSS6X6X5/8	1.1	7.33	14.93	OK	402	526.5	✔ 0.69	✔ 0.38

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	

*highly ductile

Beam

	Beam Length [ft]					Loads								
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-2.2-U	5.67	2.12	9.7	6	U-BM2.2					119	6.5	20	W8X67	1.1
BF-2.2-M	5.67	1.96	9.0	6	M-BM2.2	10	19	6	50	50	2.92	5	W12x50	1.1
BF-4-U	7.13	2.12	9.7	8	U-BM4					135	6.96	20	W8X67	1.1
BF-4-M	7.12	1.96	9.0	8	M-BM4	10	25	6	60	60	7.24	15	W12x50	1.1
BF-6-U	5.18	2.12	9.7	6	U-BM6					112	0.76	5	W8X67	1.1
BF-6-M	5.18	1.96	9.0	6	M-BM6	9	16	6	47	47	2.53	5	W12x50	1.1
BF-B-U	5.96	2.12	9.7	6	U-BMB					123	3.48	10	W8X67	1.1
BF-B-M	5.96	1.96	9.0	6	M-BMB	10	20	6	52	52	1.61	5	W12x50	1.1
BF-C-U	9.17	2.12	9.7	10	U-BMC					149	3.2	10	W8X67	1.1
BF-C-M	9.17	1.96	9.0	10	M-BMC	9	36	6	72	72	17.9	40	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-2.2-U	19.7	4.4	7	OK	11	0.12	51.7	OK	815	263	154	✓ 0.15	✓ 0.08	✓ 0.04	✓ 0.08
BF-2.2-M	14.6	6.3	7	OK	27	0.07	54.7	OK	595	269.6	135	✓ 0.08	✓ 0.02	✓ 0.02	✓ 0.03
BF-4-U	20	4	7	OK	11	0.14	51.4	OK	763	263	154	✓ 0.18	✓ 0.08	✓ 0.05	✓ 0.08
BF-4-M	15	6	7	OK	27	0.08	53.9	OK	551	269.6	135	✓ 0.11	✓ 0.06	✓ 0.05	✓ 0.08
BF-6-U	20	4	7	OK	11	0.11	51.8	OK	815	263	154	✓ 0.14	✓ 0.02	✓ 0.00	✓ 0.02
BF-6-M	15	6	7	OK	27	0.06	55.1	OK	595	269.6	135	✓ 0.08	✓ 0.02	✓ 0.02	✓ 0.03
BF-B-U	20	4	7	OK	11	0.13	51.6	OK	815	263	154	✓ 0.15	✓ 0.04	✓ 0.02	✓ 0.04
BF-B-M	15	6	7	OK	27	0.07	54.6	OK	595	269.6	135	✓ 0.09	✓ 0.02	✓ 0.01	✓ 0.04
BF-C-U	20	4	7	OK	11	0.15	51.1	OK	701	263	154	✓ 0.21	✓ 0.04	✓ 0.02	✓ 0.25
BF-C-M	15	6	7	OK	27	0.10	52.9	OK	500	405	135	✓ 0.14	✓ 0.10	✓ 0.13	✓ 0.13

BRACED FRAME CONNECTION DESIGN

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	

Max ID	ID	BF Name	Column Size	Beam Size	Brace Size	Brace Properties Compression [k] Expected	Tension [k] Expected	Width, b [in]	Brace Area, Ag [in2]	thickness, t [in]	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	HSS6x6x1/2	W8X67	HSS3X.250	90	131	3.00	2.03	0.23	6.01	7.59	9.33	50.9
2	2	BF-2.2-U	HSS6X6X5/8	W8X67	HSS3X.250	98	131	3.00	2.03	0.23	5.46	5.67	9.33	58.7
3	3	BF-2.2-M	HSS6X6X5/8	W12x50	HSS3X.250	87	131	3.00	2.03	0.23	6.19	5.67	11.00	62.7
4	4	BF-4-U	HSS6X6X5/8	W8X67	HSS3X.250	92	131	3.00	2.03	0.23	5.87	7.13	9.33	52.6
5	5	BF-4-M	HSS6X6X5/8	W12x50	HSS3X.250	81	131	3.00	2.03	0.23	6.55	7.12	11.00	57.1
6	6	BF-6-U	HSS6X6X5/8	W8X67	HSS3X.250	100	131	3.00	2.03	0.23	5.34	5.18	9.33	60.9
7	7	BF-6-M	HSS6X6X5/8	W12x50	HSS3X.250	89	131	3.00	2.03	0.23	6.08	5.18	11.00	64.8
8	8	BF-B-U	HSS6X6X5/8	W8X67	HSS3X.250	97	131	3.00	2.03	0.23	5.54	5.96	9.33	57.4
9	9	BF-B-M	HSS6X6X5/8	W12x50	HSS3X.250	86	131	3.00	2.03	0.23	6.26	5.96	11.00	61.6
10	10	BF-C-U	HSS6X6X5/8	W8X67	HSS3X.250	82	131	3.00	2.03	0.23	6.54	9.17	9.33	45.5
11	11	BF-C-M	HSS6X6X5/8	W12x50	HSS3X.250	72	131	3.00	2.03	0.23	7.16	9.17	11.00	50.2

	Gusset Properties		Design Parameters				Brace Block Shear Rupture					Brace to Gusset Weld Fracture	
BF Name	Fy, Gusset [ksi]	Fu, Gusset [ksi]	Gusset Thk, tg [in]	Brace Weld Length [in]	Gusset Weld Size [in]	ϕ (Whitmore angle at gusset)	ϕ	Rt	Anv [in2]	ϕRn [k]	DCR	ϕRn [k]	DCR
BF-1	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-2.2-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-2.2-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-4-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-4-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-6-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-6-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-B-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-B-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-C-U	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39
BF-C-M	50	65	1/2	12	5/16	30	0.75	1.3	11.18	406	✓ 0.32	334	✓ 0.39

BRACED FRAME CONNECTION DESIGN

	Block Shear Rupture of Gusset								Tensile Yielding of Gusset Plate				Buckling of Gusset Plate							
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
BF-1	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	8.32	0.14	69.20	59.77	35.23	267.23 ✓	0.49
BF-2.2-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.47	0.14	53.83	98.77	40.45	306.85 ✓	0.43
BF-2.2-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.64	0.14	46.88	130.21	42.58	322.96 ✓	0.40
BF-4-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	7.88	0.14	65.53	66.66	36.53	277.08 ✓	0.47
BF-4-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.83	0.14	56.81	88.69	39.49	299.55 ✓	0.44
BF-6-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.00	0.14	49.91	114.92	41.68	316.12 ✓	0.41
BF-6-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.24	0.14	43.55	150.90	43.53	330.16 ✓	0.40
BF-B-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	6.76	0.14	56.17	90.72	39.70	301.14 ✓	0.43
BF-B-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	5.88	0.14	48.87	119.85	41.99	318.50 ✓	0.41
BF-C-U	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	9.85	0.14	81.90	42.67	30.62	232.24 ✓	0.56
BF-C-M	0.75	12.00	1.81	468	118	360	358 ✓	0.36	0.9	16.86	379 ✓	0.34	1.2	8.50	0.14	70.71	57.25	34.69	263.15 ✓	0.50

	Connections																			
	Brace Net Section Fracture																			
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																			
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
BF-2.2-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
BF-2.2-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
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
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
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
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
BF-B-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
BF-B-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
BF-C-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
BF-C-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

*strength of cover plates have to be at least same as brace

	Connections							Beam to Column Connection								
	Welds - Cover Plates to Brace								Forces		Beam Shear		Beam Axial		Combined	
BF Name	φ	Ry	φRyFyAp [k]	Weld Size (tpl-1/16)	Lw [in]	φRn [k]	DCR	Pu,E Beam	Nu [k]	Vu [k]	φVn [k]	Vu/φVn	φPn [k]	Pu/φPn	Mu/φMn	Combine
BF-1	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	139	139	43	154 ✓	0.28	763 ✓	0.18 ✓	0.02 ✓	0.11
BF-2.2-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	119	119	52	154 ✓	0.34	815 ✓	0.15 ✓	0.08 ✓	0.15
BF-2.2-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	50	52	59	135 ✓	0.44	595 ✓	0.09 ✓	0.02 ✓	0.06
BF-4-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	135	136	49	154 ✓	0.32	763 ✓	0.18 ✓	0.08 ✓	0.17
BF-4-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	60	62	61	135 ✓	0.45	551 ✓	0.11 ✓	0.06 ✓	0.11
BF-6-U	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	112	113	47	154 ✓	0.30	815 ✓	0.14 ✓	0.02 ✓	0.09
BF-6-M	0.75	1.1	55.00	1/4	6.00	66.82 ✓	0.82	47	48	59	135 ✓	0.44	595 ✓	0.08 ✓	0.02 ✓	0.06
BF-B-U	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62	123	124	48	154 ✓	0.31	815 ✓	0.15 ✓	0.04 ✓	0.11
BF-B-M	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62	52	54	58	135 ✓	0.43	595 ✓	0.09 ✓	0.02 ✓	0.06
BF-C-U	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62	149	150	42	154 ✓	0.28	701 ✓	0.21 ✓	0.04 ✓	0.25
BF-C-M	0.75	1.1	55.00	1/4	8.00	89.09 ✓	0.62	72	74	67	135 ✓	0.50	500 ✓	0.15 ✓	0.10 ✓	0.17

BRACED FRAME CONNECTION DESIGN

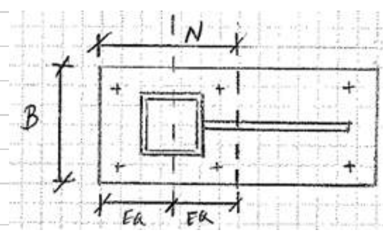
	Gusset-to-Beam Interface																				
	Forces				Shear Yielding			Tension Yielding			Weld						Web Local Yielding				
BF Name	Nu [k]	Vu [k]	Ru [k] Resultant	L_A [in]	ϕ	ϕRn [k]	DCR	ϕ	ϕRn [k]	DCR	Load angle (deg)	Directional strength Inc	Ductility Factor	Weld Size	ϕRn [k]	DCR	ϕ	tw [in]	Kdes	ϕRn [k	DCR
BF-1	41	55	69	12	1	169 ✓	0.32	0.9	254 ✓	0.16	37.09	1.23	1.25	5/16	194 ✓	0.44	1	0.57	1.33	416 ✓	0.10
BF-2.2-U	45	38	59	10	1	139 ✓	0.27	0.9	209 ✓	0.22	50.31	1.34	1.25	5/16	173 ✓	0.43	1	0.57	1.33	359 ✓	0.13
BF-2.2-M	56	32	65	10	1	139 ✓	0.23	0.9	209 ✓	0.27	60.33	1.40	1.25	5/16	181 ✓	0.45	1	0.37	1.14	224 ✓	0.25
BF-4-U	43	51	66	12	1	162 ✓	0.32	0.9	242 ✓	0.18	39.79	1.26	1.25	5/16	188 ✓	0.44	1	0.57	1.33	402 ✓	0.11
BF-4-M	54	44	70	12	1	160 ✓	0.28	0.9	240 ✓	0.23	50.58	1.34	1.25	5/16	199 ✓	0.44	1	0.37	1.14	250 ✓	0.22
BF-6-U	46	33	57	10	1	132 ✓	0.25	0.9	198 ✓	0.23	54.61	1.37	1.25	5/16	168 ✓	0.42	1	0.57	1.33	346 ✓	0.13
BF-6-M	57	28	63	10	1	132 ✓	0.21	0.9	199 ✓	0.29	64.04	1.43	1.25	5/16	175 ✓	0.45	1	0.37	1.14	216 ✓	0.26
BF-B-U	45	40	60	11	1	143 ✓	0.28	0.9	215 ✓	0.21	47.94	1.32	1.25	5/16	176 ✓	0.43	1	0.57	1.33	367 ✓	0.12
BF-B-M	56	35	66	11	1	143 ✓	0.24	0.9	215 ✓	0.26	58.22	1.39	1.25	5/16	185 ✓	0.45	1	0.37	1.14	229 ✓	0.24
BF-C-U	39	66	76	14	1	196 ✓	0.33	0.9	294 ✓	0.13	30.88	1.18	1.25	5/16	215 ✓	0.44	1	0.57	1.33	467 ✓	0.08
BF-C-M	50	59	77	14	1	192 ✓	0.31	0.9	288 ✓	0.17	39.85	1.26	1.25	5/16	224 ✓	0.43	1	0.37	1.14	290 ✓	0.17

	Gusset-to-Beam Interface								
	Web Local Yielding					Web Local Crippling			
BF Name	ϕ	tw [in]	Kdes	ϕRn [k	DCR	ϕ	tf [in]	ϕRn [k]	DCR
BF-1	1	0.57	1.33	416 ✓	0.10	0.75	0.935	838 ✓	0.05
BF-2.2-U	1	0.57	1.33	359 ✓	0.13	0.75	0.935	743 ✓	0.06
BF-2.2-M	1	0.37	1.14	224 ✓	0.25	0.75	0.64	260 ✓	0.22
BF-4-U	1	0.57	1.33	402 ✓	0.11	0.75	0.935	814 ✓	0.05
BF-4-M	1	0.37	1.14	250 ✓	0.22	0.75	0.64	280 ✓	0.19
BF-6-U	1	0.57	1.33	346 ✓	0.13	0.75	0.935	720 ✓	0.06
BF-6-M	1	0.37	1.14	216 ✓	0.26	0.75	0.64	254 ✓	0.22
BF-B-U	1	0.57	1.33	367 ✓	0.12	0.75	0.935	757 ✓	0.06
BF-B-M	1	0.37	1.14	229 ✓	0.24	0.75	0.64	264 ✓	0.21
BF-C-U	1	0.57	1.33	467 ✓	0.08	0.75	0.935	923 ✓	0.04
BF-C-M	1	0.37	1.14	290 ✓	0.17	0.75	0.64	310 ✓	0.16

	Gusset-to-Column Interface																								
	Forces							Weld						Shear Yielding			Tension Yielding			Column Shear (HSS only)					
BF Name	Nu [k]	Vu [k]	Muc [k-in]	Nu,equiv [k]	R [k] Resultant	L_g [in]	Fy [ksi]	Load angle (deg)	strength Inc	Ductility Factor	Weld Size	ϕRn [k]	DCR	ϕ	ϕRn [k]	DCR	ϕ	ϕRn [k]	DCR	h [in]	tw [in]	h/tw	Cv	ϕVn [k]	DCR
BF-1	28	60	0	28	66	12	50	25	1.14	1.25	5/16	174 ✓	0.47	1	165 ✓	0.36	0.90	248 ✓	0.11	4.61	0.47	9.91	1.00	115.76 ✓	0.24
BF-2.2-U	30	66	0	30	73	12	50	25	1.13	1.25	5/16	176 ✓	0.52	1	167 ✓	0.40	0.90	251 ✓	0.12	4.26	0.58	7.33	1.00	133.65 ✓	0.23
BF-2.2-M	28	60	0	28	66	12	50	25	1.14	1.25	5/16	173 ✓	0.48	1	164 ✓	0.36	0.90	246 ✓	0.11	4.26	0.58	7.33	1.00	133.65 ✓	0.21
BF-4-U	28	61	0	28	68	12	50	25	1.14	1.25	5/16	174 ✓	0.49	1	165 ✓	0.37	0.90	247 ✓	0.11	4.26	0.58	7.33	1.00	133.65 ✓	0.21
BF-4-M	27	56	0	27	62	12	50	26	1.14	1.25	5/16	168 ✓	0.46	1	158 ✓	0.35	0.90	237 ✓	0.11	4.26	0.58	7.33	1.00	133.65 ✓	0.20
BF-6-U	31	68	0	31	75	12	50	24	1.13	1.25	5/16	178 ✓	0.52	1	170 ✓	0.40	0.90	254 ✓	0.12	4.26	0.58	7.33	1.00	133.65 ✓	0.23
BF-6-M	28	61	0	28	67	12	50	24	1.13	1.25	5/16	176 ✓	0.48	1	168 ✓	0.37	0.90	251 ✓	0.11	4.26	0.58	7.33	1.00	133.65 ✓	0.21
BF-B-U	30	65	0	30	72	12	50	25	1.13	1.25	5/16	175 ✓	0.51	1	166 ✓	0.39	0.90	250 ✓	0.12	4.26	0.58	7.33	1.00	133.65 ✓	0.22
BF-B-M	28	59	0	28	65	12	50	25	1.14	1.25	5/16	172 ✓	0.47	1	162 ✓	0.36	0.90	244 ✓	0.11	4.26	0.58	7.33	1.00	133.65 ✓	0.21
BF-C-U	26	54	11	30	62	12	50	29	1.17	1.25	5/16	184 ✓	0.42	1	169 ✓	0.32	0.90	254 ✓	0.12	4.26	0.58	7.33	1.00	133.65 ✓	0.20
BF-C-M	24	51	0	24	56	12	50	26	1.14	1.25	5/16	168 ✓	0.42	1	158 ✓	0.32	0.90	237 ✓	0.10	4.26	0.58	7.33	1.00	133.65 ✓	0.18

BRACED FRAME BASE CONNECTION

Expected Forces on Base Plate			
Vertical (compression - column only) =	343	k	
Vertical (tension - column only) =	282	k	
Total Vertical (compression) =	420	k	
Total Vertical (tension) =	400	k	
Horizontal =	92	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 =	17.890	in.	246.188
Fillet Weld Thickness =	0.5	in.	5.527
Fillet Weld Length Required =	25.302	in.	
Size Base Plate			
Fy =	50	ksi	
B =	15	in.	
N =	15	in.	
m = (N-0.95d)/2 =	4.65	in.	
n = $(B-0.8b_f)/2$ =	5.1	in.	
$\lambda n = \sqrt{(d*b_f)}/4$	1.50	in.	
ℓ =	5.1	in.	
$T_{min} = \ell \sqrt{(2*P_u/(0.9F_yBN))}$ =	1.47	in.	
Use T =	1.50	in.	OK



Anchor Rods - Tension			
Number of Anchors =	6	(number of anchors adjacent to column)	
F1554 Grade:	55	ksi	
Tension Force per anchor:	66.7	k	
Tension Force per anchor (override):		k	(value based on eccentric layout)
Anchor Diameter =	1.5	in.	(Reference Table 3.1 for AISC Design Guide)
Anchor Area =	1.77	in^2	(Reference Table 3.1 for AISC Design Guide)
Min Anchor Bolt Spacing =	9	in.	(ACI 318: 17.7.1)
Anchor Edge Distance =	3	in	
Check Anchors	525.69	k	
Size Plate Washer for Yield Strength of Anchor Rod			
Concrete Strength, f'c =	4	ksi	
FyAg =	97.35	k	
A_{brg} =	4.35	in^2	
A_{WASHER} =	6.12	in^2	
Min square washer dim =	2.5in. x 2.5in.		

Table 3.1. Anchor Rod (Rod Only) Available Strength, kips

Rod Diameter, in.	Rod Area, A_r, in²	LRFD ϕR_n, $\phi = 0.75$			ASD R_n/Ω, $\Omega = 2.00$		
		Grade 36, kips	Grade 55, kips	Grade 105, kips	Grade 36, kips	Grade 55, kips	Grade 105, kips
5/8	0.307	10.0	12.9	21.6	6.7	8.6	14.4
3/4	0.442	14.4	18.6	31.1	9.6	12.4	20.7
7/8	0.601	19.6	25.4	42.3	13.1	16.9	28.2
1	0.785	25.6	33.1	55.2	17.1	22.1	36.8
1 1/8	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1 1/4	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1 1/2	1.77	57.7	74.6	124	38.4	49.7	82.8
1 3/4	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2 1/4	3.98	130	168	280	86.5	112	186
2 1/2	4.91	160	207	345	107	138	230
2 3/4	5.94	194	251	418	129	167	278
3	7.07	231	298	497	154	199	331
3 1/4	8.30	271	350	583	180	233	389
3 1/2	9.62	314	406	677	209	271	451
3 3/4	11.0	360	466	777	240	311	518
4	12.6	410	530	884	273	353	589

BRACED FRAME BASE CONNECTION

Size Plate Washer for Yield Strength of Anchor Rod			
Concrete Strength, f'c =	4	ksi	
FyAg =	97.35	k	
A_{brg} =	4.35	in^2	
A_{WASHER} =	6.12	in^2	
Min square washer dim =	2.5in. x 2.5in.		
Shear Lug Size			
Number of shear lugs:	2		
Grout thickness =	1	in.	
Shear Lug Plate Fy =	50	ksi	
Shear Lug Width =	15	in.	
Shear lug depth, $d = \Phi R_n/(0.8f'_c b)$ =	1.0	in.	
Use d =	2.0	in.	OK
Total d (includes grout thk.)=	3.0	in.	
Concrete pier width =	60	in.	
Clear dist from lug to edge of conc. =	24	in.	
Depth of concrete failure plane =	26.0	in.	
Av =	1530.0	in^2	
$Vu = \Phi 4\sqrt{(f'_c)}A_v$ =	355.5	k	OK
Mu =	91.6	k-in	
$t_{min} = \sqrt{(M_u*4/(\Phi F_y b))}$ =	0.74	in.	
Use t =	1.0	in.	OK
Check base plate T >= $t/\sqrt{(2)}$ =	0.71	in.	OK
Shear lug dimensions:	1 thk. x 3 dp		
Shear Lug Welds			
C = T = Mu/t =	91.6	k	
$V_1 = V_2$ =	22.90970468	k	
R_n =	6.3	k/in	
Min. fillet weld size (each side) =	0.141	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		
	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
BF2.2	25.9	18.2	10.8	62.8	38.0	39.30	42.01	26.06
BF4	31.1	27.2	18.6	88.7	49.2	53.42	60.65	33.62
BF6	29.8	5.3	5.2	42.6	33.3	27.01	25.45	23.16
BFB	21.1	9.3	4.9	39.3	27.3	25.58	25.13	18.82
BFC	42.9	15.6	7.8	73.1	53.3	48.17	45.78	36.82

Footing Size		Grade Beam		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				

BRACED FRAME FOUNDATIONS - UPLIFT CHECK

				Footing			Adjacent Grade Beam		
		EQ [k]	Col DL	Ftg [klf]	trib to Col	Ftg [k]	Grade Beam [klf]	trib to Col	Grade Bm DL [k
BF2.2	Col 1	25.9	18.2	1.875	11.0	20.6	0.45	6.6	3.0
	Col 2	25.9	17.6	1.875	6.0	11.3	0.45	6.5	2.9
BF4	Col 1	31.1	15.8	1.875	10.1	18.9	0.45	6.1	2.7
	Col 2	31.1	27.2	1.875	10.1	18.9	0.45	8.1	3.6
BF6	Col 1	29.8	4.9	1.875	6.0	11.3	0.45	21.0	9.5
	Col 2	29.8	5.3	1.875	6.0	11.3	0.45	15.3	6.9
BFB	Col 1	21.1	9.3	1.875	5.0	9.4	0.45	8.0	3.6
	Col 2	21.1	4.3	1.875	11.0	20.6	0.45	11.6	5.2
BFC	Col 1	42.9	15.6	1.875	11.6	21.7	0.45	15.0	6.8
	Col 2	42.9	11.8	1.875	8.1	15.2	0.45	35.0	15.8

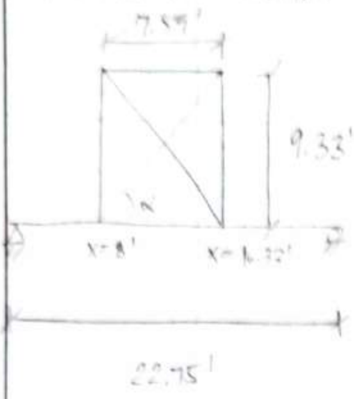
		Slab on grade				Concrete Stem/Wall				Add'l Wall Load			
		Slab DL [ksf]	Trib width [ft]	Trib length [ft]	Slab DL [k]	Weight [ksf]	Height [ft]	Trib length [	Wall DL [k]	Weight [klf]	Tag	Trib length [	Wall DL [k]
BF2.2	Col 1	0.075	3.125	11.6	2.7	0.10			0				
	Col 2	0.075	9.835	12.5	9.2	0.10			0				
BF4	Col 1	0.075	9.96	14.9	11.1	0.10			0				
	Col 2	0.075	9.96	6.8	5.0	0.10			0				
BF6	Col 1	0.075	5	16.0	6.0	0.10	2	14	2.8	0.403	FW1	9	3.627
	Col 2	0.075	7.75	11.3	6.6	0.10	2	10	2	0.336	FW1	10	3.36
BFB	Col 1	0.075	3.125	5.0	1.2	0.10	8	16	12.8				
	Col 2	0.075	3.125	11.6	2.7	0.10			0				
BFC	Col 1	0.075	2	9.1	1.4	0.10			0				
	Col 2	0.075	10.25	18.0	13.8	0.10			0				
			*point load from grade beam										

		Add'l Wall Load				Addition Pt Load					
		Weight [klf]	Tag	Trib length [	Wall DL [k]	Tag	DL [k]	Total DL	0.4359D	0.7E [k]	
BF2.2	Col 1							44.5	19.4	18.1	0.93
	Col 2							41.0	17.9	18.1	1.01
BF4	Col 1							48.5	21.2	21.8	1.03
	Col 2					RP6+FP11	6.287	61.1	26.6	21.8	0.82
BF6	Col 1	0.247	FW2	8.5	2.0995	FP1+FP4+FP1	10.212	50.4	22.0	20.9	0.95
	Col 2	0.336		10	3.36	FP8+RP3	9.2	48.0	20.9	20.9	1.00
BFB	Col 1							36.2	15.8	14.8	0.94
	Col 2							32.9	14.3	14.8	1.03
BFC	Col 1					RP6+FP11+RI	33.887	79.3	34.6	30.0	0.87
	Col 2	0.247	FW2	5	1.235	FP16+RP11+I	14.758	72.5	31.6	30.0	0.95

all within 5%, ok

REFER TO FOUNDATIONS SECTION FOR BRACED FRAME PIN PILES

TRANSFER BEAM BELOW BF-1



$\alpha = \tan^{-1}(9.33'/7.59') = 50.87°$

BRACE DEMAND = 7.1k (ρ=1.3, 1.0E0, not Ω or EXPECTED)

$V_{comp} = \sin 50.87 = V/7.1 = 5.5k$

$H_{comp} = \cos 50.87 = H/7.1 = 4.5k$

MULTIPLY BY Λ=2 (R=5/CONC SW): 5.5k(2) = 11k ↓ ↑

CASE 1: 11k↓ @ x=8'   11k ↑ @ x=16.33'

CASE 2: 11k ↑ @ x=8'   11k↓ @ x=16.33'

ADD 4.5k AXIAL x Λ=2 = 9k

L=22.5FT   9k/22.5' = 0.4 klf

GRAVITY LOADS:

ROOF:

DL = 20PSF x (3'+10'/2) = 160 plf

SL = 25 psf x (3'+10/2) = 200 plf

GLAZING: 15psf x 9.33' = 140 plf

MAIN:

DL: 80 psf x (7'/2) = 280 plf

LL: 40psf x 7'/2 = 140 plf

TOTAL DL = 580 plf

TOTAL SL/LL = 340 plf

RESULTS PER VA

DCR = 0.58

COL LOADS:

DL = 1.3k @ x=8' & x=16.33'

