



civil & structural
engineering & planning



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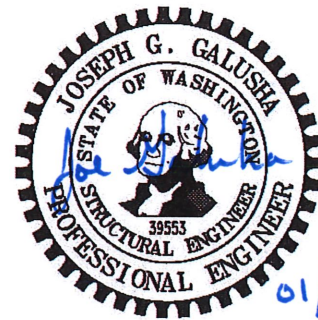
1/30/2023

STRUCTURAL CALCULATIONS

Rose Hill Mukilteo

Structural Design

3rd & Park Ave
Mukilteo, WA 98275



250 4th Ave S Ste 200
Edmonds, WA 98020
Phone: (425) 778-8500
Fax: (425) 778-5536

CG Project No.: 22332.10

Project Description

This project involves the design of a new three story 14-unit mixed use building on a 0.25-acre lot with two levels of wood framing over a partial daylight concrete parking garage. Each floor is approximately 10,600SF. The first floor includes 1900SF of commercial space, but otherwise the upper two floors consist of residential apartments. There is a roof top deck over part of the building. The subgrade garage will consist of concrete footings, walls, slab-on-grade. The main floor consists of a post-tensioned transfer slab. The upper levels will be light wood framing with TJI joists, wood beams, wood stud walls, plywood shear walls, and conventional holdowns.

Scope of Work

We will provide stamped structural calculations and construction drawings in accordance with the current building code and suitable for permit and construction.

Basis of Design

Dead Loads	Roof	16 psf
	Floor	25 psf
	Roof Deck	40 psf
	P/T Floors	30 psf+self weight

Live Loads	Roof(Snow)	25 psf
	Residential	40 psf
	Retail	100 psf
	Roof Deck	100 psf

Wind Parameters

Wind Speed, 3-Sec Gust	98	MPH
Exposure Category	C	
Kzt	1.0	
Mean Height	35	(FT Above Grade Elevation)

Seismic Parameters

$$V = Wp*[Sds/(R/Ie)] = Sds/(6.5/1) = Cs*Wp$$

Sds 1.123

Importance Factor, Ie 1 (Non-Essential Facility)

Wp = Seismic Dead Weight of Structure



250 4th Ave South
Suite 200
Edmonds, WA 98020

Description

Project Summary

Project

Rose Hill Mukilteo

By

CCC

Checked

Scale

NTS

Job No.

22332.10

Date

1/27/2023

Date

Sheet No.

0.001

SECTION NO. 1

Rose Hill Mukilteo
3rd & Park Ave
Mukilteo, WA 98275

SECTION TITLE: Rose Hill Mukilteo Gravity Design

SECTION INDEX

DESCRIPTION	PAGE
<u>Gravity Design Loads</u>	<u>1.01</u>
<u>Beam, Bearing Wall, and Column Design Tables</u>	<u>1.02-1.05</u>
<u>Key Plans</u>	<u>1.06-1.07</u>
<u>Joist Design</u>	<u>1.08-1.13</u>
<u>Beam Design</u>	<u>1.14-1.24</u>
<u>Header Design</u>	<u>1.25-1.26</u>
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Gravity Design Loads

Roof DL

Roofing Material	2.0	psf
3/4 Sheathing	2.2	psf
Insulation	1.0	psf
(2) 5/8 Gypsum	5.6	psf
11 7/8" TJI @ 24" OC	1.5	psf
M/E	1.0	psf
Sprinklers	1.5	psf
Misc	1.0	psf
	15.8	psf
USE	16.0	psf

Roof Deck DL

2" Pavers	25.0	psf
3/4 Sheathing	2.3	psf
Insulation	1.0	psf
(2) 5/8 Gypsum	5.6	psf
11 7/8" TJI @ 24" OC	1.5	psf
M/E	1.0	psf
Sprinklers	1.5	psf
Misc	1.0	psf
	38.9	psf
USE	40.0	psf

Floor DL

1" Gypcrete	8.6	psf
3/4 Sheathing	2.3	psf
Insulation	1.0	psf
(2) 5/8 Gypsum	5.6	psf
11 7/8" TJI @ 16" OC	2.3	psf
M/E	1.0	psf
Sprinklers	1.5	psf
Misc	1.5	psf
	23.8	psf
USE	25.0	psf

Exterior Walls

Siding	2.0	psf
1/2 Sheathing	1.5	psf
Insulation	1.0	psf
5/8 Gypsum	2.8	psf
2x6 @ 16" OC	1.7	psf
Misc	1.0	psf
	10.0	psf
USE	10.0	psf

Roof LL (Snow)	25.0	psf
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Floor LL	40.0	psf
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Deck LL	60.0	psf
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250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description Gravity Design Loads	By CCC	Date 11/18/22
	Checked	Date
	Scale	Sheet No.
Project Rose Hill Mukilteo	Job No. 22332.10	1.01

Beam Span Table - Roof Beams

Allowable Uniform Distributed Load in Pounds Per Lineal Foot (PLF)																	
Beam	Span Length in Feet																
	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
4x6 HF #2	937	600	417	306	234	185	150	124	104	-	-	-	-	-	-	-	-
3 1/2 x 5 1/2 LSL	1541	986	685	503	369	259	189	142	109	-	-	-	-	-	-	-	-
4x8 HF #2	1461	1038	721	529	405	320	259	214	180	154	132	115	101	-	-	-	-
3 1/2 x 7 1/4 LSL	2616	1674	1163	854	654	517	419	321	247	195	156	127	104	-	-	-	-
6x8 DF #2	2162	1384	961	706	541	427	346	286	240	205	176	154	135	120	107	-	-
2 11/16 x 9 1/4 PSL	2405	1924	1603	1374	1193	942	763	631	530	452	378	307	253	211	178	151	130
4x10 HF #2	1863	1490	1084	796	610	482	390	322	271	231	199	173	152	135	120	108	-
4x12 HF #2	2266	1812	1469	1080	827	653	529	437	367	313	270	235	207	183	163	147	132
5 1/4 x 9 1/4 PSL	5399	4319	3600	3085	2677	2115	1713	1416	1183	931	745	606	499	416	351	298	256
2 11/16 x 9 1/2 PSL	2470	1976	1647	1411	1235	991	802	663	557	475	409	334	275	229	193	164	141
3 1/2 x 9 1/2 LSL	3634	2907	2423	1893	1449	1145	927	766	643	506	405	329	271	226	191	162	139
3 1/2 x 9 1/2 PSL	3700	2960	2467	2114	1850	1482	1201	992	834	674	540	439	362	302	254	216	185
6x10 DF #2	3404	2219	1541	1132	867	685	555	458	385	328	283	247	217	192	171	154	139
5 1/4 x 9 1/2 PSL	5545	4436	3697	3169	2773	2224	1802	1489	1251	1011	810	658	543	452	381	324	278
7 x 9 1/2 PSL	7390	5912	4927	4223	3695	2966	2402	1985	1668	1349	1080	878	723	603	508	432	370
2 11/16 x 11 1/4 PSL	2925	2340	1950	1671	1463	1300	1104	912	767	653	563	491	431	382	325	276	237
3 1/2 x 11 1/4 LSL	4301	3441	2867	2458	2001	1581	1281	1058	889	758	653	547	450	375	316	269	231
3 1/2 x 11 1/4 PSL	4382	3505	2921	2504	2191	1947	1653	1366	1148	978	843	729	600	501	422	359	307
6x12 DF #2	4123	3253	2259	1660	1271	1004	813	672	565	481	415	361	318	281	251	225	203
5 1/4 x 11 1/4 PSL	6567	5253	4378	3752	3283	2918	2480	2050	1722	1468	1265	1097	904	754	635	540	463
2 11/16 x 11 7/8 PSL	3085	2468	2057	1763	1543	1371	1222	1010	849	723	624	543	478	423	377	324	278
3 1/2 x 11 7/8 LSL	4543	3634	3028	2596	2220	1754	1420	1174	986	841	725	631	530	441	372	316	271
3 1/2 x 11 7/8 PSL	4623	3698	3082	2642	2312	2055	1831	1513	1271	1083	934	814	709	591	498	423	363
5 1/4 x 11 7/8 PSL	-	5548	4623	3963	3467	3082	2747	2270	1908	1626	1402	1221	1063	887	747	635	544
7 x 11 7/8 PSL	-	-	6160	5280	4620	4107	3663	3027	2543	2167	1869	1628	1411	1176	991	842	722

Notes:

1. This table is applicable for Simple Span beams with uniformly distributed loads (no point loads)
2. Table values are based on the limiting beam shear & moment capacities, as well as deflection
3. The deflection limit used in the above table is (L/180 Total Load) and (L/240 Snow Load)
4. This table is applicable for $W_{LL}/W_{DL} \leq 3.0$
5. Table values include the Size Factor (C_F) and the Load Duration Factor (C_D)



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Beam Span Table	By	CCC	Date	12/02/22
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.	22332.10		1.02

Beam Span Table - Floor Beams

Allowable Uniform Distributed Load in Pounds Per Lineal Foot (PLF)																	
Beam	Span Length in Feet																
	8	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
4x6 HF #2	232	522	362	266	204	160	117	-	-	-	-	-	-	-	-	-	-
3 1/2 x 5 1/2 LSL	280	858	546	344	230	162	118	-	-	-	-	-	-	-	-	-	-
4x8 HF #2	401	902	627	460	353	279	226	186	155	122	-	-	-	-	-	-	-
3 1/2 x 7 1/4 LSL	633	1456	1011	743	522	367	267	201	155	122	-	-	-	-	-	-	-
6x8 DF #2	535	1203	836	614	470	371	301	249	209	178	153	134	114	-	-	-	-
2 11/16 x 9 1/4 PSL	1283	1924	1603	1374	1193	889	648	487	375	295	236	192	158	132	111	-	-
4x10 HF #2	603	1296	942	692	530	419	339	280	236	201	173	151	133	113	-	-	-
3 1/2 x 9 1/4 PSL	1669	2504	2087	1789	1553	1169	852	640	493	388	310	252	208	173	146	124	106
5 1/4 x 9 1/4 PSL	2504	3756	3130	2683	2328	1753	1278	960	739	582	466	379	312	260	219	186	160
2 11/16 x 9 1/2 PSL	1317	1976	1647	1411	1235	965	704	529	407	320	256	209	172	143	121	103	-
3 1/2 x 9 1/2 LSL	1434	2528	2107	1646	1260	953	694	522	402	316	253	206	170	141	119	101	-
3 1/2 x 9 1/2 PSL	1715	2572	2143	1837	1608	1270	926	696	536	421	337	274	226	188	159	135	116
6x10 DF #2	858	1930	1340	984	754	596	482	399	335	285	246	214	188	167	149	134	118
5 1/4 x 9 1/2 PSL	2573	3860	3217	2757	2413	1905	1389	1043	804	632	506	412	339	283	238	202	174
7 x 9 1/2 PSL	3429	5144	4287	3674	3215	2540	1852	1391	1072	843	675	549	452	377	318	270	231
2 11/16 x 11 1/4 PSL	1560	2340	1950	1671	1463	1300	1104	890	686	539	432	351	289	241	203	173	148
3 1/2 x 11 1/4 LSL	1980	2992	2493	2137	1740	1375	1114	866	667	525	420	342	281	235	198	168	144
3 1/2 x 11 1/4 PSL	2032	3048	2540	2177	1905	1693	1438	1155	889	700	560	455	375	313	264	224	192
6x12 DF #2	1257	2829	1964	1443	1105	873	707	584	491	418	361	314	276	245	218	196	177
5 1/4 x 11 1/4 PSL	3045	4568	3807	3263	2855	2538	2157	1739	1340	1054	844	686	565	471	397	337	289
2 11/16 x 11 7/8 PSL	1645	2468	2057	1763	1543	1371	1222	1010	804	632	506	412	339	283	238	202	174
3 1/2 x 11 7/8 LSL	2107	3160	2633	2257	1930	1525	1235	1018	784	617	494	402	331	276	232	198	169
3 1/2 x 11 7/8 PSL	2144	3216	2680	2297	2010	1787	1592	1316	1050	826	661	538	443	369	311	265	227
5 1/4 x 11 7/8 PSL	-	4824	4020	3446	3015	2680	2389	1974	1575	1239	992	807	665	554	467	397	340
7 x 11 7/8 PSL	-	-	5357	4591	4018	3571	3185	2632	2090	1644	1316	1070	882	735	619	526	451

Notes:

1. This table is applicable for Simple Span beams with uniformly distributed loads (no point loads)
2. Table values are based on the limiting beam shear & moment capacities, as well as deflection
3. The deflection limit used in the above table is (L/240 Total Load) and (L/360 Live Load)
4. This table is applicable for $W_{LL}/W_{DL} \leq 4.0$
5. Table values include the Size Factor (C_F)



Description	Beam Span Table		By	CCC	Date	12/02/22
			Checked		Date	
			Scale		Sheet No.	
	Project	Rose Hill Mukilteo	Job No.	22332.10		1.03

DF Column & DF Sill Plate Capacity TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

	6	7	8	9	10	11	12	13	14	15	16
C1 (2) 2x4 DF stud	5,781	4,680	3,786	3,092	2,559	2,146	1,823	1,565	1,358	1,188	1,048
(4") P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
C2 (3) 2x4 DF Stud	10,202	8,671	7,239	6,029	5,050	4,270	3,645	3,143	2,734	2,398	2,119
(4") P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
C3 (4) 2x4 DF Sud	13,603	11,561	9,652	8,039	6,734	5,693	4,860	4,190	3,645	3,197	2,825
(4") P _{SILL}	13,125	-	-	-	-	-	-	-	-	-	-
(2) 3x4 DF Stud	11,336	9,634	8,043	6,699	5,612	4,744	4,050	3,492	3,038	2,664	2,354
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
(3) 3x4 DF Stud	17,004	14,452	12,065	10,048	8,417	7,116	6,076	5,238	4,556	3,996	3,531
P _{SILL}	16,406	-	-	-	-	-	-	-	-	-	-
C1 (2) 2x6 DF Stud	8,908	7,263	5,899	4,830	4,003	3,361	2,856	2,454	2,129	1,864	1,645
(6") P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
C2 (3) 2x6 DF Stud	18,111	16,739	15,105	13,362	11,677	10,160	8,848	7,734	6,796	6,005	5,337
(6") P _{SILL}	16,758	-	-	-	-	-	-	-	-	-	-
C3 (4) 2x6 DF Stud	25,659	24,573	23,202	21,559	19,722	17,815	15,964	14,252	12,718	11,367	10,190
(6") P _{SILL}	20,625	20,625	20,625	20,625	-	-	-	-	-	-	-
C4 4x6 HF #2	14,409	11,327	9,009	7,286	5,993	5,006	4,239	3,633	3,147	2,751	2,425
P _{SILL}	12,852	-	-	-	-	-	-	-	-	-	-
C5 (4") 4x8 HF #2	18,744	14,808	11,809	9,566	7,876	6,583	5,577	4,782	4,142	3,622	3,193
P _{SILL}	15,859	-	-	-	-	-	-	-	-	-	-
C6 (4") 4x10 HF #2	23,562	18,717	14,972	12,150	10,015	8,377	7,101	6,090	5,277	4,615	4,069
P _{SILL}	20,234	-	-	-	-	-	-	-	-	-	-
C5 (6") 6x6 DF #2	19,595	18,889	17,995	16,908	15,659	14,315	12,960	11,665	10,475	9,407	8,463
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
C6 (6") 6x8 DF #2	25,830	24,899	23,721	22,288	20,642	18,870	17,083	15,377	13,808	12,400	11,156
P _{SILL}	24,922	-	-	-	-	-	-	-	-	-	-
C7 6x10 DF #2	28,621	27,790	26,739	25,450	23,929	22,224	20,420	18,614	16,885	15,285	13,835
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-



Description

Wood Column Capacity Table

Project

Rose Hill Mukilteo

By

CCC

Date

01/24/23

Checked

Date

Scale

Sheet No.

Job No.

22332.10

1.04

BEARING WALL TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

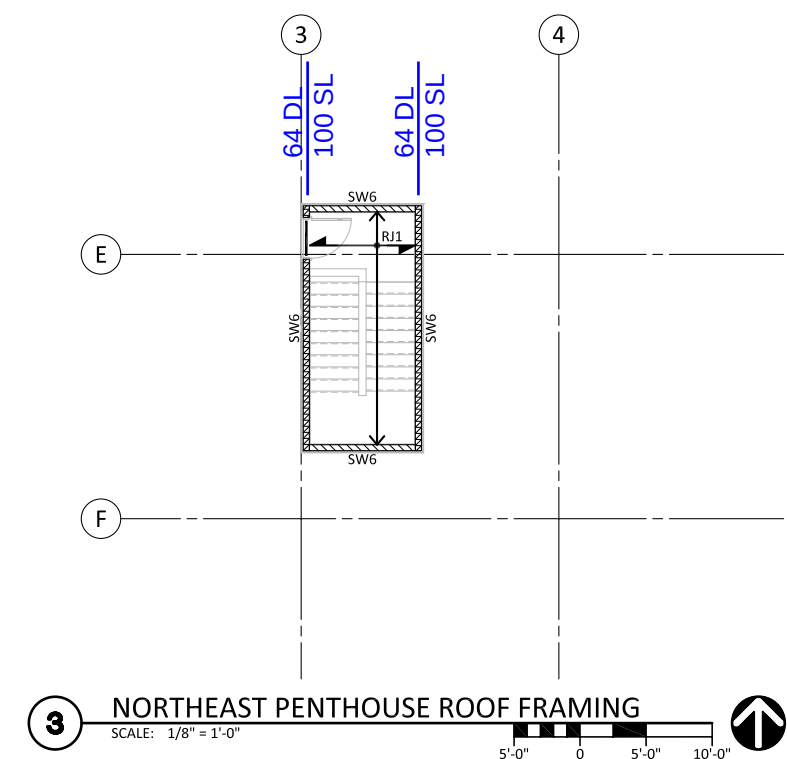
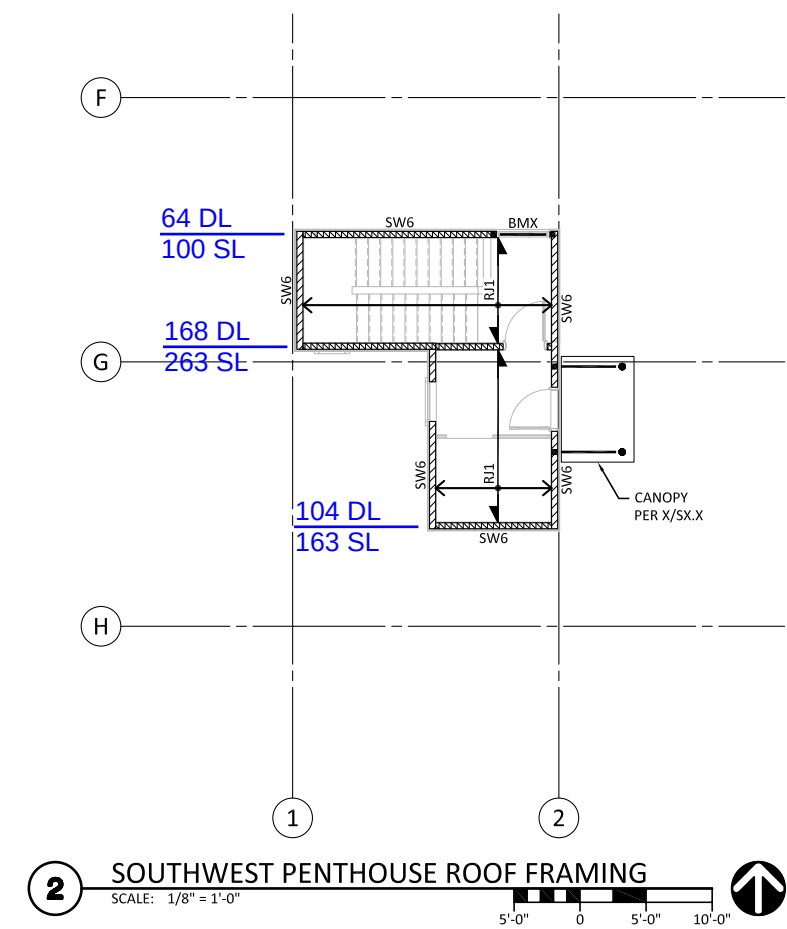
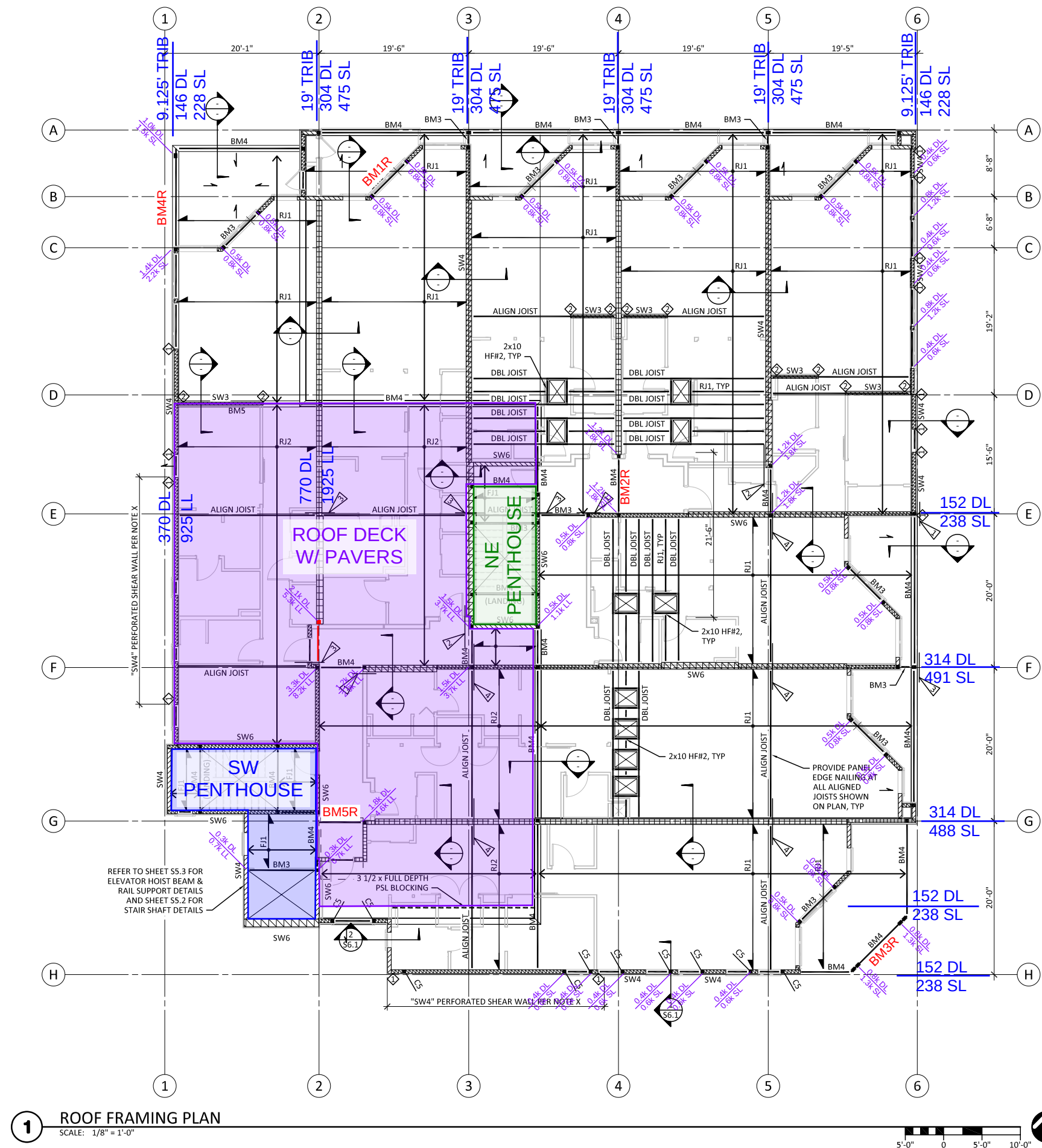
STUD	ALLOWABLE LOAD (PLF)										
	8ft				9ft				10ft		
	8" O.C.	12" O.C.	16" O.C.		8" O.C.	12" O.C.	16" O.C.		8" O.C.	12" O.C.	16" O.C.
2x4 HF Stud Grade	3191	2127	1596		2640	1760	1320		2203	1469	1101
2x4 HF #2	3704	2469	1852		2991	1994	1496		2458	1639	1229
2x4 HF #1	3987	2658	1993		3465	2310	1733		2855	1903	1427
2x4 DF Stud Grade	3620	2413	1810		3014	2010	1507		2525	1683	1263
2x4 DF #2	4474	2983	2237		3635	2424	1818		2999	1999	1499
2x4 DF #1	4808	3205	2404		3901	2601	1950		3215	2143	1607
3x4 HF Stud Grade	5318	3546	2659		4401	2934	2200		3672	2448	1836
3x4 HF #2	6113	4075	3056		4986	3324	2493		4097	2732	2049
3x4 HF #1	6113	4075	3056		5775	3850	2888		4758	3172	2379
3x4 DF Stud Grade	6033	4022	3016		5024	3349	2512		4209	2806	2104
3x4 DF #2	7457	4971	3728		6059	4039	3030		4998	3332	2499
3x4 DF #1	8013	5342	4007		6501	4334	3251		5358	3572	2679
2x6 HF Stud Grade	6265	4177	3132		6265	4177	3132		6265	4177	3132
2x6 HF #2	6265	4177	3132		6265	4177	3132		6265	4177	3132
2x6 HF #1	6265	4177	3132		6265	4177	3132		6265	4177	3132
2x6 DF Stud Grade	8701	5800	4350		8084	5390	4042		7396	4930	3698
2x6 DF #2	9668	6445	4834		9668	6445	4834		9668	6445	4834
2x6 DF #1	9668	6445	4834		9668	6445	4834		9668	6445	4834

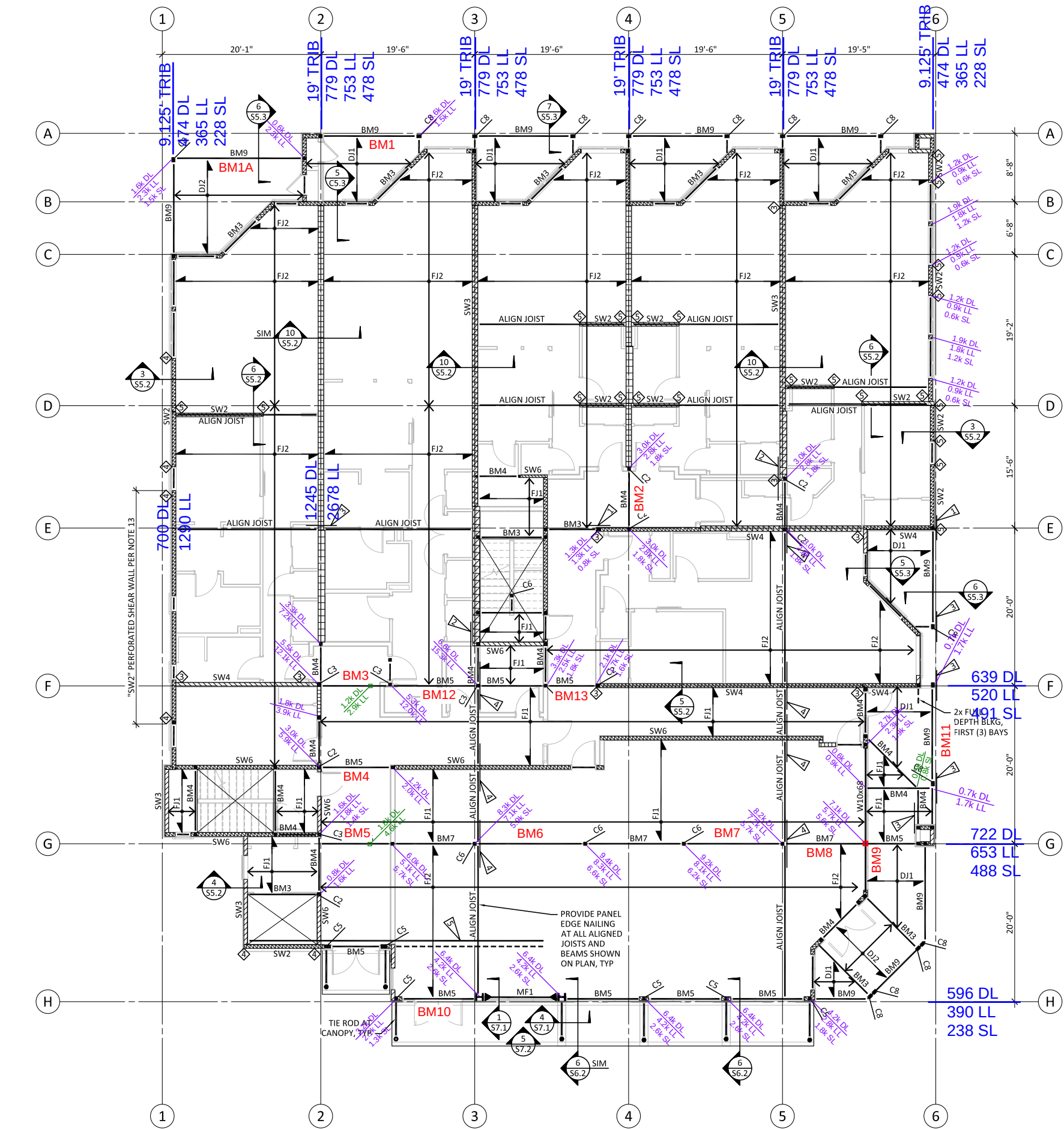
Notes:

1. This table assumes that the studs are braced by either sheathing or gypsum wall board.
2. Values shown are in plf and represent 100% bearing capacity based on the February 2018 report by American Wood Council for "Fire-Resistance-Rated Wood-Frame Wall and Floor/Ceiling Assemblies"
3. **Bold** and *italicized* values are controlled by bottom plate bearing capacity.
4. All DF studs assume a DF bottom plate.
5. All appropriate C_F and C_b factors have been included.
6. Engineer should apply the C_i factor to the allowable loads shown when pressure treated studs are necessary.
7. Engineer should consider out of Plane loads where appropriate.