SL = 1.7k

SWENSON SAY FAGET

SEATTLE
TACOMA

STRUCTURAL ENGINEERING

8480

PROJECT TRANSFER BEAM

DATE

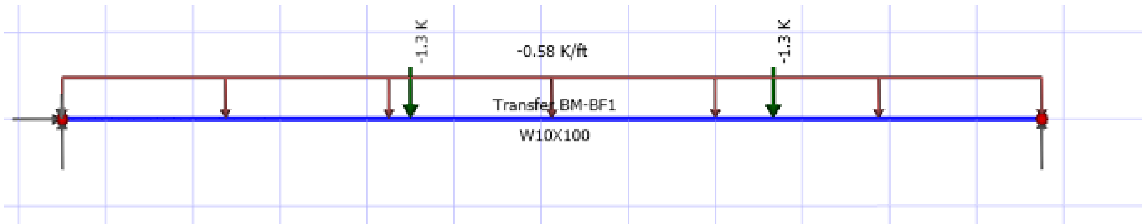
PROJ #

DESIGN

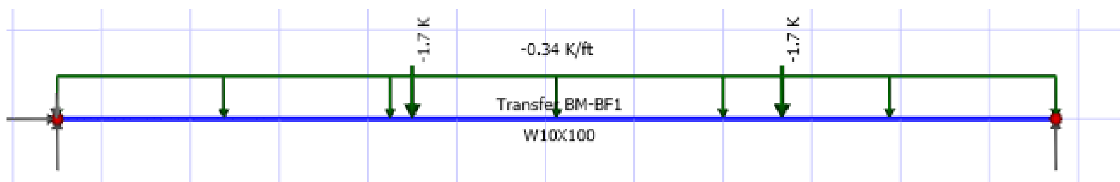
SHEET

TRANSFER BEAM AT BRACED FRAME

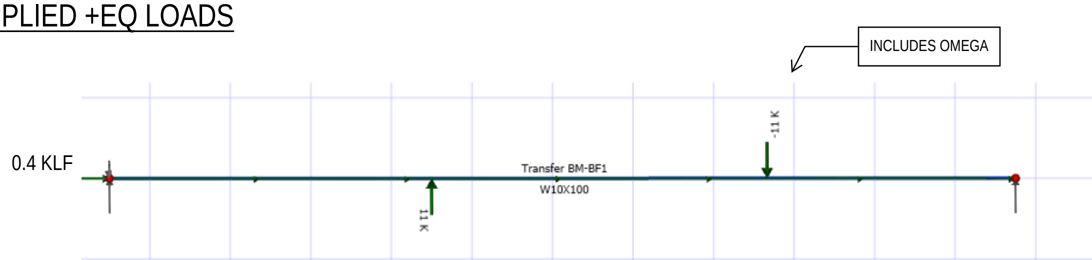
APPLIED DEAD LOADS



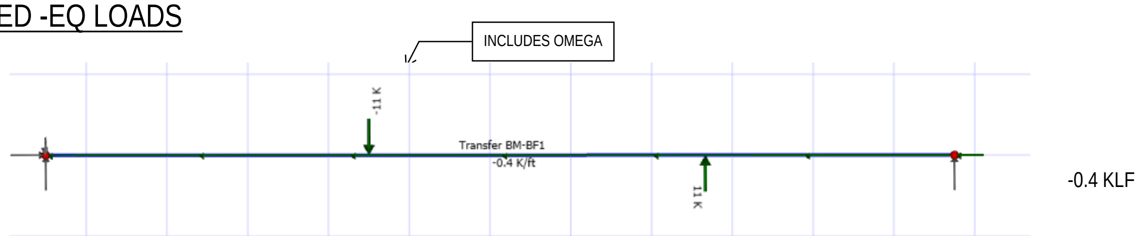
APPLIED LIVE LOADS



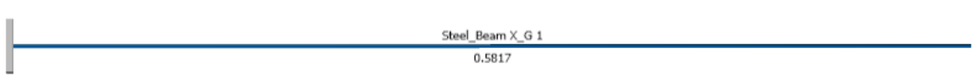
APPLIED +EQ LOADS



APPLIED -EQ LOADS

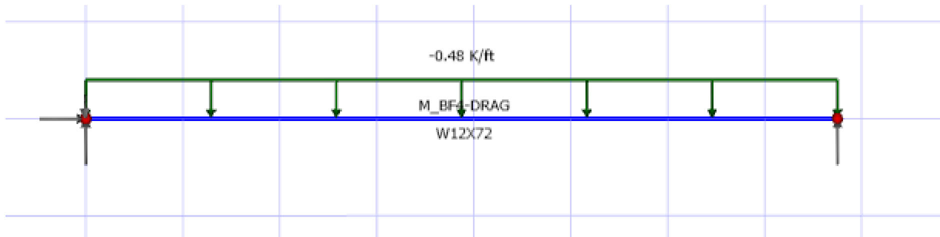


DCR

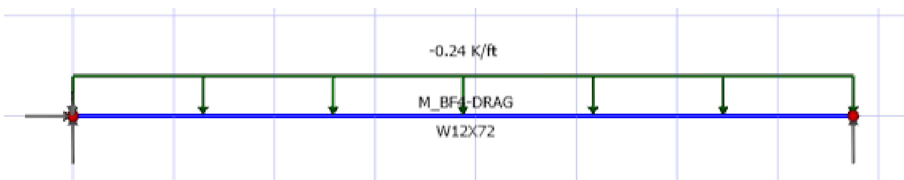


DRAG STRUT AT BRACED FRAME 4

APPLIED DEAD LOADS

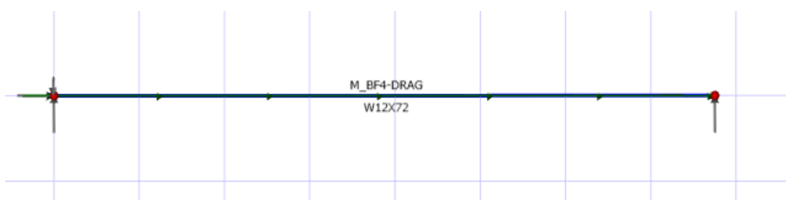


APPLIED LIVE LOADS

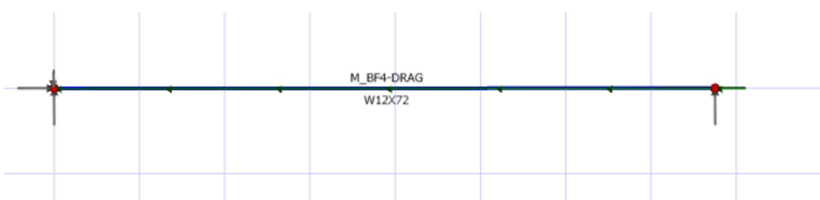


APPLIED +EQ LOADS

3.5 KLF

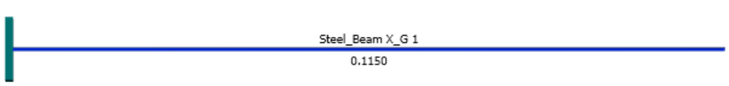


APPLIED -EQ LOADS



-3.5 KLF

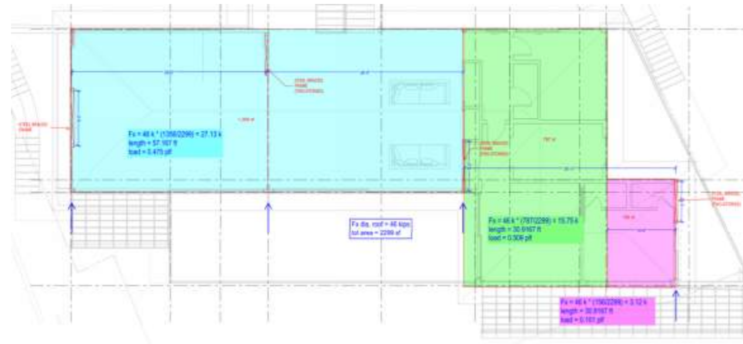
DCR



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



Ω =	2	
Nail size =	10d	
Nominal w =	3	in, of nailed faced at panel edges
Diaphragm t =	19/32"	PLY sheating
Capacity =	384	plf, LRFD for unblocked
	576	plf, LRFD for 6" spacing blkg
	768	plf, LRFD for 4" spacing blkg
	1152	plf, LRFD for 2.5" spacing blkg
	1312	plf, LRFD for 2" spacing blkg

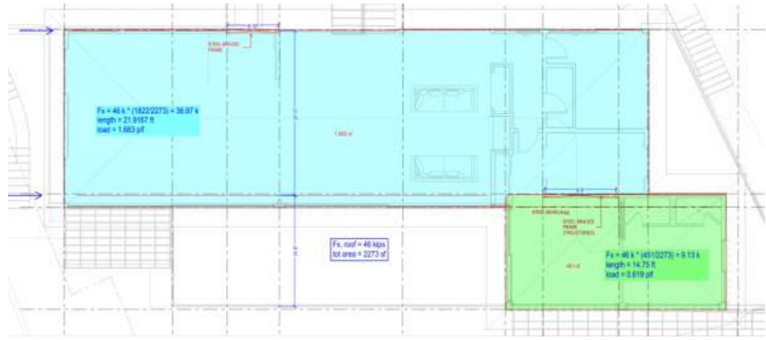
BF Line	Vx dist (plf)	Trib (ft)	V to BF (lbs)	BF L (Ft)	BF V (plf)	V*1.25 (plf)	Strut L have (ft)	Total L have (ft)	V w/ Strut (plf)	Dia Check	Blkg L (ft)	Blkg Nailing	Strut Load * Ω (lbs)
Grid 1	475	14.2	6749	8.2	826	1033	12.8	20.9	403	add blkg	0.9	6in spacing	954
Grid 2.2	475	28.6	13577	5.8	2361	2952	17.0	22.8	746	add blkg	17.3	4in spacing	18322
Grid 4 L	475	14.4	6828	7.1	964	1205	8.1	11.6	734	add blkg	8.6	4in spacing	2776
Grid 4 R	509	15.5	7868	7.1	1111	1389	15.1	18.6	528	add blkg	5.3	6in spacing	4857
Grid 6	509	5.5	2778	-	-	-	-	-	-	-	-	-	-
Grid 6	101	10.0	1010	-	-	-	-	-	-	-	-	-	-
Grid 6	varies	15.5	3788	5.6	679	848	8.3	13.9	340	OK	-	-	-

Span	Grids Btwn	Span L** (ft)	Span Type	Moment (k-ft)	Moment Arm (ft)	Chord T/C (kips)	Chord
1	1-2.2	28.4	s.s.	47.9	21.9	2.2	Steel PL
2	2.2-4	28.8	s.s.	49.1	21.9	2.2	Steel PL
3	4-6	30.8	s.s.	60.5	36.8	1.6	Steel PL
3 (red)	4-6	20.5	s.s.	5.3	14.9	0.4	Steel PL

** Span L is the location checking moment at for 3 (red)

Main House Roof Diaphragm -- East-West Direction

Ω =	2	
Nail size =	10d	
Nominal w =	3	in, of nailed faced at panel edges
Diaphragm t =	19/32"	PLY sheating
Capacity =	384	plf, LRFD for unblocked
	576	plf, LRFD for 6" spacing blkg
	768	plf, LRFD for 4" spacing blkg
	1152	plf, LRFD for 2.5" spacing blkg
	1312	plf, LRFD for 2" spacing blkg



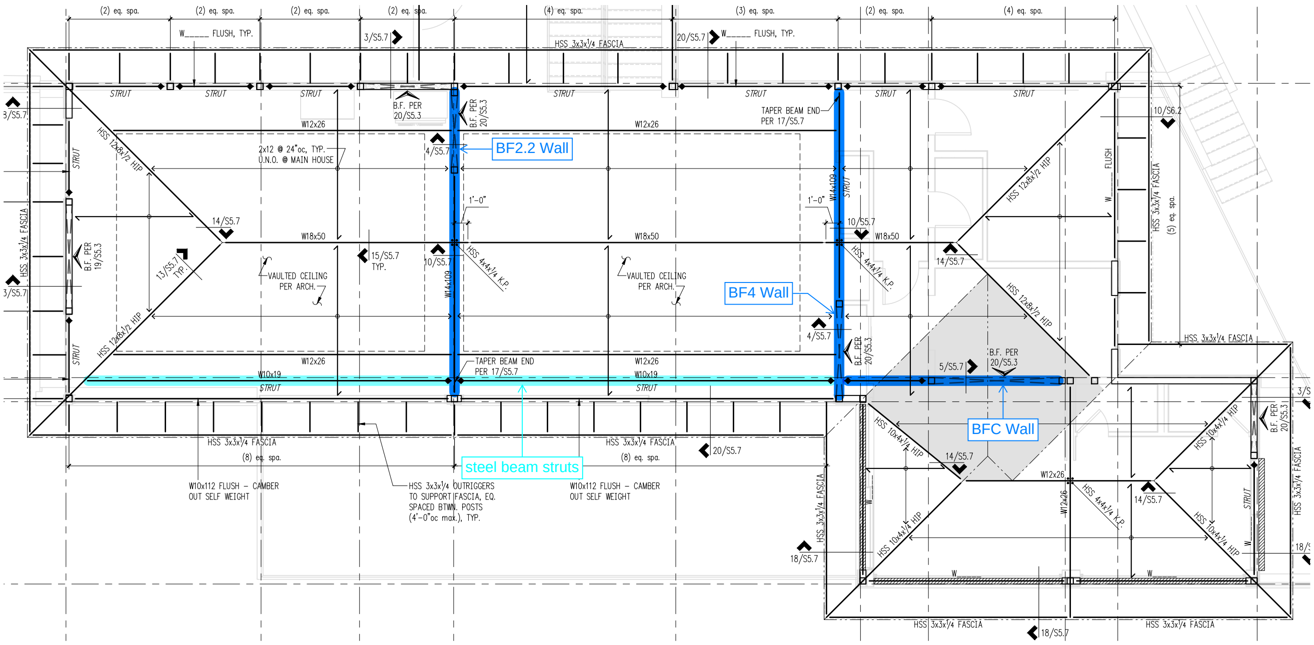
BF Line	Vx dist (plf)	Trib (ft)	V to BF (lbs)	BF L (Ft)	BF V (plf)	V*1.25 (plf)	Strut L have (ft)	Total L have (ft)	V w/ Strut (plf)	Dia Check	Blkg L (ft)	Blkg Nailing	Strut Load * Ω (lbs)
Grid B	1683	11.0	18443	8.2	2258	2823	70.8	78.9	292	OK	-	-	24342
Grid C L	1683	11.0	18443	5.6	3303	4129	64.1	66.9	345	OK	-	-	28310
Grid C R	619	14.8	9130	5.6	1635	2044	14.0	16.8	680	add blkg	8.0	4in spacing	9685

Span	Grids Btwn	Span L (ft)	Span Type	Moment (k-ft)	Moment Arm (ft)	Chord T/C (kips)	Chord
1	B-C	21.9	s.s.	101.1	77.8	1.3	Steel PL
2	C-E	14.9	cant.	68.9	29.1	2.4	Steel PL

SSF STRUCTURAL ENGINEERING

8480 Residence PROJECT	02/24/2022 DATE
	01519-2021-09 PROJ. #
	LAN DESIGN
	SHEET

Cripple Shearwall Design



	BF2.2 WALL	BF4 WALL	BFC WALL
V (k) EQ	37.3	19.2	36.0
V (k) EQ ASD	26.1	13.4	25.2
L (ft) EQ	22.75	23.25	17.17
V (plf) EQ	1148	576	480
SW type	2W2	W3	W2
H (ft)	2.25	2.25	2.25
OT (k)	2.58	1.29	1.08
DL (k)	0.71	0.73	1.05
Net Up (k)	2.28	0.99	0.45
HD	HDU2	-	-

cripple wall L = 17.17 ft so 25.2/17.17 = 1468 plf BUT limited by how much load can get in through diaphragm (capacity = 480 plf) so 8.24 k can go through cripple--> rest goes through steel beam struts West of Grid 4

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

DATE 2/23/2022
PROJ. # 01519-2021-09
DESIGN LAN
SHEET L-42

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	0.235
V (ELF), ρ=1.0	102.6
K	1
Sds	1.172
I	1

ELF

Vertical Distribution

Strength		ρ= 1.3				Story Shear		Diaphragm				
						Strength		Force (ρ not included)				ELF Scaling
Level	hx (ft)	Wx	hx^k (ft)	$Wxhx^k$	Cvx (%)	Fx (k)	ΣV (k)	Min Fpx (k)	Max Fpx (k)	Fpx (k)	γ=Fpx/Fx	Fpx/Fx
Roof	23.5	161.0	23.5	3783.5	0.448	59.82	59.8	37.7	75.5	46.0	0.77	1.00
Main	11.0	423.0	11.0	4653.0	0.552	73.56	133.4	99.2	198.3	99.2	1.35	1.35
	Σ	584.0		8436.5		133.38						

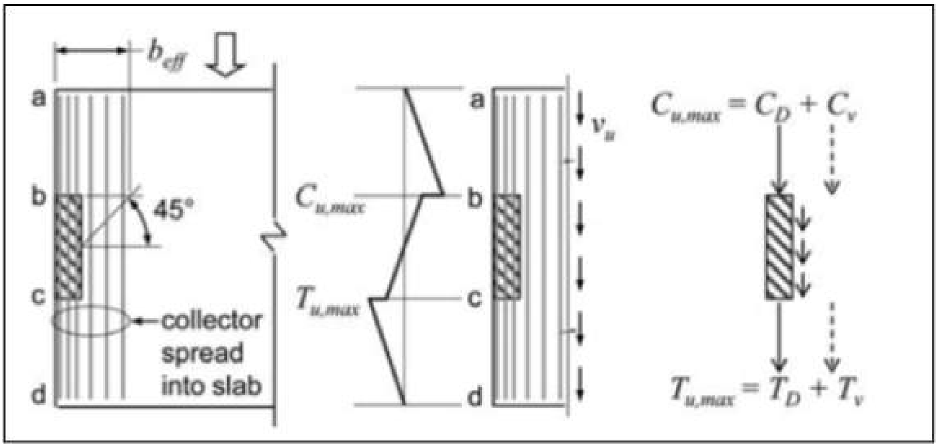
STRUCTURAL ENGINEERING

8480 Residence
PROJECT
Lateral Design

2/23/22
DATE
PROJ. #
SRW
DESIGN
L-44
SHEET

CONCRETE DIAPHRAGMS

Fy	60	ksi	
Fc'	4	ksi	
φb	0.9		
φv	0.6		
μ, at shear wall	0.6	will detail as roughened surface - but conservatively assume not roughened	
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	143.66	179.58	1.35	148.95	179.58	23.50	7.64
2	2	SC-WB-1-R	X	Wall	190.38	237.97	1.35	197.38	237.97	24.67	9.65
3	3	SC-WB-2-R	X	Wall	68.91	86.13	1.35	71.44	86.13	33.23	2.59
4	4	SC-BF-B-R	X	Brace	46.30	57.87	1.35	48.00	57.87	6.00	9.64
8	8	SC-BF-C-R	X	Brace	15.58	19.47	1.35	16.15	19.47	9.17	2.12
9	9	SC-BF-C-L	X	Brace	40.44	50.55	1.35	41.93	50.55	9.17	5.51
12	12	SC-W1-R	Y	Wall	37.73	47.17	1.35	39.12	47.17	23.25	2.03
13	13	SC-W2-R	Y	Wall	139.87	174.84	1.35	145.02	174.84	47.42	3.69
14	14	SC-W3-L	Y	Wall	181.65	227.06	1.35	188.33	227.06	35.00	6.49
18	18	SC-BF-2.2-L	Y	Brace	19.49	24.36	1.35	20.21	24.36	5.67	4.30
20	20	SC-BF-2.2-R	Y	Brace	27.51	34.39	1.35	28.52	34.39	5.67	6.07
22	22	SC-BF-4-L	Y	Brace	44.35	55.44	1.35	45.98	55.44	7.13	7.78
23	23	SC-BF-4-R	Y	Brace	71.26	89.07	1.35	73.88	89.07	7.13	12.50
26	26	SC-BF-6-L	Y	Brace	17.17	21.46	1.35	17.80	21.46	5.18	4.14
27	27	SC-BF-6-R	Y	Brace	2.90	3.63	1.35	3.01	3.63	5.18	0.70
0	0										

STRUCTURAL ENGINEERING

8480 Residence	2/23/22
PROJECT	DATE
Lateral Design	PROJ. #
	SRW
	DESIGN
	L-45
	SHEET

CONCRETE DIAPHRAGMS

Section Cut	X or Y?	Floor type	Conc	Roof Deck 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	Decking	Main Floo									Decking				
SC-WB-1-R	X	Decking	Main Floo									Decking				
SC-WB-2-R	X	Decking	Main Floo									Decking				
SC-BF-B-R	X	Decking	Main Floo	8.62	yes		7	ok	9	0	9	Decking			1.34	OK
SC-BF-C-R	X	Decking	Main Floo	8.62	no							Decking				
SC-BF-C-L	X	Decking	Main Floo	8.62	no							Decking				
SC-W1-R	Y	Decking	Main Floo									Decking				
SC-W2-R	Y	Decking	Main Floo									Decking				
SC-W3-L	Y	Decking	Main Floo									Decking				
SC-BF-2.2-L	Y	Decking	Main Floo	8.62	no							Decking				
SC-BF-2.2-R	Y	Decking	Main Floo	8.62	no							Decking				
SC-BF-4-L	Y	Decking	Main Floo	8.62	no							Decking				
SC-BF-4-R	Y	Decking	Main Floo	8.62	yes	16		ok	43	43	0	Decking	2.74	OK		
SC-BF-6-L	Y	Decking	Main Floo	8.62	no							Decking				
SC-BF-6-R	Y	Decking	Main Floo	8.62	no							Decking				

MaxID	ID	Section Cut	X or Y?	Wall or Brace?	Conc to Wall (limit capacity to shear friction) Bar size	Bar Spg	shear capacity, ϕVn [klf]	(shear friction) [klf]	ϕVn limit [klf]	*checks min of shear friction, concrete shear capacity, and limit Collector reqd?	Collector Length L(A-B) [ft]	Collector Length L(C-D) [ft]	Check Diaphragm Shear	Total Collector force [k]	Vu (A-B) [k]	Vu (C-D) [k]	Deck type at collectors	Collector A-B klf	Check Collector A-B	Collector C-D klf	Check Collector C-D
1	1	SC-WA-R	X	Wall	#4	6	12.32	8.64	43.20	no							Decking				
2	2	SC-WB-1-R	X	Wall	#4	6	12.32	8.64	43.20	yes		8	ok	38.19	0.00	38.19	Decking			4.77	OK
3	3	SC-WB-2-R	X	Wall	#4	6	12.32	8.64	43.20	no							Decking				
4	4	SC-BF-B-R	X	Brace													Decking				
8	8	SC-BF-C-R	X	Brace													Decking				
9	9	SC-BF-C-L	X	Brace													Decking				
12	12	SC-W1-R	Y	Wall	#4	6	12.32	8.64	43.20	no							Decking				
13	13	SC-W2-R	Y	Wall	#4	6	12.32	8.64	43.20	no							Decking				
14	14	SC-W3-L	Y	Wall	#4	6	12.32	8.64	43.20	no							Decking				
18	18	SC-BF-2.2-L	Y	Brace													Decking				
20	20	SC-BF-2.2-R	Y	Brace													Decking				
22	22	SC-BF-4-L	Y	Brace													Decking				
23	23	SC-BF-4-R	Y	Brace													Decking				
26	26	SC-BF-6-L	Y	Brace													Decking				
27	27	SC-BF-6-R	Y	Brace													Decking				

DRAG AT BF 'B'

$V_u = 57.87$ K

CONNECTION TO BF W/ SHEAR STUDS @ 24" o.c. 8.62 KLF

57.87K / 6FT = 9.64 KLF

USE COLLECTOR ⇒ W12X72

8.62 KLF × 6 FT = 51.72K

(57.87 − 51.72) = 6.15 K × 2 = 12.3 K
℄

COLLECTOR L = 7'
12.3K / 7 FT = 1.7 KLF
STUDS OK

TRANSFER AXIAL LOAD THROUGH SHEAR TAB

PL½ OK IN TENSION BY INSPECTION

$\phi R_{nw} = 1.392(5) = 6.96$ K/IN × 6IN(2) = 83.52K OK

DRAG AT BF "4"

$V_u = 89.1$K

8.62 KLF SHEAR STUDS DIAPHRAGM TO B4

89.1 K / 7.13FT = 12.5 KLF > 8.62KLF

USE COLLECTOR ⇒ W12X72

8.62KLF × 7.13FT = 61.8 K

89.1K − 61.8K = 27.3K × 2 = 54.6 K
℄

COLLECTOR L = 15.5'
54.6K / 15.5' = 3.52 KLF
STUDS OK

PL½
$\phi P_n = 0.9(50 \text{ KSI})(0.5 \text{IN}) 6 \text{IN} = 135$K OK

5/16" FILLET $\phi R_{nw} = 6.96$ K/IN (6IN)2 = 83.5 K OK

8480
PROJECT LATERAL - MH
DRAGS

2/23/22
DATE
PROJ #
SKW
DESIGN
SHEET

STRUCTURAL ENGINEERING

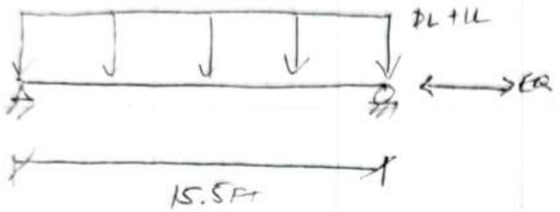
SWENSON SAY FAGET

SEATTLE TACOMA

DRAG AT EF "4" (CONT.)

W12x72 COLLECTOR BM

DL = 80psf (6ft) = 480plf
LL = 40psf (6ft) = 240plf
EQX = +/- 55K OVER 15.5' AXIAL ⇒ 3.5 K/LF +/- X DIRECTION



M_u = 31.4 K·FT ϕM_n = 405 K·FT   DCR = 0.08
V_u = 8.1 K ϕV_n = 158.7 K   DCR = 0.05   PER VA
P_u = 55 K ϕP_n = 722 K   DCR = 0.07

COMBINED AXIAL/FLEXURE   DCR = 0.07

8480 PROJECT LATERAL - MH	2/23/22 DATE
DRAGS	PROJ. # SRW DESIGN
	SHEET

STRUCTURAL ENGINEERING

SWENSON SAY FAGET SEATTLE TACOMA

CONCRETE SHEARWALLS

SWENSON SAY FAGET

Project: 8480
Title: Concrete Wall Shear Design
Date: 2/24/2022
Design: SRW

Pier Info						Seismic Demand		Design						
Pier ID	Story	Wall Thickness	Wall Length	Area	f'c	Vu	Governing Load	Vu / √f'c Ac	ΦVc	ΦVs req'd	rho	Reinf	ΦVn	DCR
		(in)	(ft)	(in^2)	(ksi)	(kips)	Case		Φ=0.6	(kips)	req'rd	Provide	Φ=0.6	(ratio)
P1	Main	8	23.4	2242.0	4	23.7	EQXNTNYN	0.17	170.2	0.0	0.0025	#4 @ 12" EF	506	0.05
P2	Main	8	46.6	4476.0	4	66.1	EQYNTNXN	0.23	339.7	0.0	0.0025	#4 @ 12" EF	1011	0.07
P3	Main	8	46.6	4476.0	4	198.3	EQYNTNXP	0.70	339.7	0.0	0.0025	#4 @ 12" EF	1011	0.20
PA	Main	8	22.9	2193.9	4	39.7	EQXNTNYP	0.29	166.5	0.0	0.0025	#4 @ 12" EF	496	0.09
PB-1	Main	8	23.6	2264.1	4	101.2	EQXNTNYN	0.71	171.8	0.0	0.0025	#4 @ 12" EF	511	0.20
PB-2	Main	8	33.2	3190.0	4	60.0	EQXNTNYP	0.30	242.1	0.0	0.0025	#4 @ 12" EF	721	0.09
0	0	0	0.0	0.0	0	0.0	0	#DIV/0!	0.0	0.0	#DIV/0!	#4 @ 0" EF	#####	####