Description	By	CCC	Date	01/24/23
Bearing Wall Capacity Table	Checked		Date	
Project	Scale		Sheet No.	
Rose Hill Mukilteo	Job No.			
	22332.10			1.05





Joists Table:

Call	Joist Type	Spacing	Loading (psf)	Span (ft)	M	Δ_T	Rating
RJ1	11 ⁷ / ₈ " TJI 110	24" OC	16 DL + 25 SL	18'-3"	96%	L/258	
★ RJ2	11 ⁷ / ₈ " TJI 360	24" OC	40 DL + 100 LL				
FJ1	11 ⁷ / ₈ " TJI 110	16" OC	25 DL + 40 LL	16'-6"	95%	L/398	43
FJ2	11 ⁷ / ₈ " TJI 230	16" OC	25 DL + 40 LL	18'-0"	85%	L/383	40
DJ1	2x6 HF #12	16" OC	25 DL + 60 LL	7'-0"	91%	L/346	-



250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
www.cgengineering.com

Description Joists

Project Rose Hill Mukiteo

By CCC

Checked

Scale

Job No.

22332.10

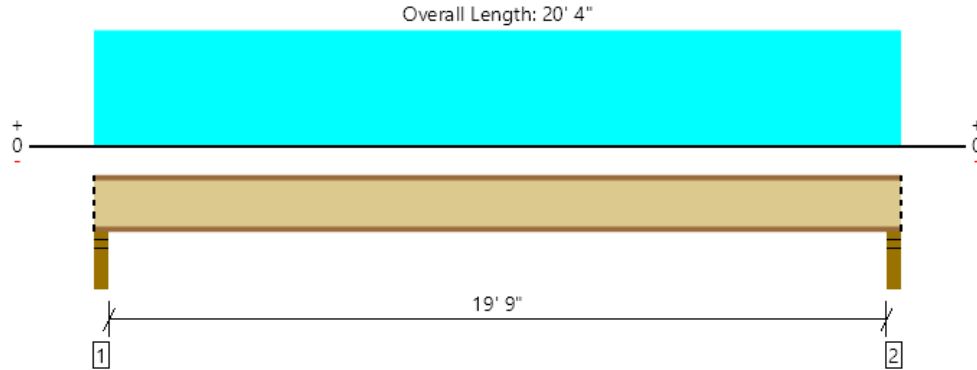
Date 12/2/22

Date

Sheet No.

1.08

Level, RJ1
1 piece(s) 11 7/8" TJI® 210 @ 24" 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)	834 @ 2 1/2"	1679 (3.50")	Passed (50%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	810 @ 3 1/2"	1903	Passed (43%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4066 @ 10' 2"	4364	Passed (93%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.607 @ 10' 2"	0.664	Passed (L/394)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.995 @ 10' 2"	0.996	Passed (L/240)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - SPF	3.50"	3.50"	1.75"	325	508	834	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.75"	325	508	834	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	20' 4" 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)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20' 4"	24"	16.0	25.0	Default Load

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

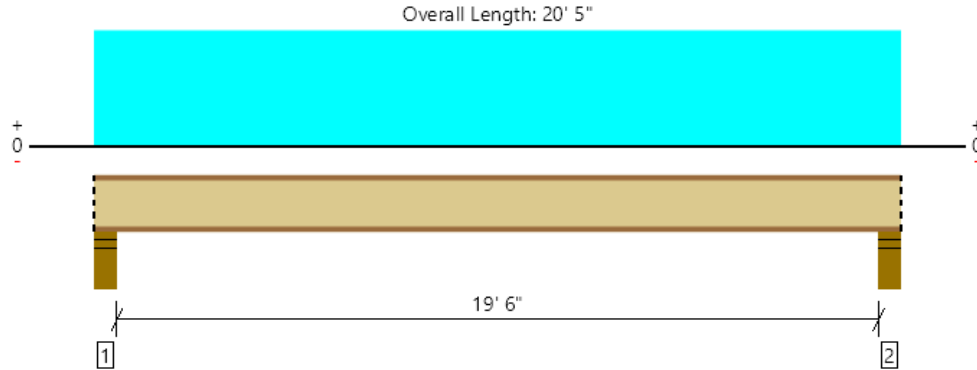
ForteWEB Software Operator	Job Notes
Colin Chase CG Engineering (425) 778-8500 colinc@cgengineering.com	



12/16/2022 10:36:02 PM UTC
ForteWEB v3.5, Engine: V8.2.3.63, Data: V8.1.3.6

File Name: Rose Hill

Level, RJ2
1 piece(s) 14" TJI® 360 @ 12" 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)	1429 @ 4 1/2"	1731 (3.50")	Passed (83%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1365 @ 5 1/2"	2248	Passed (61%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	6769 @ 10' 2 1/2"	8435	Passed (80%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.624 @ 10' 2 1/2"	0.656	Passed (L/378)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.873 @ 10' 2 1/2"	0.983	Passed (L/270)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - HF	5.50"	5.50"	2.42"	408	1021	1429	Blocking
2 - Stud wall - HF	5.50"	5.50"	2.42"	408	1021	1429	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 9" o/c	
Bottom Edge (Lu)	20' 5" 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)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20' 5"	12"	40.0	100.0	Default Load

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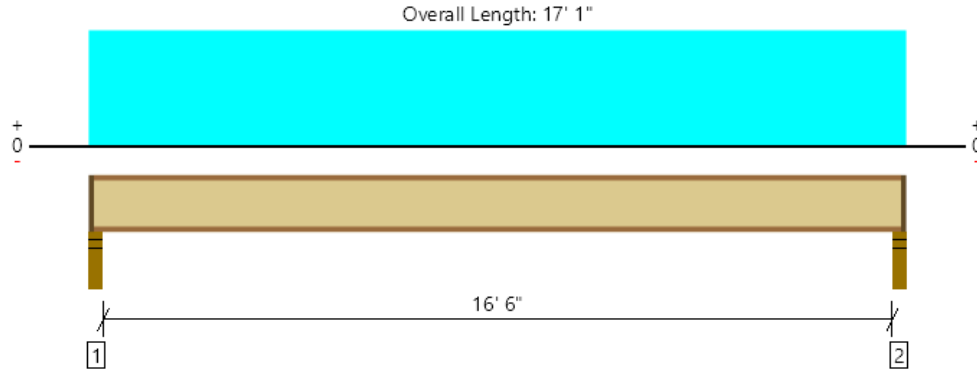
ForteWEB Software Operator	Job Notes
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12/16/2022 10:31:53 PM UTC
ForteWEB v3.5, Engine: V8.2.3.63, Data: V8.1.3.6

File Name: Rose Hill

Level, FJ1
1 piece(s) 11 7/8" TJI® 110 @ 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)	731 @ 2 1/2"	1041 (2.25")	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	715 @ 3 1/2"	1560	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3009 @ 8' 6 1/2"	3160	Passed (95%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.309 @ 8' 6 1/2"	0.417	Passed (L/647)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.502 @ 8' 6 1/2"	0.833	Passed (L/398)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	41	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) 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	Factored	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	285	456	740	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	285	456	740	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)	3' 1" o/c	
Bottom Edge (Lu)	16' 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)	Comments
1 - Uniform (PSF)	0 to 17' 1"	16"	25.0	40.0	Default Load

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

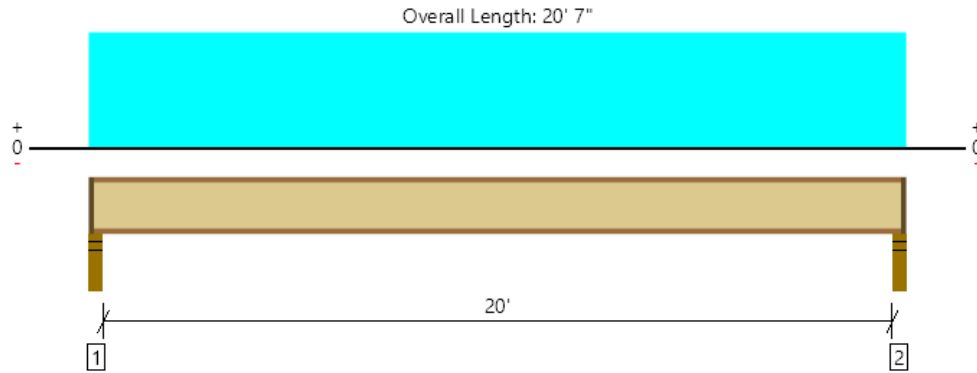
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Colin Chase CG Engineering (425) 778-8500 colinc@cgengineering.com	



12/16/2022 10:40:18 PM UTC
ForteWEB v3.5, Engine: V8.2.3.63, Data: V8.1.3.6

File Name: Rose Hill

Level, FJ2
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)	883 @ 2 1/2"	1202 (2.25")	Passed (73%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	867 @ 3 1/2"	1705	Passed (51%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4406 @ 10' 3 1/2"	6180	Passed (71%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.448 @ 10' 3 1/2"	0.504	Passed (L/540)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.728 @ 10' 3 1/2"	1.008	Passed (L/332)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	35	40	Failed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) 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	Factored	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	343	549	892	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	343	549	892	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)	4' 4" o/c	
Bottom Edge (Lu)	20' 5" 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)	Comments
1 - Uniform (PSF)	0 to 20' 7"	16"	25.0	40.0	Default Load

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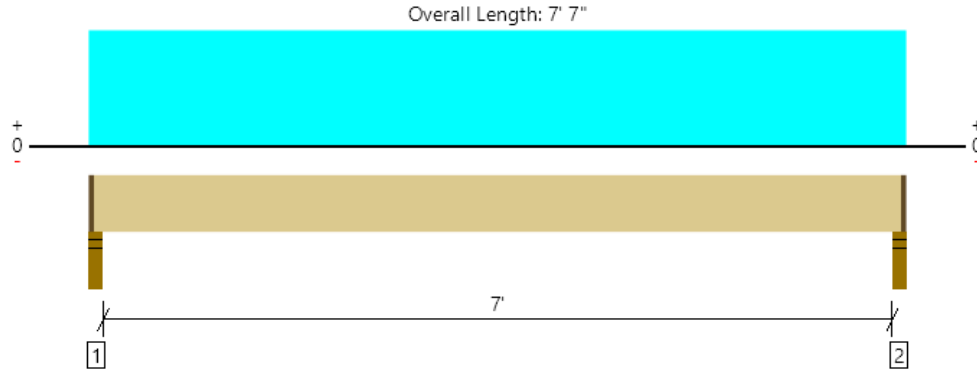
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12/16/2022 10:44:07 PM UTC
ForteWEB v3.5, Engine: V8.2.3.63, Data: V8.1.3.6

File Name: Rose Hill

Level, DJ1
1 piece(s) 2 x 6 HF No.2 @ 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)	418 @ 2 1/2"	1367 (2.25")	Passed (31%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	345 @ 9"	825	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	728 @ 3' 9 1/2"	801	Passed (91%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.176 @ 3' 9 1/2"	0.179	Passed (L/490)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.249 @ 3' 9 1/2"	0.358	Passed (L/346)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	126	303	430	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	126	303	430	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)	6' o/c	
Bottom Edge (Lu)	7' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 7' 7"	16"	25.0	60.0	Default Load

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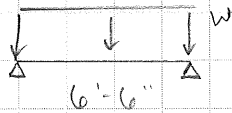


12/16/2022 10:50:33 PM UTC
ForteWEB v3.5, Engine: V8.2.3.63, Data: V8.1.3.6

File Name: Rose Hill

Roof Beams:

BM1R:

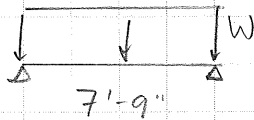


$$W = (161 \text{ DL} + 252 \text{ SL}) \text{ plf}$$

$$R = 523 \text{ DL} + 819 \text{ SL}$$

Use 3 1/2" x 9 1/2" PSL * Per Beam Table

BM2R:

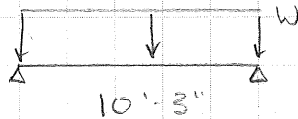


$$W = (304 \text{ DL} + 475 \text{ SL}) \text{ plf}$$

$$R = 1178 \text{ DL} + 1841 \text{ SL}$$

Use 3 1/2" x 9 1/2" PSL * Per Beam Table

BM3R:

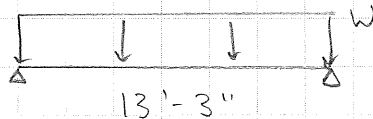


$$W = (157 \text{ DL} + 246 \text{ SL}) \text{ plf}$$

$$R = 805 \text{ DL} + 1260 \text{ SL}$$

Use 3 1/2" x 9 1/2" PSL * Per Beam Table

BM4R:

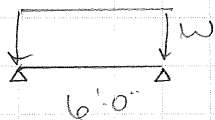


$$W = (144 \text{ DL} + 225 \text{ SL})$$

$$R = 954 \text{ DL} + 1491 \text{ SL}$$

Use 3 1/2" x 9 1/2" PSL * Per Beam Table

BM5R: @ Deck



$$W = 613 \text{ DL} + 1533 \text{ LL}$$

$$R = 1839 \text{ DL} + 4600 \text{ LL}$$

Use 3 1/2" x 11 7/8" PSL * Per Beam Table

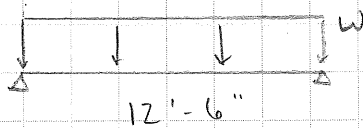


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Description	Roof Beams	By	CCC	Date	12/2/22
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.	22352.10		1.14

Floor Beams:

BM1:

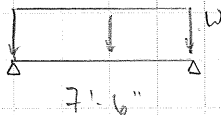


$$w = (64 \text{ DL} + 255 \text{ LL})$$

$$R = 402 \text{ DL} + 1600 \text{ LL}$$

Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL * From Table

BM2:

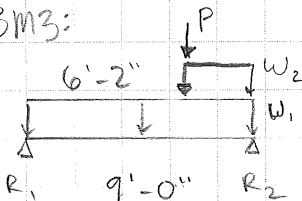


$$w = 475 \text{ DL} + 753 \text{ LL}$$

$$R = 1781 \text{ DL} + 2824 \text{ LL}$$

Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL * From Table

BM3:



$$w_1 = 128 \text{ DL} + 205 \text{ LL}$$

$$w_2 = 380 \text{ DL} + 950 \text{ LL}$$

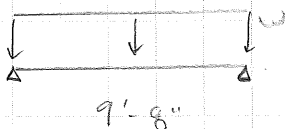
$$P = 1.2 \text{ K DL} + 2.9 \text{ K LL}$$

$$R_1 = 1019 \text{ DL} + 2000 \text{ LL}$$

$$R_2 = 2434 \text{ DL} + 5474 \text{ LL}$$

$M = 65\%$ $V = 76\%$ $\Delta_T = 0.17$ Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL

BM4:

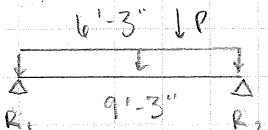


$$w = 253 \text{ DL} + 405 \text{ LL}$$

$$R = 1.2 \text{ K DL} + 2.0 \text{ K LL}$$

Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL * From Table

BM5:



$$w = 260 \text{ DL} + 415 \text{ LL}$$

$$P = 1.8 \text{ K DL} + 4.6 \text{ K LL}$$

$$R_1 = 1.8 \text{ K DL} + 2.0 \text{ K LL} + 1.5 \text{ K SL}$$

$$R_2 = 2.4 \text{ K DL} + 2.0 \text{ K LL} + 3.1 \text{ K SL}$$

$M = 73\%$ $V = 60\%$

$\Delta_T = 0.24$

Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL



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Description

Project Rose Hill Mukiteo

By CCC

Checked

Scale

Job No.

22352.10

Date 12/6/22

Date

Sheet No.

1.15



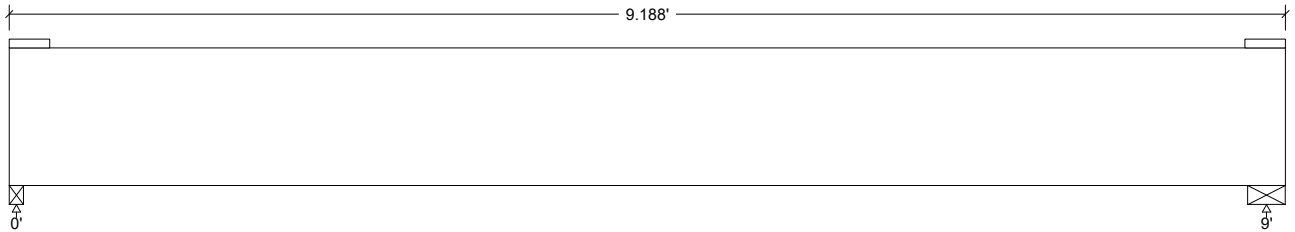
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			128.0	plf
LL	Live	Full UDL			205.0	plf
Dlp	Dead	Point		7.05	1200	lbs
LLp	Live	Point		7.05	2900	lbs
DLw	Dead	Partial UDL		6.22 9.05	380.0 380.0	plf
LLw	Live	Partial UDL		6.22 9.05	950.0 950.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	1019		2434
Live	2001		5474
Factored:			
Total	3020		7908
Bearing:			
Capacity			
Beam	3273		8571
Support	3020		7908
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#2		#2
Length	1.25		3.27**
Min req'd	1.25**		3.27**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 3-1/2"x11-7/8"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 9.19'; Clear span: 8.813'; Volume = 2.7 cu.ft.

Lateral support: top = at supports, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	$f_v = 220$	$F_v' = 290$	psi	$f_v/F_v' = 0.76$
Bending(+)	$f_b = 1817$	$F_b' = 2801$	psi	$f_b/F_b' = 0.65$
Live Defl'n	$0.10 < L/999$	$0.30 = L/360$	in	0.34
Total Defl'n	$0.17 = L/621$	$0.45 = L/240$	in	0.39

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
F_v'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
F_b'	2900	1.00	-	1.00	0.965	1.001	-	1.00	1.00	-	2
F_{cp}'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	2
E_{min}'	1.14 million	-	-	1.00	-	-	-	-	1.00	-	2

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D + L
 Bending(+): LC #2 = D + L
 Deflection: LC #2 = D + L (live)
 LC #2 = D + L (total)
 Bearing : Support 1 - LC #2 = D + L
 Support 2 - LC #2 = D + L

D=dead L=live

All LC's are listed in the Analysis output

Load Patterns: s=S/2, X=L+S or L+Lr, _=no pattern load in this span

Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

$V_{max} = 7862$, $V_{design} = 6087$ (NDS 3.4.3.1(a)) lbs; $M(+) = 12456$ lbs-ft
 $EI = 1074.51e06$ lb-in² Apparent E approximates the effect of shear deflection.
 "Live" deflection is due to all non-dead loads (live, wind, snow...)
 Total deflection = 1.50 permanent + "live"
 Lateral stability(+): $L_u = 9.00'$ $L_e = 17.63'$ $R_B = 14.3$

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.



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Dec. 7, 2022 15:23

BM5

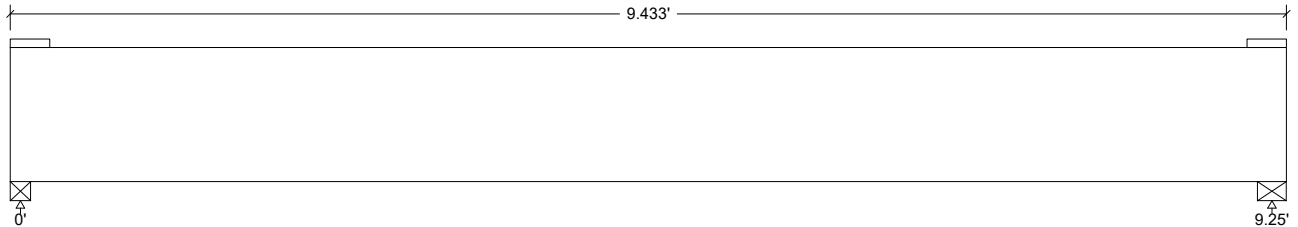
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 4)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			260.0	plf
LL	Live	Full UDL			415.0	plf
Dlp	Dead	Point		6.33	1800	lbs
SLp	Snow	Point		6.33	4600	lbs

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	1806		2447
Live	1951		1964
Snow	1492		3108
Factored:			
Total	4388		6251
Bearing:			
Capacity			
Beam	4756		6775
Support	4388		6251
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#3		#3
Length	1.81		2.58
Min req'd	1.81**		2.58**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 3-1/2"x11-7/8"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 9.44'; Clear span: 9.063'; Volume = 2.7 cu.ft.

Lateral support: top = at supports, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 201	Fv' = 334	psi	fv/Fv' = 0.60
Bending(+)	fb = 2334	Fb' = 3186	psi	fb/Fb' = 0.73
Live Defl'n	0.12 = L/890	0.31 = L/360	in	0.40
Total Defl'n	0.24 = L/453	0.46 = L/240	in	0.53

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	3
Fb'+	2900	1.15	-	1.00	0.954	1.001	-	1.00	1.00	-	3
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	3
Eminy'	1.14 million	-	-	1.00	-	-	-	-	1.00	-	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #3 = D + 0.75(L + S)
 Bending(+): LC #3 = D + 0.75(L + S)
 Deflection: LC #3 = D + 0.75(L + S) (live)
 LC #3 = D + 0.75(L + S) (total)
 Bearing : Support 1 - LC #3 = D + 0.75(L + S)
 Support 2 - LC #3 = D + 0.75(L + S)

D=dead L=live S=snow

All LC's are listed in the Analysis output

Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.1

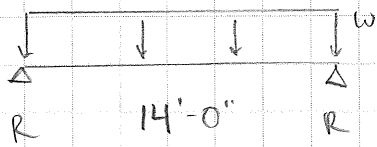
CALCULATIONS:

V max = 6189, V design = 5565 lbs; M(+) = 15997 lbs-ft
 Ely = 1074.51 lb-in^2 Apparent E approximates the effect of shear deflection.
 "Live" deflection is due to all non-dead loads (live, wind, snow...)
 Total deflection = 1.5 dead + "live"
 Lateral stability(+): Lu = 9.25' Le = 18.06' RB = 14.5

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2018) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.

BM6:



Use 7" x 14" PSL

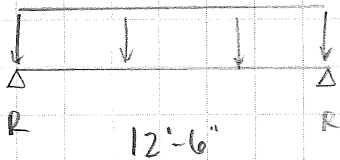
$$W = 680 \text{ DL} + 585 \text{ LL} + 488 \text{ SL}$$

$$R = 4.8 \text{ k DL} + 4.2 \text{ k LL} + 3.5 \text{ k SL}$$

$$M = 58\% \quad V = 39\%$$

$$\Delta_T = L/375 \quad 0.45 \text{ in}$$

BM7:



Use 7" x 14" PSL

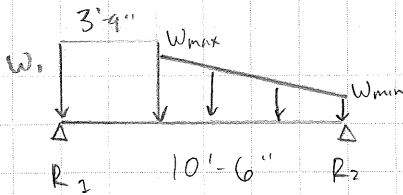
$$W = 719 \text{ DL} + 648 \text{ LL} + 488 \text{ SL}$$

$$R = 4.6 \text{ k DL} + 4.1 \text{ k LL} + 3.1 \text{ k SL}$$

$$M = 49\% \quad V = 36\%$$

$$\Delta_T = L/498 \quad 0.30 \text{ in}$$

BM8:



Use 7" x 14" PSL

$$W_1 = 719 \text{ DL} + 648 \text{ LL} + 488 \text{ SL}$$

$$W_{2\text{MAX}} = 642 \text{ DL} + 525 \text{ LL} + 488 \text{ SL}$$

$$W_{2\text{MIN}} = 551 \text{ DL} + 386 \text{ LL} + 488 \text{ SL}$$

$$R_1 = 3.6 \text{ k DL} + 3.1 \text{ k LL} + 2.6 \text{ k SL}$$

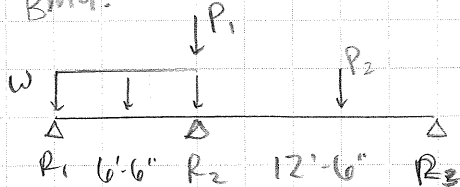
$$R_2 = 3.2 \text{ k DL} + 2.5 \text{ k LL} + 2.6 \text{ k SL}$$

$$M = 31\%$$

$$V = 27\%$$

$$\Delta_T = L/943 \quad 0.13 \text{ in}$$

BM9:



Use 3 1/2" x 11 7/8" PSL

$$W = 94 \text{ p/ft DL} + 150 \text{ p/ft LL}$$

$$P_1 = 6305 \text{ DL} + 4400 \text{ LL} + 5600 \text{ LL}$$

$$P_2 = 600 \text{ DL} + 900 \text{ LL}$$

$$R_1 = 140 \text{ DL} + 240 \text{ LL}$$

$$R_2 = 7.1 \text{ k DL} + 5.7 \text{ k LL} + 5.6 \text{ k SL}$$

$$R_3 = 230 \text{ DL} + 340 \text{ LL}$$

$$M = 17\%$$

$$V = 12\%$$

$$\Delta_T = L/999 \quad 0.10 \text{ in}$$



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www.cgeengineering.com

Description Gravity Design

By CCC

Date 1/12/23

Checked

Date

Scale

Sheet No.

Project Rose Hill Mukilteo

Job No.