TABLE: Pier Forces													
Story	Pier	Output Case	Case Type	Step Type	Step Number	Step Label	Location	P	V2	V3	T	M2	M3
								kip	kip	kip	kip-ft	kip-ft	kip-ft
Main	P1	EQXNTNYN	Combination				Top	-0.582	23.733	0.071	-2.6454	-0.0162	5.9479
Main	P1	EQXNTNYN	Combination				Bottom	-8.053	23.078	-0.009	-3.207	0.052	348.5054
Main	P1	EQXNTNYP	Combination				Top	-0.741	17.703	0.183	-1.1396	-0.0318	8.614
Main	P1	EQXNTNYP	Combination				Bottom	-8.06	18.823	-0.023	-2.4931	0.0288	297.7436
Main	P1	EQXNTPYN	Combination				Top	-0.582	23.733	0.071	-2.6454	-0.0162	5.9479
Main	P1	EQXNTPYN	Combination				Bottom	-8.053	23.078	-0.009	-3.207	0.052	348.5054
Main	P1	EQXNTPYP	Combination				Top	-0.741	17.703	0.183	-1.1396	-0.0318	8.614
Main	P1	EQXNTPYP	Combination				Bottom	-8.06	18.823	-0.023	-2.4931	0.0288	297.7436
Main	P1	EQXPTNYN	Combination				Top	0.633	-16.415	-0.173	0.8958	0.0277	-7.3979
Main	P1	EQXPTNYN	Combination				Bottom	6.822	-17.515	0.024	2.1934	-0.0225	-269.0283
Main	P1	EQXPTNYP	Combination				Top	0.474	-22.445	-0.061	2.4017	0.0122	-4.7317
Main	P1	EQXPTNYP	Combination				Bottom	6.815	-21.77	0.01	2.9073	-0.0456	-319.7901
Main	P1	EQXPTPYN	Combination				Top	0.633	-16.415	-0.173	0.8958	0.0277	-7.3979
Main	P1	EQXPTPYN	Combination				Bottom	6.822	-17.515	0.024	2.1934	-0.0225	-269.0283
Main	P1	EQXPTPYP	Combination				Top	0.474	-22.445	-0.061	2.4017	0.0122	-4.7317
Main	P1	EQXPTPYP	Combination				Bottom	6.815	-21.77	0.01	2.9073	-0.0456	-319.7901
Main	P1	EQYNTNXN	Combination				Top	-0.027	18.225	-0.136	-3.427	0.014	-1.2277
Main	P1	EQYNTNXN	Combination				Bottom	-3.253	15.36	0.015	-2.5129	0.0576	213.0891
Main	P1	EQYNTNXP	Combination				Top	0.337	6.181	-0.21	-2.3646	0.0272	-5.2315
Main	P1	EQYNTNXP	Combination				Bottom	1.209	3.182	0.025	-0.8928	0.0353	27.829
Main	P1	EQYNTPXN	Combination				Top	-0.027	18.225	-0.136	-3.427	0.014	-1.2277
Main	P1	EQYNTPXN	Combination				Bottom	-3.253	15.36	0.015	-2.5129	0.0576	213.0891
Main	P1	EQYNTPXP	Combination				Top	0.337	6.181	-0.21	-2.3646	0.0272	-5.2315
Main	P1	EQYNTPXP	Combination				Bottom	1.209	3.182	0.025	-0.8928	0.0353	27.829
Main	P1	EQYPTNXN	Combination				Top	-0.557	-1.875	0.239	1.5925	-0.0378	7.6595
Main	P1	EQYPTNXN	Combination				Bottom	-3.275	1.175	-0.03	-0.1334	-0.0195	43.8831
Main	P1	EQYPTNXP	Combination				Top	-0.192	-13.92	0.166	2.6549	-0.0246	3.6558
Main	P1	EQYPTNXP	Combination				Bottom	1.187	-11.003	-0.02	1.4867	-0.0419	-141.377
Main	P1	EQYPTPXN	Combination				Top	-0.557	-1.875	0.239	1.5925	-0.0378	7.6595
Main	P1	EQYPTPXN	Combination				Bottom	-3.275	1.175	-0.03	-0.1334	-0.0195	43.8831
Main	P1	EQYPTPXP	Combination				Top	-0.192	-13.92	0.166	2.6549	-0.0246	3.6558
Main	P1	EQYPTPXP	Combination				Bottom	1.187	-11.003	-0.02	1.4867	-0.0419	-141.377
Main	P2	EQXNTNYN	Combination				Top	12.655	-46.21	1.583	-34.389	-1.3889	331.6566
Main	P2	EQXNTNYN	Combination				Bottom	40.584	-46.658	0.073	-3.7866	0.613	750.3152
Main	P2	EQXNTNYP	Combination				Top	13.043	-21.708	1.662	-34.3983	-1.1801	318.5845
Main	P2	EQXNTNYP	Combination				Bottom	39.486	-21.882	-0.01	-2.8631	0.632	888.6695
Main	P2	EQXNTPYN	Combination				Top	12.655	-46.21	1.583	-34.389	-1.3889	331.6566
Main	P2	EQXNTPYN	Combination				Bottom	40.584	-46.658	0.073	-3.7866	0.613	750.3152
Main	P2	EQXNTPYP	Combination				Top	13.043	-21.708	1.662	-34.3983	-1.1801	318.5845
Main	P2	EQXNTPYP	Combination				Bottom	39.486	-21.882	-0.01	-2.8631	0.632	888.6695
Main	P2	EQXPTNYN	Combination				Top	-11.471	20.728	-1.324	29.3274	0.9772	-270.993
Main	P2	EQXPTNYN	Combination				Bottom	-35.041	20.997	-0.023	2.713	-0.5395	-676.5469
Main	P2	EQXPTNYP	Combination				Top	-11.082	45.23	-1.244	29.3181	1.1861	-284.065
Main	P2	EQXPTNYP	Combination				Bottom	-36.139	45.772	-0.107	3.6366	-0.5205	-538.1925
Main	P2	EQXPTPYN	Combination				Top	-11.471	20.728	-1.324	29.3274	0.9772	-270.993
Main	P2	EQXPTPYN	Combination				Bottom	-35.041	20.997	-0.023	2.713	-0.5395	-676.5469
Main	P2	EQXPTPYP	Combination				Top	-11.082	45.23	-1.244	29.3181	1.1861	-284.065
Main	P2	EQXPTPYP	Combination				Bottom	-36.139	45.772	-0.107	3.6366	-0.5205	-538.1925
Main	P2	EQYNTNXN	Combination				Top	4.021	-65.302	0.365	-12.3746	-0.8849	147.7109
Main	P2	EQYNTNXN	Combination				Bottom	16.72	-66.077	0.2	-3.0031	0.1911	-14.437
Main	P2	EQYNTNXP	Combination				Top	-3.216	-45.221	-0.507	6.7403	-0.175	-33.084
Main	P2	EQYNTNXP	Combination				Bottom	-5.968	-45.781	0.171	-1.0532	-0.1547	-442.4956
Main	P2	EQYNTPXN	Combination				Top	4.021	-65.302	0.365	-12.3746	-0.8849	147.7109
Main	P2	EQYNTPXN	Combination				Bottom	16.72	-66.077	0.2	-3.0031	0.1911	-14.437
Main	P2	EQYNTPXP	Combination				Top	-3.216	-45.221	-0.507	6.7403	-0.175	-33.084
Main	P2	EQYNTPXP	Combination				Bottom	-5.968	-45.781	0.171	-1.0532	-0.1547	-442.4956
Main	P2	EQYPTNXN	Combination				Top	5.316	16.369	0.63	-12.4059	-0.1886	104.1373
Main	P2	EQYPTNXN	Combination				Bottom	13.06	16.507	-0.078	0.0755	0.2545	446.7443
Main	P2	EQYPTNXP	Combination				Top	-1.921	36.451	-0.242	6.709	0.5213	-76.6575
Main	P2	EQYPTNXP	Combination				Bottom	-9.627	36.803	-0.107	2.0254	-0.0913	18.6856
Main	P2	EQYPTPXN	Combination				Top	5.316	16.369	0.63	-12.4059	-0.1886	104.1373
Main	P2	EQYPTPXN	Combination				Bottom	13.06	16.507	-0.078	0.0755	0.2545	446.7443
Main	P2	EQYPTPXP	Combination				Top	-1.921	36.451	-0.242	6.709	0.5213	-76.6575
Main	P2	EQYPTPXP	Combination				Bottom	-9.627	36.803	-0.107	2.0254	-0.0913	18.6856
Main	P3	EQXNTNYN	Combination				Top	1.426	-35.827	2.601	-39.8855	-1.7968	28.6425
Main	P3	EQXNTNYN	Combination				Bottom	-12.922	-36.036	-0.745	10.6048	-1.3671	-806.1843
Main	P3	EQXNTNYP	Combination				Top	-5.369	57.605	2.23	-26.4223	-3.6615	-143.8117
Main	P3	EQXNTNYP	Combination				Bottom	-45.642	63.146	-0.578	8.0974	-3.1822	-634.247
Main	P3	EQXNTPYN	Combination				Top	1.426	-35.827	2.601	-39.8855	-1.7968	28.6425
Main	P3	EQXNTPYN	Combination				Bottom	-12.922	-36.036	-0.745	10.6048	-1.3671	-806.1843

Main	P3	EQXNTPYP	Combination	Top	-5.369	57.605	2.23	-26.4223	-3.6615	-143.8117
Main	P3	EQXNTPYP	Combination	Bottom	-45.642	63.146	-0.578	8.0974	-3.1822	-634.247
Main	P3	EQXPTNYN	Combination	Top	6.383	-79.84	-1.116	11.4536	4.0957	158.5005
Main	P3	EQXPTNYN	Combination	Bottom	50.51	-86.758	0.137	-1.1646	3.2444	406.037
Main	P3	EQXPTNYP	Combination	Top	-0.411	13.592	-1.487	24.9168	2.2309	-13.9537
Main	P3	EQXPTNYP	Combination	Bottom	17.79	12.425	0.305	-3.672	1.4293	577.9743
Main	P3	EQXPTPYN	Combination	Top	6.383	-79.84	-1.116	11.4536	4.0957	158.5005
Main	P3	EQXPTPYN	Combination	Bottom	50.51	-86.758	0.137	-1.1646	3.2444	406.037
Main	P3	EQXPTPYP	Combination	Top	-0.411	13.592	-1.487	24.9168	2.2309	-13.9537
Main	P3	EQXPTPYP	Combination	Bottom	17.79	12.425	0.305	-3.672	1.4293	577.9743
Main	P3	EQYNTNXN	Combination	Top	13.674	-173.402	1.415	-32.0881	2.5462	347.7462
Main	P3	EQYNTNXN	Combination	Bottom	52.987	-183.12	-0.582	8.2733	2.3084	-503.8316
Main	P3	EQYNTNXP	Combination	Top	15.162	-186.606	0.3	-16.6864	4.314	386.7036
Main	P3	EQYNTNXP	Combination	Bottom	72.017	-198.337	-0.317	4.7425	3.6919	-140.1652
Main	P3	EQYNTPXN	Combination	Top	13.674	-173.402	1.415	-32.0881	2.5462	347.7462
Main	P3	EQYNTPXN	Combination	Bottom	52.987	-183.12	-0.582	8.2733	2.3084	-503.8316
Main	P3	EQYNTPXP	Combination	Top	15.162	-186.606	0.3	-16.6864	4.314	386.7036
Main	P3	EQYNTPXP	Combination	Bottom	72.017	-198.337	-0.317	4.7425	3.6919	-140.1652
Main	P3	EQYPTNXN	Combination	Top	-8.974	138.037	0.176	12.7892	-3.6695	-227.101
Main	P3	EQYPTNXN	Combination	Bottom	-56.081	147.488	-0.023	-0.0847	-3.7419	69.2929
Main	P3	EQYPTNXP	Combination	Top	-7.487	124.833	-0.939	28.1909	-1.9018	-188.1436
Main	P3	EQYPTNXP	Combination	Bottom	-37.051	132.271	0.241	-3.6155	-2.3585	432.9592
Main	P3	EQYPTPXN	Combination	Top	-8.974	138.037	0.176	12.7892	-3.6695	-227.101
Main	P3	EQYPTPXN	Combination	Bottom	-56.081	147.488	-0.023	-0.0847	-3.7419	69.2929
Main	P3	EQYPTPXP	Combination	Top	-7.487	124.833	-0.939	28.1909	-1.9018	-188.1436
Main	P3	EQYPTPXP	Combination	Bottom	-37.051	132.271	0.241	-3.6155	-2.3585	432.9592
Main	PA	EQXNTNYN	Combination	Top	0.535	-36.005	-0.133	-2.4437	0.0156	12.0815
Main	PA	EQXNTNYN	Combination	Bottom	4.3	-36.357	0.023	0.1735	-0.0368	-291.998
Main	PA	EQXNTNYP	Combination	Top	-0.469	-39.329	0.005	-2.5325	-0.0442	12.5475
Main	PA	EQXNTNYP	Combination	Bottom	-1.361	-39.714	0.002	0.3421	-0.0058	-317.3412
Main	PA	EQXNTPYN	Combination	Top	0.535	-36.005	-0.133	-2.4437	0.0156	12.0815
Main	PA	EQXNTPYN	Combination	Bottom	4.3	-36.357	0.023	0.1735	-0.0368	-291.998
Main	PA	EQXNTPYP	Combination	Top	-0.469	-39.329	0.005	-2.5325	-0.0442	12.5475
Main	PA	EQXNTPYP	Combination	Bottom	-1.361	-39.714	0.002	0.3421	-0.0058	-317.3412
Main	PA	EQXPTNYN	Combination	Top	0.508	18.39	-0.091	0.7729	0.0375	-7.5972
Main	PA	EQXPTNYN	Combination	Bottom	2.074	18.51	0.018	-0.2021	-0.0083	134.9183
Main	PA	EQXPTNYP	Combination	Top	-0.495	15.066	0.048	0.6841	-0.0223	-7.1312
Main	PA	EQXPTNYP	Combination	Bottom	-3.587	15.153	-0.003	-0.0336	0.0228	109.5751
Main	PA	EQXPTPYN	Combination	Top	0.508	18.39	-0.091	0.7729	0.0375	-7.5972
Main	PA	EQXPTPYN	Combination	Bottom	2.074	18.51	0.018	-0.2021	-0.0083	134.9183
Main	PA	EQXPTPYP	Combination	Top	-0.495	15.066	0.048	0.6841	-0.0223	-7.1312
Main	PA	EQXPTPYP	Combination	Bottom	-3.587	15.153	-0.003	-0.0336	0.0228	109.5751
Main	PA	EQYNTNXN	Combination	Top	2.093	-4.966	-0.382	-0.6392	0.1322	2.8999
Main	PA	EQYNTNXN	Combination	Bottom	12.706	-5.012	0.067	-0.2623	-0.0762	-42.0566
Main	PA	EQYNTNXP	Combination	Top	2.085	11.353	-0.369	0.3258	0.1387	-3.0037
Main	PA	EQYNTNXP	Combination	Bottom	12.038	11.448	0.065	-0.375	-0.0676	86.0183
Main	PA	EQYNTPXN	Combination	Top	2.093	-4.966	-0.382	-0.6392	0.1322	2.8999
Main	PA	EQYNTPXN	Combination	Bottom	12.706	-5.012	0.067	-0.2623	-0.0762	-42.0566
Main	PA	EQYNTPXP	Combination	Top	2.085	11.353	-0.369	0.3258	0.1387	-3.0037
Main	PA	EQYNTPXP	Combination	Bottom	12.038	11.448	0.065	-0.375	-0.0676	86.0183
Main	PA	EQYPTNXN	Combination	Top	-1.253	-16.046	0.081	-0.9351	-0.0672	4.4532
Main	PA	EQYPTNXN	Combination	Bottom	-6.165	-16.202	-0.002	0.2995	0.0274	-126.534
Main	PA	EQYPTNXP	Combination	Top	-1.261	0.272	0.093	0.0299	-0.0606	-1.4504
Main	PA	EQYPTNXP	Combination	Bottom	-6.833	0.258	-0.004	0.1868	0.0359	1.5409
Main	PA	EQYPTPXN	Combination	Top	-1.253	-16.046	0.081	-0.9351	-0.0672	4.4532
Main	PA	EQYPTPXN	Combination	Bottom	-6.165	-16.202	-0.002	0.2995	0.0274	-126.534
Main	PA	EQYPTPXP	Combination	Top	-1.261	0.272	0.093	0.0299	-0.0606	-1.4504
Main	PA	EQYPTPXP	Combination	Bottom	-6.833	0.258	-0.004	0.1868	0.0359	1.5409
Main	PB-1	EQXNTNYN	Combination	Top	-11.702	-99.747	-1.421	-0.1471	3.2314	176.0098
Main	PB-1	EQXNTNYN	Combination	Bottom	-38.2	-101.236	-0.228	-1.6456	-0.2981	-447.8074
Main	PB-1	EQXNTNYP	Combination	Top	-13.847	-90.715	0.598	-18.2339	-2.4921	152.6464
Main	PB-1	EQXNTNYP	Combination	Bottom	-37.166	-92.432	-0.16	-1.3015	-0.291	-414.7717
Main	PB-1	EQXNTPYN	Combination	Top	-11.702	-99.747	-1.421	-0.1471	3.2314	176.0098
Main	PB-1	EQXNTPYN	Combination	Bottom	-38.2	-101.236	-0.228	-1.6456	-0.2981	-447.8074
Main	PB-1	EQXNTPYP	Combination	Top	-13.847	-90.715	0.598	-18.2339	-2.4921	152.6464
Main	PB-1	EQXNTPYP	Combination	Bottom	-37.166	-92.432	-0.16	-1.3015	-0.291	-414.7717
Main	PB-1	EQXPTNYN	Combination	Top	12.133	78.635	-0.612	17.3076	2.6329	-130.3379
Main	PB-1	EQXPTNYN	Combination	Bottom	31.394	80.045	0.104	0.8491	0.2506	375.7645
Main	PB-1	EQXPTNYP	Combination	Top	9.988	87.667	1.407	-0.7792	-3.0907	-153.7014
Main	PB-1	EQXPTNYP	Combination	Bottom	32.428	88.849	0.172	1.1933	0.2577	408.8002
Main	PB-1	EQXPTPYN	Combination	Top	12.133	78.635	-0.612	17.3076	2.6329	-130.3379
Main	PB-1	EQXPTPYN	Combination	Bottom	31.394	80.045	0.104	0.8491	0.2506	375.7645
Main	PB-1	EQXPTPYP	Combination	Top	9.988	87.667	1.407	-0.7792	-3.0907	-153.7014

Main	PB-1	EQXPTPYP	Combination	Bottom	32.428	88.849	0.172	1.1933	0.2577	408.8002
Main	PB-1	EQYNTNXN	Combination	Top	-1.2	-52.116	-3.611	25.6364	9.7202	101.0287
Main	PB-1	EQYNTNXN	Combination	Bottom	-16.418	-52.246	-0.174	-1.1586	-0.1335	-218.9252
Main	PB-1	EQYNTNXP	Combination	Top	5.951	1.398	-3.368	30.8729	9.5406	9.1244
Main	PB-1	EQYNTNXP	Combination	Bottom	4.46	2.139	-0.075	-0.4102	0.0311	28.1464
Main	PB-1	EQYNTPXN	Combination	Top	-1.2	-52.116	-3.611	25.6364	9.7202	101.0287
Main	PB-1	EQYNTPXN	Combination	Bottom	-16.418	-52.246	-0.174	-1.1586	-0.1335	-218.9252
Main	PB-1	EQYNTPXP	Combination	Top	5.951	1.398	-3.368	30.8729	9.5406	9.1244
Main	PB-1	EQYNTPXP	Combination	Bottom	4.46	2.139	-0.075	-0.4102	0.0311	28.1464
Main	PB-1	EQYPTNXN	Combination	Top	-8.35	-22.009	3.118	-34.6529	-9.3582	23.1506
Main	PB-1	EQYPTNXN	Combination	Bottom	-12.971	-22.9	0.051	-0.0114	-0.1099	-108.8062
Main	PB-1	EQYPTNXP	Combination	Top	-1.199	31.505	3.36	-29.4165	-9.5378	-68.7537
Main	PB-1	EQYPTNXP	Combination	Bottom	7.908	31.485	0.151	0.737	0.0547	138.2654
Main	PB-1	EQYPTPXN	Combination	Top	-8.35	-22.009	3.118	-34.6529	-9.3582	23.1506
Main	PB-1	EQYPTPXN	Combination	Bottom	-12.971	-22.9	0.051	-0.0114	-0.1099	-108.8062
Main	PB-1	EQYPTPXP	Combination	Top	-1.199	31.505	3.36	-29.4165	-9.5378	-68.7537
Main	PB-1	EQYPTPXP	Combination	Bottom	7.908	31.485	0.151	0.737	0.0547	138.2654
Main	PB-2	EQXNTNYN	Combination	Top	-1.16	-48.319	0.868	-40.0347	-7.7718	-21.0404
Main	PB-2	EQXNTNYN	Combination	Bottom	15.462	-51.415	0.831	-22.7302	-0.0085	-302.3659
Main	PB-2	EQXNTNYP	Combination	Top	6.156	-57.426	6.546	-132.8047	-18.0275	98.6794
Main	PB-2	EQXNTNYP	Combination	Bottom	51.517	-60.01	0.821	-16.3602	0.6598	198.6604
Main	PB-2	EQXNTPYN	Combination	Top	-1.16	-48.319	0.868	-40.0347	-7.7718	-21.0404
Main	PB-2	EQXNTPYN	Combination	Bottom	15.462	-51.415	0.831	-22.7302	-0.0085	-302.3659
Main	PB-2	EQXNTPYP	Combination	Top	6.156	-57.426	6.546	-132.8047	-18.0275	98.6794
Main	PB-2	EQXNTPYP	Combination	Bottom	51.517	-60.01	0.821	-16.3602	0.6598	198.6604
Main	PB-2	EQXPTNYN	Combination	Top	-6.972	58.527	-7.454	141.9505	17.7233	-111.7658
Main	PB-2	EQXPTNYN	Combination	Bottom	-54.545	59.746	-0.529	11.7374	-0.7305	-247.3656
Main	PB-2	EQXPTNYP	Combination	Top	0.345	49.42	-1.775	49.1804	7.4676	7.954
Main	PB-2	EQXPTNYP	Combination	Bottom	-18.49	51.151	-0.539	18.1075	-0.0622	253.6607
Main	PB-2	EQXPTPYN	Combination	Top	-6.972	58.527	-7.454	141.9505	17.7233	-111.7658
Main	PB-2	EQXPTPYN	Combination	Bottom	-54.545	59.746	-0.529	11.7374	-0.7305	-247.3656
Main	PB-2	EQXPTPYP	Combination	Top	0.345	49.42	-1.775	49.1804	7.4676	7.954
Main	PB-2	EQXPTPYP	Combination	Bottom	-18.49	51.151	-0.539	18.1075	-0.0622	253.6607
Main	PB-2	EQYNTNXN	Combination	Top	-14.593	9.075	-9.112	137.443	13.7689	-239.9461
Main	PB-2	EQYNTNXN	Combination	Bottom	-58.773	7.241	0.361	-20.7297	-1.1405	-888.7642
Main	PB-2	EQYNTNXP	Combination	Top	-16.336	41.129	-11.608	192.0386	21.4174	-267.1637
Main	PB-2	EQYNTNXP	Combination	Bottom	-79.775	40.589	-0.047	-10.3894	-1.3571	-872.2641
Main	PB-2	EQYNTPXN	Combination	Top	-14.593	9.075	-9.112	137.443	13.7689	-239.9461
Main	PB-2	EQYNTPXN	Combination	Bottom	-58.773	7.241	0.361	-20.7297	-1.1405	-888.7642
Main	PB-2	EQYNTPXP	Combination	Top	-16.336	41.129	-11.608	192.0386	21.4174	-267.1637
Main	PB-2	EQYNTPXP	Combination	Bottom	-79.775	40.589	-0.047	-10.3894	-1.3571	-872.2641
Main	PB-2	EQYPTNXN	Combination	Top	9.797	-21.28	9.816	-171.7905	-20.4166	159.12
Main	PB-2	EQYPTNXN	Combination	Bottom	61.411	-21.411	0.327	0.5038	1.087	781.3235
Main	PB-2	EQYPTNXP	Combination	Top	8.053	10.774	7.32	-117.195	-12.7681	131.9024
Main	PB-2	EQYPTNXP	Combination	Bottom	40.409	11.938	-0.081	10.8441	0.8704	797.8235
Main	PB-2	EQYPTPXN	Combination	Top	9.797	-21.28	9.816	-171.7905	-20.4166	159.12
Main	PB-2	EQYPTPXN	Combination	Bottom	61.411	-21.411	0.327	0.5038	1.087	781.3235
Main	PB-2	EQYPTPXP	Combination	Top	8.053	10.774	7.32	-117.195	-12.7681	131.9024
Main	PB-2	EQYPTPXP	Combination	Bottom	40.409	11.938	-0.081	10.8441	0.8704	797.8235

Main House Gravity

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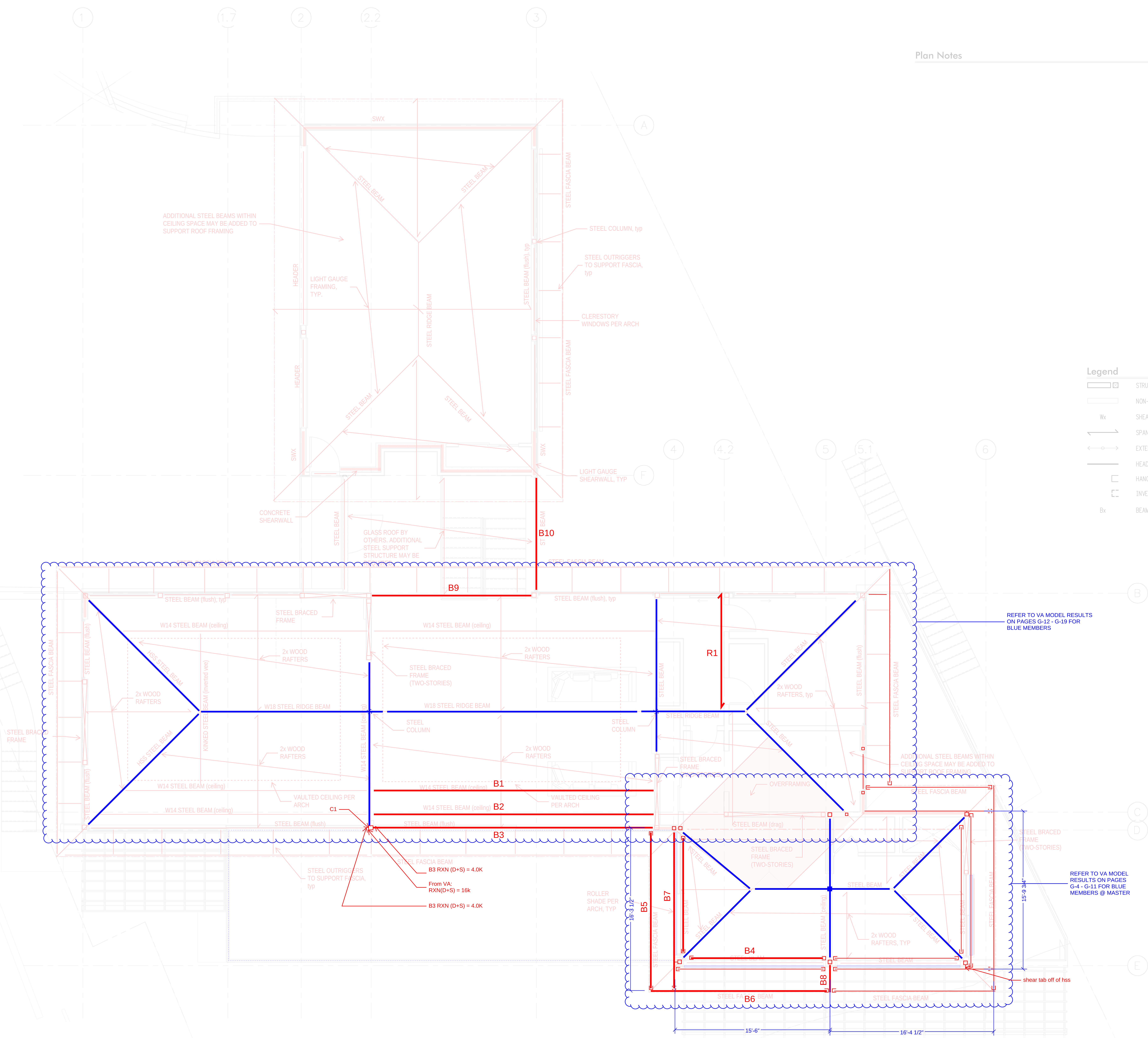
TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402 O 253.284.9470
ssfengineers.com



Plan Notes

Legend

- STRUCTURAL WALL OR POST BELOW
- NON-STRUCTURAL WALL BELOW
- SHEARWALL PER
- SPAN DIRECTION
- EXTENT OF JOISTS
- HEADER/BEAM PER PLAN
- HANGER
- INVERTED HANGER
- BEAM PER SCHEDULE, THIS SHEET

Loads:
DL= 20psf (40psf w/ steel framing weight)
SL= 25psf

Deflection Limits:
Total L/360 (0.75" Max)
Snow L/480
Glazing 1/8" (Total)

Key Plan - Roof Framing

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

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DESIGN: HAA, SRW
DRAWN: NHD
CHECKED: SRW
APPROVED: BDM

REVISIONS:

JURISDICTIONAL APPROVAL STAMP:

PROJECT TITLE:
8480 Residence
8480 85th Ave SE
Mercer Island, WA 98040

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

ISSUE:
PRELIMINARY

SHEET TITLE:
Roof Framing Plan

SCALE: 1/4" = 1'-0" U.N.O.
DATE:
PROJECT NO: 01519-2021-09
SHEET NO:
S2.3

Gravity Design
Roof Framing

R1

L= 12'
w= 90 plf
R= 0.5 k
M= 1.6 k-ft

f_b= 614 psi
f_v= 41 psi
Δ= 0.18"
L/793

2x12@24"oc

B9

L= 17' w= 163 plf Rxn= 1.4 k

DCR_M: 0.09 Δ= 0.11"
DCR_V: 0.03 L/1889

W8x28

B1

L= 29' w= 322 plf Rxn= 4.1 k

DCR_M: 0.27 Δ= 0.48"
DCR_V: 0.07 L/628

MAX 0.5" TO ACCOUNT FOR SOME COMPOUND DEFLECTION DUE TO GIRDER/HIP SUPPORTS

W12x26

B2

L= 29' w= 110 plf Rxn= 1.4 k

DCR_M: 0.17 Δ= 0.44"
DCR_V: 0.03 L/738

MAX 0.5" TO ACCOUNT FOR SOME COMPOUND DEFLECTION DUE TO GIRDER/HIP SUPPORTS

W10x19

B3

L= 29' w= 146 plf Rxn= 2.1 k

DCR_M: 0.04 Δ= 0.112"
DCR_V: 0.01 L/3107

W10x112

note: camber out dead weight to limit deflection to 1/8" at glazing

B4

L= 15' w= 236 plf Rxn= 1.8 k

DCR_M: 0.12 Δ= 0.1"
DCR_V: 0.04 L/1875

W10x19

B5

L= 16.25' w= 62 plf P= .12k (@ 2.75') Rxn= 0.6 k

DCR_M: 0.1 Δ= 0.17"
DCR_V: 0.02 L/1154

HSS 5-1/2x5-1/2x1/4

B6

L= 15' (+2.75' cant.) w= 62 plf P= 1.2 k (@ end) Rxn= 2.0 k

DCR_M: 0.12 Δ= 0.04"
DCR_V: 0.04 2L/1492

HSS 5-1/2x5-1/2x1/4

B7

L= 14.75' (+2.75' cant.) w= 180 plf P= 2.5 k (@ end) Rxn= 3.6 k

DCR_M: 0.28 Δ= 0.13"
DCR_V: 0.07 2L/492

HSS 5-1/2x5-1/2x1/4

B8

L= 2.75' P= 0.93k (@1.75') P= 0.93 k (@ end) R= 1.9 k M= 3.5 k-ft

DCR_M: 0.16 Δ= 0.02"
DCR_V: 0.05 2L/3142

HSS 5-1/2x5-1/2x1/4

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TACOMA 934 Broadway, Suite 100, Tacoma, WA 98402 O 253.284.9470