22332.10

1.18



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PROJECT

Jan. 26, 2023 16:58

BM6

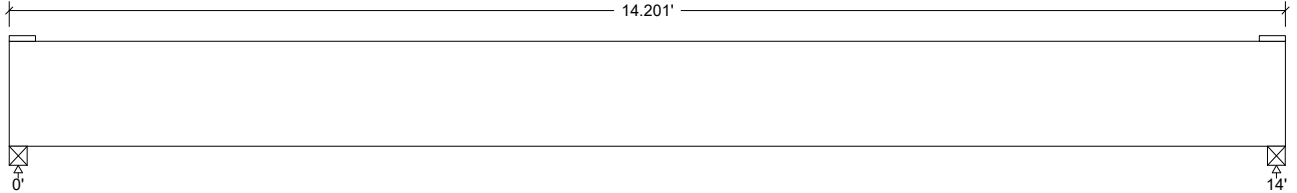
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			680.0	plf
LL	Live	Full UDL			585.0	plf
SL	Snow	Full UDL			488.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	4828		4828
Live	4154		4154
Snow	3465		3465
Factored:			
Total	10542		10542
Bearing:			
Capacity			
Beam	12651		12651
Support	10542		10542
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#3
Length	2.41		2.41
Min req'd	2.41**		2.41**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 7"x14"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 14.19'; Clear span: 13.813'; Volume = 9.7 cu.ft.
Lateral support: top = at supports, bottom = at supports;
This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	$f_v = 130$	$F_v' = 334$	psi	$f_v/F_v' = 0.39$
Bending(+)	$f_b = 1909$	$F_b' = 3278$	psi	$f_b/F_b' = 0.58$
Live Defl'n	$0.20 = L/850$	$0.47 = L/360$	in	0.42
Total Defl'n	$0.45 = L/375$	$0.70 = L/240$	in	0.64

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
F_v'	290	1.15	-	1.00	-	-	-	-	1.00	-	3
$F_b' +$	2900	1.15	-	1.00	0.986	0.983	-	1.00	1.00	-	3
F_{cp}'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	3
E_{miny}'	1.14 million	-	-	1.00	-	-	-	-	1.00	-	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #3 = $D + 0.75(L + S)$
Bending(+): LC #3 = $D + 0.75(L + S)$
Deflection: LC #3 = $D + 0.75(L + S)$ (live)
LC #3 = $D + 0.75(L + S)$ (total)
Bearing : Support 1 - LC #3 = $D + 0.75(L + S)$
Support 2 - LC #3 = $D + 0.75(L + S)$
D=dead L=live S=snow

All LC's are listed in the Analysis output
Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

$V_{max} = 10393$, $V_{design} = 8512$ (NDS 3.4.3.1(a)) lbs; $M(+)$ = 36376 lbs-ft
 $EI = 3521.47e06$ lb-in² Apparent E approximates the effect of shear deflection.
"Live" deflection is due to all non-dead loads (live, wind, snow...)
Total deflection = 1.50 permanent + "live"
Lateral stability(+): $L_u = 14.00'$ $L_e = 26.31'$ $RB = 9.5$

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.



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Jan. 26, 2023 16:59

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BM7

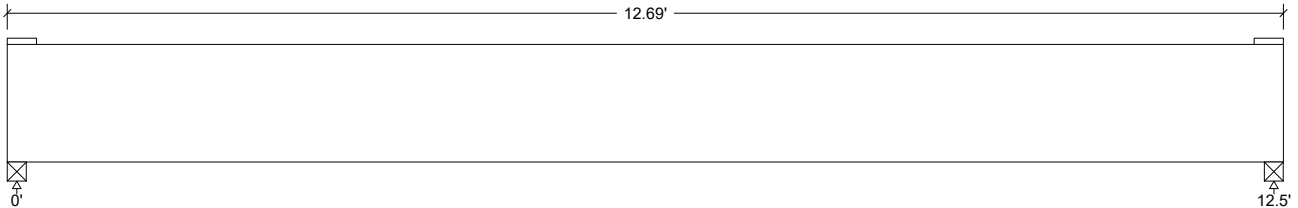
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			719.0	plf
LL	Live	Full UDL			648.0	plf
SL	Snow	Full UDL			488.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	4562		4562
Live	4112		4112
Snow	3096		3096
Factored:			
Total	9968		9968
Bearing:			
Capacity			
Beam	11962		11962
Support	9968		9968
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#3
Length	2.28		2.28
Min req'd	2.28**		2.28**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 7"x14"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 12.69'; Clear span: 12.313'; Volume = 8.6 cu.ft.
Lateral support: top = at supports, bottom = at supports;
This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	$f_v = 104$	$F_v' = 290$	psi	$f_v/F_v' = 0.36$
Bending(+)	$f_b = 1401$	$F_b' = 2851$	psi	$f_b/F_b' = 0.49$
Live Defl'n	$0.13 < L/999$	$0.42 = L/360$	in	0.32
Total Defl'n	$0.30 = L/498$	$0.62 = L/240$	in	0.48

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
F_v'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
$F_b' +$	2900	1.00	-	1.00	0.990	0.983	-	1.00	1.00	-	2
F_{cp}'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	1.00	-	-	-	-	-	1.00	-	3
E_{min}'	1.14 million	-	1.00	-	-	-	-	-	1.00	-	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D + L
Bending(+): LC #2 = D + L
Deflection: LC #3 = D + 0.75(L + S) (live)
LC #3 = D + 0.75(L + S) (total)
Bearing : Support 1 - LC #3 = D + 0.75(L + S)
Support 2 - LC #3 = D + 0.75(L + S)

D=dead L=live S=snow

All LC's are listed in the Analysis output

Load Patterns: s=S/2, X=L+S or L+Lr, _=no pattern load in this span

Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

$V_{max} = 8544$, $V_{design} = 6819$ (NDS 3.4.3.1(a)) lbs; $M(+) = 26699$ lbs-ft
 $EI = 3521.47e06$ lb-in² Apparent E approximates the effect of shear deflection.
"Live" deflection is due to all non-dead loads (live, wind, snow...)
Total deflection = 1.50 permanent + "live"
Lateral stability(+): $L_u = 12.50'$ $L_e = 23.88'$ $RB = 9.0$

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.



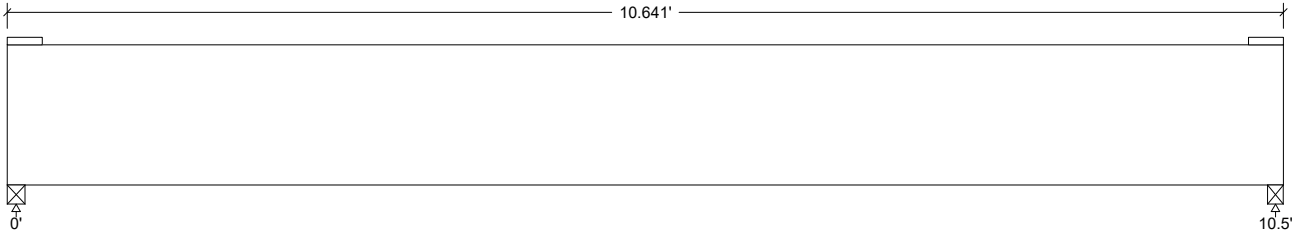
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL1	Dead	Partial UDL		0.07 3.82	727.0 727.0	plf
LL1	Live	Partial UDL		0.07 3.82	660.0 660.0	plf
SL	Snow	Full UDL			488.0	plf
DL2	Dead	Trapezoidal		3.82 10.57	642.0 551.0	plf
LL2	Live	Trapezoidal		3.82 10.57	525.0 380.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	3567		3186
Live	3067		2462
Snow	2598		2594
Factored:			
Total	7816		6979
Bearing:			
Capacity			
Beam	9379		8374
Support	7816		6979
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#3
Length	1.79		1.60
Min req'd	1.79**		1.60**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 7"x14"

Supports: All - Timber-soft Beam, D.Fir-L No.2
 Total length: 10.63'; Clear span: 10.375'; Volume = 7.2 cu.ft.
 Lateral support: top = at supports, bottom = at supports;
This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	$f_v = 89$	$F_v' = 334$	psi	$f_v/F_v' = 0.27$
Bending(+)	$f_b = 1010$	$F_b' = 3278$	psi	$f_b/F_b' = 0.31$
Live Defl'n	$0.06 < L/999$	$0.35 = L/360$	in	0.17
Total Defl'n	$0.13 = L/943$	$0.52 = L/240$	in	0.25

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
F_v'	290	1.15	-	1.00	-	-	-	-	1.00	-	3
$F_b' +$	2900	1.15	-	1.00	0.990	0.983	-	1.00	1.00	-	3
F_{cp}'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	1.00	-	-	-	-	-	1.00	-	3
E_{min}'	1.14 million	-	1.00	-	-	-	-	-	1.00	-	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #3 = $D + 0.75(L + S)$
 Bending(+): LC #3 = $D + 0.75(L + S)$
 Deflection: LC #3 = $D + 0.75(L + S)$ (live)
 LC #3 = $D + 0.75(L + S)$ (total)
 Bearing : Support 1 - LC #3 = $D + 0.75(L + S)$
 Support 2 - LC #3 = $D + 0.75(L + S)$

D=dead L=live S=snow

All LC's are listed in the Analysis output

Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

$V_{max} = 7788$, $V_{design} = 5846$ (NDS 3.4.3.1(a)) lbs; $M(+)$ = 19252 lbs-ft
 $EI = 3521.47e06$ lb-in² Apparent E approximates the effect of shear deflection.
 "Live" deflection is due to all non-dead loads (live, wind, snow...)
 Total deflection = 1.50 permanent + "live"
 Lateral stability(+): $L_u = 10.50'$ $L_e = 20.63'$ $RB = 8.4$

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Wood Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: **BM9**

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design

Load Combination : IBC 2021

Wood Species : iLevel Truss Joist

Wood Grade : Parallam PSL 2.0E

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

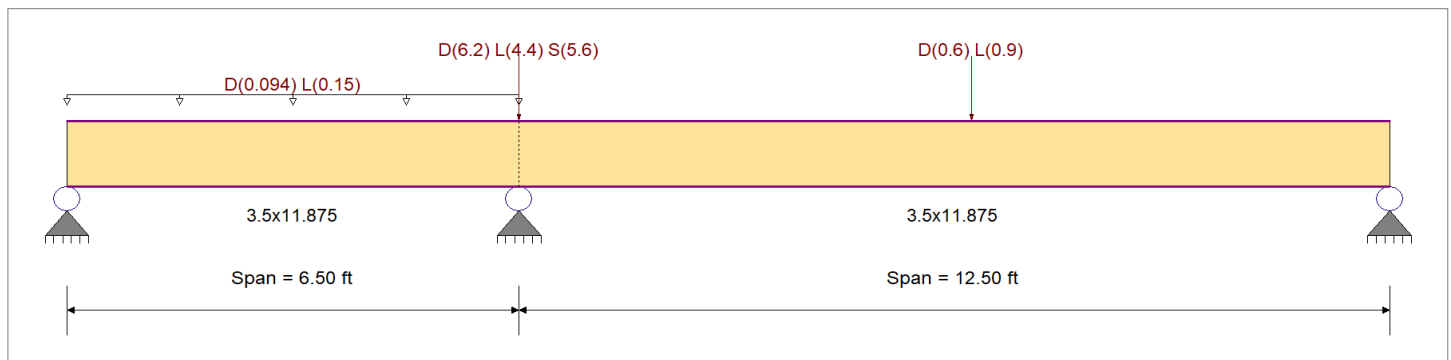
Fb + 2900 psi
 Fb - 2900 psi
 Fc - Prll 2900 psi
 Fc - Perp 750 psi
 Fv 290 psi
 Ft 2025 psi

E : Modulus of Elasticity

Ebend- xx 2000 ksi

Eminbend - xx 1016.535 ksi

Density 45.07 pcf



Applied Loads

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

Beam self weight NOT internally calculated and added

Load for Span Number 1

Uniform Load : D = 0.0940, L = 0.150 , Tributary Width = 1.0 ft

Point Load : D = 6.20, L = 4.40, S = 5.60 k @ 6.50 ft

Load for Span Number 2

Point Load : D = 0.60, L = 0.90 k @ 6.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.169	1	Maximum Shear Stress Ratio	=	0.121	1
Section used for this span		3.5x11.875		Section used for this span		3.5x11.875	
fb: Actual	=	491.55psi		fv: Actual	=	35.08 psi	
F'b	=	2,903.37psi		F'v	=	290.00 psi	
Load Combination		+D+L		Load Combination		+D+L	
Location of maximum on span	=	6.494ft		Location of maximum on span	=	5.520 ft	
Span # where maximum occurs	=	Span # 2		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.037 in	Ratio =	4057	>=360	Span: 2 : L Only		
Max Upward Transient Deflection	-0.002 in	Ratio =	33841	>=360	Span: 1 : L Only		
Max Downward Total Deflection	0.062 in	Ratio =	2427	>=240	Span: 2 : +D+L		
Max Upward Total Deflection	-0.004 in	Ratio =	19596	>=240	Span: 1 : +D+L		

Vertical Reactions

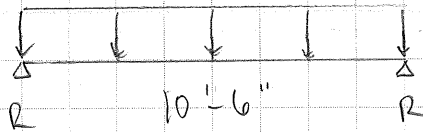
Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.375	15.523	0.562
Max Upward from Load Combinations	0.375	15.523	0.562
Max Upward from Load Cases	0.235	7.046	0.337
D Only	0.139	7.046	0.226
+D+L	0.375	12.749	0.562
+D+S	0.139	12.646	0.226
+D+0.750L	0.316	11.323	0.478
+D+0.750L+0.750S	0.316	15.523	0.478
+0.60D	0.083	4.228	0.135
L Only	0.235	5.703	0.337

Floor Beams Cont.

BM10:



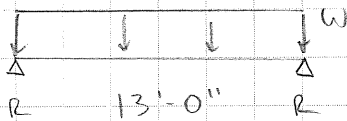
$$M = 46\% \quad V = 34\% \\ \Delta_T = 0.23" \quad L/544$$

$$W = 596 \text{ DL} + 390 \text{ LL} + 238 \text{ SL}$$

$$R = 3.2 \text{ K DL} + 2.1 \text{ K LL} + 1.3 \text{ SL}$$

Use 5 1/4" x 11 7/8" PSL

BM11:

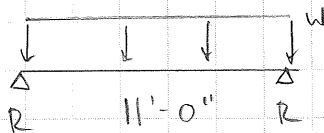


$$W = 106 \text{ DL} + 255 \text{ LL}$$

$$R = 689 \text{ DL} + 1658 \text{ LL}$$

Use 3 1/2" x 9 1/2" PSL * From Table

BM12:

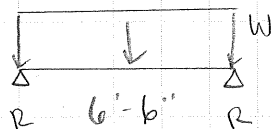


$$W = 521 \text{ DL} + 1185 \text{ LL}$$

$$R = 2.9 \text{ K DL} + 6.5 \text{ K}$$

Use 3 1/2" x 11 7/8" PSL * From Table

BM13:



$$W = 639 \text{ DL} + 520 \text{ LL} + 491 \text{ SL}$$

$$R = 2.1 \text{ K DL} + 1.7 \text{ K LL} + 1.6 \text{ K SL}$$

Use 3 1/2" x 11 7/8" PSL * From Table



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Description	By CCC	Date 12/14/22
	Checked	Date
	Scale	Sheet No.
Project Rose Hill Mukiteo	Job No. 22332.10	1.23



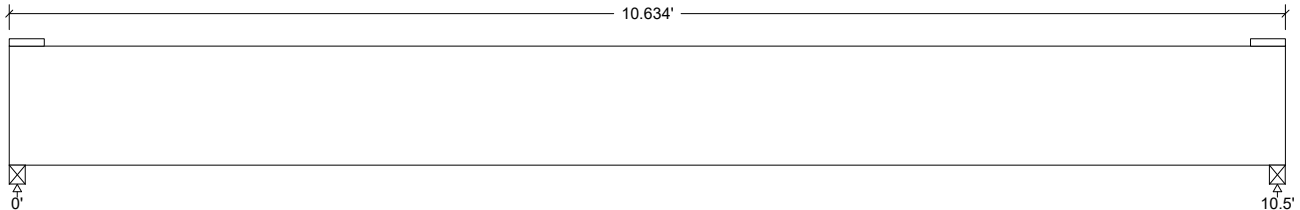
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			596.0	plf
LL	Live	Full UDL			390.0	plf
SL	Snow	Full UDL			236.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	3169		3169
Live	2074		2074
Snow	1255		1255
Factored:			
Total	5665		5665
Bearing:			
Capacity			
Beam	6345		6345
Support	5665		5665
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	1.61		1.61
Min req'd	1.61**		1.61**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.07		1.07
Fcp sup	625		625

**Minimum bearing length governed by the required width of the supporting member.

PSL, PSL, 2.2E, 5-1/4"x11-7/8"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 10.63'; Clear span: 10.375'; Volume = 4.6 cu.ft.

Lateral support: top = at supports, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 99	Fv' = 290	psi	fv/Fv' = 0.34
Bending(+)	fb = 1322	Fb' = 2863	psi	fb/Fb' = 0.46
Live Defl'n	0.08 = < L/999	0.35 = L/360	in	0.23
Total Defl'n	0.23 = L/544	0.52 = L/240	in	0.44

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cft	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	0.986	1.001	-	1.00	1.00	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	1.00	-	-	-	-	-	1.00	-	3
Eminy'	1.14 million	-	1.00	-	-	-	-	-	1.00	-	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D + L
 Bending(+): LC #2 = D + L
 Deflection: LC #3 = D + 0.75(L + S) (live)
 LC #3 = D + 0.75(L + S) (total)
 Bearing : Support 1 - LC #3 = D + 0.75(L + S)
 Support 2 - LC #3 = D + 0.75(L + S)

D=dead L=live S=snow

All LC's are listed in the Analysis output

Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

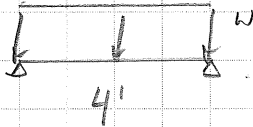
V max = 5176, V design = 4135 (NDS 3.4.3.1(a)) lbs; M(+) = 13588 lbs-ft
 EI = 1611.76e06 lb-in² Apparent E approximates the effect of shear deflection.
 "Live" deflection is due to all non-dead loads (live, wind, snow...)
 Total deflection = 1.50 permanent + "live"
 Lateral stability(+): Lu = 10.50' Le = 20.06' RB = 10.2

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
- SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
- SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.

Header Design:

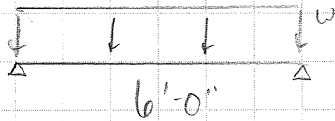
Upper Floor @ Roof Deck:



$$W = 370 \text{ DL} + 925 \text{ LL}$$

Use 4x8 HF #2 * Per Beam Table

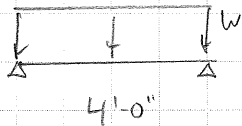
Typical Loading @ Roof



$$W = 146 \text{ DL} + 226 \text{ SL}$$

Use 4x8 HF #2 * Per Beam Table

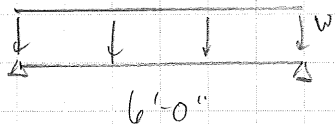
Main Floor Typ Header 4'-0"



$$W = 474 \text{ DL} + 365 \text{ LL} + 228 \text{ SL}$$

Use 4x8 HF #2 * Per Woodworks

Main Floor Typ Header 6'-0"



$$W = 474 \text{ DL} + 365 \text{ LL} + 228 \text{ SL}$$

Use 4x10 HF #2 * Per Beam Table



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Description Header Design

Project Rose Hill Mukilteo

By CCC

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22332.10

Date 1/27/23

Date

Sheet No.

1.25



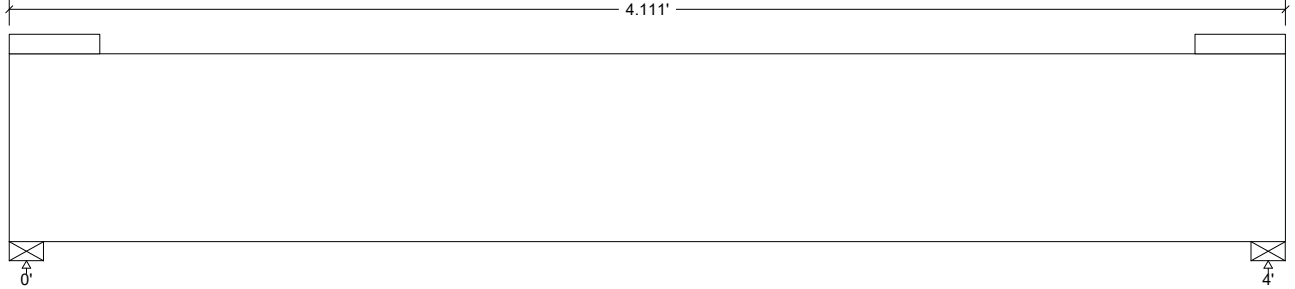
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full UDL			474.0	plf
SL	Snow	Full UDL			228.0	plf
LL	Live	Full UDL			365.0	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	974		974
Live	750		750
Snow	469		469
Factored:			
Total	1889		1889
Bearing:			
Capacity			
Beam	1889		1889
Support	3227		3227
Des ratio			
Beam	1.00		1.00
Support	0.59		0.59
Load comb	#3		#3
Length	1.33		1.33
Min req'd	1.33		1.33
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

Lumber-soft, Hem-Fir, No.2, 4x8 (3-1/2"x7-1/4")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 4.13'; Clear span: 3.875'; Volume = 0.7 cu.ft.

Lateral support: top = at supports, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	$f_v = 66$	$F_v' = 150$	psi	$f_v/F_v' = 0.44$
Bending(+)	$f_b = 657$	$F_b' = 1098$	psi	$f_b/F_b' = 0.60$
Live Defl'n	$0.02 < L/999$	$0.13 = L/360$	in	0.13
Total Defl'n	$0.05 < L/999$	$0.20 = L/240$	in	0.23

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrr	Ci	LC#
F_v'	150	1.00	1.00	1.00	-	-	-	-	1.00	1.00	2
F_b'	850	1.00	1.00	1.00	0.994	1.300	-	1.00	1.00	1.00	2
F_{cp}'	405	-	1.00	1.00	-	-	-	-	1.00	1.00	-
E'	1.3 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	3
E_{min}'	0.47 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	3

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D + L
 Bending(+): LC #2 = D + L
 Deflection: LC #3 = D + 0.75(L + S) (live)
 LC #3 = D + 0.75(L + S) (total)
 Bearing : Support 1 - LC #3 = D + 0.75(L + S)
 Support 2 - LC #3 = D + 0.75(L + S)

D=dead L=live S=snow

All LC's are listed in the Analysis output

Load Patterns: s=S/2, X=L+S or L+Lr, _=no pattern load in this span

Load combinations: ASD Basic from ASCE 7-16 2.4

CALCULATIONS:

V max = 1678, V design = 1125 (NDS 3.4.3.1(a)) lbs; M(+) = 1678 lbs-ft

EI = 144.49e06 lb-in²

"Live" deflection is due to all non-dead loads (live, wind, snow...)

Total deflection = 1.50 permanent + "live"

Lateral stability(+): Lu = 4.00' Le = 8.25' RB = 7.6

Design Notes:

- Analysis and design are in accordance with the ICC International Building Code (IBC 2021) and the National Design Specification (NDS 2018), using Allowable Stress Design (ASD). Design values are from the NDS Supplement.
- Please verify that the default deflection limits are appropriate for your application.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

SECTION NO. 2

Rose Hill Mukilteo
3rd & Park Ave
Mukilteo, WA 98275

SECTION TITLE: Wood and Steel Lateral Design

SECTION INDEX

DESCRIPTION	PAGE
Seismic and Wind Properties and Site Data	2.01-2.07
Wind and Seismic Forces	2.08-2.11
Lateral Key Plans	2.12-2.14
Shear Wall Design	2.15-2.20
Open Front Diaphragm Analysis	2.21
Perforated Shear Wall Design	2.22-2.23
Diaphragm Strapping Design	2.24-2.25
Moment Frame Design	2.26-2.33
Moment Frame Anchorage	2.33-2.35

Seismic Analysis

Design Per 2018 IBC & ASCE 7-16

Seismic Coefficients

Soil Site Class	D (assumed / default)
Occupancy Category	II (non-essential facility)
Seismic Design Category	D

$S_S =$	1.404	Lat. =	47.947
$S_1 =$	0.500	Long. =	-122.302

$S_{MS} = F_a S_S =$	1.685	$F_a =$	1.200
$S_{M1} = F_v S_1 =$	0.900	$F_v =$	1.800

$S_{DS} = (2/3)S_{MS} =$	1.123
$S_{D1} = (2/3)S_{M1} =$	0.600

$T_a = C_t h_n^x =$	0.22	$C_t =$	0.02
		$h_n =$	24
		$x =$	0.75

R Factor =	6.5	(Wood shear walls / steel moment frame)
I_E Factor =	1.0	

From Computer Program:

Short & 1-Sec Period Mapped
Acceleration Parameters (MCE)

ASCE 7-16 (Eq. 11.4-1)

ASCE 7-16 (Eq. 11.4-2)

ASCE 7-16 (Eq. 11.4-3)

ASCE 7-16 (Eq. 11.4-4)

ASCE 7-16 (Table 12.8-2)

ASCE 7-16 (Table 12.8-2)

ASCE 7-16 (Table 12.2-1)

ASCE 7-16 (Table 1.5-2)

Seismic Base Shear

$V = 0.044 S_{DS} I =$	0.049 W	(Minimum Force)	ASCE 7-16 (Eq. 12.8-5)
------------------------	---------	-----------------	------------------------

$V = (S_{DS} I W) / R =$	0.173 W	(Governing Force)	ASCE 7-16 (Eq. 12.8-2)
--------------------------	----------------	--------------------------	------------------------

$V = (S_{D1} I W) / R T_a =$	0.426 W	(Maximum Force)	ASCE 7-16 (Eq. 12.8-3)
------------------------------	---------	-----------------	------------------------



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Description	Seismic Base Shear	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

Wind Design (ASCE 28.5 Enclosed Simple Diaphragm)**2018 IBC****ASCE 7-16**

Building Exposure Exp.= **C**
 Basic Wind Speed V= **98**
 Risk Category I_w= **II**
 Top of Roof Height (feet) h= **24**
 Mean Roof Height (feet) h_{mean}= **24**
 Building Length (feet) L= **110**
 Building Width (feet) W= **98**
 End Zone Width, a (feet) a= **9.6**

Section 1609.4

Section 26.7.3
 Per Jurisdiction
 Table 1.5-1

Figure 28.6-1

Roof Angle Angle= **0.0**
 Design Wind Pressure, p_{s3} p_{s30A}= **15.3**
 Design Wind Pressure, p_{s3} p_{s30B}= **-7.9**
 Design Wind Pressure, p_{s3} p_{s30C}= **10.1**
 Design Wind Pressure, p_{s3} p_{s30D}= **-4.7**

Figure 28.6-1

Figure 28.6-1

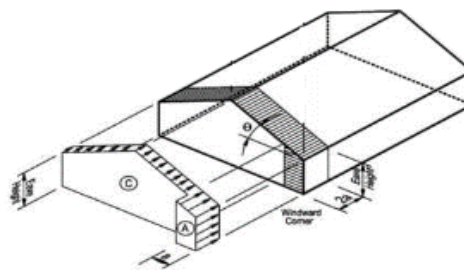
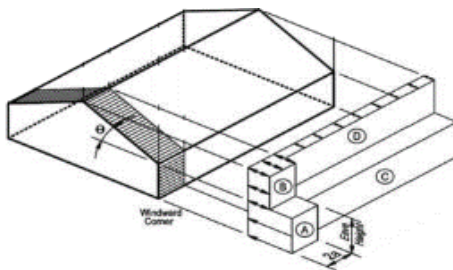
Figure 28.6-1

Figure 28.6-1

Height/Exposure Adjustm λ_{max}= **1.29**
 Topo. Effect Coeff., K_{zt} K_{zt}= **1.00**

$$V_{asd} = V_{ult} * 0.6$$

Section 1609.3.1



Design Wind Load	ULT	ASD
	$p_s = \lambda * K_{zt} * p_{s30}$	$p_s = \lambda * K_{zt} * p_{s30} * 0.6$
p _{s30A} =	19.7	11.8
p _{s30B} =	-10.2	-6.1
p _{s30C} =	13.0	7.8
p _{s30D} =	-6.1	-3.6

Minimum Wind Load	ULT	ASD
	$p_s = \lambda * K_{zt} * p_{s30}$	$p_s = \lambda * K_{zt} * p_{s30} * 0.6$
p _{s30A,C} =	16.0	9.6
p _{s30B,D} =	8.0	4.8



250 4th Ave South
 Suite 200
 Edmonds, WA 98020

Description

Wind Summary

Project

Rose Hill Apartments

By

LS

Checked

Scale

NTS

Job No.