Gravity Design
Roof Framing

B9

L= 17' w= 163 plf Rxn= 1.4 k

DCR_M: 0.09 Δ= 0.108"
DCR_V: 0.03 L/1889 W8x28

B10

L= 13' w= 478 plf Rxn= 3.1 k

DCR_M: 0.15 Δ= 0.108"
DCR_V: 0.07 L/1444 W8x28

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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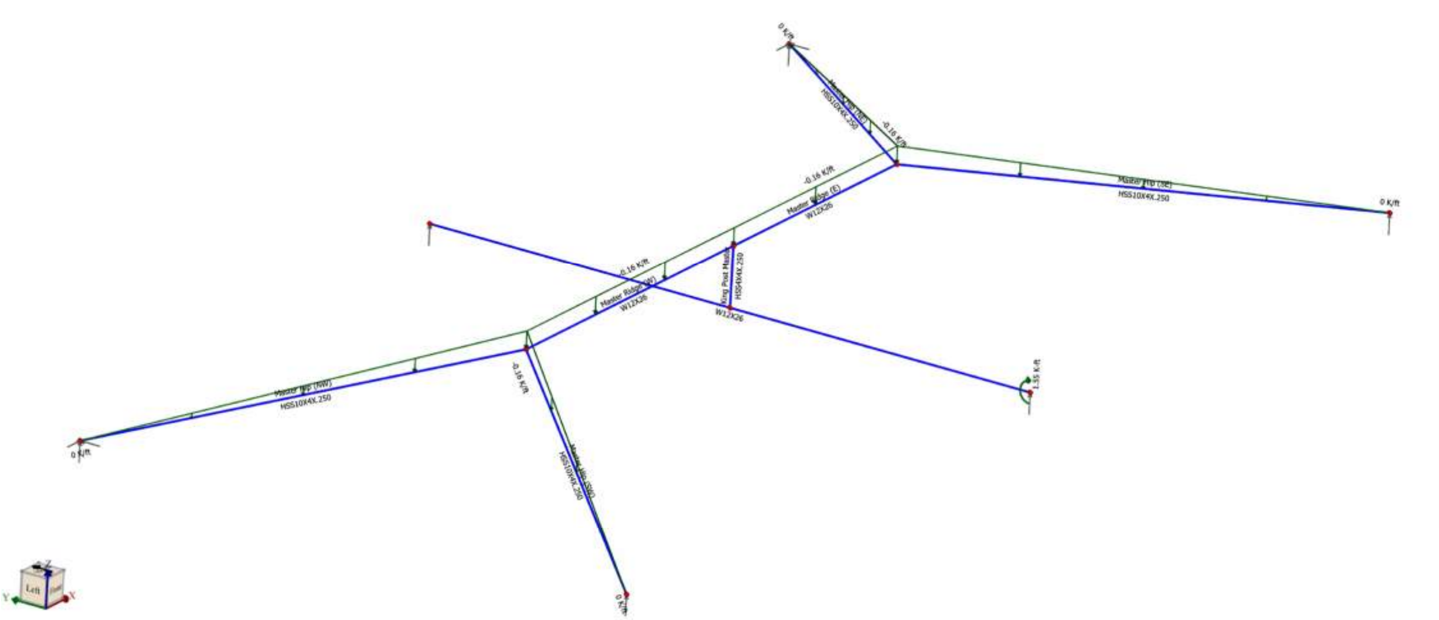
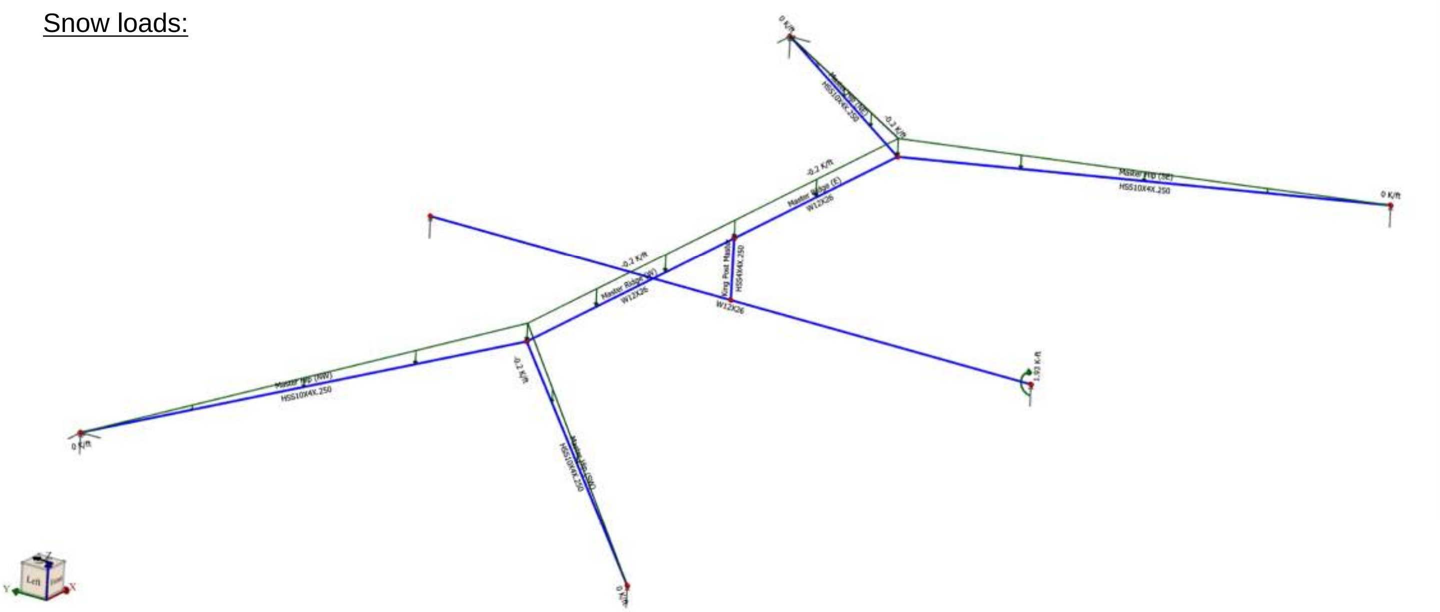
Dead loads:



Snow loads:



Members

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
King Post Master	N026	N032	HSS4X4X.250	1.3750	0.0158	Column
Master Girder	N019	N027	W12X26	14.8333	0.3867	Beam
Master Hip (NE)	N033	N024	HSS10X4X.250	10.5787	0.2224	Bracing
Master Hip (NW)	N023	N030	HSS10X4X.250	9.6899	0.2038	Bracing

Members (continued)

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
Master Hip (SE)	N033	N028	HSS10X4X.250	10.5787	0.2224	Bracing
Master Hip (SW)	N025	N030	HSS10X4X.250	10.5780	0.2224	Bracing
Master Ridge (E)	N032	N033	W12X26	6.1660	0.1608	Beam
Master Ridge (W)	N030	N032	W12X26	7.8340	0.2042	Beam

Result Cases

Name	ID	Design Checks	Result Type
1. D	3	Allowable (ASD)	Static
3. D+S	4	Allowable (ASD)	Static
4. D+0.75(L+S)	5	Allowable (ASD)	Static
7. 0.6D+0.6W	6	Allowable (ASD)	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
King Post Master	**-0.0070 (8)**	**-0.0013 (6)**	**0.0087 (6)**	0.0296 (8)
Master Girder	-0.1667 (8)	0.0000 (8)	**-0.0072 (8)**	**-0.0020 (6)**
Master Hip (SW)	-0.2228 (8)	-0.0048 (6)	0.0020 (6)	**0.1847 (8)**
Master Ridge (E)	-0.1904 (8)	-0.0506 (6)	**0.0087 (6)**	0.0395 (8)
Master Ridge (W)	**-0.2231 (8)**	**-0.0508 (6)**	0.0087 (6)	0.0452 (8)

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
Master Ridge (W)	**0.0069 (6)**	0.0236 (8)	-4.4558 (8)	0.0011 (8)	**-0.0009 (8)**	0.0446 (6)	0.1601 (8)	**-14.0251 (8)**	9.0348 (8)
Master Ridge (E)	**0.0069 (6)**	0.0236 (8)	**4.3830 (8)**	0.0011 (8)	**0.0002 (8)**	**0.0471 (6)**	0.1668 (8)	**-14.0251 (8)**	5.6611 (8)
Master Hip (SW)	-0.1680 (8)	0.1084 (8)	1.2711 (8)	**0.1652 (8)**	0.0000 (5)	0.0000 (8)	**0.8648 (8)**	0.0000 (9)	6.6526 (8)
Master Hip (SE)	-0.1463 (8)	0.1301 (8)	-1.1066 (8)	**-0.1438 (8)**	0.0000 (8)	0.0000 (6)	0.6973 (8)	0.0000 (8)	**5.3649 (8)**
Master Hip (NW)	-0.1742 (8)	0.1022 (8)	1.3320 (8)	-0.1391 (8)	0.0000 (8)	**-0.6769 (8)**	0.0000 (7)	0.0000 (9)	6.8771 (8)
Master Hip (NE)	-0.1290 (8)	**0.1474 (8)**	-1.1108 (8)	-0.1434 (8)	0.0000 (5)	-0.5838 (8)	0.0000 (6)	0.0000 (8)	5.3963 (8)
Master Girder	0.0000 (5)	0.0000 (5)	**-4.8553 (8)**	0.0000 (8)	0.0000 (8)	0.0000 (8)	0.0000 (8)	**-3.4800 (8)**	**31.8121 (8)**
King Post Master	**-8.8546 (8)**	**-2.5682 (6)**	0.0000 (8)	0.0000 (8)	0.0000 (8)	0.0003 (6)	**0.0012 (8)**	0.0000 (9)	0.0000 (8)

Node Results (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K
N023	3. D+S	0.0000	0.0000	0.0000	**-0.0236**	**-0.0011**	1.3504

Node Results (continued) (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K
N023	D+S	0.0000	0.0000	0.0000	**-0.0236**	**-0.0011**	1.3504
N024	3. D+S	0.0000	0.0000	0.0000	**0.0236**	**0.0011**	1.1253
N024	7. 0.6D+0.6W	0.0000	0.0000	0.0000	0.0069	0.0003	**0.3484**
N024	D+S	0.0000	0.0000	0.0000	**0.0236**	**0.0011**	1.1253
N025	3. D+S	**0.0899**	**-0.1721**	0.0000	0.0000	0.0000	1.2927
N025	D+S	**0.0899**	**-0.1721**	0.0000	0.0000	0.0000	1.2927
N026	7. 0.6D+0.6W	0.0021	-0.0089	**-0.0503**	0.0000	0.0000	0.0000
N027	3. D+S	0.0069	-0.0296	0.0000	0.0000	0.0000	**4.8553**
N027	D+S	0.0069	-0.0296	0.0000	0.0000	0.0000	**4.8553**
N028	3. D+S	**-0.0663**	-0.1454	0.0000	0.0000	0.0000	1.1255
N028	D+S	**-0.0663**	-0.1454	0.0000	0.0000	0.0000	1.1255
N030	3. D+S	0.0043	-0.0452	**-0.2231**	0.0000	0.0000	0.0000
N030	D+S	0.0043	-0.0452	**-0.2231**	0.0000	0.0000	0.0000
N032	7. 0.6D+0.6W	0.0013	**-0.0087**	-0.0508	0.0000	0.0000	0.0000

Member Unity Checks

Member	Section	Unity Check	Status	Result Case	Code Reference	Type	Design Group
King Post Master	HSS4X4X.250	0.0961	Pass	3. D+S	E3-2	Axial Check	Steel_Column_G 2
Master Girder	W12X26	0.4071	Pass	3. D+S	F2-2	Strong Flexure Check	Steel_Beam Y_G 2
Master Hip (NE)	HSS10X4X.250	0.3534	Pass	Snow	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
Master Hip (NW)	HSS10X4X.250	0.4572	Pass	Snow	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
Master Hip (SE)	HSS10X4X.250	0.3548	Pass	Snow	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
Master Hip (SW)	HSS10X4X.250	0.4241	Pass	Snow	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
Master Ridge (E)	W12X26	0.1590	Pass	3. D+S	H1-1b	Combined Check	Steel_Beam X_G 2
Master Ridge (W)	W12X26	0.1590	Pass	3. D+S	H1-1b	Combined Check	Steel_Beam X_G 3

Steel_V Brace_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_V Brace_G 1: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 ASD 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: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Master Hip (NW)	HSS10X4X.250	9.6899	Snow	-0.1108	0.2422	IBC 1604.3.1	**0.4572**	L/Δ = 1049.8

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

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Master Hip (SW)	HSS10X4X.250	8.0393	3. D+S	0.1902	1.0000	H1-1b	**0.1902**	KLz = 10.578 ft, KLy = 10.578 ft, KL(torsion) = 10.578 ft, Lb = 10.578 ft, Axial Unity = 0.00003, Mz Unity = 0.15254, My Unity = 0.03768, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.2129

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
Master Hip (SW)	HSS10X4X.250	0.0000	3. D+S	0.1680	116.4765	E3-2	**0.0014**	KLz = 10.578 ft, KLy = 10.578 ft, KL(torsion) = 10.578 ft, Fcr = 31.526 Ksi, Fe (E3-4) = 50.958 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
Master Hip (NW)	HSS10X4X.250	7.9457	3. D+S	6.8771	43.6128	F7-1	**0.1577**	Lb = 9.6899 ft, Cb = 1.2478

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
Master Hip (SW)	HSS10X4X.250	8.0393	3. D+S	0.8648	22.9541	F7-1	**0.0377**	

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
Master Hip (NW)	HSS10X4X.250	0.0000	3. D+S	1.3320	71.6322	G4-1	**0.0186**	Shear Area = 4.3343 in^2, Cv = 1

Steel_Beam X_G 2: Results

Deflections Strong (dy): Member 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 ASD 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 2: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Master Ridge (E)	W12X26	1.9731	Snow	0.0025	0.1542	IBC 1604.3.1	**0.0160**	L/Δ = 30081

Steel_Beam X_G 2: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Master Ridge (E)	W12X26	0.0000	3. D+S	0.1590	1.0000	H1-1b	**0.1590**	KLz = 6.0275 ft, KLy = 6.166 ft, KL(torsion) = 6.166 ft, Lb = 6.166 ft, Axial Unity = 0.0001, Mz Unity = 0.15111, My Unity = 0.00785, Kz = 0.97754, Ky = 1, K(torsion) = 1, Cb = 2.3697

Steel_Beam X_G 2: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Master Ridge (E)	W12X26	0.0000	3. D+S	0.0236	229.0419	D2-1	**0.0001**	

Steel_Beam X_G 2: 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
Master Ridge (E)	W12X26	0.0000	3. D+S	-14.0251	92.8144	F2-1	**0.1511**	Lb = 6.166 ft, Cb = 2.3697

Steel_Beam X_G 2: 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
Master Ridge (E)	W12X26	6.1660	3. D+S	0.1668	20.3842	F6-1	**0.0082**	

Steel_Beam X_G 2: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Master Ridge (E)	W12X26	0.0000	3. D+S	4.3830	56.1200	G2-1	**0.0781**	Shear Area = 2.806 in^2, Cv = 1, h/tw = 47.13

Steel_Beam Y_G 2: 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 ASD 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 2: 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
Master Girder	W12X26	7.4166	3. D+S	31.8121	78.1390	F2-2	**0.4071**	Lp = 5.3118 ft, Lr = 14.879 ft, Lb = 14.833 ft, Cb = 1.3341

Steel_Beam Y_G 2: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Master Girder	W12X26	14.8333	3. D+S	-4.8553	56.1200	G2-1	**0.0865**	Shear Area = 2.806 in^2, Cv = 1, h/tw = 47.13

Steel_Column_G 2: 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_Column_G 2: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 ASD 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_Column_G 2: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
King Post Master	HSS4X4X.250	0.0000	3. D+S	0.0482	1.0000	H1-1b	**0.0482**	KLz = 1.0003 ft, KLy = 1.375 ft, KL(torsion) = 1.375 ft, Lb = 1.375 ft, Axial Unity = 0.09615, Mz Unity = 0, My Unity = 0.00011, Kz = 0.72754, Ky = 1, K(torsion) = 1, Cb = 1

Steel_Column_G 2: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
King Post Master	HSS4X4X.250	0.0000	3. D+S	8.8546	92.0948	E3-2	**0.0961**	KLz = 1.0003 ft, KLy = 1.375 ft, KL(torsion) = 1.375 ft, Fcr = 45.637 Ksi, Fe (E3-4) = 2433.4 Ksi, Kz = 0.72754, Ky = 1, K(torsion) = 1

Steel_Column_G 2: 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
King Post Master	HSS4X4X.250	0.0000	3. D+S	0.0012	10.7655	F7-1	**0.0001**	

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_Beam X_G 3: Results (continued)

Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 ASD 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
Master Ridge (W)	W12X26	0.0000	3. D+S	0.0141	17.9641	H3-8	**0.0008**	Tr = -0.00093 K-ft, Venant Shear = 0.01414 Ksi

Steel_Beam X_G 3: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Master Ridge (W)	W12X26	7.8340	3. D+S	0.1590	1.0000	H1-1b	**0.1590**	KLz = 7.658 ft, KLy = 7.834 ft, KL(torsion) = 7.834 ft, Lb = 7.834 ft, Axial Unity = 0.0001, Mz Unity = 0.15111, My Unity = 0.00785, Kz = 0.97754, Ky = 1, K(torsion) = 1, Cb = 2.4513

Steel_Beam X_G 3: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Master Ridge (W)	W12X26	0.0000	3. D+S	0.0236	229.0419	D2-1	**0.0001**	

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
Master Ridge (W)	W12X26	7.8340	3. D+S	-14.0251	92.8144	F2-1	**0.1511**	Lb = 7.834 ft, Cb = 2.4513

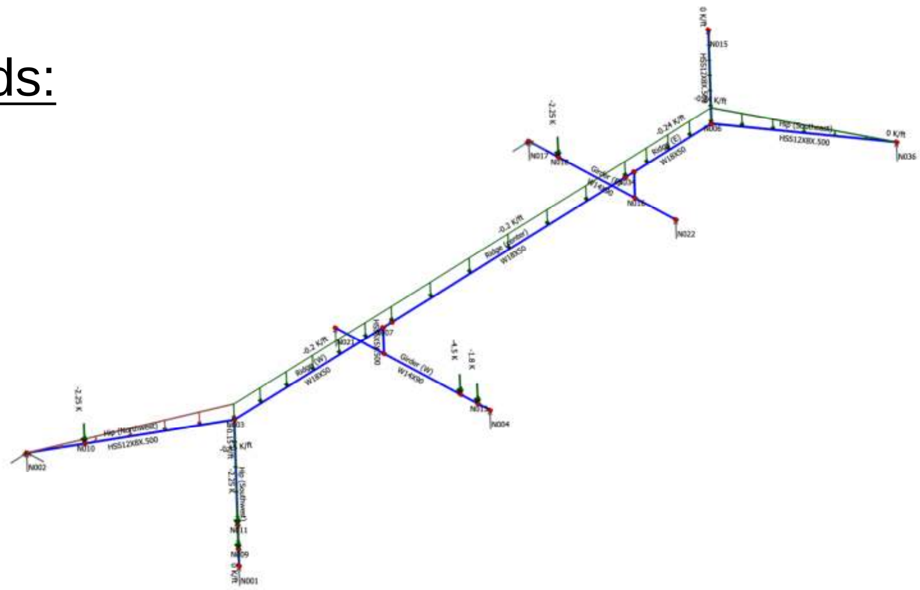
Steel_Beam X_G 3: Weak Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand My K-ft	Capacity My K-ft	Code Reference	Unity Check	Details
Master Ridge (W)	W12X26	7.8340	3. D+S	0.1601	20.3842	F6-1	**0.0079**	

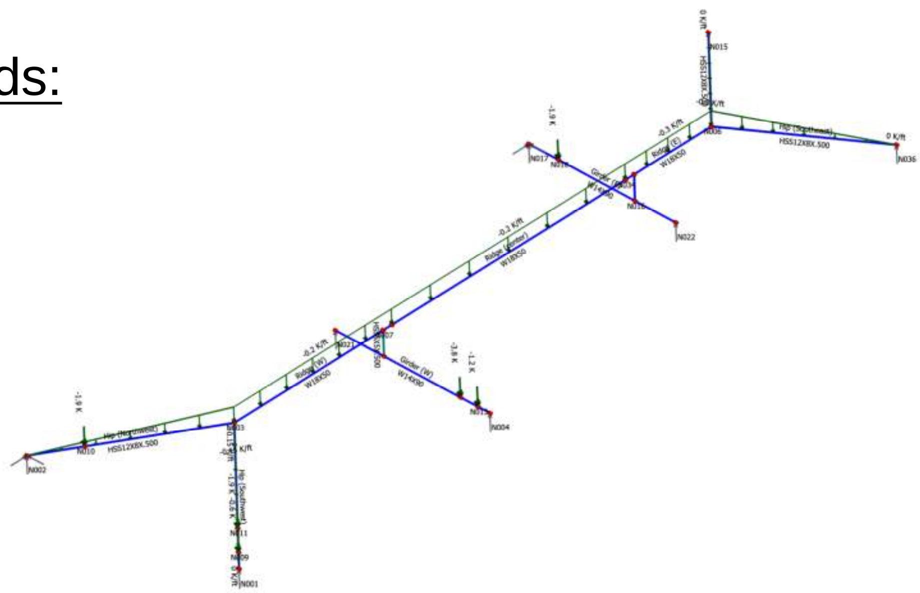
Steel_Beam X_G 3: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Master Ridge (W)	W12X26	7.8340	3. D+S	-4.4558	56.1200	G2-1	**0.0794**	Shear Area = 2.806 in^2, Cv = 1, h/tw = 47.13

Dead Loads:



Snow Loads:



Members

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
Girder (E)	N022	N017	W14X90	16.1663	1.4600	Beam
Girder (W)	N004	N021	W14X90	17.0000	1.5353	Beam
Hip (Northeast)	N006	N015	HSS12X8X.500	16.6931	0.9785	Bracing
Hip (Northwest)	N002	N003	HSS12X8X.500	16.6935	0.9785	Bracing
Hip (Southeast)	N006	N036	HSS12X8X.500	14.8053	0.8678	Bracing

Members (continued)

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
Hip (Southwest)	N001	N003	HSS12X8X.500	16.6938	0.9785	Bracing
King Post1	N005	N007	HSS5X5X.500	2.5417	0.0683	Column
King Post2	N016	N008	HSS5X5X.500	2.5417	0.0683	Column
Ridge (E)	N034	N006	W18X50	9.8340	0.4927	Beam
Ridge (W)	N003	N012	W18X50	18.0003	0.9018	Beam
Ridge (center)	N012	N034	W18X50	26.6660	1.3359	Beam

Result Cases

Name	ID	Design Checks	Result Type	P-Delta?	Seismic Type
1. D	3	Allowable (ASD)	Static	No	N.A.
3. D+S	4	Allowable (ASD)	Static	No	N.A.
4. D+0.75(L+S)	5	Allowable (ASD)	Static	No	N.A.
7. 0.6D+0.6W	6	Allowable (ASD)	Static	No	N.A.
D+L	7	Deflections	Static	No	N.A.
D+S	8	Deflections	Static	No	N.A.
Snow	9	Deflections	Static	No	N.A.

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
Hip (Northeast)	-0.2590 (8)	-0.0043 (6)	**-0.1691 (8)**	0.0031 (8)
Hip (Southwest)	**-0.6665 (8)**	-0.0059 (6)	0.0015 (6)	**0.3028 (8)**
King Post1	**-0.0111 (8)**	**0.0400 (8)**	0.0154 (6)	0.0750 (8)
Ridge (E)	-0.2309 (8)	-0.0183 (6)	-0.0542 (8)	**-0.0067 (6)**
Ridge (W)	-0.6645 (8)	**-0.0311 (6)**	**0.0251 (6)**	0.1335 (8)

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
Girder (W)	0.0000 (9)	0.0000 (8)	**15.7342 (8)**	0.0000 (8)	0.0000 (8)	0.0000 (5)	0.0000 (8)	0.0000 (6)	**76.5621 (8)**
Hip (Northeast)	**-0.4584 (8)**	**0.3768 (8)**	-2.9416 (8)	0.4479 (8)	0.0000 (8)	**-3.3606 (8)**	0.0000 (7)	0.0000 (7)	22.0723 (8)
Hip (Northwest)	-0.9788 (8)	0.1833 (8)	6.2815 (8)	**-0.9564 (8)**	0.0000 (8)	**-5.5621 (8)**	0.0000 (7)	0.0000 (7)	36.5329 (8)
Hip (Southwest)	-1.1811 (8)	0.2094 (8)	7.5797 (8)	**1.1540 (8)**	0.0000 (8)	0.0000 (7)	**5.6643 (8)**	0.0000 (8)	37.2037 (8)
King Post1	**-17.4960 (8)**	**-5.9429 (6)**	0.0000 (8)	0.0000 (8)	0.0000 (8)	0.0000 (6)	0.0000 (8)	0.0000 (8)	0.0000 (7)
Ridge (E)	0.0000 (5)	0.0000 (8)	9.5317 (8)	0.0000 (8)	**-0.0012 (8)**	0.0000 (6)	0.0000 (8)	**-38.3194 (8)**	23.7759 (8)
Ridge (W)	0.0000 (9)	0.0000 (8)	**-10.2307 (8)**	0.0000 (8)	**-0.0003 (8)**	0.0000 (6)	**0.0000 (8)**	**-59.6260 (8)**	49.2569 (8)
Ridge (center)	0.0000 (6)	0.0000 (8)	6.7470 (8)	0.0000 (8)	**-0.0012 (8)**	0.0000 (6)	0.0000 (8)	-52.6539 (8)	**-0.5305 (6)**

Node Results (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K
N001	3. D+S	**0.0800**	**-0.3470**	0.0000	0.0000	0.0000	7.7575
N001	D+S	**0.0800**	**-0.3470**	0.0000	0.0000	0.0000	7.7575
N003	3. D+S	0.0111	-0.1335	**-0.6645**	0.0000	0.0000	0.0000
N003	D+S	0.0111	-0.1335	**-0.6645**	0.0000	0.0000	0.0000
N004	3. D+S	0.0002	-0.0440	0.0000	0.0000	0.0000	**15.7342**
N004	D+S	0.0002	-0.0440	0.0000	0.0000	0.0000	**15.7342**
N013	7. 0.6D+0.6W	-0.0015	-0.0154	**-0.0107**	0.0000	0.0000	0.0000
N015	3. D+S	**-0.0619**	**0.1774**	0.0000	0.0000	0.0000	3.0106
N015	D+S	**-0.0619**	**0.1774**	0.0000	0.0000	0.0000	3.0106
N036	7. 0.6D+0.6W	0.0061	0.0042	0.0000	0.0000	0.0000	**0.9669**

Member Unity Checks

Member	Section	Unity Check	Status	Result Case	Code Reference	Type	Design Group
Girder (E)	W14X90	0.1484	Pass	3. D+S	F3-1	Strong Flexure Check	Steel_Beam Y_G 1
Girder (W)	W14X90	0.2006	Pass	3. D+S	F3-1	Strong Flexure Check	Steel_Beam Y_G 1
Hip (Northeast)	HSS12X8X.500	0.1696	Pass	3. D+S	H1-1b	Combined Check	Steel_H Brace_G 1
Hip (Northwest)	HSS12X8X.500	0.7928	Pass	D+S	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
Hip (Southeast)	HSS12X8X.500	0.1546	Pass	3. D+S	H1-1b	Combined Check	Steel_H Brace_G 1
Hip (Southwest)	HSS12X8X.500	0.7985	Pass	D+S	IBC 1604.3.1	Strong Deflection Check	Steel_V Brace_G 1
King Post1	HSS5X5X.500	0.0822	Pass	3. D+S	E3-2	Axial Check	Steel_Column_G 1
King Post2	HSS5X5X.500	0.0730	Pass	3. D+S	E3-2	Axial Check	Steel_Column_G 1
Ridge (E)	W18X50	0.4697	Pass	D+S	IBC 1604.3.1	Strong Deflection Check	Steel_Beam X_G 1
Ridge (W)	W18X50	0.7383	Pass	D+S	IBC 1604.3.1	Strong Deflection Check	Steel_Beam X_G 1
Ridge (center)	W18X50	0.2216	Pass	3. D+S	F2-3	Strong Flexure Check	Steel_Beam X_G 1

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_Beam X_G 1: Results (continued)

Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 ASD 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: 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	-0.6645	0.9000	IBC 1604.3.1	**0.7383**	L/Δ = 325.09

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	17.0003	3. D+S	-59.6260	251.9960	F2-1	**0.2366**	Lb = 18 ft, Cb = 2.0743

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	17.0003	3. D+S	-10.2307	127.8000	G2-1	**0.0801**	Shear Area = 6.39 in^2, Cv = 1, h/tw = 45.228

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
Ridge (E)	W18X50	0.0000	3. D+S	0.0068	17.9641	H3-8	**0.0004**	Tr = -0.00124 K-ft, Venant Shear = 0.00684 Ksi
Ridge (center)	W18X50	0.0000	3. D+S	0.0068	17.9641	H3-8	**0.0004**	Tr = -0.00124 K-ft, Venant Shear = 0.00684 Ksi

Steel_V Brace_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_V Brace_G 1: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 ASD 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: 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	14.5260	D+S	-0.6665	0.8347	IBC 1604.3.1	**0.7985**	L/Δ = 300.57

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	12.3582	3. D+S	0.2859	1.0000	H1-1b	**0.2859**	KLz = 16.694 ft, KLy = 16.694 ft, KL(torsion) = 16.694 ft, Lb = 16.694 ft, Axial Unity = 0, Mz Unity = 0.238, My Unity = 0.04792, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.0866

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

Member	Section	Offset ft	Result Case	Demand Fx	Capacity Fx K	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	0.0000	3. D+S	1.1811	364.9932	E3-2	**0.0032**	KLz = 16.694 ft, KLy = 16.694 ft, KL(torsion) = 16.694 ft, Fcr = 35.438 Ksi, Fe (E3-4) = 73.81 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	Capacity Mz K-ft	Code Reference	Unity Check	Details
Hip (Southwest)	HSS12X8X.500	12.3582	3. D+S	37.2037	156.3174	F7-1	**0.2380**	Lb = 16.694 ft, Cb = 1.0866

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	12.3582	3. D+S	5.6643	118.2136	F7-1	**0.0479**	

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. D+S	7.5797	162.9995	G4-1	**0.0465**	Shear Area = 9.8627 in^2, Cv = 1

Steel_Beam Y_G 1: Results

Deflections Strong (dy): Member 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 ASD 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 1: Strong Deflection Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
Girder (W)	W14X90	8.8613	D+S	-0.1253	1.0000	IBC 1604.3.1	**0.1253**	L/Δ = 1628

Steel_Beam Y_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
Girder (W)	W14X90	11.6670	3. D+S	76.5621	381.6062	F3-1	**0.2006**	Lb = 17 ft, Cb = 1.2212

Steel_Beam Y_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
Girder (W)	W14X90	0.0000	3. D+S	15.7342	123.2000	G2-1	**0.1277**	Shear Area = 6.16 in^2, Cv = 1, h/tw = 25.864

Steel_Column_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_Column_G 1: Results (continued)

Steel Material: ASTM A500 Grade B (Fy = 46ksi) Specification: AISC 360-16 ASD 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_Column_G 1: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
King Post1	HSS5X5X.500	0.0000	3. D+S	17.4960	212.9762	E3-2	**0.0822**	KLz = 2.5417 ft, KLy = 2.5417 ft, KL(torsion) = 2.5417 ft, Fcr = 45.136 Ksi, Fe (E3-4) = 1015.2 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

Steel_H 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 ASD 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_H Brace_G 1: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Hip (Northeast)	HSS12X8X.500	4.6741	3. D+S	0.1696	1.0000	H1-1b	**0.1696**	KLz = 16.693 ft, KLy = 16.693 ft, KL(torsion) = 16.693 ft, Lb = 16.693 ft, Axial Unity = 0, Mz Unity = 0.1412, My Unity = 0.02843, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.1825

Steel_H 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 (Northeast)	HSS12X8X.500	16.6931	3. D+S	0.4584	365.0012	E3-2	**0.0013**	KLz = 16.693 ft, KLy = 16.693 ft, KL(torsion) = 16.693 ft, Fcr = 35.439 Ksi, Fe (E3-4) = 73.817 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

Steel_H Brace_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
Hip (Northeast)	HSS12X8X.500	4.6741	3. D+S	22.0723	156.3174	F7-1	**0.1412**	Lb = 16.693 ft, Cb = 1.1825

Steel_H 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 (Northeast)	HSS12X8X.500	4.6741	3. D+S	-3.3606	118.2136	F7-1	**0.0284**	

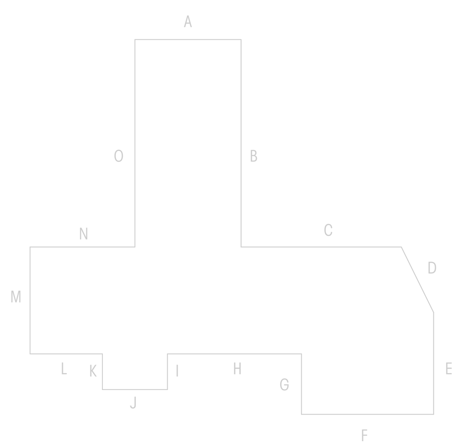
Steel_H Brace_G 1: Strong Shear Check (extreme rows: max)

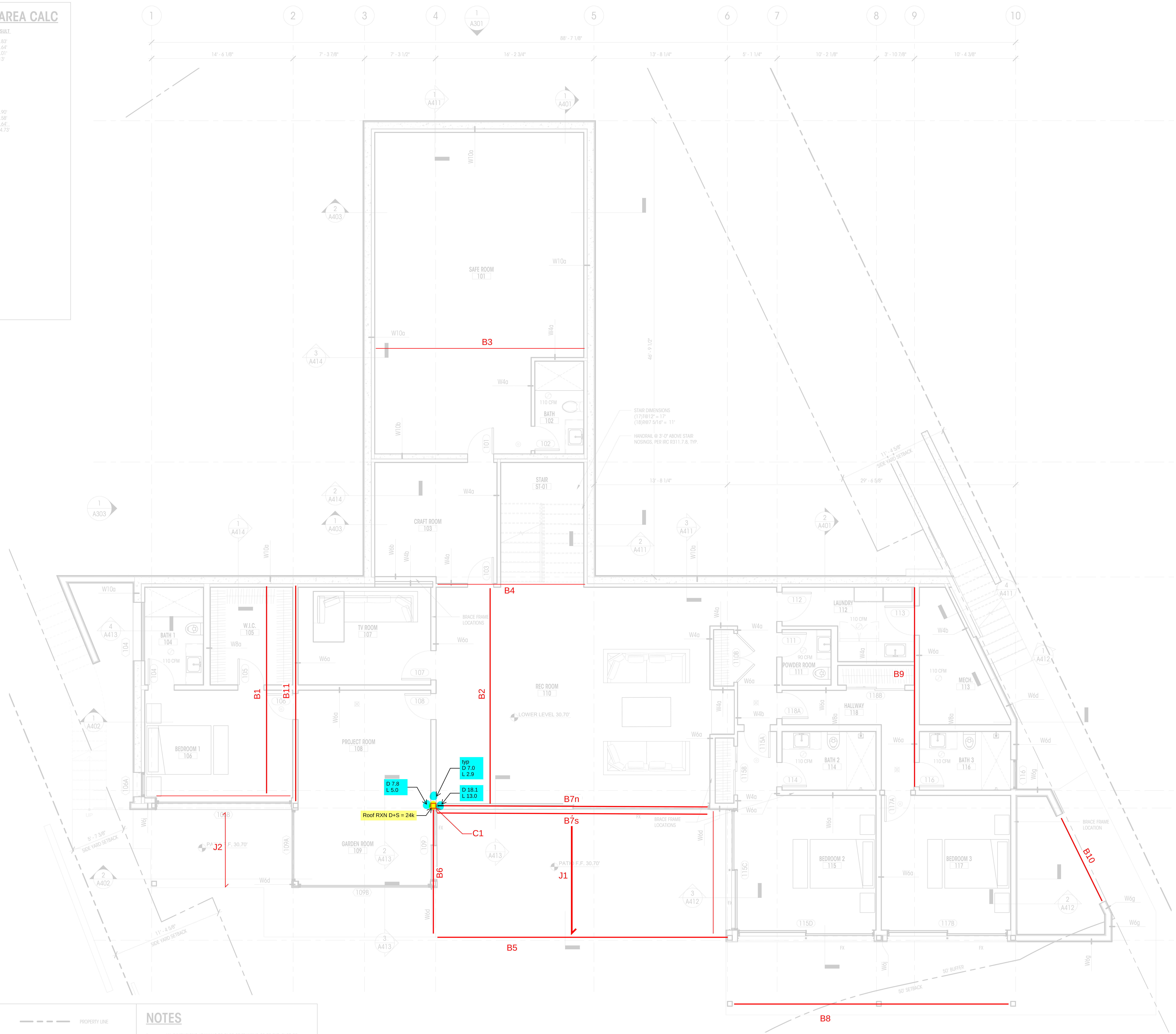
Member	Section	Offset ft	Result Case	Demand Vy	Capacity Vy K	Code Reference	Unity Check	Details
Hip (Northeast)	HSS12X8X.500	16.6931	3. D+S	-2.9416	162.9995	G4-1	**0.0180**	Shear Area = 9.8627 in^2, Cv = 1

BASEMENT LEVEL BELOW GRADE AREA CALC

WALL SEGMENT	LENGTH	COVERAGE	RESULT
A	23.83'	100%	23.83'
B	46.64'	100%	46.64'
C	36.01'	100%	36.01'
D	16.40'	25%	4.13'
E	22.84'	0%	0'
F	29.66'	0%	0'
G	13.56'	0%	0'
H	30.10'	0%	0'
I	8.00'	0%	0'
J	14.63'	0%	0'
K	8.00'	0%	0'
L	16.27'	0%	0'
M	24.00'	58%	13.90'
N	23.58'	100%	23.58'
O	46.64'	100%	46.64'
TOTALS	360.16'		194.73'

TOTAL BASEMENT GSF = 3,750.83 SQ.FT.
PORTION OF EXCLUDED BASEMENT FLOOR AREA: (194.73/360.16) X 3750.83 = 2028 SF
NET BASEMENT GFA: (3,750.83 - 2,028) = **1722.83 SF**





Loads:
Main Floor:
DL= 80psf (90 w/ steel framing weight)
LL= 40psf

Garage:
DL= 80psf (90 w/ steel framing weight)
LL= 40psf or 3000lbs point load

Decks:
DL=25psf (pavers)
LL=60psf

Trellis:
DL= 10psf
LL= 25psf (snow)

Deflection Limits:
L/800 total for steel/concrete

Wood Framing:
L/480 Total
L/600 Live

LEGEND

NOTES

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Design Group
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Seattle, WA
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brandtdesigninc.com

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8480 RESIDENCE
8480 85TH AVE SE
MERCER ISLAND, WA 98040

PROGRESS SET

REVISIONS

PLAN

A211

Key Plan - Floor Framing

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

J2

L= 8'   w= 70 plf   Rxn= 0.3 k

DCR_M: 0.08   Δ= 0.03"
DCR_V: 0.01   L/3200   HSS6x2x1/8

B1

L= 24'   w= 792 plf   Rxn= 9.5 k

DCR_M: 0.19   Δ= 0.34"
DCR_V: 0.08   L/845   W12x72

B2

L= 24'   w= 820 plf   Rxn= 9.8 k

DCR_M: 0.18   Δ= 0.34"
DCR_V: 0.07   L/850   W10x100

B3

L= 24'   w= 612 plf   Rxn= 7.3 k

DCR_M: 0.15   Δ= 0.26"
DCR_V: 0.06   L/1091   W12x72

B3a: Garage concentrated live load

L= 24'   P= 3.0 k (@ mid-span)   Rxn= 6.7 k

DCR_M: 0.16   Δ= 0.27"
DCR_V: 0.06   L/1055   W12x72

B4

L= 16'   w= 2232 plf   Rxn= 17.9 k

DCR_M: 0.26   Δ= 0.19"
DCR_V: 0.16   L/1011   W12x72

B5

L= 31'   w= 694 plf   Rxn= 10.8 k

DCR_M: 0.17   Δ= 0.47"
DCR_V: 0.06   L/800   W12x120

B6

L= 8' (+5.5' cant.)   w= 113 plf   P= 10.5 k @ end   Rxn= 19 k

DCR_M: 0.22   Δ= 0.15"
DCR_V: 0.10   2L/900   W12x72

B7n

note: camber out dead weight to limit deflection to 1/8" at glazing

L= 29'   w= 1600 plf   Rxn= 31 k

DCR_M: 0.31   Δ= 0.482"-3/8"=0.107"
DCR_V: 0.14   L/3252   W18x106 w/ 3/8" camber

B7s

L= 29'   w= 646 plf   Rxn= 9.3 k

DCR_M: 0.25   Δ= 0.567"-1/4"=0.317"
DCR_V: 0.09   L/1098   W12x72 w/ 1/4" camber

B8

L= 15.25'   w= 135 plf   Rxn= 1.0 k

DCR_M: 0.17   Δ= 0.2"
DCR_V: 0.02   L/924   HSS8x2x1/4

B9

L= 24'   w= 1055 plf   Rxn= 12.7 k

DCR_M: 0.21   Δ= 0.33"
DCR_V: 0.09   L/883   W12x96

B10

L= 10' f_b= 775 psi
w= 425 plf f_v= 62 psi
R= 2.1 k   Δ= 0.13"
M= 5.3 k-ft   L/950

LSL 3-1/2x11-7/8

B11

L= 24'   w= 672 plf   P= 10.6 k @ 23ft   Rxn= 18.2 k

DCR_M: 0.20   Δ= 0.33"
DCR_V: 0.17   2L/878   W12x72

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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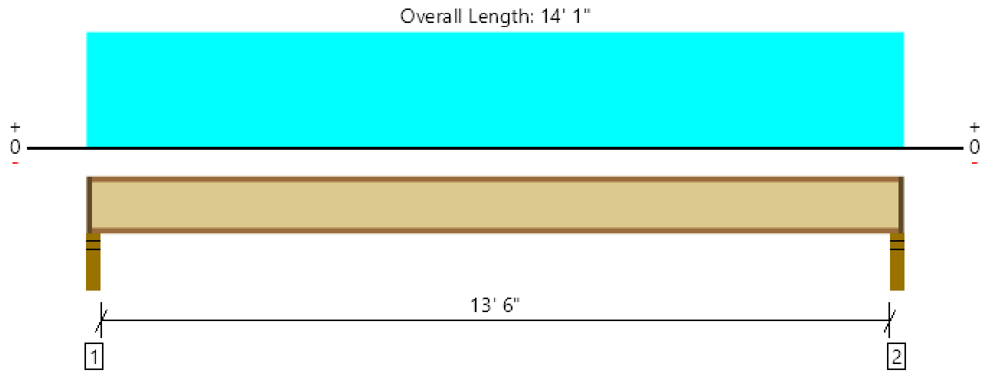
FORTE WEB

MEMBER REPORT

PASSED

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

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

with 4.5 in. 150 pcf 2500 psi NWC

VERCO DECKING, INC.
a NUCOR company

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							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	46.1	3.74	7.30	5.52	4.21	3.88	0.028	15
20	46.5	4.25	7.57	5.91	4.93	3.88	0.028	15
18	47.1	5.14	8.08	6.61	6.30	3.88	0.028	15
16	47.7	5.92	8.55	7.24	7.58	3.88	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 Lateral

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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 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 =	11 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

Roofing	4.6	psf
1/2" Sheathing (+1/8" for fire)	2.3	psf
LG Rafters @ 24"oc	2.0	psf
Steel Framing (seismic only)	10	psf
Ceiling	4.8	psf
(N) Solar Panels & Misc	6.0	psf
	29.7	psf
Use	30	psf

Main Floor

3" Gypcrete	37.5	psf
3.5" Concrete	53.1	psf
1.5" Metal Deck	2.69	psf
Steel Framing	10	psf
Ceiling	2.8	psf
Misc./Mech.	1.5	psf
	107.6	psf
Use	110.0	psf

Concrete Wall

8" Concrete	100	psf
Siding	2.5	psf
	102.5	psf
Use	103.0	psf

Brick Veneer Wall

LG Studs @ 16"o.c.	1.4	psf
vood (1/2" + 1/8" fire)	2.3	psf
Insulation	1.5	psf
5/8" GWB	2.8	psf
Thin Stone Veneer	15.0	psf
Misc./Mech.	1.5	psf
	24.5	psf
Use	25.0	psf

Live Loads:

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

footnote a, over 4.5"x4.5" area

Soils:

Soils Report Provided? Yes

Allowable Bearing	n/a	psf	Active	55/40	pcf (Restrained/Unrestrained)
Sliding, µ	n/a		Seismic Surcharge	9H	
Passive	300	pcf			

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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 (Default)	Assumed default soil properties, per 11.4.3.
Diaphragm Flexibility	Flexible	

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	11.3 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.12 sec	
T	0.12 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
	0.927	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	63.5 k	
$V = C_S W^*$	11.2 k	Eq. 12.8-1, Strength Level Base Shear
$V = C_{Sasd} W^*$	7.8 k	Eq. 12.8-1 ASD Base Shear

*75% base shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	1.2
F_v	

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

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

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

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

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

Vertical Distribution ASD   ρ= 1.3   k= 1.000

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear ASD			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	$y=F_{px}/F_x$
Roof	11.3	64	11.3	717	1.000	10.2	**10.2**	7.8	10.4	20.8	**10.4**	1.03
Σ		63.5		717		**10.2**						

STRUCTURAL ENGINEERING

8480 Residence - Garage		DATE	*2/23/2022*
Seismic Criteria		PROJ. #	01519-2021-09
		DESIGN	HAA/SRW
		SHEET	2

Wind Design - MWFRS

ASCE 7 Chapter 27 - Directional Procedure

Design Method	ASD

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

Location and Building Dimensions

Calculate Kzt?	No	
Kzt	1.00	
Roof Type	Hip	
Roof Angle - Transverse Dir	14	degrees
Roof Angle - Long Dir	14	degrees
Ground to top of roof	13.25	ft
Bot of roof to top of roof	3.92	ft
Mean Roof Height, h	11.29	ft
Short Plan Dimension	23.83	ft
Long Plan Dimension	46.67	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade		ft

Velocity Pressure at Mean Roof Height, q_h =	17.7	psf

Transverse Wind Pressures

L/B = 0.51 h/L = 0.47

Pressure Coefficients from Figure 27.3-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.72 / -0.16
Leeward Roof	-0.50

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	17.74	12.06	7.53	**11.8**
15-20	0.9	18.78	12.77	7.53	**12.2**
20-25	0.94	19.62	13.34	7.53	**12.5**
25-30	0.98	20.45	13.91	7.53	**12.9**
30-40	1.04	21.71	14.76	7.53	**13.4**
41-50	1.09	22.75	15.47	7.53	**13.8**
51-60	1.13	23.59	16.04	7.53	**14.1**
61-70	1.17	24.42	16.61	7.53	**14.5**
71-80	1.21	25.26	17.17	7.53	**14.8**
81-90	1.24	25.88	17.60	7.53	**15.1**
91-100	1.26	26.30	17.88	7.53	**15.2**

Roof Pressures (Unfactored) ASD

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.5	-10.8	-7.5	**4.80**

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Seismic Weight

Level	Area 1	A_{TOTAL} (ft^2)	$W_{AREA\ 1}$ (psf)	Area 2	A_{TOTAL} (ft^2)	$W_{AREA\ 2}$ (psf)	W_{AREA} (k)
Roof	Roof	1162	30	Glass Roof	117	25	37.79
			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

h

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	Concrete Wall	103	27.8	4.8	Brick Veneer Wall	25	95.3	4.8
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

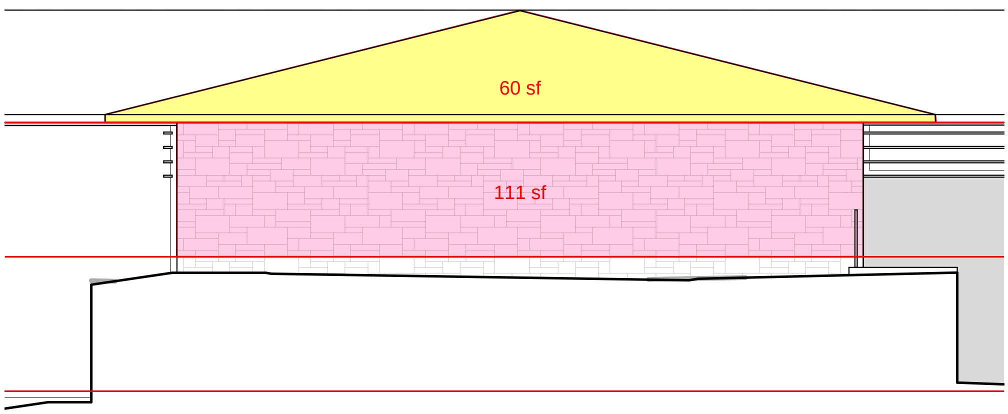
Cladding 3	$W_{CLAD\ 3}$ (psf)	$L_{CLAD\ 3}$ (ft)	$H_{CLAD\ 3}$ (ft)	W_{CLAD} (k)	$W_{DIST\ TO\ LVL}$ (k)
xterior Glazir	15	11.50	4.8	25.72	64
	0		0.0	0.00	0
	0		0.0	0.00	0
	0		0.0	0.00	0
	0		0.0	0.00	0
	0		0.0	0.00	0
	0		0.0	0.00	0

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

DATE 2/23/2022
PROJ. # 01519-2021-09
DESIGN LAN
SHEET GL-4

Wind Base Shear Calc - NS Direction



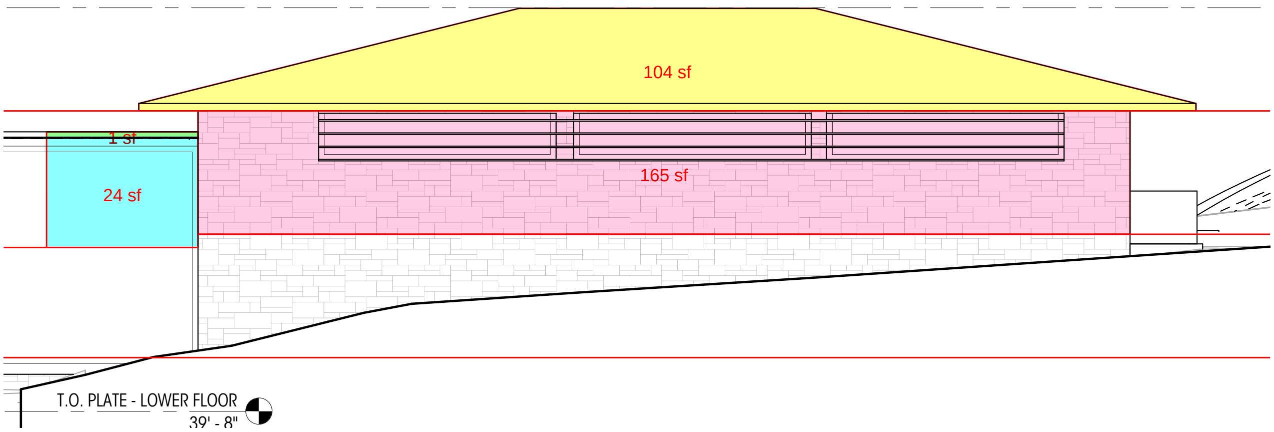
North elevation governs for NS direction

ROOF
roof: 60*4.8 = 288 lbs
wall: 111*12.3 = 1366 lbs

TOTAL WIND BASE SHEAR
1.65 kips

seismic governs NS

Wind Base Shear Calc - EW Direction



East and West elevations are the same

ROOF
roof: 105*4.8 = 499 lbs
wall: 189*12.3 = 2325 lbs

TOTAL WIND BASE SHEAR
2.8 kips

seismic governs NS

GARAGE North-South Shearwalls

ROOF - WOOD WALL DESIGN

Wind shear =	1.7	kips, ASD
Wind length =	23.3	ft
Wind dist. =	71	plf
Seismic shear =	10.2	kips, ASD
Area =	796	sf
Seismic. dist. =	13	psf
Ω =	2.0	

Wall Line	West Line		East Line	
Wind/ Seis.	Wind	Seis.	Wind	Seis.
Trib (ft)	11.6	11.6	11.6	11.6
Trib depth (ft)	-	34.3	-	34.3
V (k)	0.8	5.1	0.8	5.1
Other V (k)	0.0	0.0	0.0	0.0
Total V (k)	0.8	5.1	0.8	5.1
L (ft)	7.1	7.1	6.9	6.9
L red (ft)	5.9	5.9	5.9	5.9
V (plf)	139	862	139	860
SW	SW4		SW4	
H (ft)	9.33	9.33	8.08	8.08
V*Ω (k)	1.7	10.2	1.7	10.2
OT (lb)	2174	13440	1928	11920
Design OT (lb)	1961	13226	1715	11707
Above OT (lb)	0	0	0	0
Total OT (lb)	1961	13226	1715	11707
Holdown	(2) S/HDU9		S/HDU6	
HD Comp. (lb)	2530	13796	2284	12276
Attach. L (ft)	35.0	35.0	35.0	35.0
Diaph. V (plf)	47	291	47	291
Unblocked OK?	YES	NO	YES	NO
Blkg L (ft)	-	0.3	-	0.3
Blkg Nailing	-	6in spac.	-	6in spac.
Strut L (ft)	-	-	-	-
Strut load (k)	-	-	-	-

Span	1	
Wind/ Seis.	Wind	Seis.
Dist. V (plf)	71	439
Span L (ft)	23.3	23.3
Moment (kip-ft)	4.8	29.6
Dia. depth (ft)	34.3	34.3
Chord T/C (lb)	280	1731
Chord	CS14	CS14

GARAGE East-West Shearwalls

ROOF - WOOD WALL DESIGN

Wind shear =	2.8	kips, ASD
Wind length =	34.3	ft
Wind dist. =	82	plf
Seismic shear =	10.2	kips, ASD
Area =	796	sf
Seismic. dist. =	13	psf
Ω =	2.0	

Wall Line	South Line		North Line	
Wind/ Seis.	Wind	Seis.	Wind	Seis.
Trib (ft)	17.1	17.1	17.1	17.1
Trib depth (ft)	-	23.3	-	23.3
V (k)	1.4	5.1	1.4	5.1
Other V (k)	0.0	0.0	0.0	0.0
Total V (k)	1.4	5.1	1.4	5.1
L (ft)	12.7	12.7	23.5	23.5
L red (ft)	12.7	12.7	23.5	23.5
V (plf)	111	403	60	217
SW	Concrete		SW1	
H (ft)	9.33	9.33	6.25	6.25
V*Ω (k)	2.8	10.2	2.8	10.2
OT (lb)	2081	7516	751	2713
Design OT (lb)	-	-	557	2519
Above OT (lb)	-	-	0	0
Total OT (lb)	-	-	557	2519
Holdown	N/A		S/HDU4	
HD Comp. (lb)	-	-	1074	3036
Attach. L (ft)	23.5	23.5	23.5	23.5
Diaph. V (plf)	120	434	120	434
Unblocked OK?	YES	NO	YES	NO
Blkg L (ft)	-	16.7	-	16.7
Blkg Nailing	-	4in spac.	-	4in spac.
Strut L (ft)	-	-	-	-
Strut load (k)	-	-	-	-

Span	1	
Wind/ Seis.	Wind	Seis.
Dist. V (plf)	82	298
Span L (ft)	34.3	34.3
Moment (kip-ft)	12.1	43.7
Dia. depth (ft)	23.3	23.3
Chord T/C (lb)	1040	3756
Chord	CMSTC16	CMSTC16

GARAGE North-South Shearwalls

ROOF - CONCRETE WALL DESIGN

h = 9.33 ft

f'c = 2500 psi
fy = 60 ksi

IN PLANE SHEAR

Shear (EQ gov) = 7.3 kips, LRFD

Wall ID	V (k)	L (ft)	b (in)	h/L	α_c	ϕV_c (k)	DCR
Wall 1	2.3	3.9	6.0	2.4	2.0	16.9	13%
Wall 2	5.0	8.8	6.0	1.1	3.0	56.7	9%

Wall ID	A_c (in^2)	ρ min	min steel (in^2/ft)	steel provide	ϕV_s (k)	ϕV_n (k)	Max ϕV_n (k)
Wall 1	282	0.002	0.144	#4 @ 12"oc	28.2	45.1	84.6
Wall 2	630	0.002	0.144	#4 @ 12"oc	63	119.7	189

OUT OF PLANE SHEAR

S_{DS} = 1.17
I_e = 1.00

Wall ID	Wt (lbs)	Fp (k)	Mu (K-ft)	Vu (k)
Wall 1	2742	1.3	3.0	0.6
Wall 2	6125	2.9	6.7	1.4

Wall ID	Nu (k)	A_g (in^2)	bw (in)	d (in)	ϕV_c (k)	DCR
Wall 1	3.9	282	47.0	3.0	14.2	5%
Wall 2	8.6	630	105.0	3.0	31.7	5%

OUT OF PLANE SHEAR ANCHORAGE

L_F = 34.25 ft
ka = 1.34 < 2
trib = 4 ft
wt = 2800 lbs
F_p eq = 1.8 kips (LRFD)
F_p min = 0.8 kips (LRFD)
F_p = 1.8 kips
S/LTT20 cap = 1.9 kips OK

Garage Gravity

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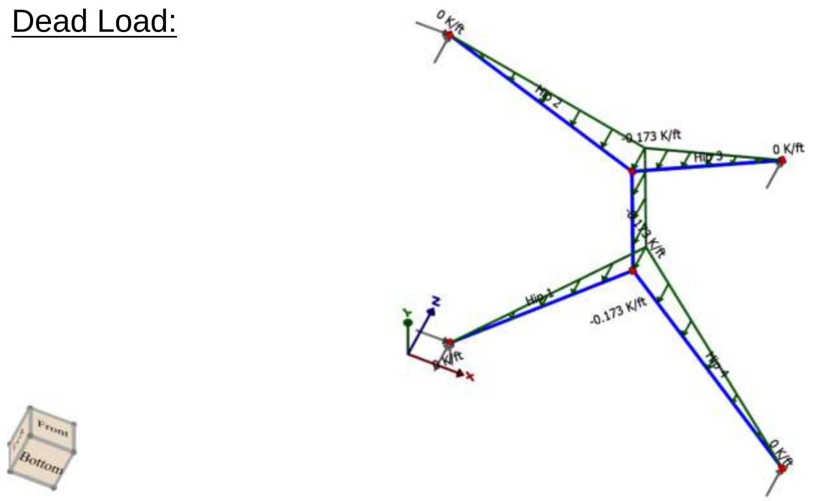
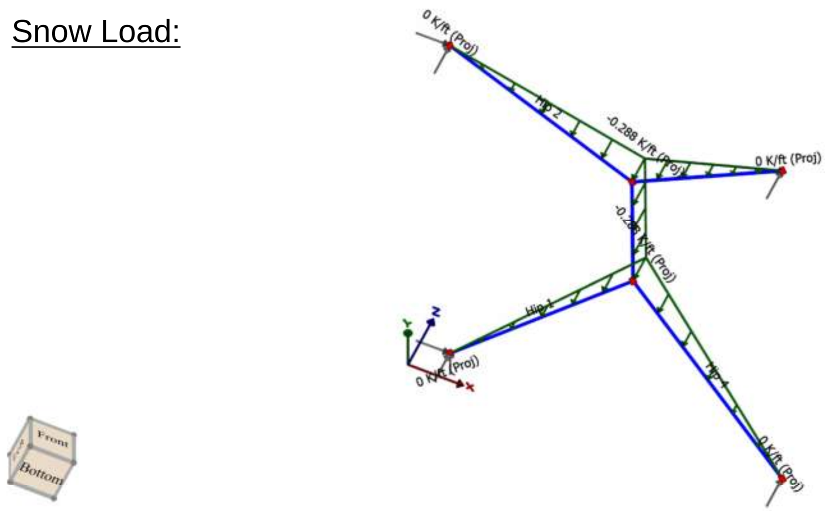
Garage Roof Gravity