22332.10

Date

1/25/2023

Date

Sheet No.

 This is a beta release of the new ATC Hazards by Location website. Please [contact us](#) with feedback.

 The ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

 Hazards by Location

Search Information

Address:

3rd St & Park Ave, Mukilteo, WA 98275, USA

Coordinates:

47.9467166, -122.3015942

Elevation:

76 ft

Timestamp:

2022-11-22T16:13:17.695Z

Hazard Type:

Seismic

Reference Document:

ASCE7-16

Risk Category:

II

Site Class:

D-default



Basic Parameters

Name	Value	Description
S _S	1.404	MCE _R ground motion (period=0.2s)
S ₁	0.5	MCE _R ground motion (period=1.0s)
S _{MS}	1.685	Site-modified spectral acceleration value
S _{M1}	* null	Site-modified spectral acceleration value
S _{DS}	1.124	Numeric seismic design value at 0.2s SA
S _{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▼Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F _a	1.2	Site amplification factor at 0.2s
F _v	* null	Site amplification factor at 1.0s
CR _S	0.898	Coefficient of risk (0.2s)
CR ₁	0.896	Coefficient of risk (1.0s)
PGA	0.607	MCE _G peak ground acceleration
F _{PGA}	1.2	Site amplification factor at PGA
PGA _M	0.729	Site modified peak ground acceleration

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

📄 The ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

ATC

Hazards by Location

Search Information

Address:

3rd St & Park Ave, Mukilteo, WA 98275, USA

Coordinates:

47.9467166, -122.3015942

Elevation:

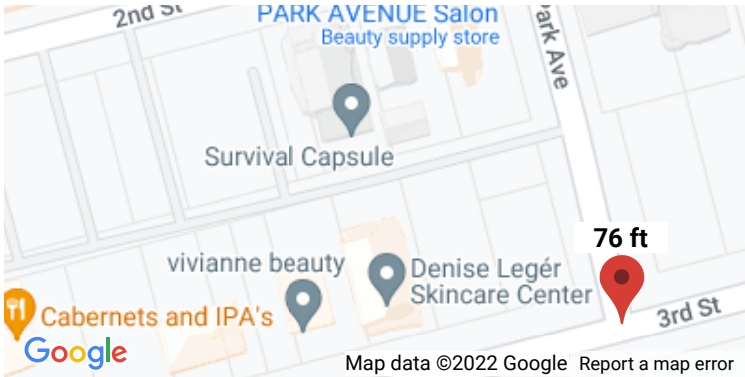
76 ft

Timestamp:

2022-11-22T16:10:14.229Z

Hazard Type:

Wind



ASCE 7-16		ASCE 7-10		ASCE 7-05	
MRI 10-Year	67 mph	MRI 10-Year	72 mph	ASCE 7-05 Wind Speed	85 mph
MRI 25-Year	73 mph	MRI 25-Year	79 mph		
MRI 50-Year	78 mph	MRI 50-Year	85 mph		
MRI 100-Year	83 mph	MRI 100-Year	91 mph		
Risk Category I	92 mph	Risk Category I	100 mph		
Risk Category II	98 mph	Risk Category II	110 mph		
Risk Category III	105 mph	Risk Category III-IV	115 mph		
Risk Category IV	109 mph				

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

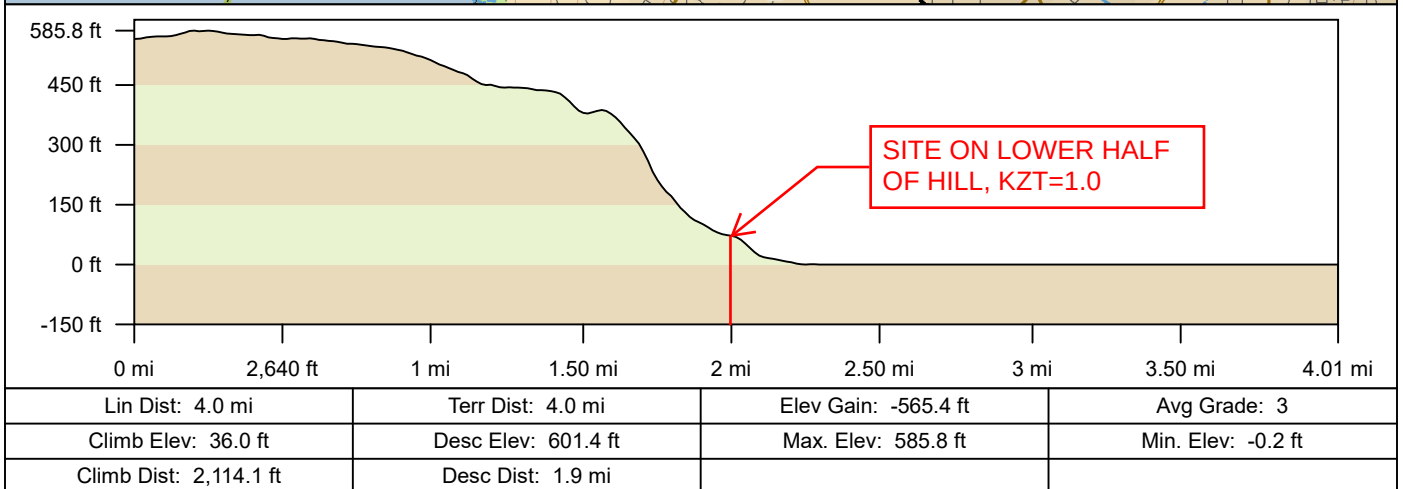
Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

Disclaimer

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

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

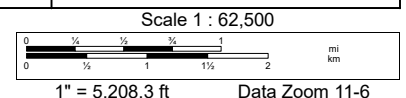
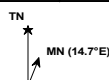
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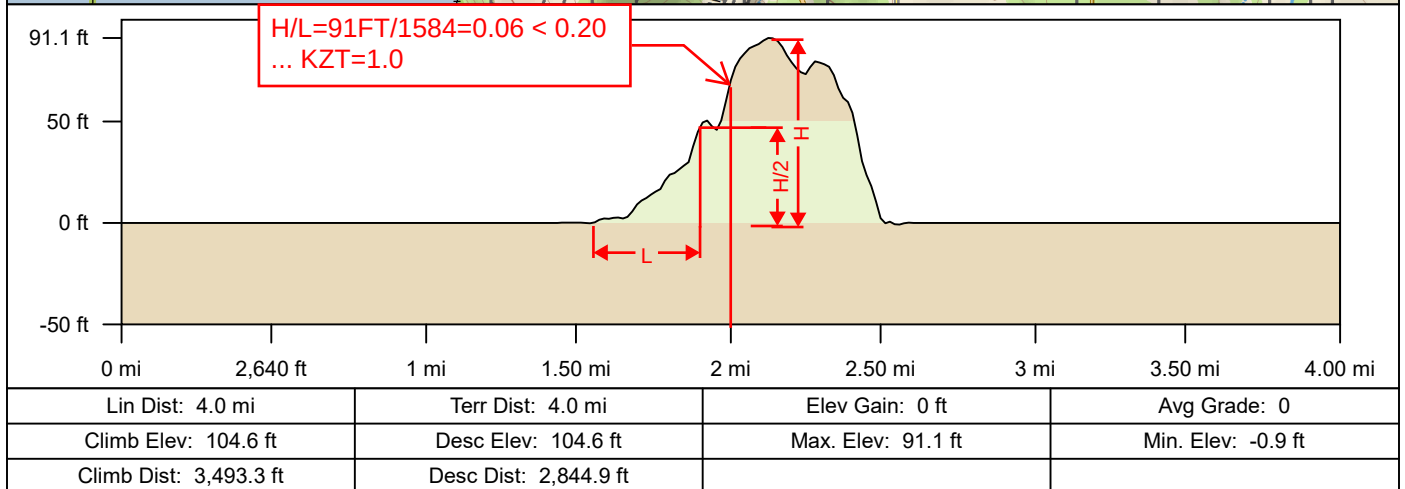
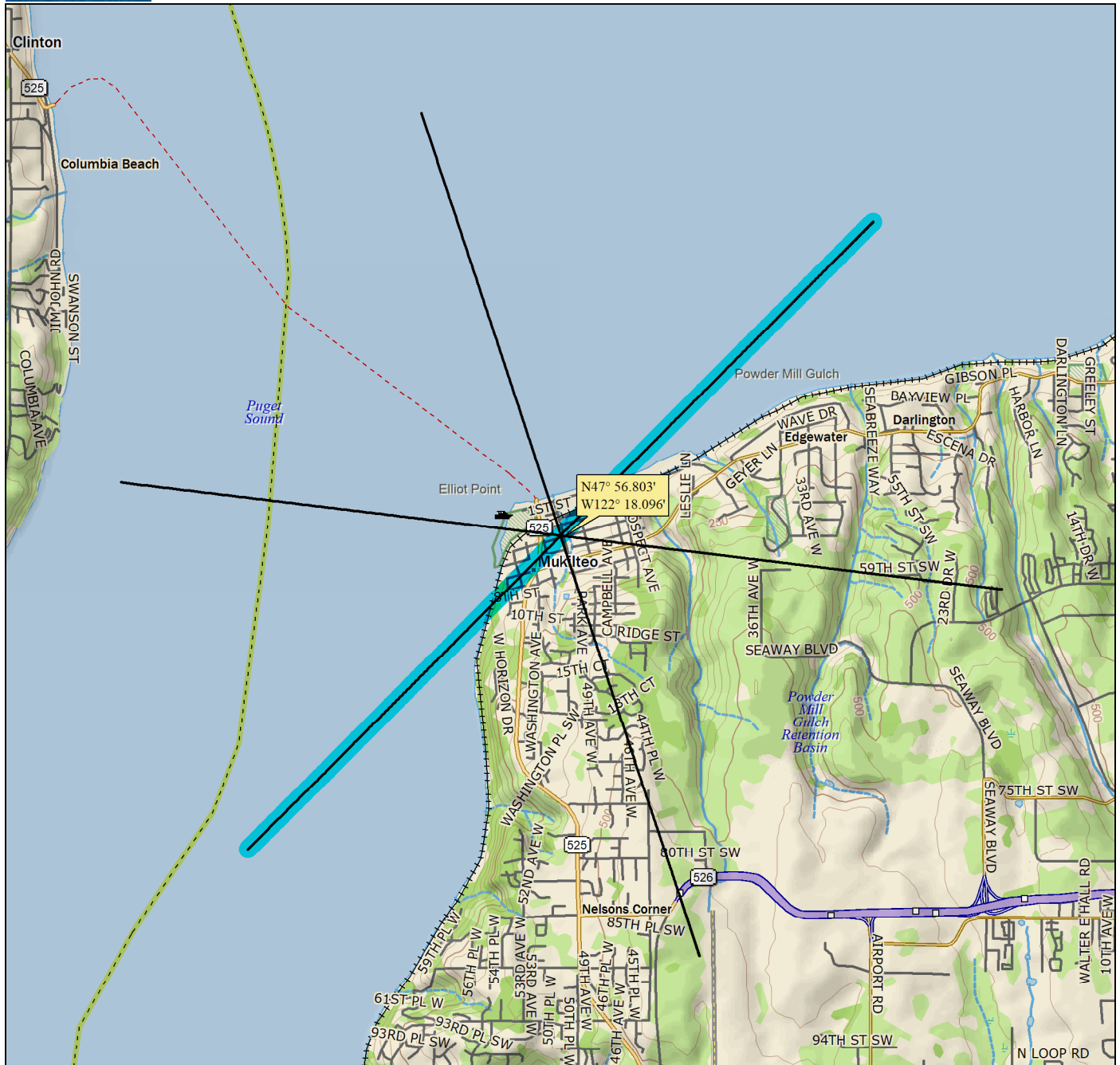


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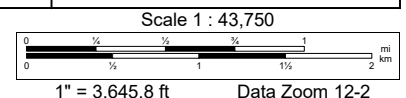
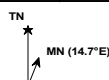


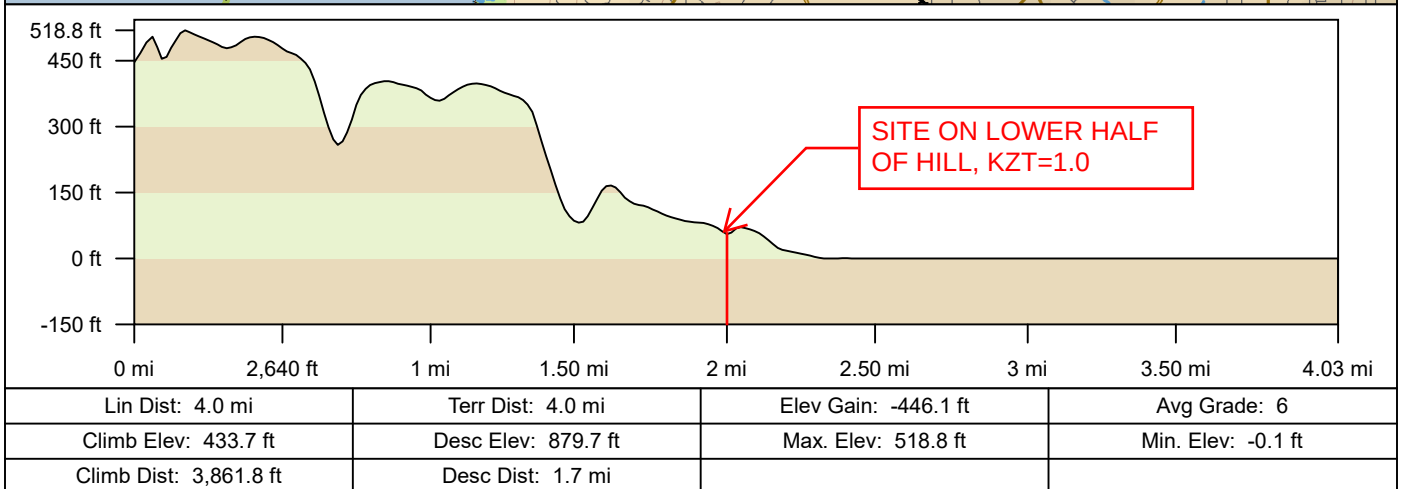
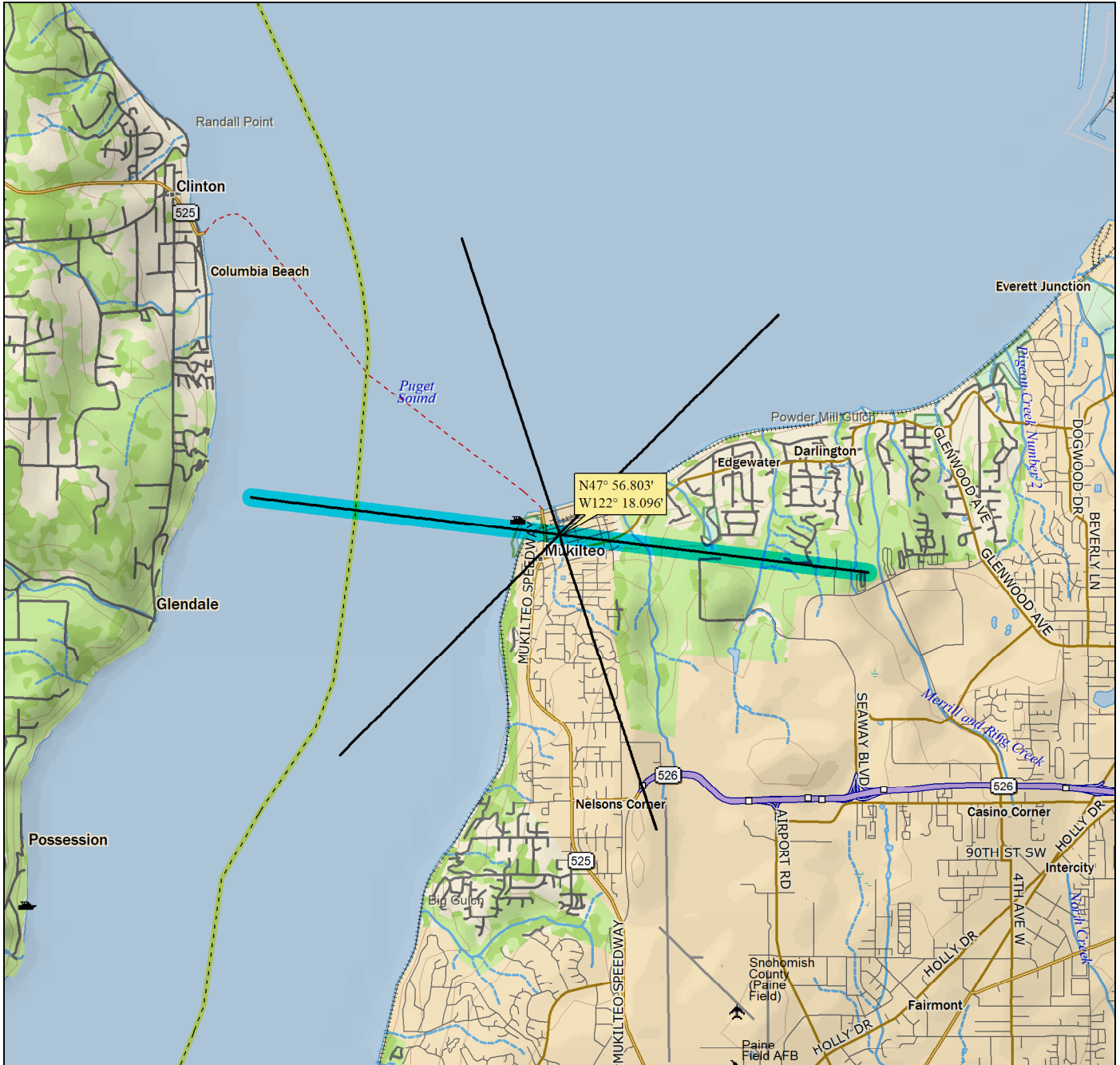


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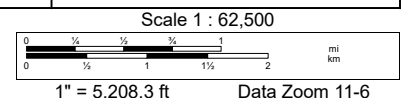
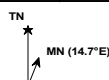




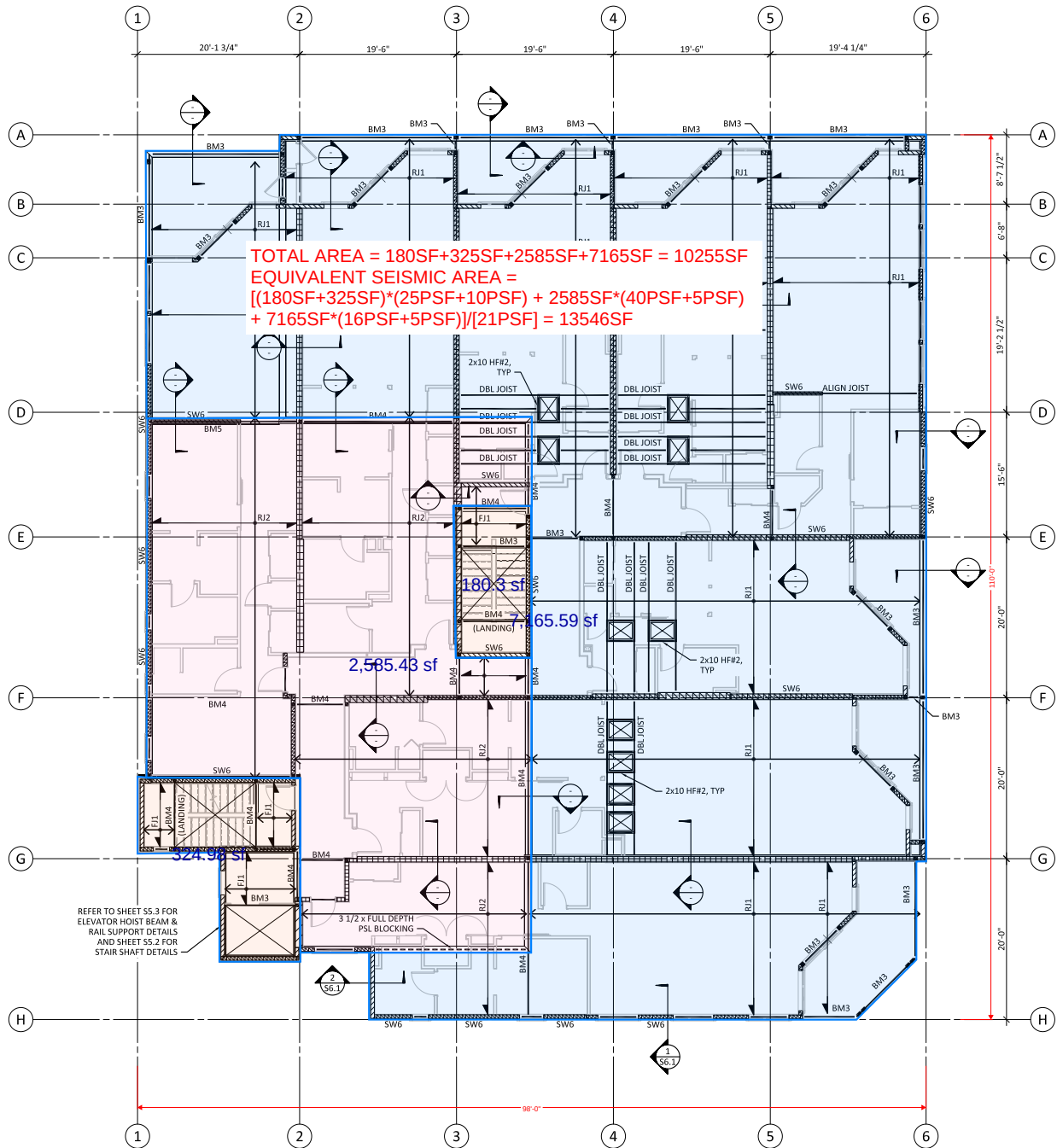
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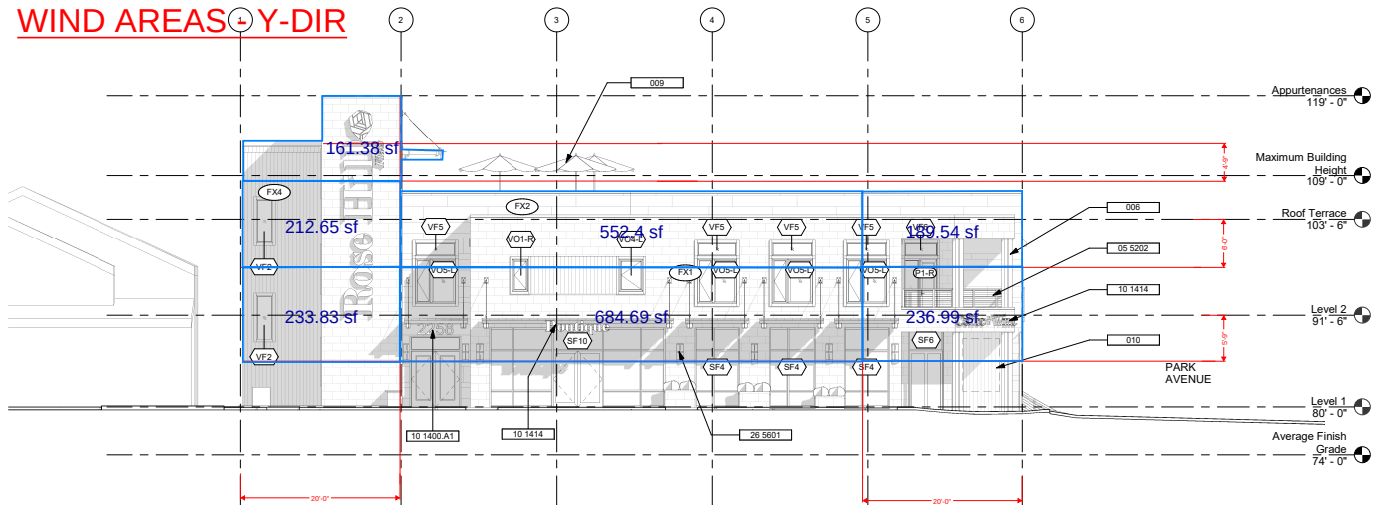


LOADING PLAN - ROOF



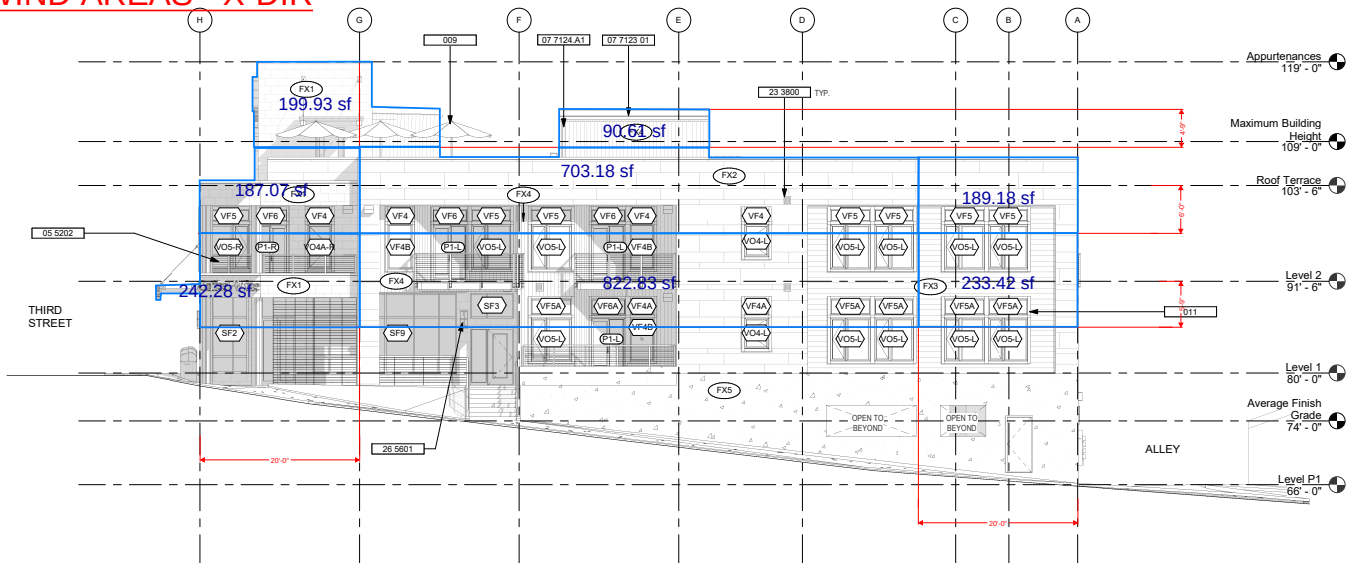
Description	Wind Areas	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

WIND AREAS - Y-DIR



C1 Elevation - South (3rd Street)