Dead Load:



Snow Load:



Members

Name	Node 1	Node 2	Shape	Length ft	Weight K	Framing
Hip 1	N001	N002	HSS12X8X.500	16.4229	0.9627	Bracing
Hip 2	N004	N005	HSS12X8X.500	16.4545	0.9645	Bracing
Hip 3	N004	N006	HSS12X8X.500	16.5126	0.9679	Bracing
Hip 4	N002	N003	HSS12X8X.500	16.4811	0.9661	Bracing
Ridge	N002	N004	W18X60	11.2512	0.6749	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)	**1.8233 (4)**	-0.5586 (4)	-0.0046 (5)
Hip 4	**-1.8033 (4)**	0.0137 (4)	**-0.5998 (4)**	**-0.0064 (5)**
Ridge	**-1.9639 (4)**	**-0.2701 (5)**	**0.0039 (5)**	**0.0288 (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	**-0.9840 (4)**	**-0.0473 (5)**	5.9025 (4)	**0.9516 (4)**	0.0000 (4)	0.0000 (7)	**12.2849 (4)**	0.0000 (6)	76.1978 (4)
Hip 2	-0.9183 (4)	-0.0437 (5)	**5.9060 (4)**	0.8973 (4)	0.0000 (4)	**-11.5871 (4)**	0.0000 (4)	**-76.2641 (4)**	0.0000 (4)
Hip 3	-0.9106 (4)	-0.0425 (5)	**-5.8793 (4)**	0.8837 (4)	0.0000 (4)	-11.4253 (4)	0.0000 (3)	0.0000 (2)	76.0095 (4)
Hip 4	-0.9757 (4)	-0.0461 (5)	-5.8760 (4)	**-0.9372 (4)**	0.0000 (4)	0.0000 (4)	12.1139 (4)	0.0000 (5)	**75.9477 (4)**
Ridge	**-0.0494 (4)**	**0.0502 (4)**	3.3843 (4)	0.0000 (4)	**0.0000 (4)**	**0.0000 (4)**	**0.0000 (1)**	**16.1662 (5)**	**117.9691 (4)**

Node Results (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K
N001	3. 1.2D+1.6S+L	0.0000	0.0000	0.0000	0.0000	0.0000	**5.9980**
N002	5. 0.9D+W	0.0042	0.0591	**-0.2710**	0.0000	0.0000	0.0000
N003	3. 1.2D+1.6S+L	**0.0611**	0.0037	0.0000	0.0000	0.0000	5.9684
N003	5. 0.9D+W	0.0088	**0.0006**	0.0000	0.0000	0.0000	0.6984
N004	3. 1.2D+1.6S+L	0.0273	0.4043	**-1.8600**	0.0000	0.0000	0.0000
N004	5. 0.9D+W	**0.0039**	0.0591	-0.2714	0.0000	0.0000	0.0000
N006	3. 1.2D+1.6S+L	0.0521	**0.7833**	0.0000	0.0000	0.0000	5.9532
N006	5. 0.9D+W	0.0075	0.1144	0.0000	0.0000	0.0000	**0.6952**

Member Unity Checks

Member	Section	Unity Check	Status	Result Case	Code Reference	Type	Design Group
Hip 1	HSS12X8X.500	0.3937	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	0.3901	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Hip 3	HSS12X8X.500	0.3881	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Hip 4	HSS12X8X.500	0.3917	Pass	3. 1.2D+1.6S+L	H1-1b	Combined Check	Steel_V Brace_G 1
Ridge	W18X60	0.3040	Pass	3. 1.2D+1.6S+L	H1-1b	Combined 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	16.4229	3. 1.2D+1.6S+L	0.3937	1.0000	H1-1b	**0.3937**	KLz = 16.423 ft, KLy = 16.423 ft, KL(torsion) = 16.423 ft, Lb = 16.423 ft, Axial Unity = 0.0005, Mz Unity = 0.32432, My Unity = 0.06914, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.4784

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 1	HSS12X8X.500	1.6423	3. 1.2D+1.6S+L	0.9840	553.2109	E3-2	**0.0018**	KLz = 16.423 ft, KLy = 16.423 ft, KL(torsion) = 16.423 ft, Fcr = 35.737 Ksi, Fe (E3-4) = 76.266 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 2	HSS12X8X.500	0.0000	3. 1.2D+1.6S+L	-76.2641	234.9450	F7-1	**0.3246**	Lb = 16.454 ft, Cb = 1.4771

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	16.4229	3. 1.2D+1.6S+L	12.2849	177.6750	F7-1	**0.0691**	

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	14.8090	3. 1.2D+1.6S+L	5.9060	244.9882	G4-1	**0.0241**	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: Combined Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand	Capacity	Code Reference	Unity Check	Details
Ridge	W18X60	5.6256	3. 1.2D+1.6S+L	0.3040	1.0000	H1-1b	**0.3040**	KLz = 11.251 ft, KLy = 11.251 ft, KL(torsion) = 11.251 ft, Lb = 11.251 ft, Axial Unity = 0, Mz Unity = 0.30396, My Unity = 0, Kz = 1, Ky = 1, K(torsion) = 1, Cb = 1.0097

Steel_Beam_G 1: Axial Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Fx K	Capacity Fx K	Code Reference	Unity Check	Details
Ridge	W18X60	11.2512	3. 1.2D+1.6S+L	0.0494	495.8793	E3-2	**0.0001**	KLz = 11.251 ft, KLy = 11.251 ft, KL(torsion) = 11.251 ft, Fcr = 31.306 Ksi, Fe (E3-4) = 44.695 Ksi, Kz = 1, Ky = 1, K(torsion) = 1

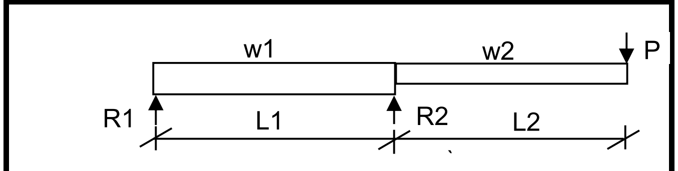
Steel_Beam_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
Ridge	W18X60	5.6256	3. 1.2D+1.6S+L	117.9691	388.1110	F2-2	**0.3040**	Lp = 5.9595 ft, Lr = 18.172 ft, Lb = 11.251 ft, Cb = 1.0097

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

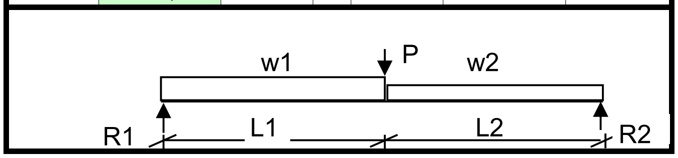
Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
Ridge	W18X60	0.0000	3. 1.2D+1.6S+L	3.3843	226.5900	G2-1	**0.0149**	Shear Area = 7.553 in^2, Cv = 1, h/tw = 38.554

Beam	fascia				
w1=	90	plf	R1=	-92	lbs
w2=	90	plf	R2=	906	lbs
L1=	3.83	ft	M+=	-	lb-ft
L2=	3.00	ft	M-=	1,010	lb-ft
X=	1.92	ft	Fb=	164	psi
P=	203	lbs	Fv=	15	psi
b=	3.50	in	Δspan=	(0.002)	in
d=	11.25	in	l span/	(27,793)	
E=	1,700	ksi	Δcant=	0.01	in
Cv=	1.00		l cant/	4,926	



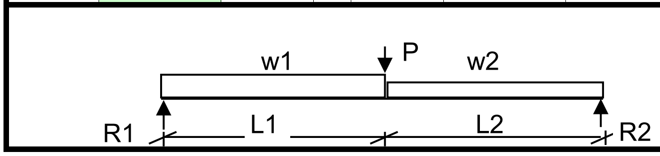
Steel Size				HSS4X4X1/2	
	Δ (in)	l /	Fy=	46	ksi
span	-0.003	-13586	Mn/Ω =	17.7	k-ft
cant.	0.03	2408	Vn/Ω =	40.0	kips

Beam	short hdr				
w1=	284	plf	R1 =	1,540	lbs
w2=	284	plf	R2 =	1,899	lbs
L1=	7.50	ft	M =	4,079	lb-ft
L2=	2.25	ft	Fb =	663	psi
X=	7.5	ft	Fv =	62	psi
P=	667	lbs	Δ=	0.09	in
b=	3.50	in	l /	1,266	
d=	11.25	in	Cv=	1.00	
E=	1,300	ksi			



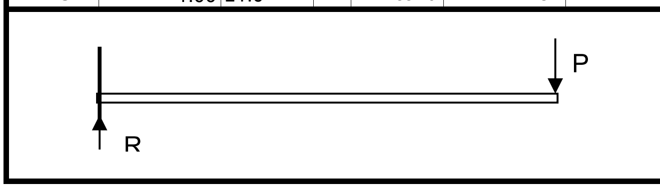
Steel Size				HSS5X3X1/4	
I =	10.7	in^4	Fy=	46	ksi
Δ =	0.161	in	Mn/Ω =	12.3	k-ft
l /	728		Vn/Ω =	33.2	kips

Beam	long hdr				
w1=	301	plf	R1 =	3,175	lbs
w2=	301	plf	R2 =	3,142	lbs
L1=	8.92	ft	M =	16,328	lb-ft
L2=	9.83	ft	Fb =	2,654	psi
X=	8.9	ft	Fv =	110	psi
P=	667	lbs	Δ=	1.84	in
b=	3.50	in	l /	122	
d=	11.25	in	Cv=	1.00	
E=	1,300	ksi			

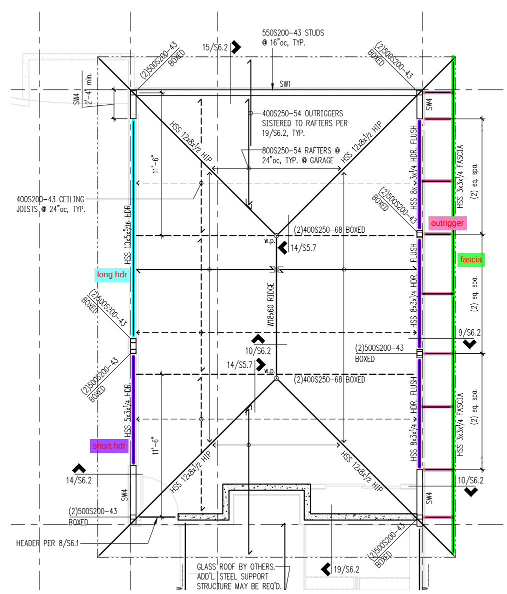


Steel Size				HSS10X5X5/16	
I =	104	in^4	Fy=	46	ksi
Δ =	0.329	in	Mn/Ω =	54.2	k-ft
l /	683		Vn/Ω =	71.8	kips

Beam	outrigger				
P	906	lbs	R=	906	lbs
L=	3.00	ft	M=	2,718	ft-lbs
b=	1.50	in	Fb=	1,031	psi
d=	11.25	in	Fv=	81	psi
E=	1300	ksi	Δ=	0.06	in
Cv=	1.00	≤1.0	l cant/	1182	



Steel Size				HSS4X4X1/2	
I =	11.9	in^4	Fy=	46	ksi
Δ =	0.04	in	Mn/Ω =	17.7	k-ft
l /	882		Vn/Ω =	40.0	kips



SSF STRUCTURAL ENGINEERING

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Project: 8480 Residence

Garage Roof Gravity

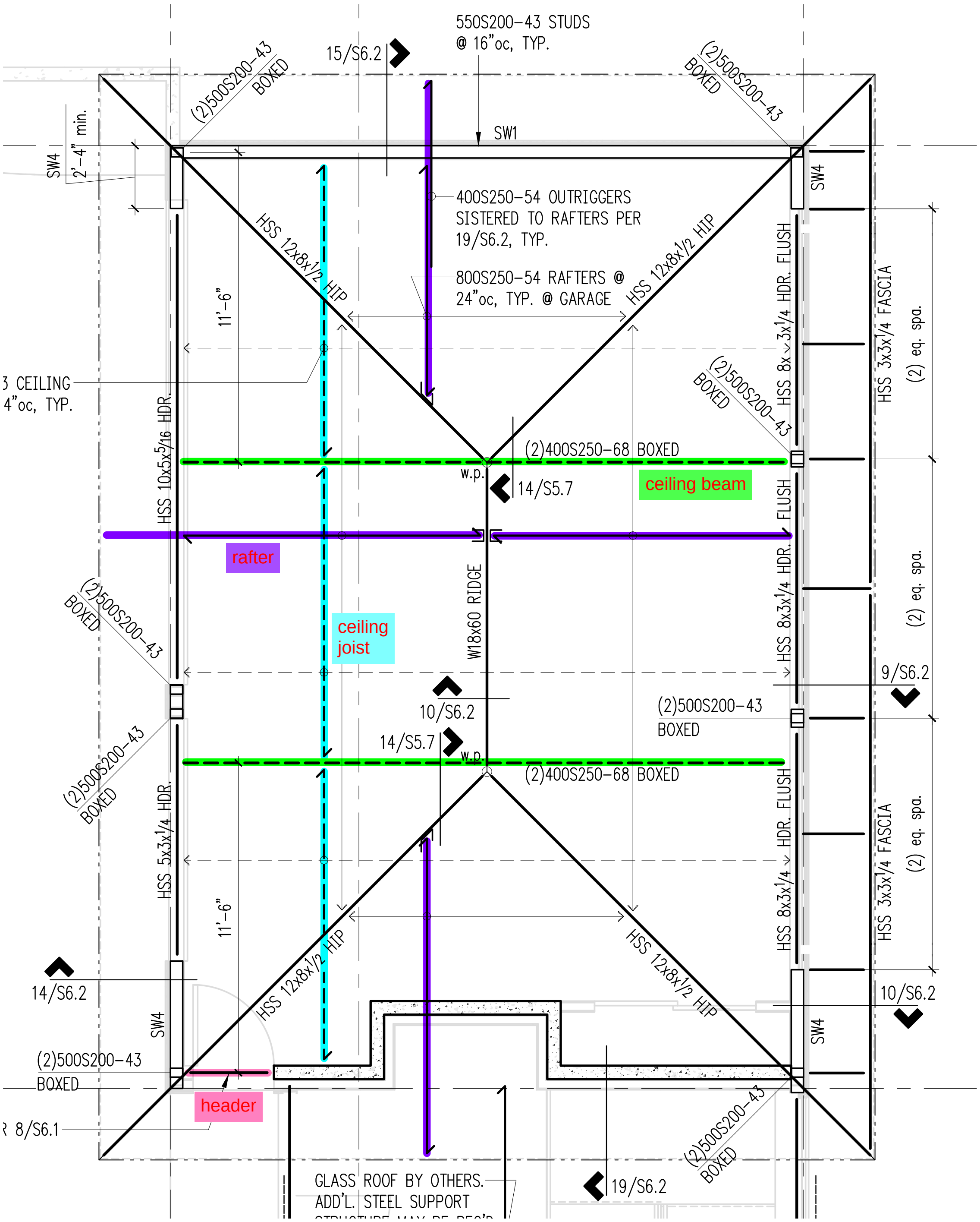
Date: 02/24/22

Project #: 01519-2021-09

Design: LAN

Sheet: GG-6

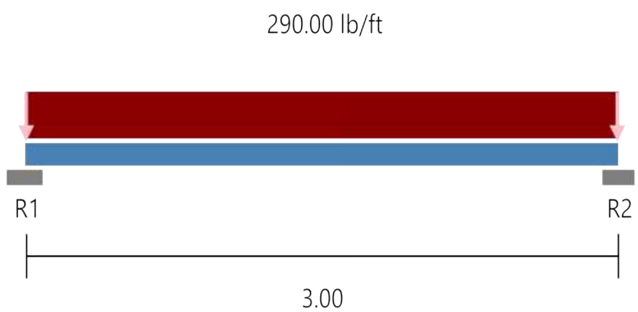
Garage Light Gauge Key Plan



8480 Residence
PROJECT

02/24/2022
DATE
01519-2021-09
PROJ. #
LAN
DESIGN
GG-7
SHEET

STRUCTURAL ENGINEERING



Reactions

Support	Reactions (lb)
R2	435.00
R1	435.00

Shear and Web Crippling Checks

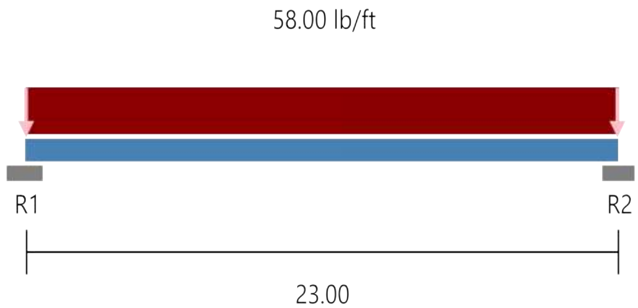
Bending and Shear (Unstiffened):	14.6% Stressed @R2
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section: 550T200-54 (50 ksi) Single Track (unpunched)
Maxo = 1571.5 ft-lb **Va =** 2980.3 lb **I =** 2.15 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	326.2	0.208	326.2	None	1567.8	0.208	0.008	L/4325



Reactions

Support	Reactions (lb)
R2	667.00
R1	667.00

Shear and Web Crippling Checks

Bending and Shear (Unstiffened):	24.6% Stressed @R1
Bending and Shear (Stiffened):	NA
Web Stiffeners Required?:	No

Section: (2) 400S250-68 (50 ksi) Boxed C Stud (punched)
Maxo = 3865.7 ft-lb **Va =** 9741.8 lb **I =** 3.73 in^4

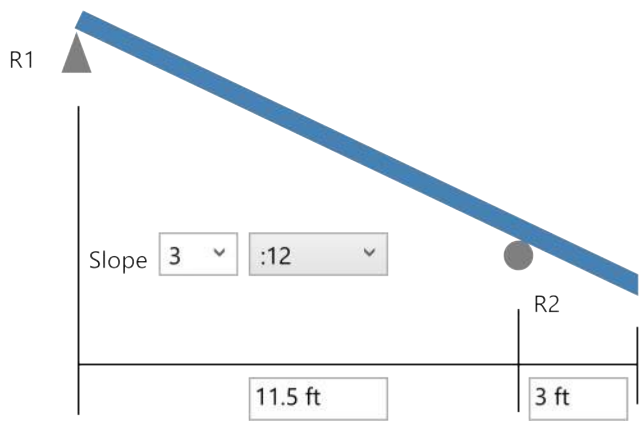
Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

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	3835.3	0.992	3835.3	None	3865.7	0.992	3.320	L/83

Project Name: garage LG
Model: Rafter
Code: 2012 NASPEC [AISI S100-2012]

Date: 02/24/2022
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	439.91	3	-94.87	4	73.95	2	-164.50	4
R2	774.08	3	-294.10	4	0.00	1	0.00	1

Rafter Flexural and Deflection

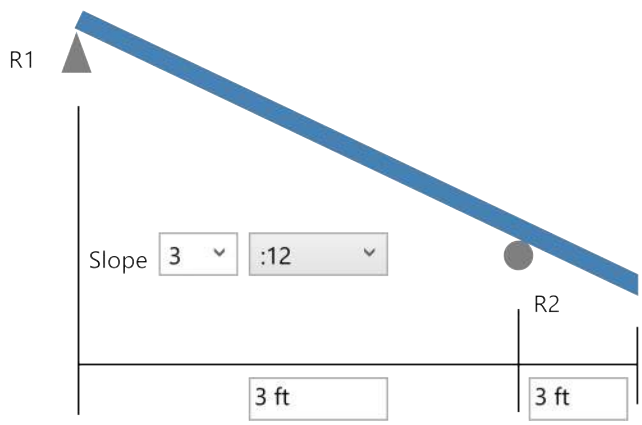
Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
1216	1914	0.64	3	L/1044	3	L/810	3

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	440.2	3	1.00	574.6	1005.5	0.40	3	NO
R2	751.0	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	440	3	1.000	0.21	0.00	0.21	3	N/A	N/A
R2	505	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	41.55	4	-7.65	2	30.60	2	-74.40	4
R2	508.08	3	-227.83	4	0.00	1	0.00	1

Rafter Flexural and Deflection

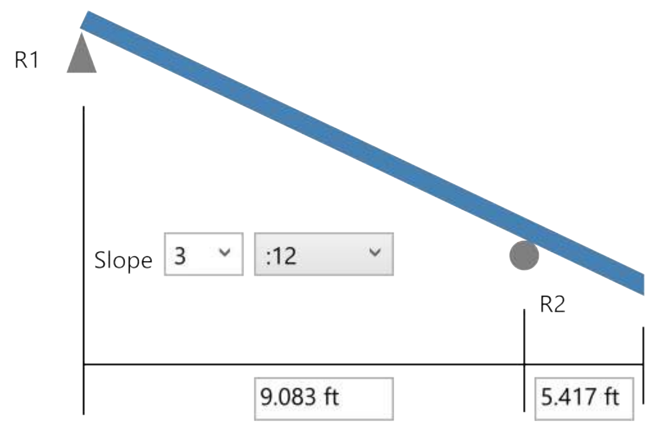
Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
381	3261	0.12	3	L/32577	3	L/5131	3

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	22.3	4	1.00	574.6	1005.5	0.02	4	NO
R2	492.9	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	22	4	1.000	0.01	0.00	0.01	4	N/A	N/A
R2	246	3	1.000	0.12	0.10	0.15	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	236.80	1	-9.52	4	73.95	2	-173.20	4
R2	980.07	3	-414.25	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.
1242	3261	0.38	3	L/9114	3	L/1047	3

Rafter Bending and Web Crippling

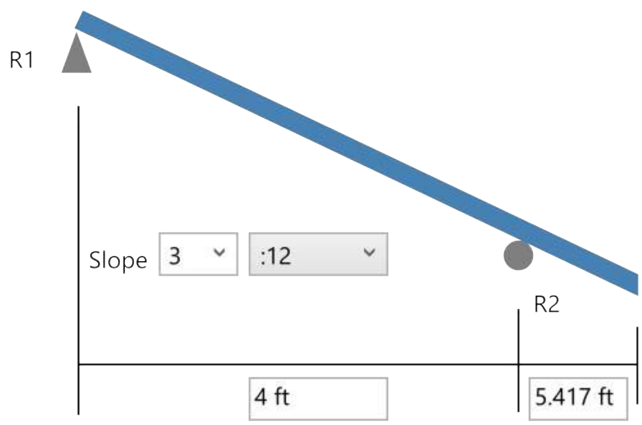
Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	240.4	3	1.00	574.6	1005.5	0.22	3	NO
R2	950.8	3	6.00	1741.4	2873.4	0.50	3	NO

Rafter Bending and Shear

Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Load Comb.	Intr. Stiffen	Load Comb.
R1	240	3	1.000	0.11	0.00	0.11	3	N/A	N/A
R2	506	3	1.000	0.24	0.33	0.41	3	N/A	N/A

Project Name: garage LG
Model: Rafter (sistered member, long backspan)
Code: 2012 NASPEC [AISI S100-2012]

Date: 02/24/2022
Simpson Strong-Tie® CFS Designer™ 4.0.0.16



Section : 400S250-54 (50 ksi) @ 24" o.c. Single C Stud (punched)
Maxo = 1436.3(ft-lb) **Va =** 3371.6 **I =** 1.506 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	130.15	4	-150.25	3	48.03	2	-119.32	4
R2	938.67	3	-432.71	4	0.00	1	0.00	1

Rafter Flexural and Deflection

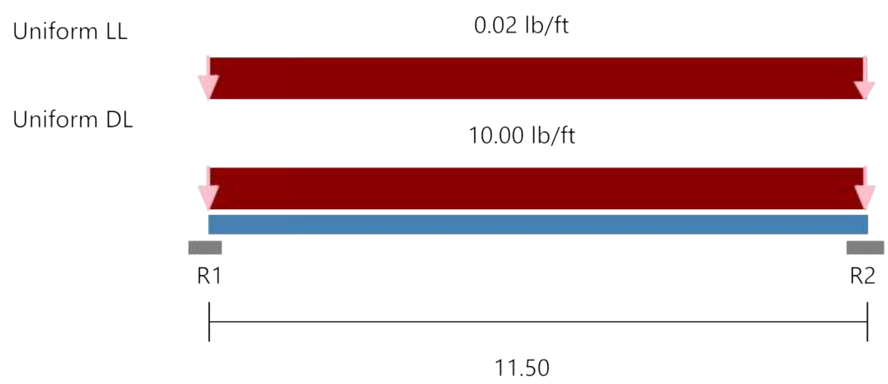
Mmax (ft-lb)	Ma (ft-lb)	Mmax/ Ma	Load Comb.	Span Defl	Load Comb.	Overhang Defl	Load Comb.
1242	1436	0.86	3	L/1193	3	L/192	3

Rafter Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Req'd
R1	-137.0	3	1.00	628.1	1099.2	0.11	3	NO
R2	910.6	3	6.00	1791.9	2956.6	0.80	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	137	3	1.000	0.04	0.00	0.04	3	N/A	N/A
R2	466	3	0.363	0.38	0.86	0.95	3	N/A	N/A



Section :	400S200-43 (33 ksi) @ 24 in" o.c. Single C Stud (punched)		
Maxo =	787.6 ft-lb	**Va =** 1739.1 lb	**I =** 1.047 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	166	0.0	138.0	811.3	0.210	1	L/1081	1	L/5418 08	4

Joist Bending and Web Crippling

Support	Load (lb)	Load Comb.	Bearing (in)	Pa (lb)	Pn (lb)	Max Intr.	Load Comb.	Stiffeners Required
R1	57.6	1	1.00	273.6	478.8	0.11	1	NO
R2	57.6	1	1.00	273.6	478.8	0.11	1	NO

Joist Bending and Shear

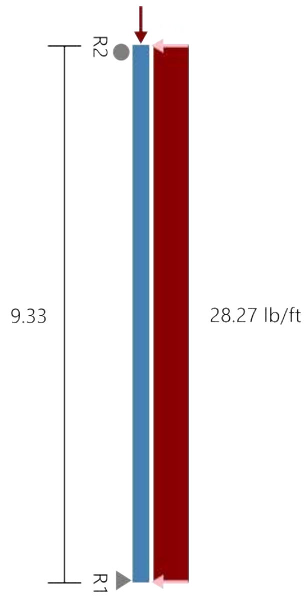
Support	Vmax (lb)	Load Comb.	Va Factor	V/Va	M/Ma	Intr. Unstiffened	Load Comb.	Intr. Stiffened	Load Comb.
R1	57.6	1	1.000	0.03	0.00	0.03	1	N/A	N/A
R2	57.6	1	1.000	0.03	0.00	0.03	1	N/A	N/A

Joist Reaction and Connections

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	57.6	SSC2.25 Min (3#10) & (2) #10 to Carrying (20/33) (Side	25.61 %	34.92 %

			Attached)		
R2	0.0	57.6	SSC2.25 Min (3#10) & (2) #10 to Carrying (20/33) (Side Attached)	25.61 %	34.92 %

* 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	0.00plf
P1y	883lb @ 9.33ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	883.0(c)	1977.0(c)	45%	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	0.81	1.00	81%	Axial 883.0(c), Moment 307.6
	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	883.0	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

Foundations

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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
ssfengineers.com

GARAGE Basement Walls - North and East (typ walls)

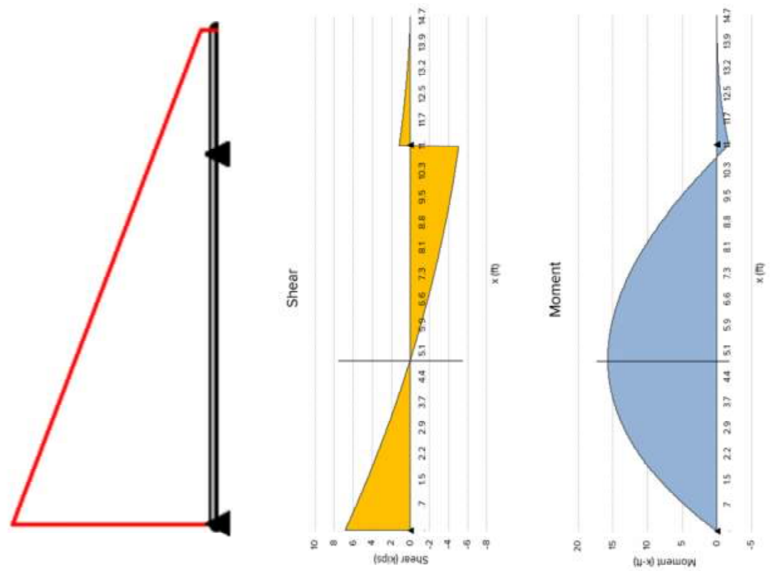
Soil info:

backspan H =	11.00	ft
cantilever H =	3.67	ft
total H =	14.67	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 =	1525	plf (max)
factored sur =	132	plf