WIND AREAS - X-DIR



A1 Elevation - East (Park Avenue)



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Roof Loading	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

Seismic Forces - Vertical Distribution

Refer to ASCE 7-16 Section 12.8.3

k = 1.0

Diaphragm Level	DL (psf)	Area (ft ²)	w _{DL} (kips)	Story Elev. (h)	w _i * h _i ^k (k-ft)	w _x * h _x ^k Σw _i * h _i ^k	Shear F _x	Sum F _x
Penthouse Framing	21	540	11.3	33	374	0.03	2.7	2.7
Roof Framing	21	13545	284.4	23.5	6684	0.59	48.5	51.2
2nd Framing	35	10500	367.5	11.5	4226	0.37	30.7	81.9
	0	0	0.0	0	0	0.00	0.0	81.9
	0	0	0.0	0	0	0.00	0.0	81.9
		Σ =	663.3	-	11285	1.00	81.9	-

Base Shear (ULT) 114.6 kips

Base Shear (ASD) **81.9 kips** * note that all table forces are ASD

Seismic Forces - Vertical Distribution Including Rho

Refer to ASCE 7-16 Section 12.3.4.2

Diaphragm Level	Rho ρ	Shear F _x	Sum F _x
Penthouse Framing	1.0	2.7	2.7
Roof Framing	1.0	48.5	51.2
2nd Framing	1.0	30.7	81.9
	1.0	0.0	81.9
	1.0	0.0	81.9
		Σ =	81.9 -

Diaphragm Forces - Vertical Distribution

Refer to ASCE 7-16 Section 12.10.1.1

Diaphragm Level	w _i (kips)	Σ w _i (kips)	F _i (kips)	Σ F _i (kips)	Σ F _i * w _{px} Σ w _i	F _{px} (Min) 0.2S _{DS} Iw _{px}	F _{px} (Max) 0.4S _{DS} Iw _{px}	F _{px} Govern
Penthouse Framing	11.3	11.3	2.7	2.7	2.7	1.8	3.6	2.7
Roof Framing	284.4	295.8	48.5	51.2	49.2	45.6	91.3	49.2
2nd Framing	367.5	663.3	30.7	81.9	45.4	59.0	117.9	59.0
	0.0	663.3	0.0	81.9	0.0	0.0	0.0	0.0
	0.0	663.3	0.0	81.9	0.0	0.0	0.0	0.0



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Seismic & Diaphragm Force Distribution	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

Wind Forces

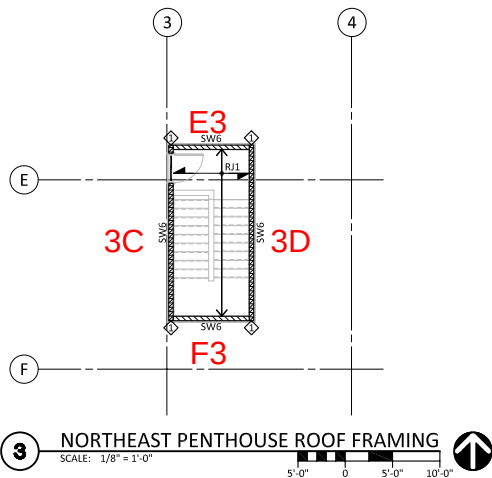
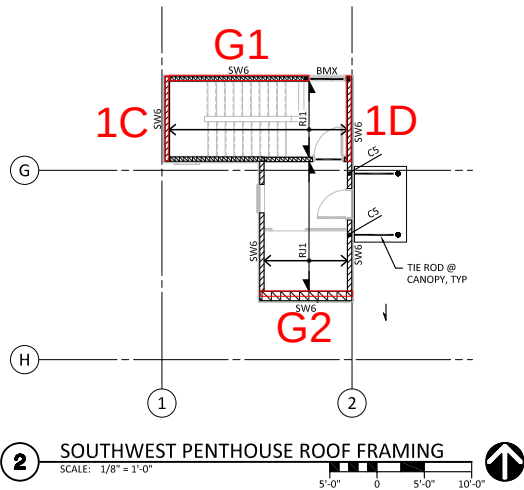
					X-Dir	Y-Dir
Diaphragm	Story Height	Elevation	B	L	p,asd	p,asd
Level					(ft)	(ft)
PH Framing	9.5	33	110	98	3.4	2.4
Roof Framing	12	23.5	110	98	11.2	10.1
2nd Framing	11.5	11.5	110	98	13.5	12.1
	0	0			0.0	0.0
	0	0			0.0	0.0

ROOF TYPE	FLAT
-----------	------

STORY	TYPE	X-DIR				
		A,A (SF)	A,B (SF)	A,C (SF)	A,D (SF)	FX (LBS)
PH Framing		290	0	0	0	3425
Roof Framing		376	0	703	0	11190
2nd Framing		475	0	823	0	13511
		0	0	0	0	0
		0	0	0	0	0

STORY	TYPE	Y-DIR				
		A,A (SF)	A,B (SF)	A,C (SF)	A,D (SF)	FY (LBS)
PH Framing		200	0	0	0	2362
Roof Framing		403	0	552	0	10059
2nd Framing		471	0	685	0	12139
		0	0	0	0	0
		0	0	0	0	0

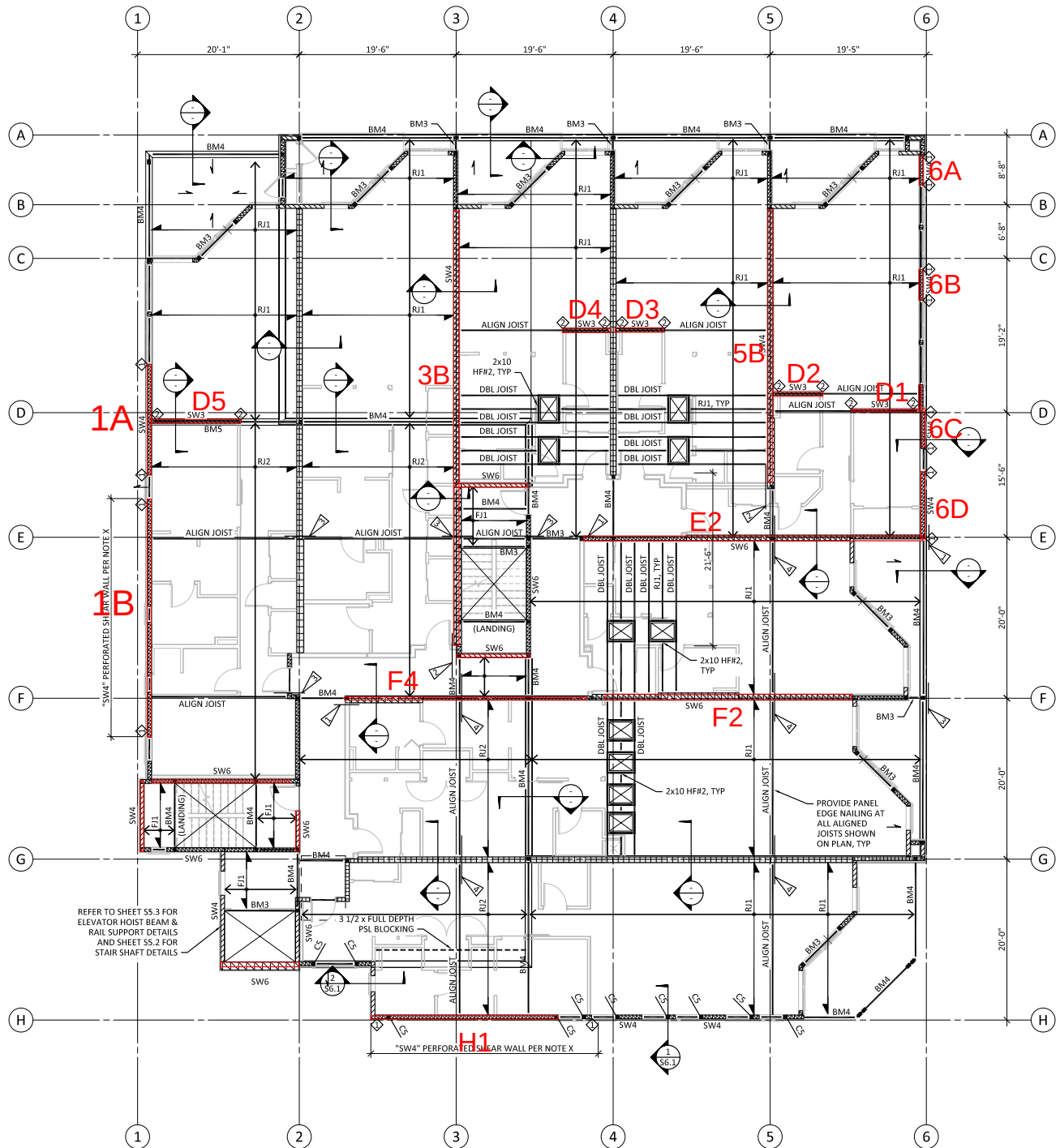
 ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Wind Design	By	LS	Date	01/25/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.			
				22332.10		



LATERAL KEY PLAN - PENTHOUSE

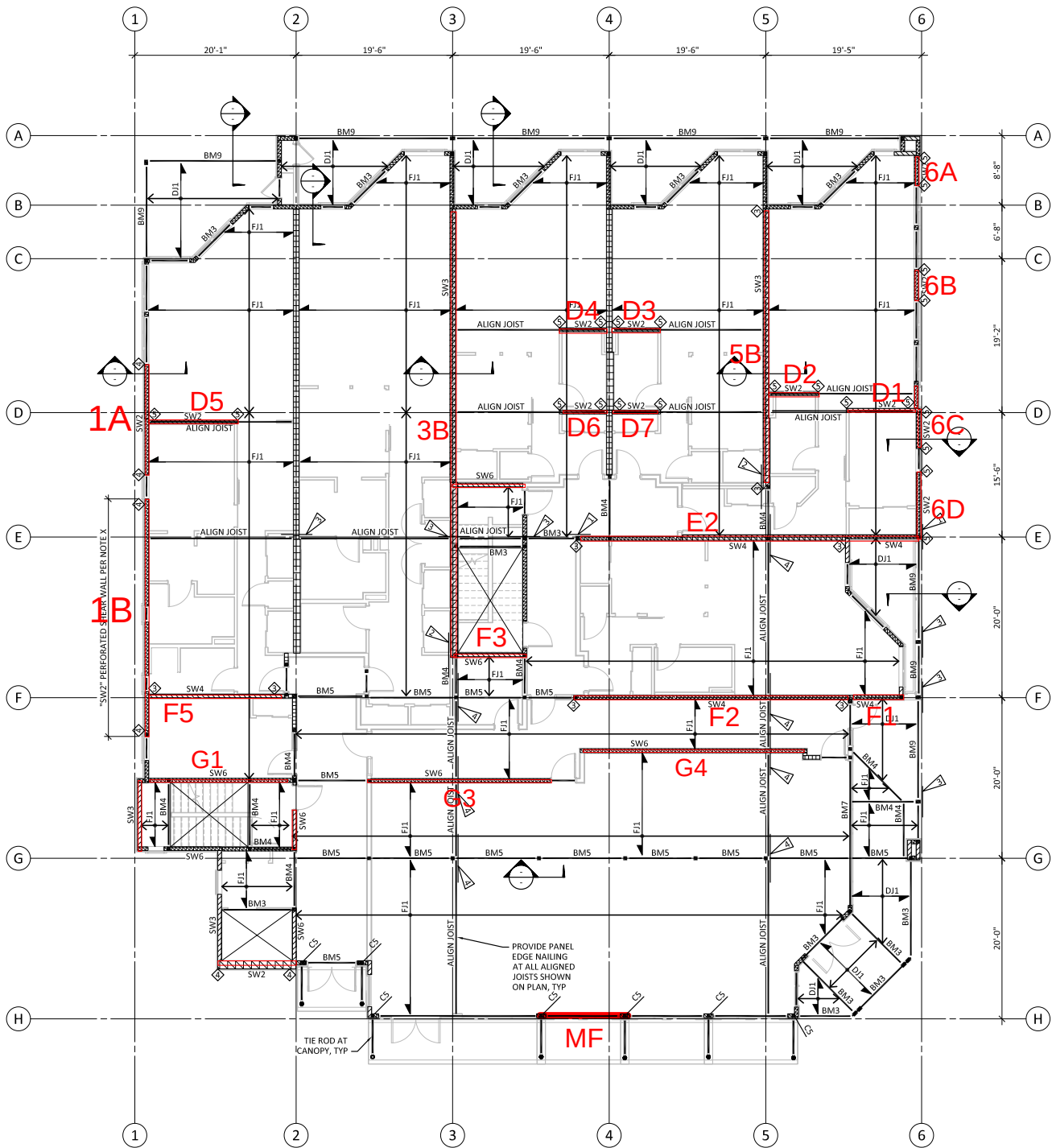
 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Lateral Key Plan	By	LS	Date	01/25/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.	22332.10		

LATERAL KEY PLAN - ROOF



Description	Lateral Key Plan	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
	Project	Job No.	22332.10		
Rose Hill Apartments					

LATERAL KEY PLAN - SECOND



250 4th Ave. South
Suite 200
Edmonds, WA 98020

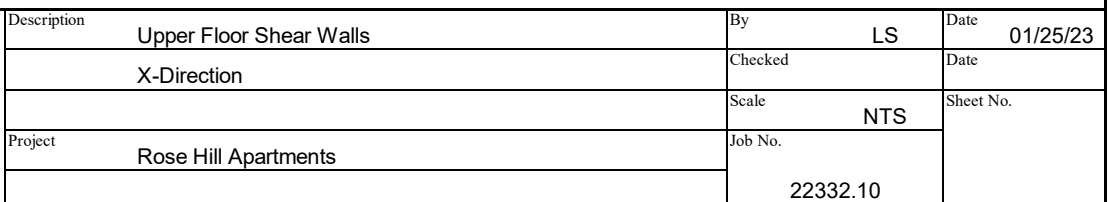
Description	Lateral Key Plan	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.			
		22332.10			

X - Direction Walls

Wx =	2714.8 PLF	seismic
Wx =	3425.3 PLF	wind

Σ	1.0
----------------------------	------------

- Input Cell
- Input Cell w/ Formula



Upper Floor Shear Walls - Walls Below the Roof Framing

Y - Direction Walls

Fy (EQ) = 2.7 kips (Story Shear)
 Fy (wind) = 2.4 kips (Story Shear)

Story HT = 9.875
 Wall HT = 9
 Max h/w = 3.5
 S_{DS} = 1.12

Wy = 2715 PLF seismic
 Wy = 2362 PLF wind

									EQ	Wind	EQ		Wind		Governing				EQ	Wind		
Wall Line	Wall Mark	SW Length	Trib Width	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	Gross Uplift	Gross Uplift	(0.6-0.14S _{DS})DL	0.6 * DL	Net Uplift	Hold-down		Line Load	Line Load	DL Trib				
1	C D	9 9	0.5	1.0 1.0	75 75	47 47	SW6 SW6	-0.5	0.8 0.8	0.7 0.7	0.0	0.0	0.0	0.0	0.6 0.6	0.6 0.6	MST37 MST37	MST37 MST37	1.4	1.2	0.0	
			-					-			-	-	-	-					-	-	-	-
			-					0.2			0.2	0.2	0.2	-					-	-	-	-
			-					0.2			0.2	0.2	0.2	-					-	-	-	-
			-					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	-					-	-	-	-
3	C D	18.5 18.5	0.5	1.0 1.0	37 37	23 23	SW6 SW6	-0.5	0.4 0.4	0.3 0.3	0.0	0.0	0.0	0.0	0.0 0.0	0.0 0.0	None None	None None	1.4	1.2	0.0	
			-					-			-	-	-	-					-	-	-	-
			-					0.7			0.7	0.9	0.9	-					-	-	-	4.5
			-					0.7			0.7	0.9	0.9	-					-	-	-	4.5
			-					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
4			-					-0.5			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	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
5			-					-0.5			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	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
6			-					-0.5			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	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
			-					0.0			0.0	0.0	0.0	-					-	-	-	0.0
Σ			1.0													2.7	2.4					

Input Cell
 Input Cell w/ Formula



250 4th Ave. South
 Suite 200
 Edmonds, WA 98020

Description	Upper Floor Shear Walls	By LS	Date 01/25/23
	Y-Direction		Date
		Scale NTS	Sheet No.
	Project Rose Hill Apartments	Job No. 22332.10	

Main Floor Shear Walls - Walls Below the Upper Floor Framing

X - Direction Walls

Fx (EQ) = 48.5 kips (Story Shear)
Fx (wind) = 11.2 kips (Story Shear)

Story HT = 10.92
Wall HT = 9.92
Max h/w = 3.5
S_{ES} = 1.12

Wx = 441 PLF seismic
Wx = 102 PLF wind

Wall Line	Wall Mark	SW Length	Trib Width	Line Load		EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ		Wind		Net Uplift		Governing		Hold-down		EQ Line Load	Wind Line Load	DL Trib	
				From Above									0.6-0.145 _{2w} DL		0.6 * DL		From Above		Net Uplift		Hold-down					
				EQ	Wind								End i	End j	End i	End j	End i	End j	End i	End j	End i	End j				End i
D	1	8.33	42.25	0.0	0.0	1.0	499	82	SW3	7.8	5.8	1.3	0.2	0.2	0.2	0.2	0.0	0.0	5.6	5.6	MST60	MST60	18.6	4.3	0.0	
	2	6.17	-	-	-	1.0	499	82	SW3	5.7	5.9	1.4	0.1	0.1	0.2	0.2	0.0	0.0	5.8	5.8	MST60	MST60	-	-	0.0	
	3	5.92	-	-	-	1.0	499	82	SW3	5.4	5.9	1.4	0.1	0.1	0.2	0.2	0.0	0.0	5.8	5.8	MST60	MST60	-	-	0.0	
	4	5.92	-	-	-	1.0	499	82	SW3	5.4	5.9	1.4	0.1	0.1	0.2	0.2	0.0	0.0	5.8	5.8	MST60	MST60	-	-	0.0	
	5	11	-	-	-	1.0	499	82	SW3	10.5	5.7	1.3	0.2	0.2	0.3	0.3	0.0	0.0	5.5	5.5	MST60	MST60	-	-	0.0	
											-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0
											-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0
											-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0
											-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0
											-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0
										-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0	
E	1	0	17.75	0.7	0.9	1.0	258	58	SW6	-0.5			0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3	None	None	8.5	2.7	0.0	
	2	33	-	-	-	1.0	258	58	SW6	32.5	2.9	0.9	2.6	2.6	3.5	3.5	0.0	0.0	0.8	0.8	None	None	-	-	10.0	
	3	0	-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
F	1		30	0.7	0.9					-0.5			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	None	None	13.9	3.9	20.0	
	2	30.83	-	-	-	1.0	229	46	SW6	30.3	2.5	0.7	4.1	4.1	5.5	5.5	0.0	0.0	0.8	0.8	None	None	-	-	20.0	
	3		-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
	4	30	-	-	-	1.0	229	46	SW6	29.5	2.5	0.7	2.3	2.3	3.1	3.1	0.0	0.0	0.2	0.2	None	None	-	-	10.0	
	5		-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.8	0.8			-	-	0.0		
G	1	17.25	0	1.4	1.7	1.0	50	45	SW6	16.8	0.6	0.7	0.4	0.4	0.5	0.5	0.0	0.0	0.2	0.2	None	None	1.4	1.7	0.0	
	2	10	-	-	-	1.0	50	45	SW6	9.5	0.6	0.7	0.6	0.6	0.8	0.8	0.7	0.7	0.6	0.6	MST37	MST37	-	-	6.8	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.7	0.7			-	-	0.0		
H	1	23.33	20	0.0	0.0	1.0	265	44	SW6	22.8	3.0	0.7	1.8	1.8	2.4	2.4	0.0	0.0	1.1	1.1	MST37	MST37	8.8	2.0	10.0	
	2	10	-	-	-	1.0	265	44	SW6	9.5	3.0	0.7	0.0	0.0	0.0	0.0	0.0	0.0	3.5	3.5	MST48	MST48	-	-	0.0	
			-	-	-					-0.5			0.2	0.2	0.3	0.3	0.7	0.7			MST48	MST48	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0			-	-	-	-	0.0	
Σ				110.0	2.7	3.4																	51.2	14.6		

Shearwalls: 1/2" sheathing w/ DF studs			
Nil	-	0	plf
SW6	10d@6"o.c.	310	plf
SW4	10d@4"o.c.	460	plf
SW3	10d@3"o.c.	600	plf
SW2	10d@2"o.c.	770	plf
2SW4	10d@4"o.c.	920	plf
2SW3	10d@3"o.c.	1200	plf
2SW2	10d@2"o.c.	1540	plf
Re-Calc	-	1550	plf

Holdown Table w/ DF (Floor Span = 16")			
Nil	-	0	kips
None	-	0.5	kips
MST37	(2)-2x DF	2.705	kips
MST48	(2)-2x DF	4.200	kips
MST60	(2)-2x DF	6.235	kips
MST72	(2)-2x DF	6.730	kips
			kips
			kips
			kips
Re-Calc	-	6.731	kips

Input Cell
Input Cell w/ Formula



Description	Main Floor Shear Walls	By	LS	Date	01/25/23
	X-Direction	Checked		Date	
	Project	Scale	NTS	Sheet No.	
	Rose Hill Apartments	Job No.	22332.10		

Main Floor Shear Walls - Walls Below the Upper Floor Framing

Y - Direction Walls

Fy (EQ) = 48.5 kips (Story Shear)
 Fy (wind) = 10.1 kips (Story Shear)

Story HT = 10.92
 Wall HT = 9.92
 Max h/w = 3.5
 S_{DS} = 1.12

Wy = 610 PLF seismic
 Wy = 127 PLF wind

Wall Line	Wall Mark	SW Length	Trib Width	Line Load		EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ		Wind		Net Uplift		Governing		Hold-down		EQ Line Load	Wind Line Load	DL Trib		
				From Above									0.6-0.14S _{DS} DL	End i	End j	0.6 * DL	End i	End j	From Above	Net Uplift	End i	End j				End i	End j
				EQ	Wind																						
1	A B	13.75	20	1.4	1.2	1.0	312	61	SW4	13.3	3.5	1.0	1.0	1.0	1.3	1.3	0.0	0.0	2.6	2.6	MST37	MST37	13.6	3.7	9.0		
		29.67	-	-	-	1.0	395	77	SW4	29.2	4.4	1.2	2.1	2.1	2.9	2.9	0.0	0.0	2.3	2.3	MST37	MST37	-	-	9.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	9.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
3	A B C D	0	30	1.4	1.2	1.0	362	66	SW4	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	None	None	19.7	5.0	19.5		
		54.25	-	-	-	-	-	-	-	53.8	4.0	1.0	4.1	4.1	5.6	5.6	0.0	0.0	0.0	0.0	-	-	-	-	9.8		
	0	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	0	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
4				0.0	0.0					-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0					0.0	0.0	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0				
5	A B	0	19.5	0.0	0.0	1.0	350	52	SW4	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	None	None	11.9	2.5	0.0		
		34	-	-	-	-	-	-	-	33.5	3.9	0.8	4.4	4.4	6.0	6.0	0.0	0.0	0.0	0.0	-	-	-	-	19.5		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
6	A B C D	3.75	10	0.0	0.0	0.8	389	58	SW4	3.3	3.7	0.8	0.3	0.3	0.4	0.4	0.0	0.0	3.4	3.4	MST48	MST48	6.1	1.3	8.7		
		3.83	-	-	-	0.8	381	56	SW4	3.3	3.7	0.8	0.3	0.3	0.4	0.4	0.0	0.0	3.4	3.4	MST48	MST48	-	-	8.7		
	4.75	-	-	-	1.0	307	45	SW6	4.3	3.6	0.7	0.3	0.3	0.5	0.5	0.0	0.0	3.3	3.3	MST48	MST48	-	-	8.7			
	8.42	-	-	-	1.0	294	44	SW6	7.9	3.4	0.7	0.6	0.6	0.8	0.8	0.0	0.0	2.8	2.8	MST48	MST48	-	-	8.7			
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	-	-	-	-	-	-	0.0		
Σ		79.5	2.7	2.4																		51.2	12.4				

Input Cell
 Input Cell w/ Formula
 Co=0.79 Applied for Perforated Shear Wall



Description	Main Floor Shear Walls	By	LS	Date	01/25/23
	Y-Direction	Checked		Date	
	Project	Scale	NTS	Sheet No.	
	Rose Hill Apartments	Job No.	22332.10		

Lower Floor Shear Walls - Walls Below the Main Floor Framing

X - Direction Walls

F_x (EQ) = 30.7 kips (Story Shear)
F_x (wind) = 13.5 kips (Story Shear)

Story HT = 11.5
Wall HT = 10.33
Max h/w = 3.5
S₀₅ = 1.12

W_x = 279 PLF seismic
W_x = 123 PLF wind

						Line Load							EQ		Wind		EQ		Wind		Net Uplift		Governing				EQ		Wind	
Wall Line	Wall Mark	SW Length	Trib Width	From Above		EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	Gross Uplift	Gross Uplift	EQ		Wind		Net Uplift		Governing		Hold-down		Line Load	Line Load	DL Trib					
				EQ	Wind								End i	End j	End i	End j	End i	End j	End i	End j	End i	End j				End i	End j			
D	1	8.33	42.25	18.6	4.3	1.0	629	140	SW2	7.8	7.7	2.4	0.2	0.2	0.3	0.3	5.6	5.6	13.1	13.1	HDU14	HDU14	30.4	9.5	0.0					
	2	6.17	-	-	-	1.0	629	140	SW2	5.7	7.9	2.5	0.1	0.1	0.2	0.2	5.8	5.8	13.5	13.5	HDU14	HDU14	-	-	0.0					
	3	5.92	-	-	-	1.0	629	140	SW2	5.4	7.9	2.5	0.1	0.1	0.2	0.2	5.8	5.8	13.6	13.6	HDU14	HDU14	-	-	0.0					
	4	5.92	-	-	-	1.0	629	140	SW2	5.4	7.9	2.5	0.1	0.1	0.2	0.2	5.8	5.8	13.6	13.6	HDU14	HDU14	-	-	0.0					
	5	11	-	-	-	1.0	629	140	SW2	10.5	7.6	2.4	0.3	0.3	0.3	0.3	5.5	5.5	12.8	12.8	HDU14	HDU14	-	-	0.0					
	6	5.5	-	-	-	1.0	629	140	SW2	5.0	8.0	2.5	0.1	0.1	0.2	0.2	0.0	0.0	7.8	7.8	HDU11	HDU11	-	-	0.0					
	7	5.5	-	-	-	1.0	629	140	SW2	5.0	8.0	2.5	0.1	0.1	0.2	0.2	0.0	0.0	7.8	7.8	HDU11	HDU11	-	-	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				
E	1	0	17.75	8.5	2.7					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					13.5	4.8	0.0					
	2	33	-	-	-	1.0	408	105	SW4	32.5	4.8	1.7	2.6	2.6	3.5	3.5	0.3	0.3	2.5	2.5	HDU2	HDU2	-	-	10.0					
	3	0	-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
F	1	0	14.5	13.9	3.9					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					17.9	5.7	0.0					
	2	34.17	-	-	-	1.0	349	79	SW4	33.7	4.1	1.3	3.1	3.1	4.3	4.3	0.0	0.0	0.9	0.9	HDU2	HDU2	-	-	12.5					
	3	0	-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
	4	0	-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
	5	17.25	-	-	-	1.0	349	79	SW4	16.8	4.1	1.3	0.4	0.4	0.5	0.5	0.0	0.0	3.7	3.7	HDU4	HDU4	-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
G	1	0	20	1.4	1.7					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					6.9	4.2	0.0					
	2	0	-	-	-					-0.5			0.0	0.0	0.0	0.0	0.6	0.6					-	-	0.0					
	3	22.5	-	-	-	1.0	140	60	SW6	22.0	1.6	1.0	1.8	1.8	2.4	2.4	0.0	0.0	0.0	0.0	None	None	-	-	10.0					
	4	27	-	-	-	1.0	140	60	SW6	26.5	1.6	1.0	2.1	2.1	2.9	2.9	0.0	0.0	0.0	0.0	None	None	-	-	10.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
H	1	0	15.5	8.8	2.0					-0.5			0.0	0.0	0.0	0.0	1.1	1.1					13.1	3.9	0.0					
	MF	10.5	-	-	-	1.0	609	130		10.0	7.4	2.2	0.8	0.8	1.1	1.1	0.0	0.0	6.5	6.5			-	-	10.0					
	H2	11.08	-	-	-	1.0	609	130	SW2	10.6	7.3	2.2	0.3	0.3	0.3	0.3	3.5	3.5	10.6	10.6	HDU14	HDU14	-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
			-	-	-					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					-	-	0.0					
		Σ	110.0	51.2	14.6																		81.9	28.1						

Shearwalls: 1/2" sheathing w/ DF studs			
Nil	-	0	plf
SW6	10d@6"o.c.	310	plf
SW4	10d@4"o.c.	460	plf
SW3	10d@3"o.c.	600	plf
SW2	10d@2"o.c.	770	plf
2SW4	10d@4"o.c.	920	plf
2SW3	10d@3"o.c.	1200	plf
2SW2	10d@2"o.c.	1540	plf
Re-Calc	-	1550	plf

Holdown Table (DF Studs)			
Nil	-	0	kips
None	-	0.5	kips
HDU2	(2)-2x DF	3.075	kips
HDU4	(2)-2x DF	4.6	kips
HDU5	(2)-2x DF	5.6	kips
HDU8	4x DF#2	7.0	kips
HDU11	6x6 DF#1	9.5	kips
HDU14	6x6 DF#1	14.4	kips
			kips
Re-Calc	-	14.5	kips

Input Cell
Input Cell w/ Formula



Description	Lower Floor Shear Walls	By	LS	Date	01/25/23
	X-Direction	Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.			
		22332.10			

Lower Floor Shear Walls - Walls Below the Main Floor Framing

Y - Direction Walls

Fy (EQ) = 30.7 kips (Story Shear)
Fy (wind) = 12.1 kips (Story Shear)

Story HT = 11.50
Wall HT = 10.33
Max h/w = 3.5
S_{cs} = 1.12

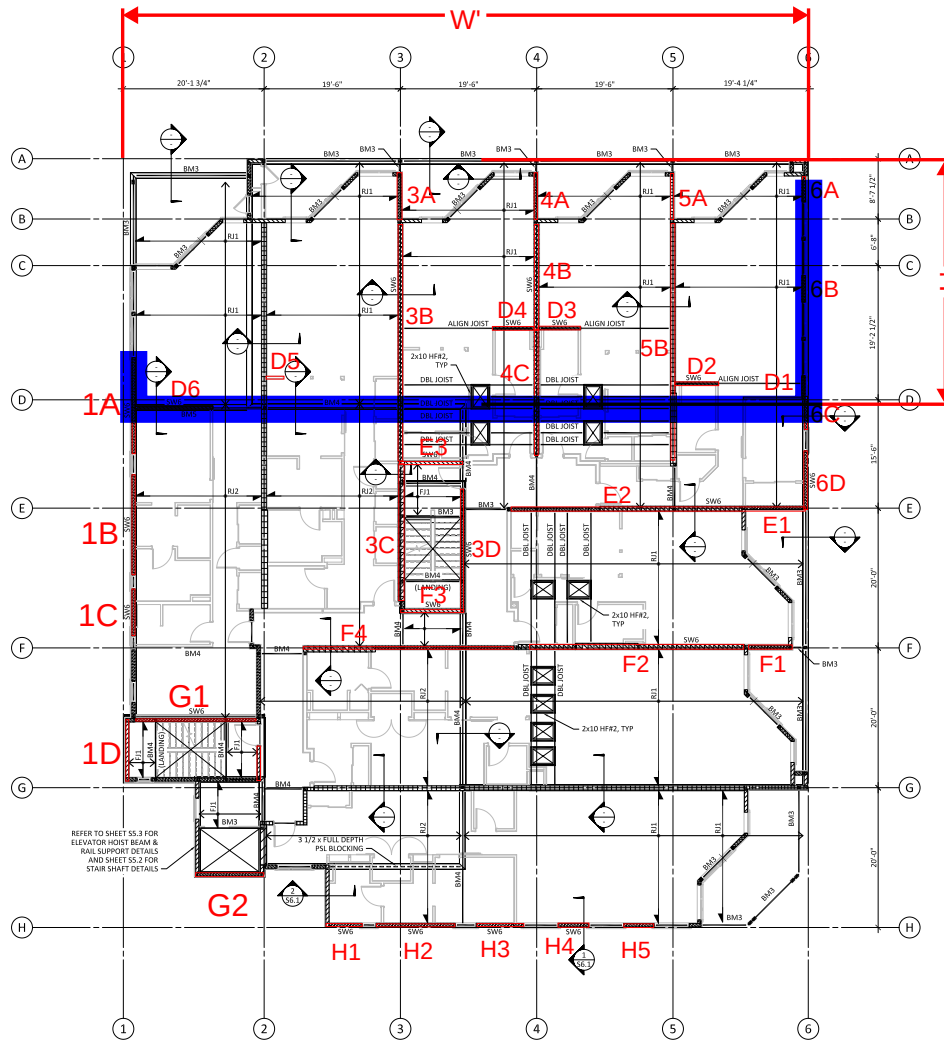
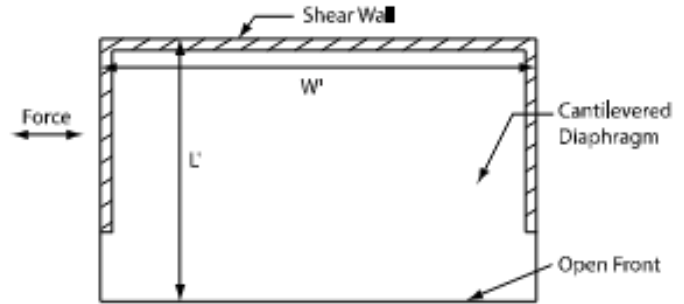
Wy = 386 PLF seismic
Wy = 153 PLF wind

Wall Line	Wall Mark	SW Length	Trib Width	Line Load		EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ		Wind		Net Uplift		Governing		Hold-down		EQ Line Load	Wind Line Load	DL Trib
				From Above									End i	End j	End i	End j	From Above	End i	End j	End i	End j				
				EQ	Wind																	(0.6-0.14S _{cs})DL			
1	A B	13.75	20	13.6	3.7	1.0	490	111	SW3	13.3	5.8	1.9	0.3	0.3	0.4	0.4	2.6	2.6	8.1	8.1	HDU11	HDU11	21.3	6.8	0.0
		29.67	-	-	-	1.0	620	141	SW2	29.2	7.3	2.3	0.7	0.7	0.9	0.9	2.3	2.3	8.8	8.8	HDU11	HDU11	-	-	0.0
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
3	A B C D	0	30	19.7	5.0					-0.5			7.3	7.3	9.9	9.9							31.2	9.6	0.0
		55.75	-	-	-	1.0	560	122	SW3	55.3	6.5	2.0	7.3	7.3	9.9	9.9	0.0	0.0	0.0	0.0	None	None	-	-	19.5
		0	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		0	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
4				0.0	0.0					-0.5			0.0	0.0	0.0	0.0	0.0	0.0					0.0	0.0	0.0
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
	-	-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0				-	-	0.0	
5	A B	0	19.5	11.9	2.5					-0.5			7.3	7.3	9.9	9.9							19.4	5.4	0.0
		34	-	-	-	1.0	571	114	SW3	33.5	6.7	1.9	4.4	4.4	6.0	6.0	0.0	0.0	2.2	2.2	HDU2	HDU2	-	-	19.5
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
6	A B C D	3.75	10	6.1	1.3	0.7	661	132	SW2	3.3	6.4	1.8	0.3	0.3	0.4	0.4	3.4	3.4	9.5	9.5	HDU14	HDU14	10.0	2.8	8.7
		3.83	-	-	-	0.7	647	130	SW2	3.3	6.3	1.8	0.3	0.3	0.4	0.4	3.4	3.4	9.5	9.5	HDU11	HDU11	-	-	8.7
		4.75	-	-	-	0.9	522	105	SW3	4.3	6.2	1.7	0.3	0.3	0.5	0.5	3.3	3.3	9.1	9.1	HDU11	HDU11	-	-	8.7
		8.42	-	-	-	1.0	480	96	SW3	7.9	5.9	1.6	0.6	0.6	0.8	0.8	2.8	2.8	8.1	8.1	HDU11	HDU11	-	-	8.7
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
		-	-	-	-	-	-	-	-	-0.5	-	-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-	-	0.0		
Σ		79.5	51.2	12.4																			81.9	24.6	

Input Cell
Input Cell w/ Formula
Co=0.79 Applied for Perforated Shear Wall

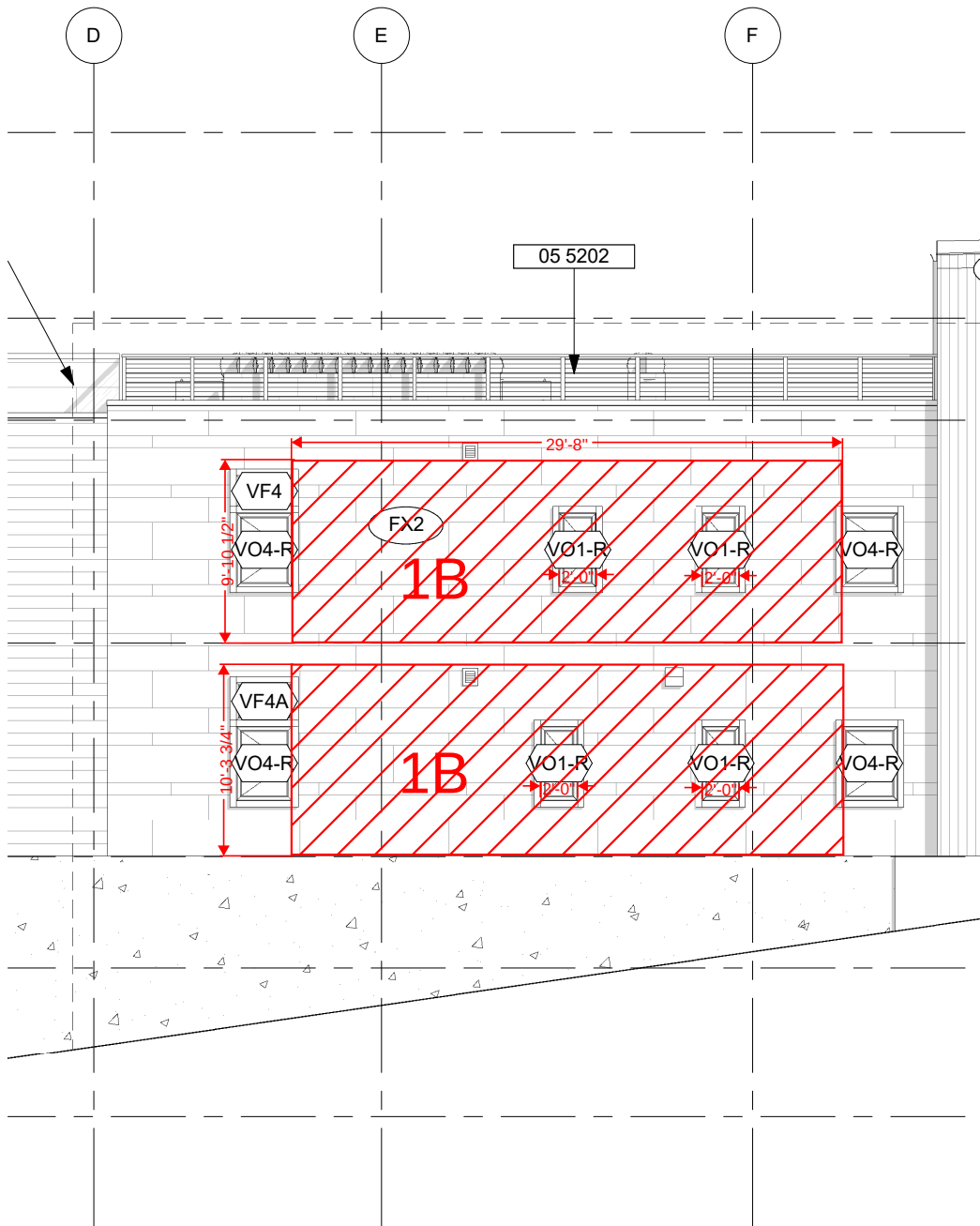
 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Lower Floor Shear Walls	By	LS	Date	01/25/23
		Y-Direction	Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.			
			22332.10			

... OPEN FRONT DIAPHRAGM OK



 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Open Front Diaphragm	By	LS	Date	01/25/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.			
			22332.10			

PERFORATED SHEAR WALL (1B)



Description	Perforated Shear Wall	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
	Project	Job No.	22332.10		
	Rose Hill Apartments				

PERFORATED SHEAR WALL (1B)

Table 4.3.3.5 Shear Capacity Adjustment Factor, C_o

Wall Height, h	Maximum Opening Height ¹				
	h/3	h/2	2h/3	5h/6	h
8' Wall	2'-8"	4'-0"	5'-4"	6'-8"	8'-0"
10' Wall	3'-4"	5'-0"	6'-8"	8'-4"	10'-0"
Percent Full-Height Sheathing ²	Effective Shear Capacity Ratio				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

1 The maximum opening height shall be taken as the maximum opening clear height in a perforated shear wall. Where areas above and/or below an opening remain unsheathed, the height of each opening shall be defined as the clear height of the opening plus the unsheathed areas.

2 The sum of the perforated shear wall segment lengths, ΣL_p , divided by the total length of the perforated shear wall, L_{tot} . Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with Section 4.3.4.3.

$$\%SHEATHING = 25.67FT/29.67FT=87\%$$

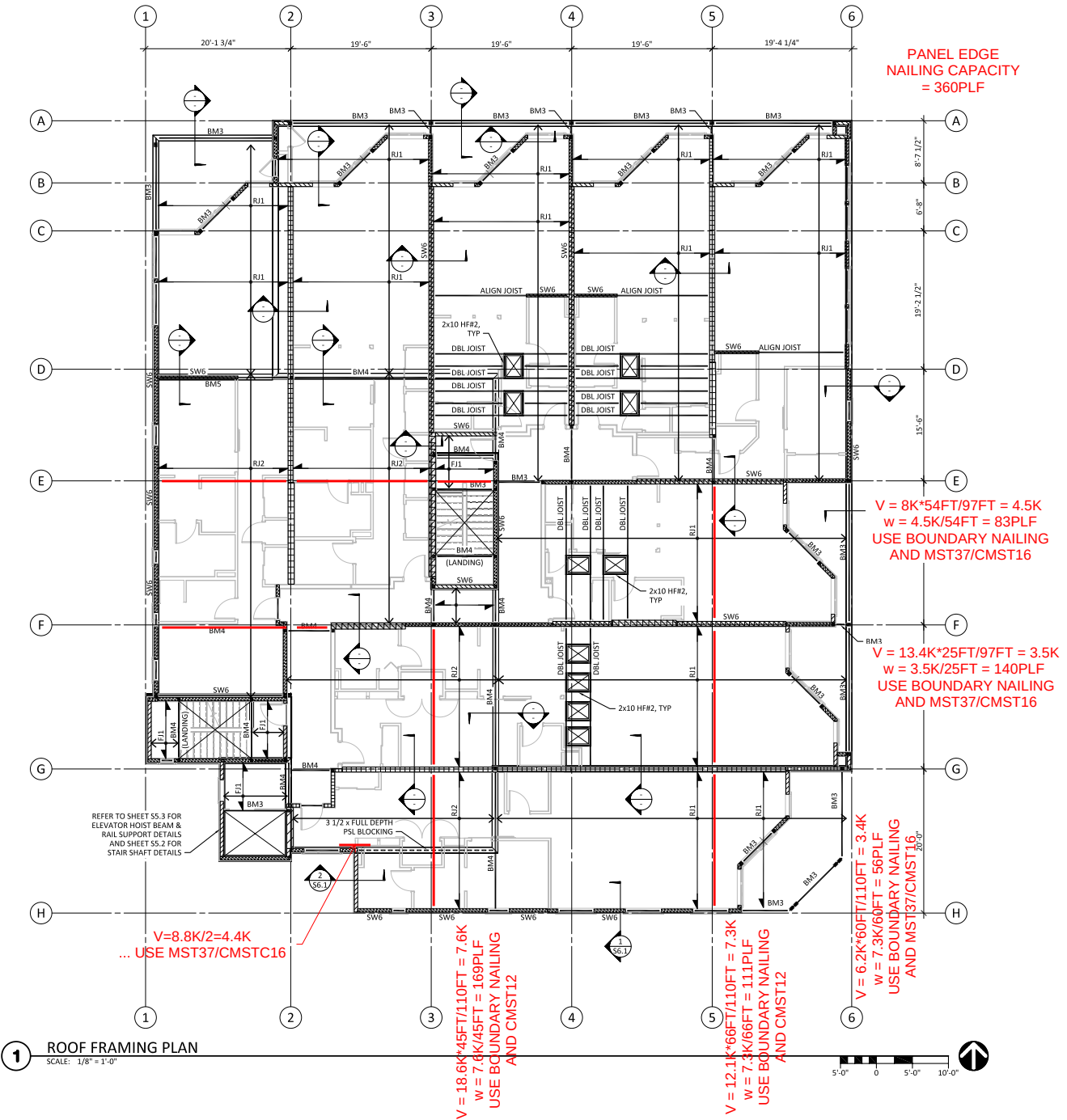
$$C_o = (r/(3-2r)) * L_{tot}/L_{wall}$$

$$C_o = (0.87/(3-2*0.87))/0.87 = 0.79$$

... SEE SHEAR WALL CALCULATIONS ON PREVIOUS PAGES

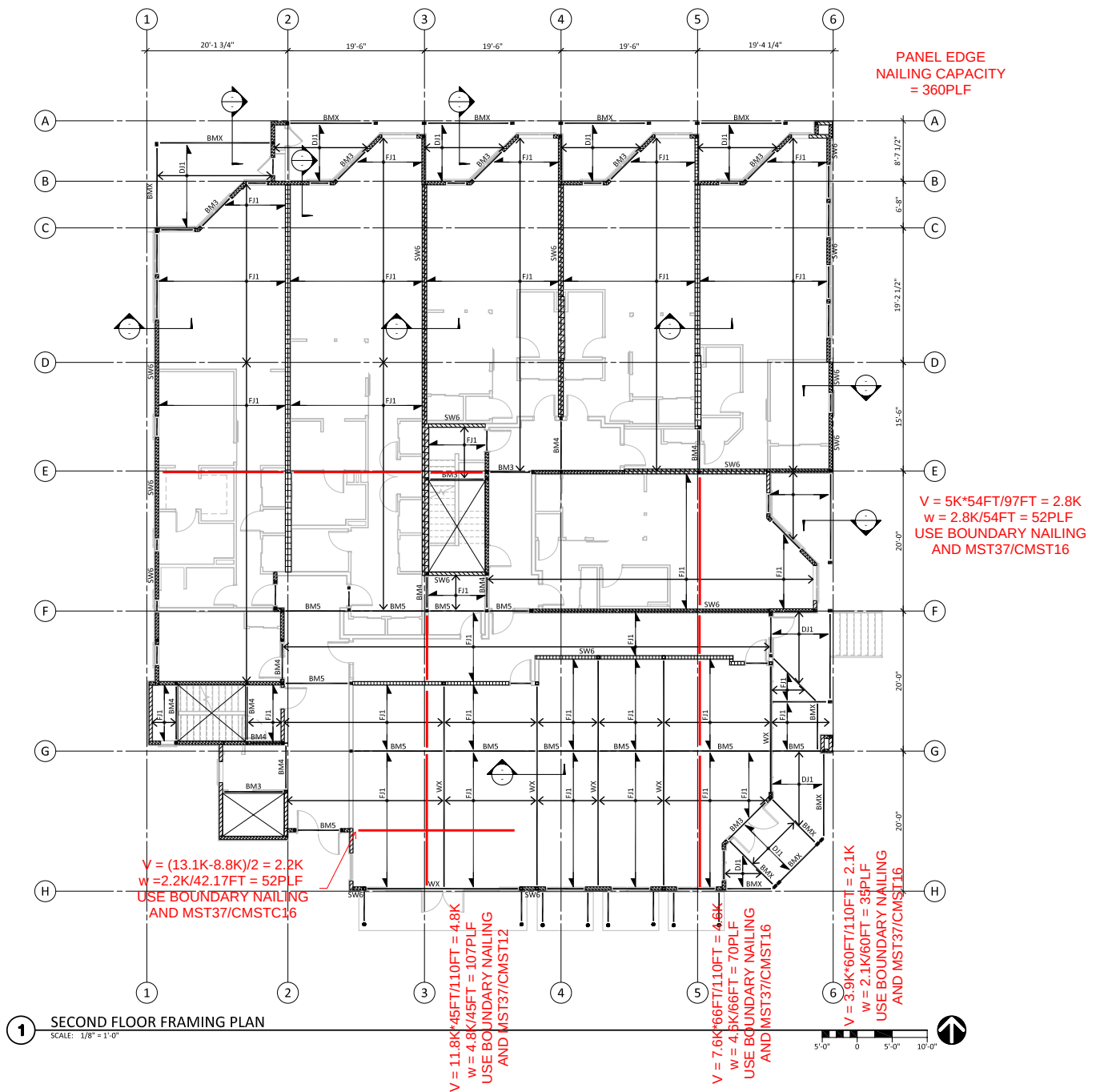
 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Perforated Shear Wall	By	LS	Date	01/25/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.	22332.10		

STRAPPING - ROOF DIAPHRAGM



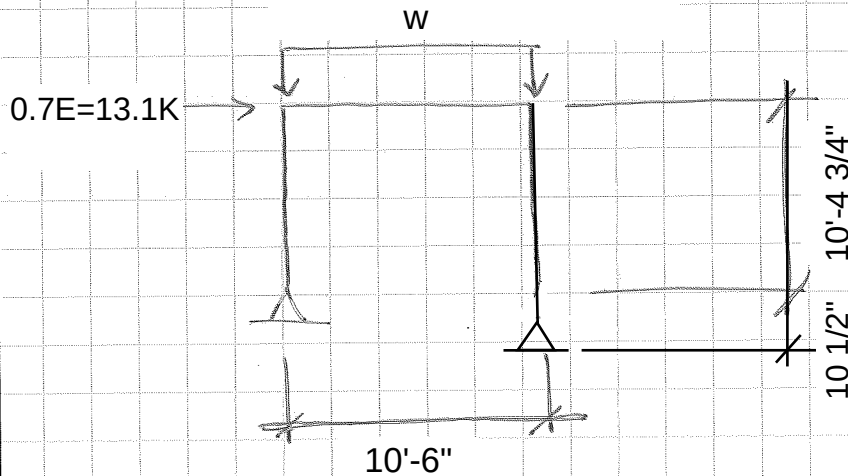
Description	Strapping Calculations	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

STRAPPING - SECOND FLOOR DIAPHRAGM



Description	Strapping Calculations	By	LS	Date	01/25/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

MOMENT FRAME



W:

ROOF:
D=16PSF
S=25PSF
TRIB=10FT

WALL:
D=10PSF
TRIB=12.5FT

FLOOR:
D=25PSF
L=40PSF
TRIB=10FT

STEEL SPECIAL MOMENT FRAME

$$R = 6.5$$

$$\Omega = 3$$

$$C_D = 5.5$$

ALLOWABLE DRIFT:

$$\Delta_a = 0.02 h_{sx} = 0.02 (11.3 \text{ FT}) (12 \text{ IN/FT})$$

$$\Delta_a = 2.72 \text{ IN}$$

$$\delta_m = \frac{C_D \delta_{max}}{I_e} = \frac{5.5 \delta_{max}}{I_e}$$

MAX MODAL DRIFT:

$$\delta_{max} = 2.72 \text{ IN} / 5.5 = 0.49 \text{ IN} = 0.44 \text{ IN} \dots \text{OK}$$