Vu max =	6.86	kips
Mu pos max =	15.80	kip-ft
Mu neg max =	1.74	kip-ft

Wall Info:

Wall t =	8.00	in
Cover =	1.50	in (outer face rebar)
	0.75	in (inner face rebar)
Rebar db =	0.50	in (#4 bar, outer face rebar)
	0.75	in (#56 bar, inner face rebar)

d =	6.25	in (outer face rebar)
	6.88	in (inner face rebar)
bw =	12.00	in

Gravity Loading:

roof DL =	20	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 =	25	psf
wall above H =	5.7	ft
brick wall DL =	115	psf
brick wall H =	3.7	ft
wall DL =	100	psf
wall H =	11.0	ft
total DL =	2176	plf
total SL =	229	plf
total LL =	120	plf
factored TL =	2777	plf (1.2D+1.0L+0.2S)
trib =	1.0	ft
Nu =	2777	lbs
	2.78	kips
Ag =	96	in^2

Out of Plane Shear Check:

ϕVc =	7.94	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 =	#6 @ 9"oc	
spacing =	9.0	in

As =	0.44	in^2 (#6 bar)	
As/ft =	0.59	in^2	
bw =	12.0	in	
db =	6.88	in	
a =	0.86	in	
ϕMn =	204.1	kip-in	
	17.01	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 =	176.0	in	
h =	8.0	in	
k =	0.8		
bw =	12.0	in	
Ag =	96	in^2	
ϕPn =	96	kips	AXIAL OK

GARAGE Basement Walls - North and West at Driveway

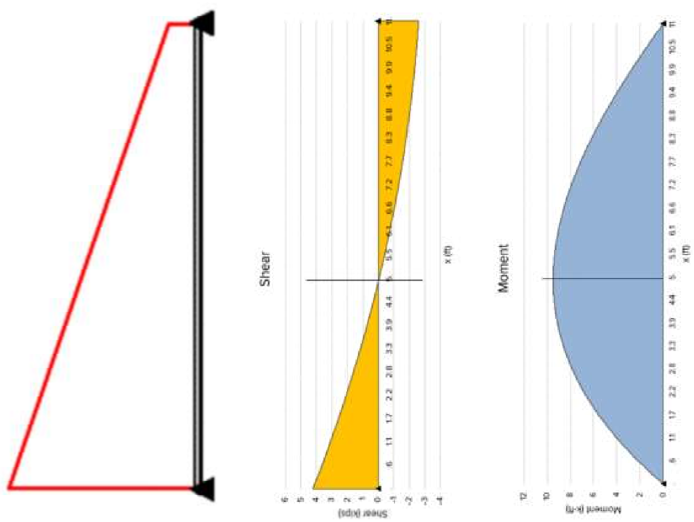
Soil info:

backspan H =	11.00	ft
cantilever H =	0.00	ft
total H =	11.00	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 =	880	plf (max)
factored seis sur =	99	plf
factored traffic sur =	80	plf
factored tot sur =	179	plf



Vu max =	4.21	kips
Mu pos max =	9.49	kip-ft

Wall Info:

Wall t =	8.00	in
Cover =	1.50	in (outer face rebar)
	0.75	in (inner face rebar)
Rebar db =	0.50	in (#4 bar, outer face rebar)
	0.75	in (#56 bar, inner face rebar)

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8480 Residence
PROJECT

02/24/2022
DATE

01519-2021-09
PROJ. #

LAN
DESIGN

F-4
SHEET

d =	6.25	in (outer face rebar)
	6.88	in (inner face rebar)
bw =	12.00	in

Gravity Loading:

roof DL =	20	psf
roof SL =	25	psf
roof trib =	9.1	ft
floor DL =	110	psf
floor LL =	40	psf
floor trib =	1.5	ft
brick above DL =	25	psf
brick above H =	9.3	ft
wall DL =	100	psf
wall H =	11.0	ft
total DL =	1681	plf
total SL =	229	plf
total LL =	60	plf
factored TL =	2123	plf (1.2D+1.0L+0.2S)
trib =	1.0	ft
Nu =	2123	lbs
	2.12	kips
Ag =	96	in^2

Out of Plane Shear Check:

ϕVc =	7.91	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 =	#6 @ 9"oc	
spacing =	9.0	in

As =	0.44	in^2 (#6 bar)
As/ft =	0.59	in^2
bw =	12.0	in
db =	6.88	in
a =	0.86	in
ϕMn =	204.1	kip-in
	17.01	kip-ft FLEXURE OK

Axial Check:

ecc =	0.63	in
Use Simplified eq?	YES	
Lc =	132.0	in
h =	8.0	in
k =	0.8	
bw =	12.0	in
Ag =	96	in^2
ϕPn =	114	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 =	5	psf
tot DL =	80	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

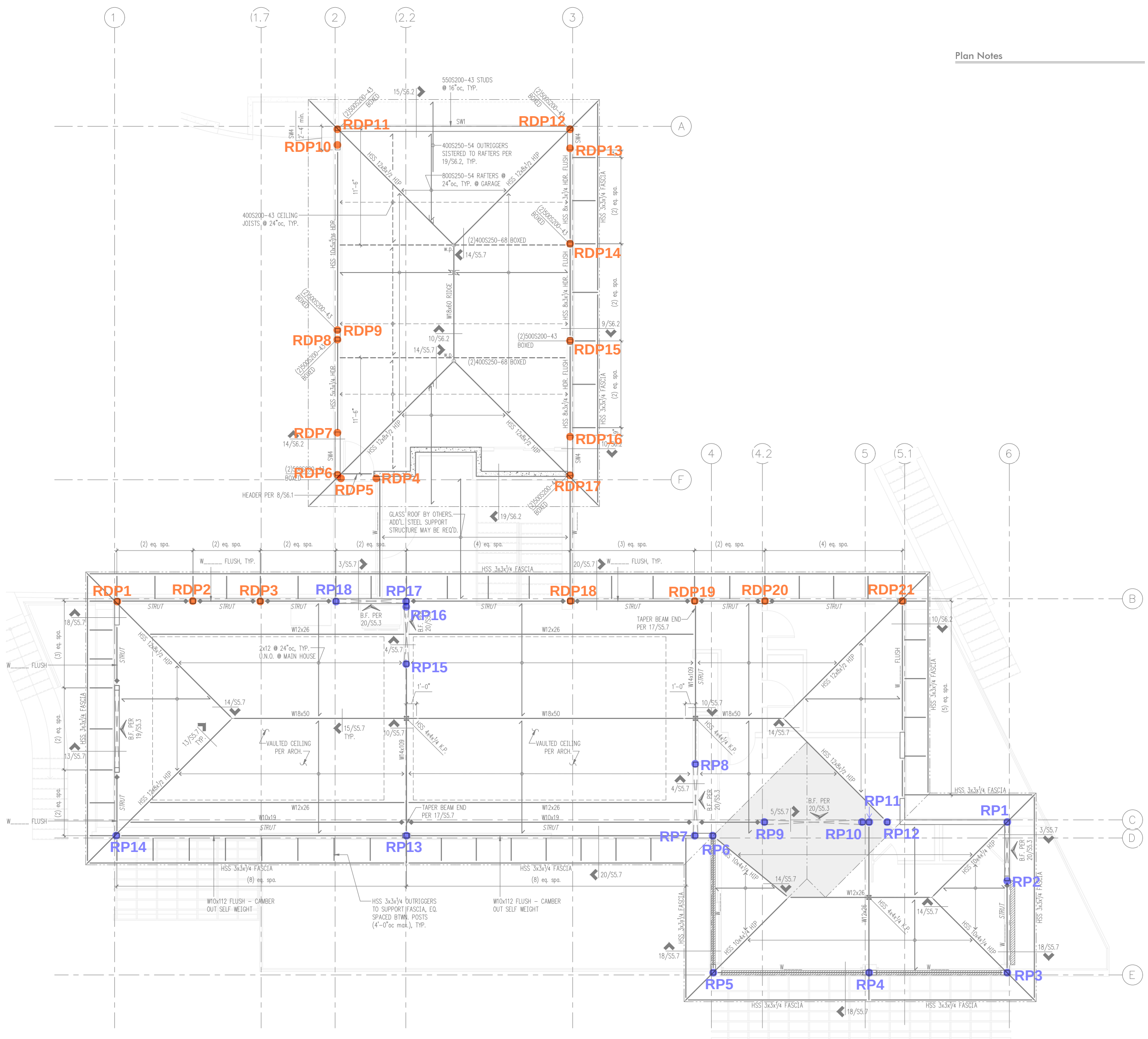
TL =	0.16	klf
L from M =	10.8	ft
L from V =	53.4	ft

at exterior

TL =	0.19	klf
L from M =	9.9	ft
L from V =	44.5	ft

Pile Load Key Plans

ROOF - POINT LOADS



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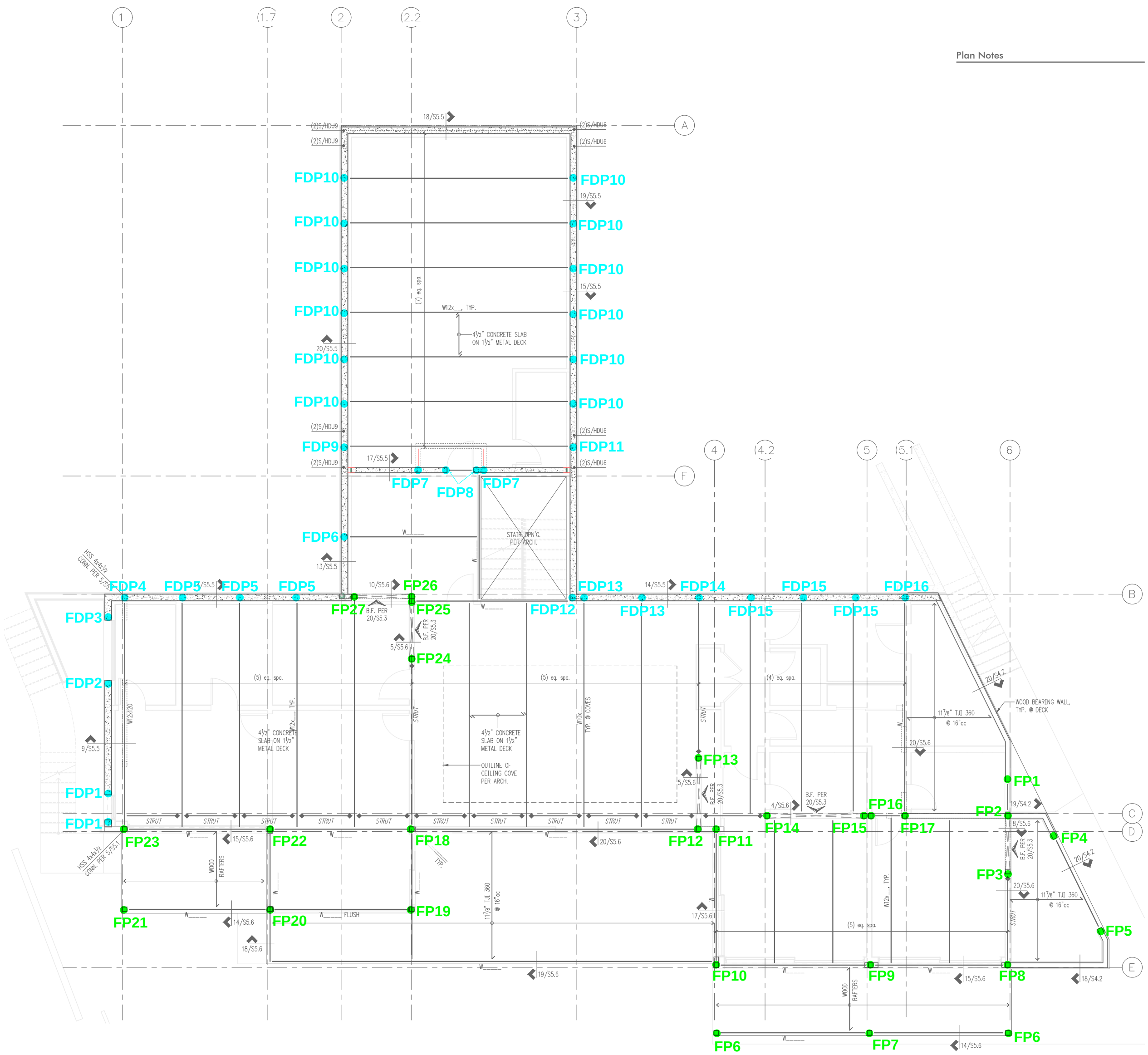
8480 Residence	02/24/2022
PROJECT	DATE
	01519-2021-09
	PROJ. #
	LAN
	DESIGN
	F-8
	SHEET

Pile Load Key Plans

FLOOR - POINT LOADS



8480 Residence
PROJECT

02/24/2022
DATE
01519-2021-09
PROJ. #
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DESIGN
F-9
SHEET

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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	1.46	146	371
15/S3.2	1.50	1.00	225	0.67	1.46	146	371
19/S3.2	1.50	1.00	225	0.67	1.46	146	371
18/S3.2	1.50	1.00	225	0.67	1.46	146	371
14/S3.2	1.75	2.00	525	0.00	0.00	0	525
13/S3.2	1.75	2.00	525	0.00	0.00	0	525
9/S3.2	1.75	2.00	525	0.00	0.00	0	525
BFB	4.00	2.50	1500	0.00	0.00	0	1500
BF2.2	4.00	2.50	1500	0.00	0.00	0	1500
BF4	4.00	2.50	1500	0.00	0.00	0	1500
BFC	4.00	2.50	1500	0.00	0.00	0	1500
BF6	4.00	2.50	1500	0.00	0.00	0	1500

**stem wall height increased by 25% from min height to be conservative

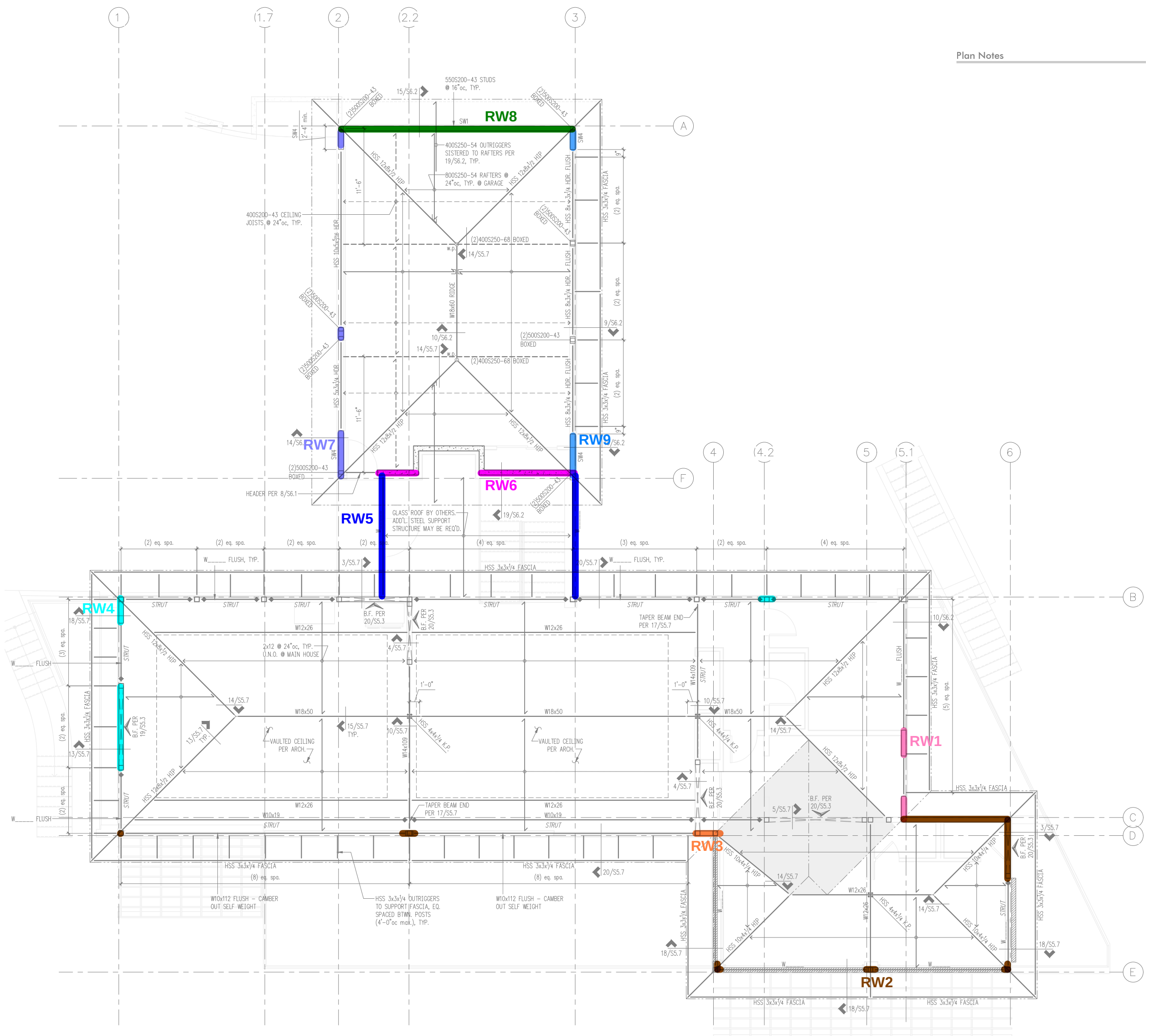
Pt Name	DL (lbs)	SL/LL (lbs)	TL (lbs)	Dist Over (ft)	DL (plf)	SL (plf)
RP1	3700	3800	7500	-	-	-
RP2	2300	2400	4700	-	-	-
RP3	2600	3100	5700	-	-	-
RP4	4900	5600	10500	-	-	-
RP5	3400	4100	7500	-	-	-
RP6	4100	4800	8900	-	-	-
RP7	6000	3400	9400	-	-	-
RP8	9300	7200	16500	-	-	-
RP9	2200	2300	4500	-	-	-
RP10	2200	2300	4500	-	-	-
RP11	2500	2600	5100	-	-	-
RP12	1600	1300	2900	-	-	-
RP13	14400	9100	23500	-	-	-
RP14	7900	5400	13300	-	-	-
RP15	11500	8000	19500	-	-	-
RP16	4700	4300	9000	-	-	-
RP17	3400	3400	6800	-	-	-
RP18	2000	1700	3700	-	-	-
RDP1	5100	4400	9500	10	510	440
RDP2	600	800	1400	11	55	73
RDP3	1500	2000	3500	11	136	182
RDP4	222	277	499	11	20	25
RDP5	246	308	554	11	22	28
RDP6	1030	3385	4414	11	94	308
RDP7	684	856	1540	11	62	78
RDP8	844	1056	1900	11	77	96
RDP9	1411	1764	3175	11	128	160
RDP10	1396	1746	3142	11	127	159
RDP11	1026	3378	4404	11	93	307
RDP12	1013	3356	4370	11	92	305

Pt Name	DL (lbs)	SL/LL (lbs)	TL (lbs)	Dist Over (ft)	DL (plf)	SL (plf)
RDP13	618	772	1390	11	56	70
RDP14	1236	1544	2780	11	112	140
RDP15	1462	1828	3290	11	133	166
RDP16	684	856	1540	11	62	78
RDP17	1052	3394	4446	11	96	309
RDP18	1900	2400	4300	11	173	218
RDP19	6500	5800	12300	11	591	527
RDP20	1700	2200	3900	11	155	200
RDP21	5500	6100	11600	11	500	555
FP1	192	461	653	11	17	42
FP2	1600	1400	3000	-	-	-
FP3	3400	2800	6200	-	-	-
FP4	391	937	1328	-	-	-
FP5	484	1163	1647	-	-	-
FP6	400	700	1100	-	-	-
FP7	800	1400	2200	-	-	-
FP8	6627	4080	10707	-	-	-
FP9	10263	5589	15852	-	-	-
FP10	9940	9263	19203	-	-	-
FP11	2187	1064	3251	-	-	-
FP12	21600	15200	36800	-	-	-
FP13	6900	3800	10700	-	-	-
FP14	13800	5500	19300	-	-	-
FP15	10000	4000	14000	-	-	-
FP16	2629	1252	3881	-	-	-
FP17	9629	8052	17681	-	-	-
FP18	28354	13100	41454	-	-	-
FP19	8947	23090	32037	-	-	-
FP20	3647	8410	12057	-	-	-
FP21	300	800	1100	-	-	-
FP22	15300	9400	24700	-	-	-
FP23	16260	8630	24890	-	-	-
FP24	6500	2700	9200	-	-	-
FP25	13900	6500	20400	-	-	-
FP26	1300	400	1700	-	-	-
FP27	7700	3200	10900	-	-	-
FDP1	75	38	113	9	8	4
FDP2	180	90	270	10	18	9
FDP3	180	90	270	11	16	8
FDP4	8860	4100	12960	11	805	373
FDP5	6600	2900	9500	11	600	264
FDP6	3882	1699	5581	11	353	154
FDP7	1192	152	1344	11	108	14
FDP8	594	247	841	11	54	22
FDP9	9440	3130	12570	11	858	285
FDP10	5200	2200	7400	11	473	200
FDP11	8770	2960	11730	11	797	269
FDP12	12000	5800	17800	11	1091	527

Pt Name	DL (lbs)	SL/LL (lbs)	TL (lbs)	Dist Over (ft)	DL (plf)	SL (plf)
FDP13	7000	2900	9900	11	636	264
FDP14	4117	1769	5886	11	374	161
FDP15	6600	2900	9500	11	600	264
FDP16	7000	6800	13800	11	636	618

Pile Load Key Plans

ROOF - WALL LOADS



8480 Residence
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02/24/2022
DATE

01519-2021-09
PROJ. #

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F-13
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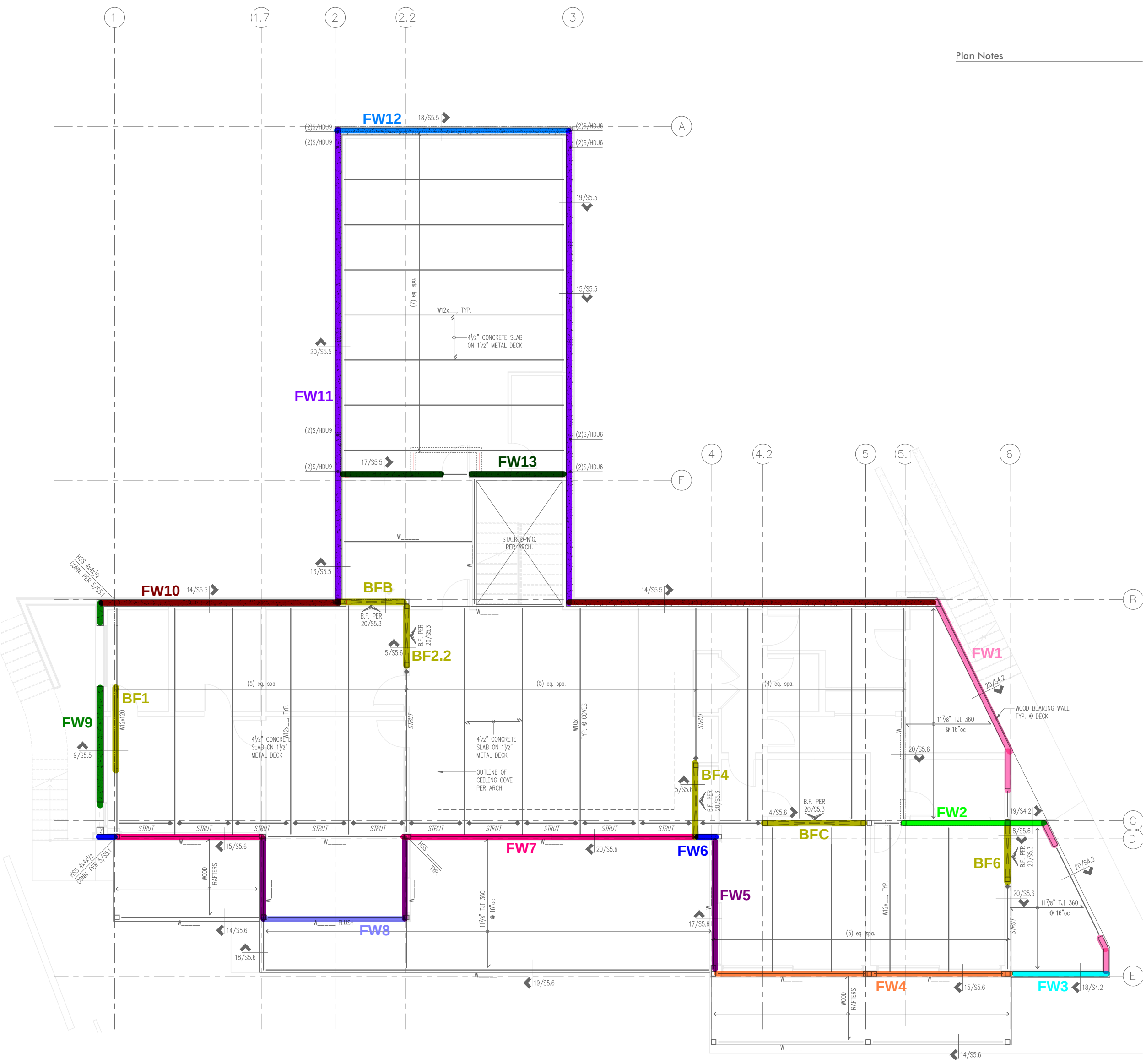
Pile Load Key Plans

FLOOR - WALL LOADS



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8480 Residence

PROJECT

DATE 02/24/2022

PROJ. # 01519-2021-09

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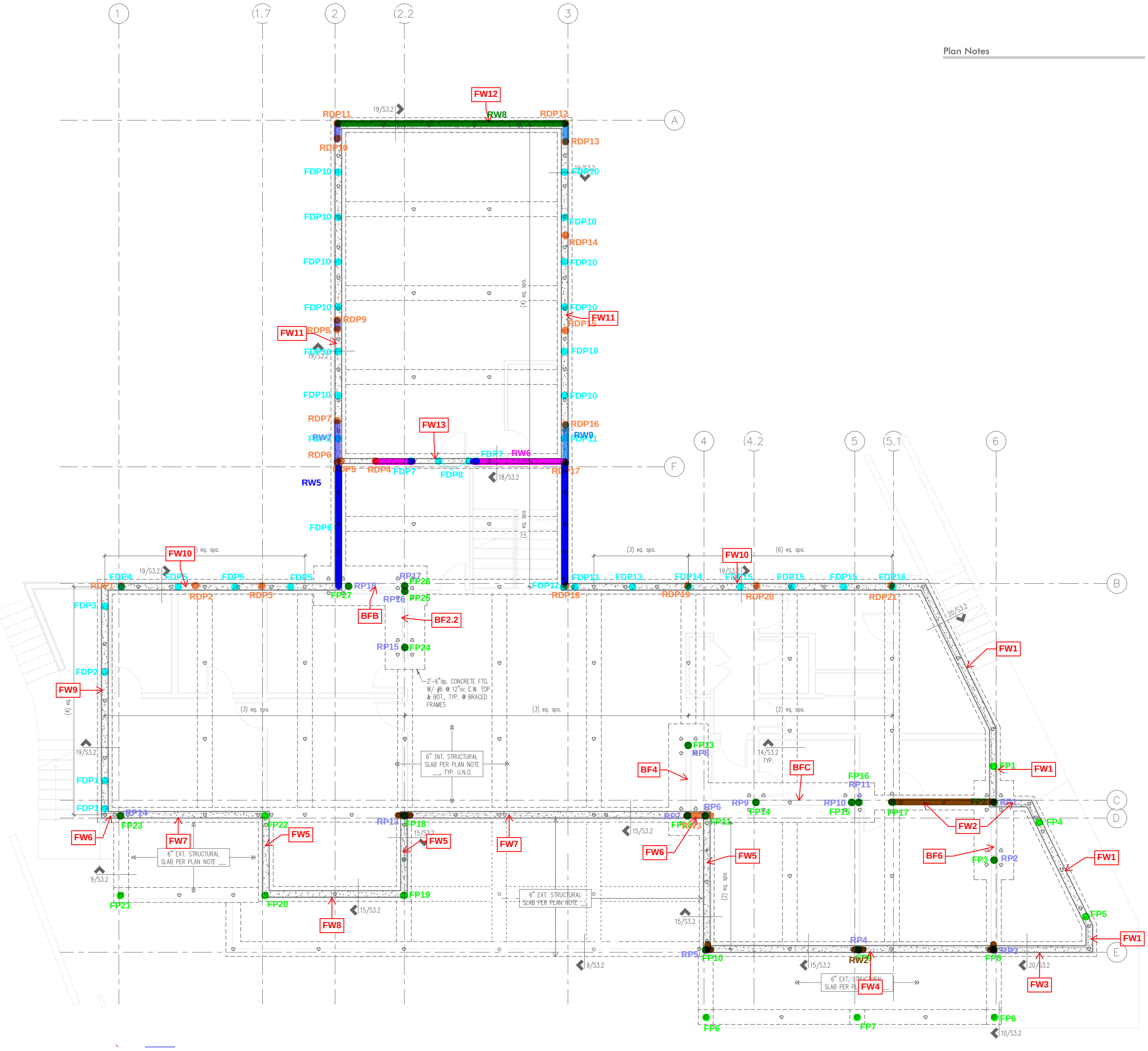
SHEET F-14