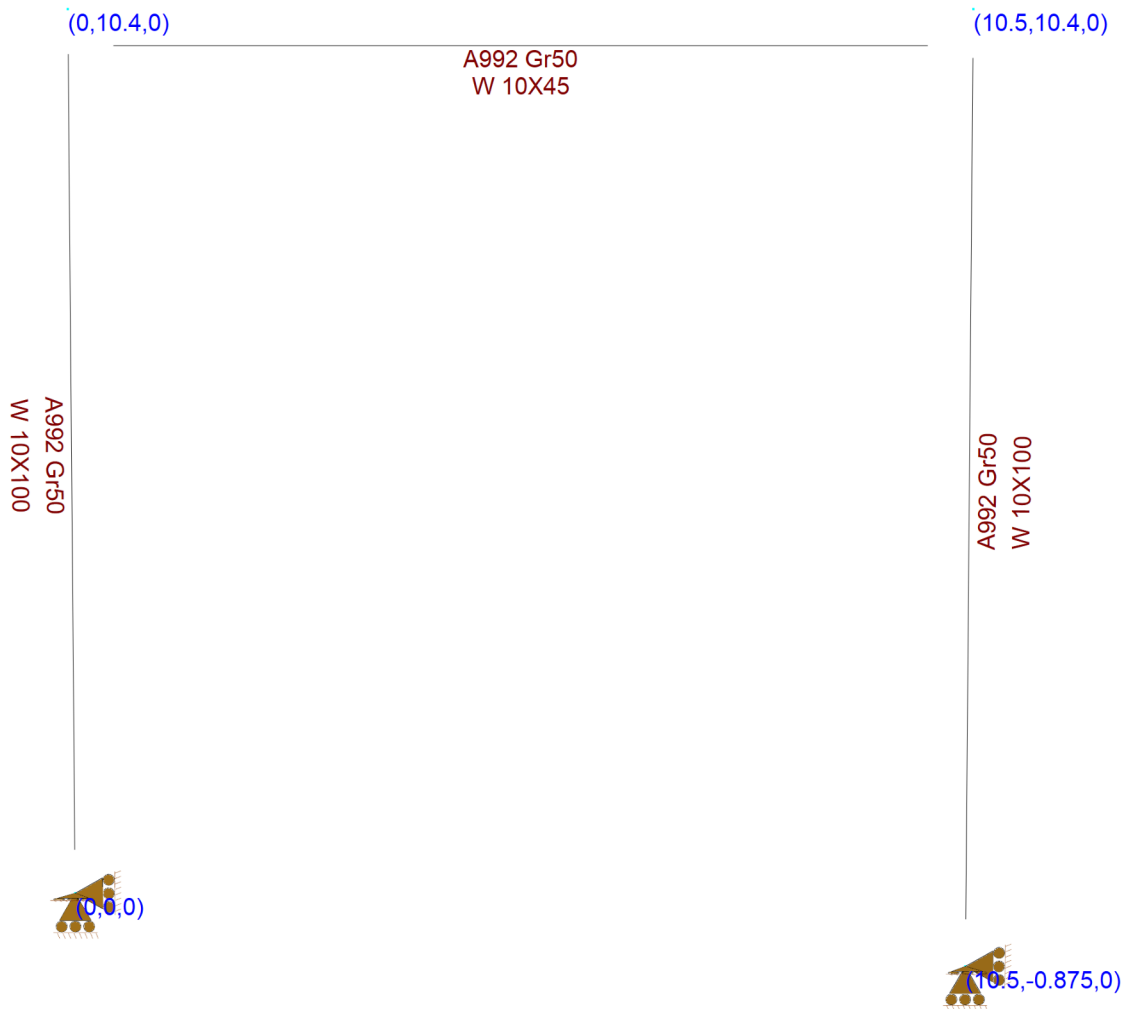
UPLIFT: SEE HILTI ANCHOR CALCULATION



250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
www.cgengineering.com

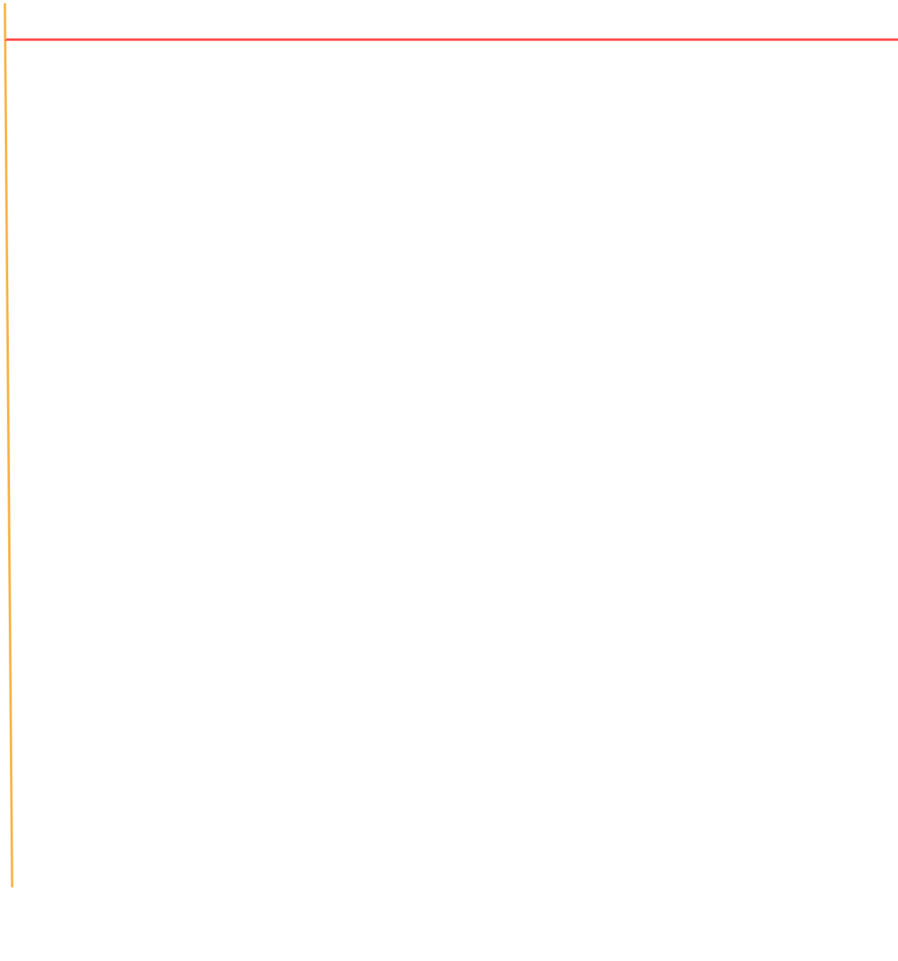
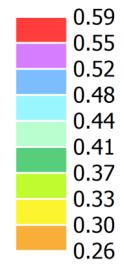
Description	MOMENT FRAME CALCULATIONS	By	LS	Date	1/20/23
		Checked		Date	
		Scale		Sheet No.	
Project	ROSE HILL MUKILTEO	Job No.	22332.10		

MF-1 PROPERTIES



STRESS RATIO

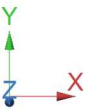
Stress ratio
AISC/AISI/BS/AS/CSA/NDS



SEISMIC DRIFT

Tx=0.438743[in]

Tx=0.437336[in]



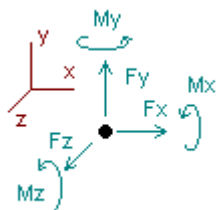
Current Date: 1/25/2023 10:34 AM

Units system: English

File name: R:_2022 Projects\22332 Rose Hill Mukilteo_Structural\Engineering\Lateral\Moment Frame\Moment Frame 2.ret

Analysis result

Reactions



Direction of positive forces and moments

Node	Forces [Kip]			Moments [Kip*ft]		
	FX	FY	FZ	MX	MY	MZ
Condition DL=Dead Load						
1	0.35987	2.77876	0.00000	0.00000	0.00000	0.00000
4	-0.35987	2.83874	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	5.61750	0.00000	0.00000	0.00000	0.00000
Condition L=Live Load						
1	0.26906	2.07758	0.00000	0.00000	0.00000	0.00000
4	-0.26906	2.12242	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	4.20000	0.00000	0.00000	0.00000	0.00000
Condition Eq=Seismic Load						
1	-4.87794	-9.00045	0.00000	0.00000	0.00000	0.00000
4	-4.22206	9.00045	0.00000	0.00000	0.00000	0.00000
SUM	-9.10000	0.00000	0.00000	0.00000	0.00000	0.00000
Condition S=Snow Load						
1	0.16816	1.29849	0.00000	0.00000	0.00000	0.00000
4	-0.16816	1.32651	0.00000	0.00000	0.00000	0.00000
SUM	0.00000	2.62500	0.00000	0.00000	0.00000	0.00000
Condition OVST=0.68DL+2.5Eq						
1	-11.95013	-20.61157	0.00000	0.00000	0.00000	0.00000
4	-10.79987	24.43147	0.00000	0.00000	0.00000	0.00000
SUM	-22.75000	3.81990	0.00000	0.00000	0.00000	0.00000

Design of Special Moment Frames (Reduced Beam Section)

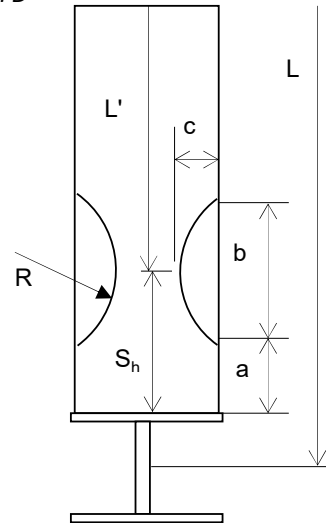
Per AISC Seismic Design Manual 2016 - LRFD

Beam Properties:

Size= W10X45
 L= 126 in (col. CL)
 d= 10.1 in
 t_w= 0.35 in
 b_f= 8.02 in
 t_f= 0.62 in
 Z_x= 54.9 in³
 r_y= 2.01 in
 R_y= 1.1 in²
 F_y= 50 ksi
 F_u= 65 ksi

Column Properties:

Size= W10X100
 H= 124 in
 A_g= 29.3 in²
 d= 11.1 in
 t_w= 0.68 in
 b_f= 10.3 in
 t_f= 1.12 in
 k_{det}= 1.8125 in
 Z_x= 130 in³
 r_y= 2.65 in



Beam Limitations (per AISC 358 5.3-1)

Depth ≤ W36
 Weight ≤ 300 lbs/ft
 t_f ≤ 1.75
 Clear Span-to-Depth Ratio ≥ 7.0 (for SMF only)

Protected Zone= 13.6 in

a = .6*b_f = 4.8 in (Eq. 5.8-1)
 b = .75*d = 7.6 in (Eq. 5.8-2)
 c = .2*b_f = 1.6 in (Eq. 5.8-3)
 Radius = 5.3 in
 (refer to RBS detail sheets by CG Eng)

Width-to-Thickness Ratios for Highly Ductile Compression Members

(AISC 341 Table D1.1)

Beam

b/t = 6.5 ≤ 7.3 **OKAY**
 h/t = 25.3 ≤ 36.1 **OKAY**

Column

b/t = 4.6 ≤ 7.3 **OKAY**
 h/t = 13.0 ≤ 36.1 **OKAY**

Compute Probable Maximum Moment @ Center of RBS

Z_{RBS} = Z_x - 2ct_f(d - t_f) = 36.0 in³ (AISC 358 Eq. 5.8-4)
 C_{pr} = (F_y + F_u)/2F_y ≤ 1.2 = 1.15 (AISC 358 Eq. 2.4.3-2)
 M_{pr} = C_{pr}R_yF_yZ_{RBS} = 2280 kip-in (AISC 358 Eq. 5.8-5)



250 4th Ave South
 Suite 200
 Edmonds, WA 98020

Description

Moment Frame Calculation

Project

Rose Hill Mukilteo

By

LS

Checked

Scale

Job No.

22332.10

Date 1/25/2023

Date

Sheet No.

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2016 - LRFD

Compute Shear Force @ Center of RBS

$$\begin{aligned}
 S_h &= a + b/2 = & 8.6 \text{ in} \\
 V_{gravity} &= & 6.8 \text{ kips (See RAM model LC: 1.2D + 1.6L)} \\
 V_{pr} &= 2 * M_{pr} / L_h = & 46.7 \text{ kips} \\
 V_{RBS} &= 1.2V_{DL} + 1.0V_{LL} + V_{pr} = & 53.5 \text{ kips}
 \end{aligned}$$

Compute Maximum Moment @ Face of Column

$$M_f = M_{pr} + V_{RBS} S_h = 2739.6 \text{ kip-in} \quad (\text{AISC 358 Eq. 5.8-6})$$

Check Flexural Strength of the Beam @ the Face of the Column

$$\begin{aligned}
 M_{pe} &= R_y F_y Z_x = 3019.5 \text{ kip-in} & (\text{AISC 358 Eq. 5.8-7}) \\
 M_f &\leq \phi_d M_{pe} & \text{OKAY (for } \phi_d = 1.0) & (\text{AISC 358 Eq. 5.8-8})
 \end{aligned}$$

Check Column-Beam Relationship Limitations

$$\begin{aligned}
 M_{uv} &= V_{RBS} * (a + b/2 + d_c/2) = 757 \text{ kip-in} \\
 \Sigma M^*_{pb} &= \Sigma (M_{pr} + M_{uv}) = 3036 \text{ kip-in} & (\text{Seismic Provisions Eq. E3-3a}) \\
 & & (\text{assuming single beam-column})
 \end{aligned}$$

$$\begin{aligned}
 P_{uc} &= 15.4 \text{ kips (See RAM model LC: 1.2D + 1.0E + 1.0L)} \\
 \Sigma M^*_{pc} &= \Sigma Z_{xc} (F_{yc} - P_{uc} / A_g) = 6432 \text{ kip-in} & (\text{Seismic Provisions Eq. E3-2a}) \\
 & & (\text{assuming single beam-column})
 \end{aligned}$$

$$\Sigma M^*_{pc} / \Sigma M^*_{pb} \geq 1.0 = 2.1 \text{ OKAY}$$

Check Shear Strength of the Beam

$$\begin{aligned}
 L_h &= 97.7 \text{ in (distance between plastic hinges)} \\
 V_{gravity} &= 6.8 \text{ kips (See RAM model LC: 1.2D + 1.0L + 0.7S)} \\
 E_{mh} &= 2 * 1.1 * M_{pr} / L_h = 51.3 \text{ kips} & (\text{AISC 341 Eq. E3-6}) \\
 V_u &= 2 * 1.1 * M_{pr} / L_h + V_{gravity} = 58.1 \text{ kips} & (\text{AISC 358 Eq. 5.8-9}) \\
 \phi V_{n(web)} &= \phi * 0.6 * F_y * A_w * C_v = 76.0 \text{ kips} & \text{OKAY (per AISC Sec G Eq. G2-1)}
 \end{aligned}$$

*note: beam web is full-penetration welded to column face

*note: assumes reduced area for (2) 13/16" erection bolt holes

 250 4th Ave South Suite 200 Edmonds, WA 98020	Description	By	LS	Date	1/25/2023
	Moment Frame Calculation	Checked		Date	
		Scale		Sheet No.	
		Job No.			
		22332.10			
Project	Rose Hill Mukilteo				

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2016 - LRFD

Check Column Panel Zone Shear Strength

$$\begin{aligned}
 R_u &= \Sigma M_f / (d - t_f) = & 289 \text{ kips} \\
 .75 * F_y * A_g &= & 1099 \text{ kips} \text{ -----> USE J10-11} \\
 R_u \leq \phi R_n &= & 331 \text{ kips} \quad \text{OKAY! DOUBLER PLATE NOT REQ'D}
 \end{aligned}$$

Check Continuity Plate Requirements

$$\begin{aligned}
 t_{cf} &\geq .4 \sqrt{1.8 b_{bf} t_{bf} F_{yb} R_{yb} / (F_{yc} * R_{yc})} = & 1.2 \text{ REQUIRED} \\
 t_{cf} &\geq b_{bf} / 6 = & 1.3 \text{ REQUIRED}
 \end{aligned}$$

If required, continuity plates shall match thickness of beam flanges

 250 4th Ave South Suite 200 Edmonds, WA 98020	Description Moment Frame Calculation	By LS	Date 1/25/2023
		Checked	Date
		Scale	Sheet No.
	Project Rose Hill Mukilteo	Job No. 22332.10	

Company:
 Specifier:
 Address:
 Phone | Fax: |
 E-Mail:

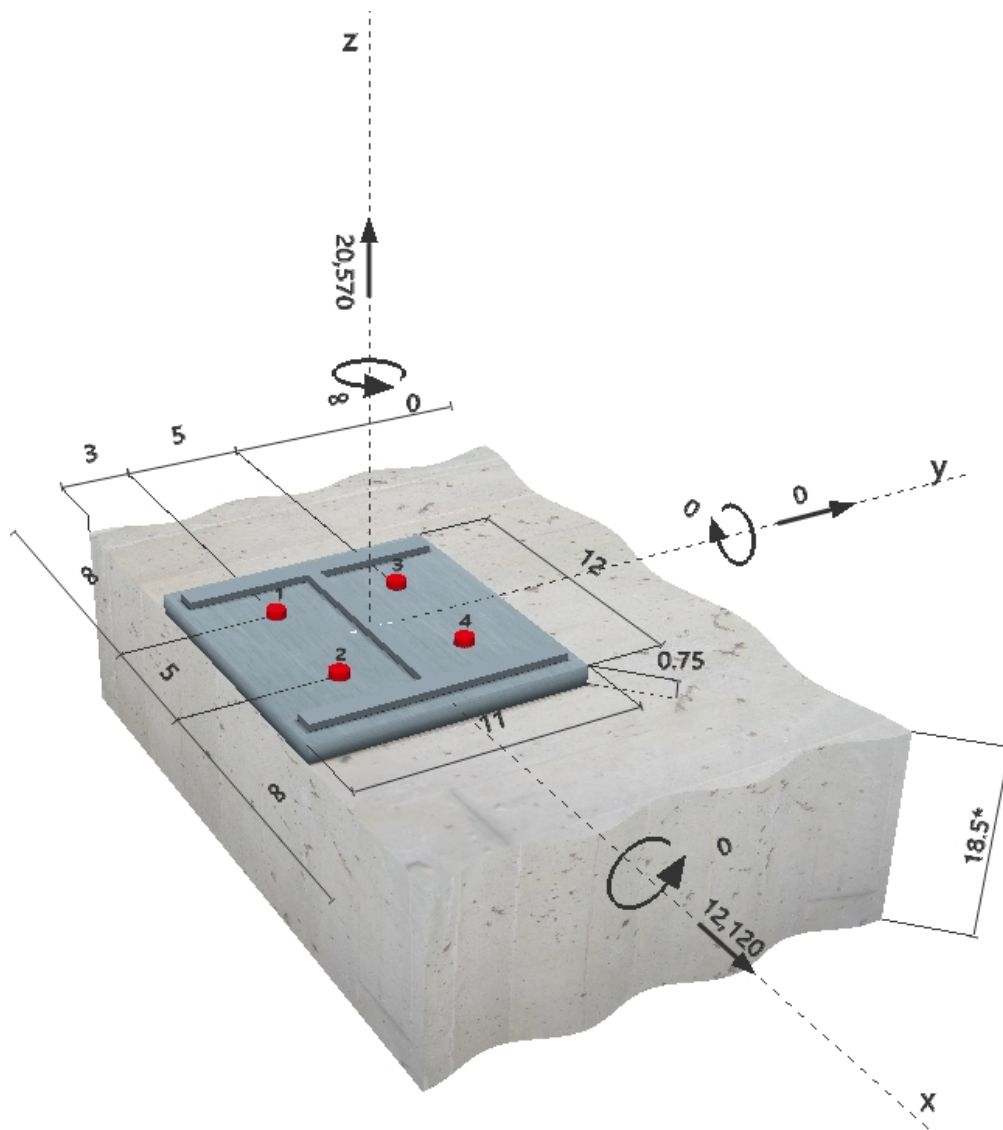
Page: 1
 Project:
 Sub-Project | Pos. No.:
 Date: 1/25/2023

Specifier's comments:

1 Input data

Anchor type and diameter:	Heavy Hex Head ASTM F 1554 GR. 36 3/4	
Effective embedment depth:	$h_{ef} = 12.000$ in.	
Material:	ASTM F 1554	
Proof:	Design method ACI 318-14 / CIP	
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 0.750$ in.	
Anchor plate:	$l_x \times l_y \times t = 12.000$ in. $\times$ 11.000 in. $\times$ 0.750 in.; (Recommended plate thickness: not calculated)	
Profile:	W shape (AISC); $(L \times W \times T \times FT) = 10.600$ in. $\times$ 10.200 in. $\times$ 0.530 in. $\times$ 0.870 in.	
Base material:	uncracked concrete, 5000, $f'_c = 5000$ psi; $h = 18.500$ in.	
Reinforcement:	tension: condition B, shear: condition A; edge reinforcement: none or $< \text{No. 4 bar}$	
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d)) Shear load: yes (17.2.3.5.3 (c))	

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



Company:
 Specifier:
 Address:
 Phone | Fax: |
 E-Mail:

Page: 2
 Project:
 Sub-Project | Pos. No.:
 Date: 1/25/2023

2 Proof | Utilization (Governing Cases)

Loading	Proof	Design values [lb]		Utilization	
		Load	Capacity	β_N / β_V [%]	Status
Tension	Concrete Breakout Strength	20570	28809	72 / -	OK
Shear	Concrete edge failure in direction y-	12120	10802	- / 113	not recommended

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.714	1.122	1.0	154	not recommended

3 Warnings

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

Fastening does not meet the design criteria!

4 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

SECTION NO. 3

Rose Hill Mukilteo
3rd & Park Ave
Mukilteo, WA 98275

SECTION TITLE: Concrete Lateral Design

SECTION INDEX

DESCRIPTION	PAGE
Site Data	3.01
Concrete Lateral Key Plan	3.02
Center of Mass Calculation	3.03
Center of Rigidity and Seismic Force Distribution	3.04-3.08
Concrete Shear Wall Design	3.09

Seismic Analysis

Design Per 2018 IBC & ASCE 7-16

Seismic Coefficients

Soil Site Class	D (assumed / default)
Occupancy Category	II (non-essential facility)
Seismic Design Category	D

$S_S =$	1.404	Lat. =	47.947
$S_1 =$	0.500	Long. =	-122.302

$S_{MS} = F_a S_S =$	1.685	$F_a =$	1.200
$S_{M1} = F_v S_1 =$	0.900	$F_v =$	1.800

$S_{DS} = (2/3)S_{MS} =$	1.123
$S_{D1} = (2/3)S_{M1} =$	0.600

$T_a = C_t h_n^x =$	0.22	$C_t =$	0.02
		$h_n =$	24
		$x =$	0.75

R Factor =	5.0	(concrete shear walls)
I_E Factor =	1.0	

From Computer Program:

Short & 1-Sec Period Mapped
Acceleration Parameters (MCE)

ASCE 7-16 (Eq. 11.4-1)

ASCE 7-16 (Eq. 11.4-2)

ASCE 7-16 (Eq. 11.4-3)

ASCE 7-16 (Eq. 11.4-4)

ASCE 7-16 (Table 12.8-2)

ASCE 7-16 (Table 12.8-2)

ASCE 7-16 (Table 12.2-1)

ASCE 7-16 (Table 1.5-2)

Seismic Base Shear

$V = 0.044 S_{DS} I =$	0.049 W	(Minimum Force)	ASCE 7-16 (Eq. 12.8-5)
------------------------	---------	-----------------	------------------------

$V = (S_{DS} I W) / R =$	0.225 W	(Governing Force)	ASCE 7-16 (Eq. 12.8-2)
--------------------------	---------	-------------------	------------------------

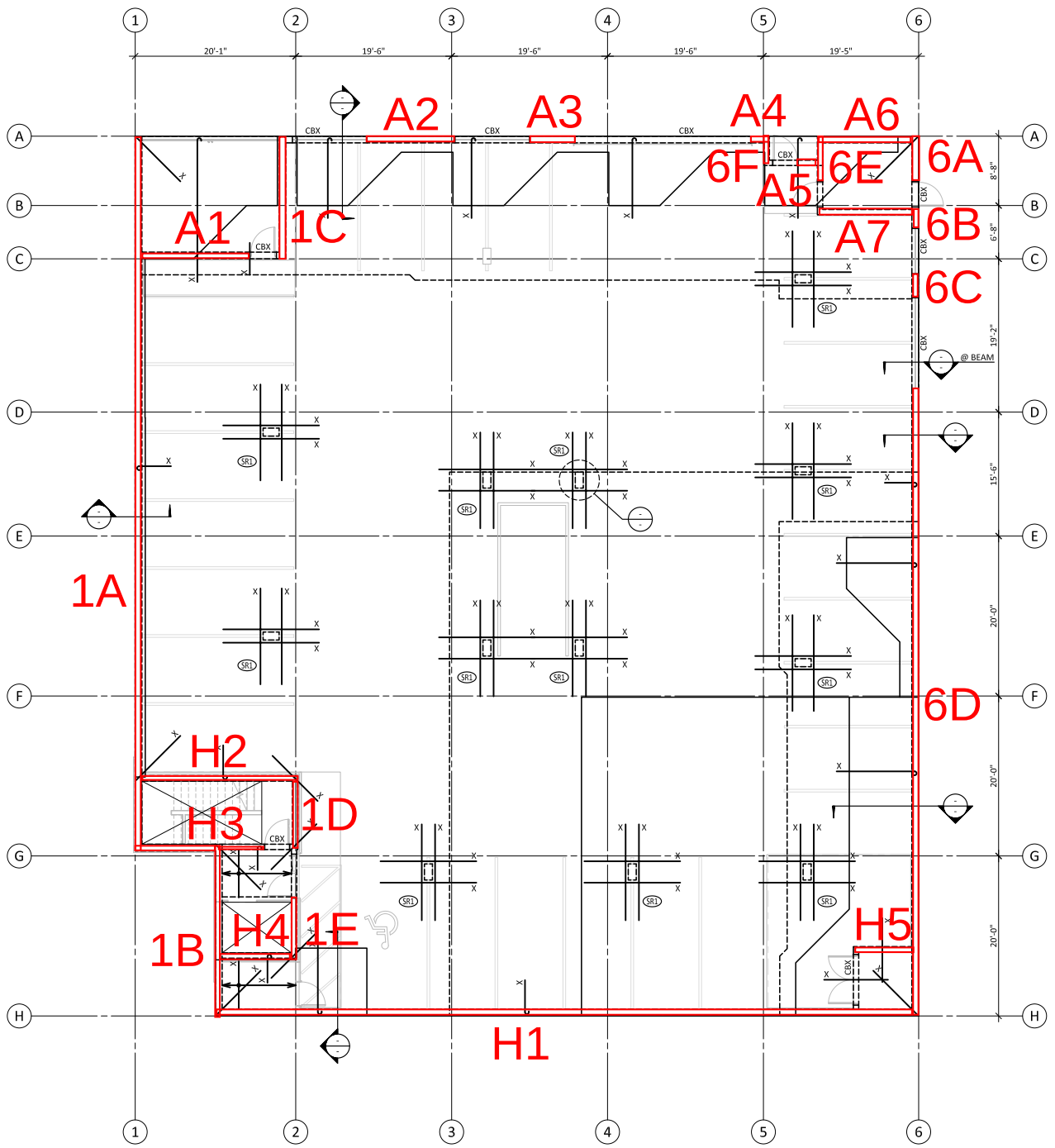
$V = (S_{D1} I W) / R T_a =$	0.553 W	(Maximum Force)	ASCE 7-16 (Eq. 12.8-3)
------------------------------	---------	-----------------	------------------------



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Suite 200
Edmonds, WA 98020

Description	Seismic Base Shear	By	LS	Date	01/27/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rose Hill Apartments	Job No.	22332.10		

LATERAL KEY PLAN - CONCRETE



 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Lateral Key Plan	By	LS	Date	01/25/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	Rose Hill Apartments	Job No.	22332.10		

Rose Hill Mukilteo

Center of Mass

Element I.D.	Area [ft ²]	Length [ft]	Weight [psf]	W _i [kips]	x _i [ft]	y _i [ft]	Wx _i [k-ft]	Wy _i [k-ft]
12" Slab	10590		150	1588.50	49.00	-55.00	77836.5	-87367.5
Drop Slab	3140		34.4	108.02	72.60	-79.10	7842.0	-8544.1
Drop Slab	806		131.3	105.83	79.60	-59.10	8423.9	-6254.4
Drop Slab	865		131.3	113.57	61.25	-79.10	6956.4	-8983.7
X-Walls			[plf]					
A1		13.50	813	10.97	7.50	-14.50	82.3	0.0
A2		11.10	813	9.02	34.50	0.00	311.1	0.0
A3		5.67	813	4.61	52.00	0.00	239.6	0.0
A4		2.33	813	1.89	78.10	0.00	147.9	0.0
A5		2.50	650	1.63	84.00	-3.00	136.5	0.0
A6		12.67	813	10.29	91.67	0.00	943.7	0.0
A7		12.67	650	8.24	91.67	-9.17	754.9	0.0
H1		88.00	813	71.50	54.00	-110.00	3861.0	0.0
H2		20.00	650	13.00	10.00	-79.00	130.0	0.0
H3		16.00	813	13.00	8.00	-89.00	104.0	0.0
H4		9.25	650	6.01	15.25	-102.50	91.7	0.0
H5		7.50	650	4.88	93.50	-101.50	455.8	0.0
Y-Walls								
1A		89.33	813	72.58	0.000	-45.000	0.0	-3266.1
1B		20.67	813	16.79	10.000	-100.000	0.0	-1679.4
1C		15.33	813	12.46	18.000	-7.670	0.0	-95.5
1D		9.25	650	6.01	19.670	-84.250	0.0	-506.6
1E		7.67	650	4.99	19.500	-98.900	0.0	-493.1
6A		5.50	813	4.47	98.000	-2.750	0.0	-12.3
6B		2.75	813	2.23	98.000	-10.250	0.0	-22.9
6C		3.00	813	2.44	98.000	-18.750	0.0	-45.7
6D		78.50	813	63.78	98.000	-70.750	0.0	-4512.5
6E		5.67	813	4.61	85.500	-2.830	0.0	-13.0
6F		3.67	813	2.98	78.750	-1.830	0.0	-5.5
Headers								
				0.00			0.0	0.0
Concrete Columns								
	2.00	13.00	300	1.95	17.00	-34.00	33.2	-66.3
	2.00	13.00	300	1.95	44.00	-43.00	85.8	-83.9
	2.00	13.00	300	1.95	55.50	-43.00	108.2	-83.9
	2.00	13.00	300	1.95	83.50	-17.75	162.8	-34.6
	2.00	13.00	300	1.95	84.00	-41.75	163.8	-81.4
	2.00	13.00	300	1.95	84.00	-65.75	163.8	-128.2
	2.00	13.00	300	1.95	16.00	-62.50	31.2	-121.9
	2.00	13.00	300	1.95	43.25	-64.00	84.3	-124.8
	2.00	13.00	300	1.95	54.67	-64.00	106.6	-124.8
	2.00	13.00	300	1.95	36.67	-92.00	71.5	-179.4
	2.00	13.00	300	1.95	62.25	-92.00	121.4	-179.4
	2.00	13.00	300	1.95	84.00	-92.00	163.8	-179.4
Wood Framing								
PH	540		21	11.34	30	-70	340.2	-793.8
Roof	13545		21	284.45	49	-55	13937.8	-15644.5
2nd Framing	10500		35	367.50	49	-55	18007.5	-20212.5
				2950.97			141899.20	-159841.05

Center of Gravity

X = 51.38 ft
Y = -57.09 ft

Cs = 0.2249

Lateral Mass X

2799.84 kips

Lateral Force X

629.68 kips (ULT)

Lateral Mass Y

2761.53 kips

Lateral Force Y

621.07 kips (ULT)



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description

Center of Mass

By

LS

Date

1/27/2023

Checked

Date

Scale

Sheet No.