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	8.3	20	9.3	12	279	25	208	-	0	0.0	0	0	0	0	0	0
RW2	7.4	20	9.3	12	259	25	184	-	0	0.0	0	0	0	0	0	0
RW3	6.8	20	9.3	12	247	25	169	-	0	0.0	0	0	0	0	0	0
RW4	9.1	20	9.3	12	295	25	229	-	0	0.0	0	0	0	0	0	0
RW5	2.0	20	9.3	15	180	25	50	-	0	0.0	0	0	0	0	0	0
RW6	9.1	20	9.3	103	1144	25	229	-	0	0.0	0	0	0	0	0	0
RW7	9.1	20	9.3	25	416	25	229	-	0	0.0	0	0	0	0	0	0
RW8	9.1	20	6.1/3.3	25/115	709	25	229	-	0	0.0	0	0	0	0	0	0
RW9	6.1	20	6.1/3.3	25/115	648	25	153	-	0	0.0	0	0	0	0	0	0
FW1	5.1	25	11.0	25	403	60	308	20/S3.2	371	5.0	378	5	25	403	40	202
FW2	1.0/1.0	25/90	11.0	12	247	60/40	100	13/S3.2	525	2.0	150	5	10	160	40	80
FW3	3.6/1.0	10/25	11.0	25	336	25/60	151	20/S3.2	371	3.4	256	5	17	273	40	137
FW4	0	0	11.0	15	165	0	0	15/S3.2	371	3.4	256	5	17	273	40	137
FW5	0	0	11.0	15	165	0	0	15/S3.2	371	5.1	384	5	26	410	40	205
FW6	6.7/1.0	25/90	11.0	25	533	60/40	442	15/S3.2	371	5.0	375	5	25	400	40	200
FW7	0	0	11.0	15	165	0	0	15/S3.2	371	5.0	375	5	25	400	40	200
FW8	0	0	11.0	15	165	0	0	15/S3.2	371	5.0	375	5	25	400	40	200
FW9	0.8	90	11.0	115	1368	40	90	19/S3.2	371	5.0	375	5	25	400	40	200
FW10	1.0	90	11.0	103	1223	40	40	19/S3.2	371	1.0	75	5	5	80	40	40
FW11	1.0	90	11.0	103	1223	40	40	19/S3.2	371	1.0	75	5	5	80	40	40
FW12	2.5	90	11.0	103	1354	40	98	19/S3.2	371	4.3	319	5	21	340	40	170
FW13	4.5	90	11.0	103	1538	40	180	18/S3.2	371	7.4	556	5	37	593	40	297
BFB	0.0	0	0.0	0	0	0	0	BFB	1500	4.2	312	5	21	333	40	167
BF2.2	0.0	0	0.0	0	0	0	0	BF2.2	1500	9.8	738	5	49	787	40	393
BF4	0.0	0	0.0	0	0	0	0	BF4	1500	10.0	750	5	50	800	40	400
BFC	0.0	0	0.0	0	0	0	0	BFC	1500	2.0	150	5	10	160	40	80
BF6	0.0	0	0.0	0	0	0	0	BF6	1500	8.8	656	5	44	700	40	350
Interior GB	0.0	0	0.0	0	0	0	0	14/S3.2	525	10.3	775	5	52	827	40	413
Exterior GB perp	0.0	0	0.0	0	0	0	0	14/S3.2	525	6.7	506	5	34	540	60	405
Exterior GB para	0.0	0	0.0	0	0	0	0	14/S3.2	525	2.0	150	5	10	160	60	120
Ext Edge GB	0.0	0	0.0	0	0	0	0	9/S3.2	525	4.0	300	5	20	320	60	240

Pile Load Key Plans

ALL LOADS AT BASE



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8480 Residence

PROJECT

02/24/2022

DATE

01519-2021-09

PROJ. #

LAN

DESIGN

F-16

SHEET

Pile Design - Distributed Load (Wall + Dist. Pt)

Pile Dia. (in)	Cap (tons)	ASD (lbs)
3	6	12000
4	10	20000
6	25	50000

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 ASD (plf)	Pile Dia. (in)	Spacing Req (ft)	Spacing Used (ft)	Spacing OK?
FW1	403	308	371	403	202	-	0	-	0	-	0	1686	4	11.9	8.00	OK
RW2+FW2	506	284	525	160	80	-	0	-	0	-	0	1555	4	12.9	10.25	OK
FW3	336	151	371	273	137	-	0	-	0	-	0	1268	4	15.8	9.58	OK
FW4	165	0	371	273	137	-	0	-	0	-	0	946	4	21.1	10.33	OK
RW2+FW4	424	184	371	273	137	-	0	-	0	-	0	1389	4	14.4	10.33	OK
FW5	165	0	371	410	205	-	0	-	0	-	0	1151	4	17.4	7.92	OK
RW3+FW6	780	611	371	400	200	-	0	-	0	-	0	2362	4	8.5	2.00	OK
FW7	165	0	371	400	200	-	0	-	0	-	0	1136	4	17.6	9.58	OK
RW2+FW7	424	184	371	400	200	-	0	-	0	-	0	1579	4	12.7	9.58	OK
FW8	165	0	371	400	200	-	0	-	0	-	0	1136	4	17.6	7.00	OK
FW9+FDP1	1368	90	371	400	200	FDP1	13	-	0	-	0	2441	4	8.2	7.67	OK
FW9+FDP2	1368	90	371	400	200	FDP2	27	-	0	-	0	2455	4	8.1	7.67	OK
FW9+FPD3	1368	90	371	400	200	FDP3	25	-	0	-	0	2453	4	8.2	7.67	OK
FW10	1223	40	371	80	40	-	0	-	0	-	0	1754	4	11.4	5.04	OK
FW10+FDP4+RDP1	1223	40	371	80	40	FDP4	1178	RDP1	950	-	0	3882	4	5.2	5.04	OK
FW10+FDP5+RDP2	1223	40	371	80	40	FDP5	864	RDP2	127	-	0	2745	4	7.3	5.04	OK
FW10+FDP5	1223	40	371	80	40	FDP5	864	-	0	-	0	2617	4	7.6	5.04	OK
FW10+FDP5+RDP3	1223	40	371	80	40	FDP5	864	RDP3	318	-	0	2936	4	6.8	5.04	OK
FW10+FDP12+RDP18	1223	40	371	80	40	FDP12	1618	RDP18	391	-	0	3763	4	5.3	2.92	OK
FW10+FDP13	1223	40	371	80	40	FDP13	900	-	0	-	0	2654	4	7.5	4.79	OK
FW10+FDP14+RDP19	1223	40	371	80	40	FDP14	535	RDP19	1118	-	0	3407	4	5.9	3.42	OK
FW10+FDP14+FDP15+RDP19	1223	40	371	80	40	FDP14	535	FDP15	864	RDP19	1118	4271	4	4.7	3.42	OK
FW10+FDP15	1223	40	371	80	40	FDP15	864	-	0	-	0	2617	4	7.6	3.42	OK
FW10+FDP15+FDP16+RDP21	1223	40	371	80	40	FDP15	864	FDP16	1255	RDP21	1055	4927	4	4.1	3.42	OK
FW10+FDP15+RDP20	1223	40	371	80	40	FDP15	864	RDP20	355	-	0	2972	4	6.7	3.42	OK
FW10+FDP16+RDP21	1223	40	371	80	40	FDP16	1255	RDP21	1055	-	0	4063	4	4.9	3.42	OK

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

02/24/2022
DATE

01519-2021-09
PROJ. #

LAN
DESIGN

SHEET

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)	Pile Dia. (in)	Spacing Req (ft)	Spacing Used (ft)	Spacing OK?
RW5+FW11	1403	90	371	80	40	-	0	-	0	-	0	1984	4	10.1	6.33	OK
RW5+FW11+FDP6	1403	90	371	80	40	FDP6	507	-	0	-	0	2491	4	8.0	6.33	OK
FW11+FDP10	1223	40	371	80	40	FDP10	673	-	0	-	0	2427	4	8.2	4.25	OK
FW11+(2)FDP10	1223	40	371	80	40	FDP10	673	FDP10	673	-	0	3099	4	6.5	4.25	OK
RW7+FW11+FDP9+FPD10+RDP6+RDP7	1639	269	371	80	40	FDP9+FDP10	1815	RDP6	401	RDP7	140	4755	4	4.2	2.83	OK
RW7+FW11+FDP9+RDP6+RDP7	1639	269	371	80	40	FDP9	1143	RDP6	401	RDP7	140	4083	4	4.9	2.83	OK
RW7+FW11+FPD10+RDP7	1639	269	371	80	40	FDP10	673	RDP7	140	-	0	3211	4	6.2	2.83	OK
RW7+FW11+FDP9+FPD10+RDP7	1639	269	371	80	40	FDP9	1143	FDP10	673	RDP7	140	4354	4	4.6	2.83	OK
RW7+FW11+FDP10+RDP8	1639	269	371	80	40	FDP10	673	FDP10	673	RDP8	173	3917	4	5.1	4.25	OK
RW7+FW11+FDP10+RDP8+RDP9	1639	269	371	80	40	(2)FDP10	1345	RDP8	173	RDP9	289	4205	4	4.8	4.25	OK
RW7+FW11+FDP10+RDP10+RDP11	1639	269	371	80	40	FDP10	673	RDP10	286	RDP11	400	3757	4	5.3	4.25	OK
RW8+FW12	2063	327	371	340	170	-	0	-	0	-	0	3271	4	6.1	5.73	OK
FW11+FDP10+RDP14	1223	40	371	80	40	FDP10	673	FDP10	673	RDP14	253	3352	4	6.0	4.25	OK
FW11+FDP10+RDP15	1223	40	371	80	40	FDP10	673	FDP10	673	RDP15	299	3398	4	5.9	4.25	OK
RW9+FW11+FDP10+RDP12+RDP13	1871	193	371	80	40	FDP10	673	RDP12	397	RDP13	126	3752	4	5.3	4.25	OK
RW9+FW11+FDP10+FDP11+RDP16	1871	193	371	80	40	FDP10+FDP11	1739	RDP16	140	-	0	4434	4	4.5	2.83	OK
RW9+FW11+FDP10+FDP11+RDP16+RDP17	1871	193	371	80	40	FDP10+FDP11	1739	RDP16	140	RDP17	404	4839	4	4.1	2.83	OK
RW6+FW13+FDP7+RDP4	2682	409	371	593	297	FDP7	122	RDP4	45	-	0	4519	4	4.4	3.82	OK
BFB	0	0	1500	333	167	-	0	-	0	-	0	2000	4	10.0	7.00	OK
BF2.2	0	0	1500	787	393	-	0	-	0	-	0	2680	4	7.5	5.75	OK
BF4	0	0	1500	800	400	-	0	-	0	-	0	2700	4	7.4	7.00	OK
BFC	0	0	1500	160	80	-	0	-	0	-	0	1740	4	11.5	6.33	OK
BF6	0	0	1500	700	350	-	0	-	0	-	0	2550	4	7.8	5.50	OK
Interior GB	0	0	525	827	413	-	0	-	0	-	0	1765	4	11.3	10.00	OK
Exterior GB perp	0	0	525	540	405	-	0	-	0	-	0	1470	4	13.6	12.00	OK
Exterior GB para	0	0	525	160	120	-	0	-	0	-	0	805	4	24.8	12.00	OK
Ext Edge GB	0	0	525	320	240	-	0	-	0	-	0	1085	4	18.4	11.00	OK

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

DATE 02/24/2022
PROJ. # 01519-2021-09
DESIGN LAN
SHEET

Pile Design - Point Load

Pile Dia. (in)	Cap (tons)	ASD (lbs)
3	6	12000
4	10	20000
6	25	50000

Rxns from grade beams with point loads on them:

Name	Rxn (kips)	Ftg (plf)	GB L (ft)	Ftg Rxn (lbs)	Total Rxn (lbs)	No. of 4" Piles
FW1+FP1 pt	6.8	371	7.4	1375	8148	1
FW1+FP4 pt	6.9	371	6.9	1275	8144	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
FP2+RP1	FP2	3000	RP1	7500	FW1	1686	7.4	8148	RW2+FW2	1555	10.3	7968	RW2+FW2	1555	3.6	2786	BF6	2550	5.5	7013	36414	2
FP3+RP2	FP3	6200	RP2	4700	BF6	2550	5.5	7013	Interior GB	1765	4.5	3971		0		0		0		0	21884	2
FW1+FW2 int	-	0	-	0	FW1	1686	7.4	7992	RW2+FW2	1555	3.6	2786		0		0		0		0	10777	1
FW1+FW1 int	-	0	-	0	FW1	1686	7.4	8143	FW1	1686	2.3	1968		0		0		0		0	10111	1
FW1+FW3 int	-	0	-	0	FW1	2	6.9	8	FW3	1268	9.58	6074		0		0		0		0	6082	1
FP8+RP3	FP8	10707	RP3	5700	Interior GB	1765	4.3	3751	FW3	1268	9.58	6074	Exterior GB para	805	6.8	2717	FW4	946	10.3	4887	33836	2
FP9+RP4	FP9	15852	RP4	10500	RW2+FW4	1389	3.3	2314	FW4	946	7.0	3310		0		0		0		0	31977	2
RW2+FW4 + Interior GB int	-	0	-	0	FW4	946	7.0	3310	FW4	946	8.4	3980	Interior GB	1765	7.4	6508		0		0	13799	1
FP7	FP7	2200	-	0	Exterior GB perp	1470	6.9	5084	Exterior GB perp	1470	7.7	5635		0		0		0		0	12919	1
FP10+RP5	FP10	19203	RP5	7500	FW4	946	8.4	3980	FW5	1151	5.6	3213	Exterior GB perp	1470	11.4	8391	Exterior GB para	805	6.8	2717	45004	3
Exterior GB perp + para int	-	0	-	0	Exterior GB perp	1470	9.6	7044	Exterior GB perp	1470	9.6	7044	Exterior GB para	805	8.2	3287	Exterior GB para	805	5.4	2180	19555	1
Exterior GB perp + para int (edge)	-	0	-	0	Exterior GB perp	1470	9.6	7044	Exterior GB perp	1470	9.6	7044	Exterior GB para	805	5.5	2214		0		0	16301	1
Exterior GB perp + FW5 int	-	0	-	0	Exterior GB perp	1470	5.7	4196	FW5	1151	5.6	3213	FW5	1151	7.2	4124		0		0	11532	1
FP11+RP6	FP11	3251	RP6	8900	BFC	1740	5.0	4350	FW5	1151	7.2	4124	RW3+FW6	2362	1.9	2263		0		0	22888	2
FP12+RP7	FP12	36800	RP7	9400	BF4	2700	7.1	9563	FW7	1136	9.6	5443	RW3+FW6	2362	1.9	2263		0		0	63468	4
FP13+RP8	FP13	10700	RP8	16500	BF4	2700	7.1	9563	Interior GB	1765	5.2	4560		0		0		0		0	41322	3
FP14+RP9	FP14	19300	RP9	4500	BFC	1740	5.0	4350	BFC	1740	3.4	2973		0		0		0		0	31123	2
Interior GB + BF5 int	-	0	-	0	BFC	1740	3.4	2973	BFC	1740	6.4	5583	Interior GB	1765	4.9	4339	Interior GB	1765	5.5	4854	17748	1
FP15+RP10+FP16 +RP11	FP15+ FP16	17881	RP10+ RP11	9600	BFC	1740	6.4	5583	BFC	1740	4.0	3480		0		0		0		0	36544	2
FP17	FP17	17681	-	0	BFC	1740	4.0	3480	RW2+FW2	1555	10.3	7968	Interior GB	1765	7.4	6508	Interior GB	1765	7.3	6423	42060	3
Interior GB + FW7 int	-	0	-	0	FW7	1136	4.8	2721	FW7	1136	4.8	2721	Interior GB	1765	8.2	7207	Interior GB	1765	5.8	5148	17798	1
Interior GB + FW7 int	-	0	-	0	FW7	1136	8.5	4827	FW7	1136	6.2	3502	Interior GB	1765	5.8	5148		0		0	13477	1
FP18+RP13	FP18	41454	RP13	23500	FW7	1136	4.8	2721	FW5	1151	4.0	2302	Interior GB	1140	5.0	2850	Interior GB	1765	4.1	3585	76412	4
FP19	FP19	32037	-	0	Exterior GB perp	1470	9.7	7105	Exterior GB para	805	5.5	2214	FW8	1136	7.0	3975	FW5	1151	4.0	2302	47633	3
FP20	FP20	12057	-	0	FW8	1136	7.0	3975	Exterior GB perp	1470	3.3	2450	FW5	1151	4.0	2302		0		0	20784	2
FP22	FP22	24700	-	0	Interior GB	1765	4.0	3530	FW5	1151	4.0	2302	FW7	1136	6.2	3502		0		0	34034	2
FP23+RP14	FP23	24890	RP14	13300	FW7	1136	8.5	4827	Exterior GB para	805	8.0	3220	RW3+FW6	2362	1.7	1968		0		0	48205	3
FP24+RP15	FP24	9200	RP15	19500	Interior GB	1765	4.0	3530	BF2.2	2680	8.8	11725		0		0		0		0	43955	3
FP25+RP16+FP26 +RP17	FP25+ FP26	22100	RP16+ RP17	15800	Interior GB	1765	9.5	8384	BF2.2	2680	10.8	14405	BFB	2000	5.8	5750		0		0	66439	4
FP27+RP18	FP27	10900	RP18	3700	BFB	2000	5.8	5750		0		0		0		0		0		0	20350	2

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

02/24/2022
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
RW3+FW6 + FW9+FDP1 int	-	0	-	0	RW3+FW6	2362	5.1	6003	FW9+FDP1	2441	5.8	7043		0		0		0		0	13046	1
FW9+FPD3 + FW10+FDP4 +RDP1 int	-	0	-	0	FW9+FPD3	2453	5.8	7078	FW10+FDP4+ RDP1	3863	3.4	6546		0		0		0		0	13623	1
FW10+FDP5 +RDP2 + Interior GB int	-	0	-	0	FW10+FDP5+ RDP2	2745	3.4	4651	FW10+FDP5+ RDP2	2745	3.4	4651	Interior GB	1765	5.8	5093		0		0	14394	1
FW10+FDP5 +RDP3 + Interior GB int	-	0	-	0	FW10+FDP5+ RDP3	2936	3.4	4974	FW10+FDP5	2617	3.2	4144	Interior GB	1765	5.8	5093		0		0	14211	1
FW10+FDP5+ RDP2+RW5+ FW11+ FDP6 int	-	0	-	0	FW10+FDP5	2617	3.2	4144	RW5+FW11+ FDP6	2491	3.2	3944		0		0		0		0	8089	1
RW5+FW11+ FDP6 + Interior GB int	-	0	-	0	RW5+FW11+ FDP6	2491	3.2	3944	RW5+FW11+ FDP6	2491	3.2	3944	Interior GB	1765	5.7	5056		0		0	12945	1
Garage SW int at concrete walls	-	0	-	0	RW5+FW11	1984	3.2	3141	RW7+FW11+ FDP9+RDP6+ RDP7	4083	2.8	5784	RW6+FW13+ FDP7+RDP4	4519	3.8	8630		0		0	17555	1
W Garage int w/ Interior GB	-	0	-	0	RW7+FW11+ FPD10+RDP7	3211	2.8	4549	RW7+FW11+ FDP10+RDP8 +RDP9	4205	4.3	8936	Interior GB	1765	5.7	5056		0		0	18541	1
W Garage int w/ Interior GB	-	0	-	0	FW11+FDP10	2427	4.3	5156	RW7+FW11+ FDP10+RDP8 +RDP9	4205	4.3	8936	Interior GB	1765	5.7	5056		0		0	19148	1
Garage NW corner int at concrete walls	-	0	-	0	RW7+FW11+ FDP10+RDP10+ RDP11	3757	4.3	7984	RW8+FW12	3271	5.7	9369		0		0		0		0	17353	1
Garage NE corner int at concrete walls	-	0	-	0	RW9+FW11+ FDP10+RDP12+ RDP13	3752	4.3	7972	RW8+FW12	3271	5.7	9369		0		0		0		0	17341	1
Garage int w/ Interior GB	-	0	-	0	FW11+FDP10+ RDP14	3352	2.8	4749	FW11+FDP10+ RDP15	3398	2.8	4814	Interior GB	1765	7.6	6741		0		0	16304	1
E Garage int w/ Interior GB	-	0	-	0	FW11+FDP10+ RDP14	3352	4.3	7123	FW11+FDP10+ RDP15	3398	4.3	7222	Interior GB	1765	5.7	5056		0		0	19401	1
E Garage int w/ Interior GB	-	0	-	0	FW11+FDP10+ RDP15	3398	4.3	7222	RW9+FW11+ FDP10+FDP11+ RDP16	4041	2.8	5724	Interior GB	1765	5.7	5056		0		0	18002	1
FW10+FDP14+ RDP19 + Interior GB int	-	0	-	0	FW10+FDP14+ RDP19	3407	3.2	5442	FW10+FDP14+ FDP15+RDP19	4271	3.4	7355	Interior GB	1765	5.2	4560		0		0	17357	1
FW10+FDP15+ RDP20 + Interior GB int	-	0	-	0	FW10+FDP15+ RDP20	2972	3.4	5118	FW10+FDP15	2617	3.4	4508	Interior GB	1765	5.8	5074		0		0	14701	1
FW10+FDP16+ RDP201 + Interior GB int	-	0	-	0	FW10+FDP16+ RDP21	3352	3.2	5307	FW10+FDP15+FD P16+RDP21	4927	3.4	8485	Interior GB	1765	5.8	5074		0		0	18866	1

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

02/24/2022
DATE

01519-2021-09
PROJ. #

LAN
DESIGN

SHEET

Pile Design - Point Load With Seismic

Pile Dia. (in)	Cap (tons)	ASD (lbs)
3	6	12000
4	10	20000
6	25	50000

Cumulative Pt Name	Gravity Col Name	BF Name	Tot-1 Pt Load w/ EQ (kips)*	Tot-2 Pt Load w/ EQ (kips)*	Other Gravity Col Name	Other DL (kips)	Other LL (kips)	Tot-1 Pt Load w/ EQ+other (kips)	Tot-2 Pt Load w/ EQ+other (kips)	No. of 4" Piles
FP2+RP1	FP2+RP1	BF6	23.7	24.6	-	0	0	23.7	24.6	2
FP3+RP2	FP3+RP2	BF6	23.7	24.6	-	0	0	23.7	24.6	2
FP12+RP7	FP12+RP7	BF4	58.7	50.8	-	0	0	58.7	50.8	3
FP13+RP8	FP13+RP8	BF4	58.7	50.8	-	0	0	58.7	50.8	3
FP14+RP9	FP14+RP9	BFC	42.4	43.7	-	0	0	42.4	43.7	3
FP15+RP10+FP16 +RP11	FP15+RP10	BFC	42.4	43.7	FP16+RP11	5.1	3.9	51.0	49.7	3
FP24+RP15	FP24+RP15	BF2.2	40.1	36.7	-	0	0	40.1	36.7	3
FP25+RP16+FP26 +RP17	FP25+RP16	BF2.2	40.1	36.7	-	0	0	63.7	60.3	4
	FP26+RP17	BFB	23.6	23.6	-	0	0			
FP27+RP18	FP27+RP18	BFB	23.6	23.6	-	0	0	23.6	23.6	2

* see lateral calcs for determination of these numbers - tot-1 = 1.1172D+0.525E+0.75L, tot-2 = 1.1641D+0.7E

STRUCTURAL ENGINEERING

8480 Residence
PROJECT

DATE 02/24/2022
PROJ. # 01519-2021-09
DESIGN LAN
SHEET

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
ϕVn =	14.8	kips

shear reinf:

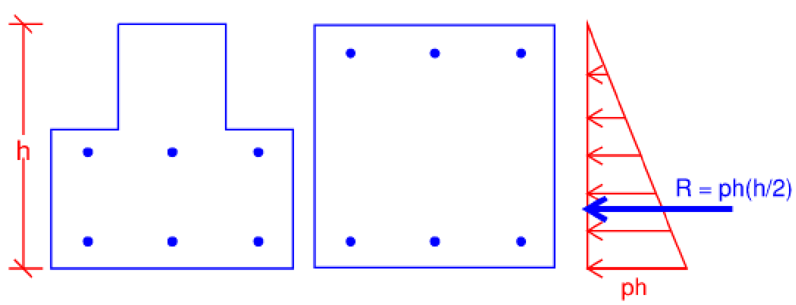
min spacing =	6.0	in, ACI 18.13.3.2	
Vu > 0.5ϕVc?	YES	so use ACI Table 9.6.3.3	
min Av/s =	0.180	in^2/ft	
use =	#3 @ 6"oc		
have =	0.220	in^2/ft	OK

** TL is factored
** span piles max 10'-4" apart

Wall Name	TL (plf)	L (ft)	Mu (k-ft)	M OK?	Vu (kips)	V OK?	Vu > 0.5ϕVc?
FW1	1782	8.0	14.3	OK	7.1	OK	NO
RW2+FW2	1381	10.3	18.1	OK	7.1	OK	NO
FW3	1191	9.6	13.7	OK	5.7	OK	NO
FW4	745	10.3	9.9	OK	3.8	OK	NO
RW2+FW4	1349	10.3	18.0	OK	7.0	OK	NO
FW5	1018	7.9	8.0	OK	4.0	OK	NO
RW3+FW6	2714	2.0	1.4	OK	2.7	OK	NO
RW2+FW7	1603	9.6	18.4	OK	7.7	OK	YES
FW8	998	7.0	6.1	OK	3.5	OK	NO
FW9+FDP2	2621	7.7	19.3	OK	10.0	OK	YES
FW10+FDP15+FDP16+ RDP21	6073	3.4	8.9	OK	10.4	OK	YES
RW9+FW11+FDP10+F DP11+RDP16+RDP17	5797	2.8	5.8	OK	8.2	OK	YES
RW6+FW13+FDP7+RD P4	5276	3.8	9.6	OK	10.1	OK	YES
Ext Edge GB	768	11.0	11.6	OK	4.2	OK	NO
FW1+FP1 pt	1782	7.4	12.9	OK	6.8	OK	NO
FW1+FP4 pt	1782	6.9	11.3	OK	6.9	OK	NO

Lateral Loads

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 =	93.4	kips, ASD

rxns from the basement wall restrained at the SOG:

soil rxn =	3.2	klf
traffic rxn =	0.4	klf
earthquake =	0.5	klf
N rxn 1 =	3.8	klf, includes active/traffic/seismic (ASD - 1.0H+0.75L+0.525E)
N rxn 2 =	3.7	klf, includes active/traffic (ASD - 1.0H+1.0L)
N rxn 1 =	3.6	klf, includes active/traffic/seismic (ASD - 1.0H+0.75L)
N rxn 2 =	3.7	klf, includes active/traffic/seismic (ASD - 1.0H+1.0L)

<u>Global Sliding Check - Pushing South (w/ Seismic)</u>

pushing south will govern over pushing north (and will govern over pushing east/ west)

L w/ N rxn 1 =	24.2	ft	
L w/ N rxn 2 =	36.5	ft	
Tot sliding =	320	kips	
Tot GB L =	368	ft	
Sliding =	871	plf	
Pressure =	208	pcf	
min h =	2.0	ft	<< use 2'-0"

<u>Global Sliding Check - Pushing South (w/OUT Seismic)</u>

L w/ N rxn 1 =	24.2	ft	
L w/ N rxn 2 =	36.5	ft	
Tot sliding =	220	kips	
Tot GB L =	368	ft	
Sliding =	598	plf	
Pressure =	167	pcf	
min h =	1.9	ft	<< 2'-0" OK

<u>Local Out of Plane Check</u>

w max =	871	plf
max L =	11.0	ft
Mu =	9.6	kip-ft, cont. span moment
Vu =	9.6	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 24"dp (no stem wall)

Grade Beam Info

h =	24.0	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 =	20.7	in

Pile Cap Punching Shear Check

pile capacity =	20	kips (Vu)	
pile cap L =	6.0	in	
pile cap W =	6.0	in	
c1 =	26.7	in	
c2 =	26.7	in	
b0 =	106.75	in	
ϕVn =	419	kips	OK

Moment and Shear Check

a =	0.91	in
ϕMn =	734	kip-in
	61.2	kip-ft
ϕVn =	35.3	kips

shear reinf:

min spacing =	9.0	in, ACI 18.13.3.2
Vu > 0.5ϕVc?	NO	
Vu > ϕVc?	NO	so don't need shear reinf.

** TL is factored

** span piles max 12'-0" apart for this grade beam

Wall Name	TL (plf)	L (ft)	Mu (k-ft)	M OK?	Vu (kips)	V OK?
Interior GB	1653	10.0	20.7	OK	8.3	OK
Exterior GB perp	1296	12.0	23.3	OK	7.8	OK
Exterior GB para	384	12.0	6.9	OK	2.3	OK

Local Out of Plane Check

w max =	871	plf	
max L =	12.0	ft	
Mu =	11.4	kip-ft, cont. span moment	
Vu =	10.4	kips, cont. span shear	
h =	24.0	in	
b =	18.0	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 =	20.7	in	
a =	0.61	in	
ϕMn =	493	kip-in	
	41.1	kip-ft	OK
ϕVn =	35.3	kips	OK