Project

Rose Hill Mukilteo

Job No.

22332.10

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Torsional Analysis of Rigid Diaphragm

Project File: lateral calcs.ec6

LIC# : KW-06015244, Build:20.22.10.25

CG ENGINEERING

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DESCRIPTION: Rigid Diaphragm Analysis

General Information

IBC 2018, CBC 2019, ASCE 7-16

Applied Lateral Force in "X" Direction	630.0 k	Center of Shear Application :	
Applied Lateral Force in "Y" Direction	621.0 k	Distance from "X" datum point	51.380 ft
		Distance from "Y" datum point	-57.090 ft
Note: These loads are resolved into X & Y components when applied to the system of elements at angular increment			
		Accidental Torsion values per ASCE 7-05 12.8.4.2	
		Ecc. as % of Maximum Dimension	5.00 %
Load Orientation Angular Increment	15.0 deg	Maximum Dimensions :	
Load Location Angular Increment	30.0 deg	Along "X" Axis	ft
		Along "Y" Axis	ft
Center of Rigidity Location (calculated) . . .			
"X" dist. from Datum	42.552 ft		
"Y" dist. from Datum	-94.822 ft		
		Accidental Eccentricity +/- from "Y" Coord. of Center of Load Application :	0.0 ft
		Accidental Eccentricity +/- from "X" Coord. of Center of Load Application :	0.0 ft

Wall Information

Label : 1A	X Wall C.G. Location	0 ft	Length	89.33 ft
	Y Wall C.G. Location	-45 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 1.2794E-005 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 7.8691E+002 in			E - Shear	1.442 Mpsi
Label : 1B	X Wall C.G. Location	10 ft	Length	20.67 ft
	Y Wall C.G. Location	-100 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 1.0753E-004 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 3.4008E+003 in			E - Shear	1.442 Mpsi
Label : 1C	X Wall C.G. Location	18 ft	Length	15.33 ft
	Y Wall C.G. Location	-7.67 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 2.0586E-004 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 4.5854E+003 in			E - Shear	1.442 Mpsi
Label : 1D	X Wall C.G. Location	19.67 ft	Length	9.25 ft
	Y Wall C.G. Location	-84.25 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	8 in
Along Wall "y" Dir 9.1600E-004 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 1.4834E+004 in			E - Shear	1.442 Mpsi
Label : 1E	X Wall C.G. Location	19.5 ft	Length	7.67 ft
	Y Wall C.G. Location	-98.9 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	8 in
Along Wall "y" Dir 1.5266E-003 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 1.7890E+004 in			E - Shear	1.442 Mpsi
Label : 6A	X Wall C.G. Location	98 ft	Length	5.5 ft
	Y Wall C.G. Location	-2.75 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 3.1263E-003 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 1.2781E+004 in			E - Shear	1.442 Mpsi
Label : 6B	X Wall C.G. Location	98 ft	Length	2.75 ft
	Y Wall C.G. Location	-10.25 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 2.3830E-002 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 2.5562E+004 in			E - Shear	1.442 Mpsi
Label : 6C	X Wall C.G. Location	98 ft	Length	3 ft
	Y Wall C.G. Location	-18.75 ft	Height	13 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "y" Dir 1.8413E-002 in	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
Along Wall "x" Dir 2.3432E+004 in			E - Shear	1.442 Mpsi

Wall Information

Project Title:
Engineer:
Project ID:
Project Descr:

Torsional Analysis of Rigid Diaphragm

Project File: lateral calcs.ec6

LIC# : KW-06015244, Build:20.22.10.25

CG ENGINEERING

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DESCRIPTION: Rigid Diaphragm Analysis

Label : 6D	X Wall C.G. Location	98 ft	Length	78.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-70.75 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : 6E	X Wall C.G. Location	85.5 ft	Length	5.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-2.83 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	90 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : 6F	X Wall C.G. Location	78.75 ft	Length	3.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-1.83 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	90 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A1	X Wall C.G. Location	7.5 ft	Length	13.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-14.5 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A2	X Wall C.G. Location	34.5 ft	Length	11.1 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	0 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A3	X Wall C.G. Location	52 ft	Length	5.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	0 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A4	X Wall C.G. Location	78.1 ft	Length	2.33 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	0 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A5	X Wall C.G. Location	84 ft	Length	2.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-3 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A6	X Wall C.G. Location	91.67 ft	Length	12.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	0 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : A7	X Wall C.G. Location	91.67 ft	Length	12.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-9.17 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : H1	X Wall C.G. Location	54 ft	Length	88 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-110 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : H2	X Wall C.G. Location	10 ft	Length	20 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-79 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi

Wall Information

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Torsional Analysis of Rigid Diaphragm

Project File: lateral calcs.ec6

LIC# : KW-06015244, Build:20.22.10.25

CG ENGINEERING

(c) ENERCALC INC 1983-2022

DESCRIPTION: Rigid Diaphragm Analysis

Label : H3	X Wall C.G. Location	8 ft	Length	16 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-89 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	10 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : H4	X Wall C.G. Location	15.25 ft	Length	9.25 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-102.5 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi
Label : H5	X Wall C.G. Location	93.5 ft	Length	7.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :	Y Wall C.G. Location	-101.5 ft	Height	13 ft
Along Wall "y" Dir	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	Wall Fixity	Fix-Pin	E - Bending	1.803 Mpsi
			E - Shear	1.442 Mpsi

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

Max Shear along Member Local "y-y" Axis					Max Shear along Member Local "x-x" Axis				
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	
1A	90	-8.83	37.73	298.735	0	-8.83	37.73	0.000	
1B	90	-8.83	37.73	35.544	0	-8.83	37.73	0.000	
1C	90	-8.83	37.73	18.566	0	-8.83	37.73	0.000	
1D	90	-8.83	37.73	4.173	0	-8.83	37.73	0.000	
1E	90	-8.83	37.73	2.504	0	-8.83	37.73	0.000	
6A	120	-8.83	37.73	1.677	0	-8.83	37.73	0.000	
6B	120	-8.83	37.73	0.220	0	-8.83	37.73	0.000	
6C	120	-8.83	37.73	0.285	0	-8.83	37.73	0.000	
6D	120	-8.83	37.73	354.542	0	-8.83	37.73	0.000	
6E	300	-8.83	37.73	1.343	0	-8.83	37.73	0.000	
6F	300	-8.83	37.73	0.450	0	-8.83	37.73	0.000	
A1	0	-8.83	37.73	36.447	90	-8.83	37.73	0.000	
A2	0	-8.83	37.73	23.936	90	-8.83	37.73	0.000	
A3	0	-8.83	37.73	3.792	90	-8.83	37.73	0.000	
A4	0	-8.83	37.73	0.279	90	-8.83	37.73	0.000	
A5	0	-8.83	37.73	0.271	90	-8.83	37.73	0.000	
A6	0	-8.83	37.73	33.423	90	-8.83	37.73	0.000	
A7	0	-8.83	37.73	25.614	90	-8.83	37.73	0.000	
H1	0	-8.83	37.73	471.887	90	-8.83	37.73	0.000	
H2	0	-8.83	37.73	48.181	90	-8.83	37.73	0.000	
H3	0	-8.83	37.73	34.450	90	-8.83	37.73	0.000	
H4	0	-8.83	37.73	6.702	90	-8.83	37.73	0.000	
H5	0	-8.83	37.73	3.779	90	-8.83	37.73	0.000	

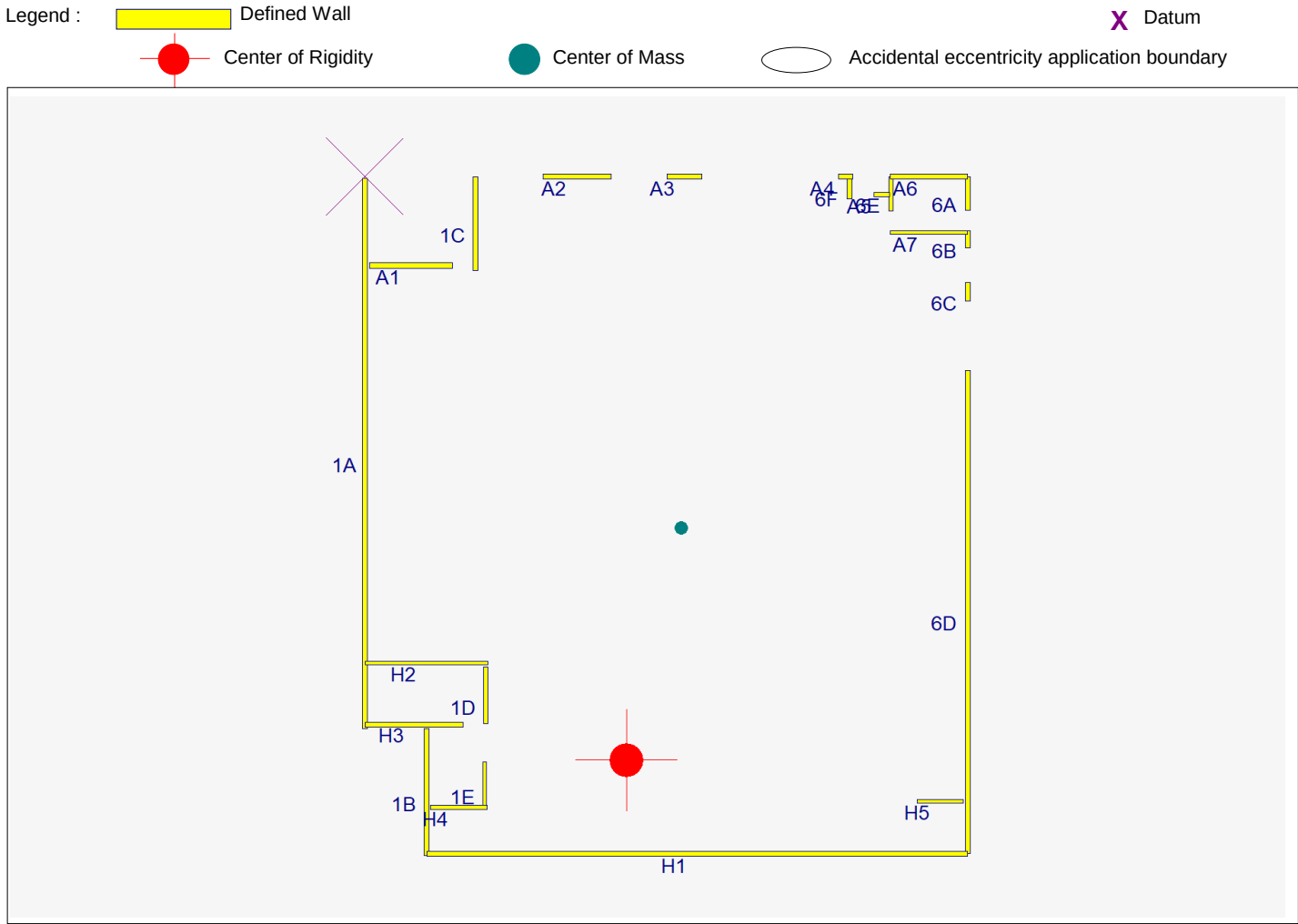
Torsional Analysis of Rigid Diaphragm

Project File: lateral calcs.ec6
(c) ENERCALC INC 1983-2022

LIC# : KW-06015244, Build:20.22.10.25 CG ENGINEERING

DESCRIPTION: Rigid Diaphragm Analysis

Layout of Resisting Elements



Project Title:
Engineer:
Project ID:
Project Descr:

Torsional Analysis of Rigid Diaphragm

Project File: lateral calcs.ec6

LIC# : KW-06015244, Build:20.22.10.25

CG ENGINEERING

(c) ENERCALC INC 1983-2022

DESCRIPTION: Rigid Diaphragm Analysis

Analysis Notes

This program is designed to distribute an applied shear load to a set of resisting elements.

Each resisting element data entry specifies a deflection along a "major" and "minor" axis due to a 1,000 lb load. Each resisting element may be entered as a wall or a column (whereby the deflection is calculated), or as a generic resisting element with specified deflection. The deflections define the stiffness of each resisting element.

Each resisting element is defined at an (X,Y) location from a datum the user has previously defined. A counter-clockwise rotation of the element can be entered with respect to a traditional "+X" axis line.

A main "shear" load and an optional orthogonal shear load are specified for distribution to the system of resisting elements. In addition the maximum orthogonal dimensions of the structure and minimum accidental eccentricity percentage are specified.

From the entered loads the program calculates resultant force vectors for each angular orientation that is requested. The force is applied to the resisting elements in angular increments to generate a series of resulting direct and torsional shear loads on each element. This application of force is then repeated at angular intervals along an elliptical path defined by the minimum accidental eccentricity.

The end result is a table of direct shear and torsional shear values for each element from the iterated angles of load application and accidental eccentricity. These values are then searched to find the maximum major and minor axis shears applied to each resisting element.

Concrete Shear Wall Design

Rose Hill Mukilteo

Formulas/Conditions

$$\phi V_c = \phi(A_{cv})(a_v)(f_c)^{1/2}$$

$$\phi V_s = \phi(A_{cv})(\rho_l)(f_y)$$

$$\phi V_n = \phi V_c + \phi V_s \leq \phi 10 A_{cv}(f_c)^{1/2}$$

Horiz. Hooks Req'd if: $V_u > (A_{cv})(f_c)^{1/2}$

Two Curtains Req'd if: $V_u > 2(A_{cv})(f_c)^{1/2}$

Boundary Elements Req'd if: $M/S + P/A > 0.2f_c$

Variables

ϕ 0.60

ρ_l (min) 0.0025

f_c , psi 4000

f_y , psi 60000

Code Reference

ACI 318-14 18.10.4.1

ACI 318-14 18.10.4.1

ACI 318-14 18.10.4.4

ACI 318-14 18.10.6.5b

ACI 318-14 18.10.2.2

ACI 318-14 18.10.6.3

ACI 318-14 21.2.4.1

ACI 318-14 18.10.2.1

Load Combinations

ASCE 7-16 Sect 12.4.2.3

Combo 5: $*(1.2+0.2S_{DS})D+pQ_E+L+0.2S$

Combo 7: $**(0.9-0.2S_{DS})D+pQ_E+1.6H$

$$S_{DS} = 1.123 \quad \rho = 1.0$$

$$C_s = 0.225$$

$$M_u = Q_E \times h_w$$

$$M_R = w_{DL}l_w^2/2$$

$$T_u = \frac{M_u - M_R}{l_w}$$

$$1.2+0.2S_{DS} = 1.4246$$

$$0.9-0.2S_{DS} = 0.6754$$

		21.9.4										(21-7)										21.9.2.2									
Wall ID	b, in	l _w , in	h _w , in	A _{cv} , in ²	Wall wt, plf	w _{DL} , plf	w _{LL} , plf	w _{SL} , plf	Q _E , kips	H, kips	V _u *, k (max)	V _u , psi (max)	Pu*, k	M _u **, k-ft (max)	T _u **, k	max comp stress, psi	a _c	φVc	φVs	φVn	(A _{cv})(f _c) ^{1/2} , k	Shear Strength?	Horiz. Hooks?	Two Curtains?	Boundary Elements?	Jamb Steel Areq	c, in	wall boundary, in	boundary area, in ²	boundary, ρ	boundary reinf?
1A	10	1072	156	10720	2063	0	0	0	299.0	0	299.0	27.9	262.5	3887.0	0.0	48.8	3.0	1220.3	964.8	2185	678.0	ok	no	no	no	0.00	255.2	148.03	120.00	0.000	no
1B	10	248	156	2480	2063	0	0	0	35.5	0	35.5	14.3	60.7	461.5	7.9	78.5	3.0	282.4	223.2	506	156.9	ok	no	no	no	0.15	59.1	34.25	120.00	0.001	no
1C	10	184	156	1840	2063	0	0	0	18.6	0	18.6	10.1	45.0	241.8	5.1	75.9	3.0	209.4	165.6	375	116.3	ok	no	no	no	0.09	43.8	25.40	120.00	0.001	no
1D	8	111	156	888	1738	0	0	0	4.2	0	4.2	4.7	22.9	54.6	0.5	65.7	3.0	101.1	79.9	181	56.2	ok	no	no	no	0.01	26.4	15.33	96.00	0.000	no
1E	8	92	156	736	1738	0	0	0	2.5	0	2.5	3.4	19.0	32.5	0.0	60.3	2.0	55.9	66.3	122	46.6	ok	no	no	no	0.00	21.9	12.71	96.00	0.000	no
6A	10	66	156	660	2063	0	0	0	1.7	0	1.7	2.6	16.2	22.1	0.2	61.0	2.0	50.1	59.4	109	41.7	ok	no	no	no	0.00	15.7	12.00	120.00	0.000	no
6B	10	33	156	330	2063	0	0	0	0.2	0	0.2	0.6	8.1	2.6	0.0	41.7	2.0	25.0	29.7	55	20.9	ok	no	no	no	0.00	7.9	12.00	120.00	0.000	no
6C	10	36	156	360	2063	0	0	0	0.3	0	0.3	0.8	8.8	3.9	0.0	46.2	2.0	27.3	32.4	60	22.8	ok	no	no	no	0.00	8.6	12.00	120.00	0.000	no
6D	10	942	156	9420	2063	0	0	0	354.0	0	354.0	37.6	230.7	4602.0	3.9	61.8	3.0	1072.4	847.8	1920	595.8	ok	no	no	no	0.07	224.3	130.09	120.00	0.001	no
6E	8	68	156	544	1738	0	0	0	1.3	0	1.3	2.4	14.0	16.9	0.0	58.6	2.0	41.3	49.0	90	34.4	ok	no	no	no	0.00	16.2	12.00	96.00	0.000	no
6F	10	44.04	156	440	2063	0	0	0	0.5	0	0.5	1.1	10.8	6.5	0.0	48.6	2.0	33.4	39.6	73	27.9	ok	no	no	no	0.00	10.5	12.00	120.00	0.000	no
A1	10	162	156	1620	2063	0	0	0	36.4	0	36.4	22.5	39.7	473.2	25.6	154.3	3.0	184.4	145.8	330	102.5	ok	no	no	no	0.47	38.6	22.37	120.00	0.004	no
A2	10	133.2	156	1332	2063	0	0	0	23.9	0	23.9	17.9	32.6	310.7	20.3	150.6	3.0	151.6	119.9	272	84.2	ok	no	no	no	0.38	31.7	18.39	120.00	0.003	no
A3	10	68	156	680	2063	0	0	0	3.8	0	3.8	5.6	16.7	49.4	4.8	101.3	2.0	51.6	61.2	113	43.0	ok	no	no	no	0.09	16.2	12.00	120.00	0.001	no
A4	10	28	156	280	2063	0	0	0	0.3	0	0.3	1.1	6.8	3.9	0.1	60.4	2.0	21.2	25.2	46	17.7	ok	no	no	no	0.00	6.7	12.00	120.00	0.000	no
A5	8	30	156	240	1738	0	0	0	0.3	0	0.3	1.3	6.2	3.9	0.1	64.8	2.0	18.2	21.6	40	15.2	ok	no	no	no	0.00	7.1	12.00	96.00	0.000	no
A6	10	152	156	1520	2063	0	0	0	33.4	0	33.4	22.0	37.2	434.2	25.4	159.7	3.0	173.1	136.8	310	96.2	ok	no	no	no	0.47	36.2	21.00	120.00	0.004	no
A7	8	152	156	1216	1738	0	0	0	25.6	0	25.6	21.0	31.4	332.8	18.8	155.4	3.0	138.5	109.5	248	76.9	ok	no	no	no	0.35	36.2	21.00	96.00	0.004	no
H1	10	1056	156	10560	2063	0	0	0	472.0	0	472.0	44.7	258.6	6136.0	8.4	64.1	3.0	1202.2	950.4	2153	667.9	ok	no	no	no	0.16	251.4	145.83	120.00	0.001	no
H2	8	240	156	1920	1738	0	0	0	48.2	0	48.2	25.1	49.5	626.6	19.6	123.7	3.0	218.6	172.8	391	121.4	ok	no	no	no	0.36	57.1	33.14	96.00	0.004	no
H3	10	192	156	1920	2063	0	0	0	34.5	0	34.5	18.0	47.0	448.5	16.9	112.1	3.0	218.6	172.8	391	121.4	ok	no	no	no	0.31	45.7	26.51	120.00	0.003	no
H4	8	111	156	888	1738	0	0	0	6.7	0	6.7	7.5	22.9	87.1	4.0	89.4	3.0	101.1	79.9	181	56.2	ok	no	no	no	0.07	26.4	15.33	96.00	0.001	no
H5	8	90	156	720	1738	0	0	0	3.8	0	3.8	5.3	18.6	49.4	2.2	80.7	2.0	54.6	64.8	119	45.5	ok	no	no	no	0.04	21.4	12.43	96.00	0.000	no

(2)#6 JAMB STEEL
ADEQUATE

SECTION NO. 4

Rose Hill Mukilteo
3rd & Park Ave
Mukilteo, WA 98275

SECTION TITLE: PT Design

SECTION INDEX

DESCRIPTION	PAGE
Structure Properties	4.1-4.5
Design Loads	4.6-4.7
Load Key Plan	4.8
RAM Concept Output	4.9-4.19
Reaction Key Plan	4.20
Shear Stud Design	4.21-4.24

Two-Way PT Slab Design

Slab Location = Level 2
Fire Resistance Rating = 2 HR

Structure Properties

Slab
f'c = 5000 psi
f'ci = 3000 psi
Fy = 60 ksi
Fpu = 270 ksi

Walls
height = 12.5 ft Measured from centroid of slab to far end of wall
f'c = 4000 psi
Fy = 60 ksi

Columns
height = 12.5 ft Measured from centroid of slab to far end of column
f'c = 4000 psi
Fy = 60 ksi
Fixed near, pinned far

Cover Requirements [ACI Chapter 20 and IBC Chapter 7]

-refer to following sheets for cover requirements

Minimum Negative Bonded Reinforcement [ACI Table 8.6.2.3]

Width = 25.5 ft (typical column trib width)
Slab t = 12 in
A_{cf} = 3672 in²
A_{s, min} = 2.754 in² Use (10) #5
Top bar length = 10.5 ft Use 12ft, staggered

-This is a typical example; RAM Concept will check these requirements; Refer to RAM Concept Output

Minimum Positive Bonded Reinforcement [ACI Table 8.6.2.3]

A_{s, min}⁺ = provided by RAM Concept where tensile stress $\geq 2\sqrt{f'c}$

-CG standard is to provide #4@24" oc each way, bottom of slab; Additional bottom bars required by RAM Concept will be added on plan



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Two-Way PT Slab Design	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	4.1
Project	Rose Hill Mukilteo	Job No.			
		22332.10			

Integrity Bars - Bottom Bars at Columns

-Integrity bottom bars are added at each column; The bars are designed to hold the DL tributary to the columns

Slab Description	DL psf	Trib Area ft ²	SDL kips	Bar Area in ²	# 5 Bars total	#5 Bars per side
12" under units	231	535	124	2.2886111	7	2
14" under units	280	535	150	2.7740741	9	2
14.75" under units	290	535	155	2.8731481	9	2
15" under units	293	535	157	2.9028704	9	2
22.25" under units	460	535	246	4.5574074	15	4



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	PT Slab Integrity Bars	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	4.2
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

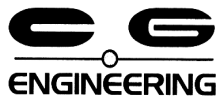
Minimum Distributed Tendon Spacing

Distributed Tendons									
Model Spacing			Plan Marks			Precompression [ksi]			
#T	[in]		#T	[in]		8.75" slab	12" slab	15" slab	25.25" slab
2.67	48	0.0556	2	36	0.0556	0.170	0.124	0.099	0.059
3	48	0.0625	3	48	0.0625	0.191	0.140	0.112	0.066
3.92	47	0.0834	4	48	0.0833	0.255	0.186	0.149	0.088
4	48	0.0833	4	48	0.0833	0.255	0.186	0.149	0.088
4.5	48	0.0938	3	32	0.0938	0.287	0.209	0.168	0.100
4.8	48	0.1000	4	40	0.1000	0.306	0.223	0.179	0.106
5	48	0.1042	5	48	0.1042	0.319	0.233	0.186	0.111
5.45	48	0.1135	5	44	0.1136	0.348	0.254	0.203	0.121
5.71	48	0.1190	5	42	0.1190	0.365	0.266	0.213	0.126
6.00	48	0.1250	5	40	0.1250	0.383	0.279	0.223	0.133
6.32	48	0.1316	5	38	0.1316	0.403	0.294	0.235	0.140
6.67	48	0.1390	5	36	0.1389	0.425	0.310	0.248	0.147

*min precompression per ACI 8.6.2.1 is 125psi; CG Standard is to use 150psi

*max precompression is 300psi

*maximum tendon spacing is 8H or 5ft per ACI 8.7.2.3



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Minimum Distributed Tendon Spacing	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

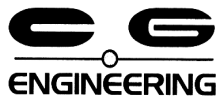
4.3

Minimum Banded Tendons

Banded Tendons							
#T	Tributary Width [feet]	Precompression [ksi]					
		8.75" Slab	12" Slab	15" Slab	19" Slab	25.25" slab	26" Slab
22	21	0.267	0.195	0.156	0.123	0.093	0.090
34	22	0.394	0.288	0.230	0.182	0.137	0.133
38	25	0.388	0.283	0.226	0.179	0.134	0.131
24	26	0.236	0.172	0.137	0.109	0.082	0.079

*min precompression per ACI 8.6.2.1 is 125psi; CG Standard is to use 150psi

*max precompression is 300psi



250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Minimum Banded Tendons	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

4.4

2hr PT Slab Tendon CGS Summary

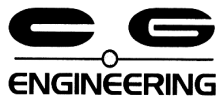
Reinforcing	Cover	
	End Span	Interior Span
PT Tendons	2	1
Mild Reinforcing	1	1

*use 1" minimum top cover for non bearing wall attachments

*values shown meet siliceous aggregate requirements

Distributed Direction Reinforcing	CGS	
	End Span	Interior Span
Top Profile	slab CG	1.75
Bottom Profile	2.25	1.25

Banded Direction Reinforcing	CGS	
	End Span	Interior Span
Top Profile	slab CG	1.25
Bottom Profile	2.25	1.25



250 4th Ave. South
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Edmonds, WA 98020

Description	PT Cover Requirements	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

4.5

PT Slab Design Loads

- Self weight of slab is included in RAM Concept model
- Determine total area loads supported on slab from wood building above
- Cumulative area loads are used where wood framing above consists of stacking unit framing
- Line and point loads are used in the common areas where there are large steel beams and column loads
- Refer to Load Key Plans for layout of area, line and point loads

Live Load Reduction [IBC 1607.11.2]

Trib Area = 535 ft²
R = 30.8 % Use 20%

Live Load Patterning [ACI 6.4.3.2]

LL/DL Ratio = 61 % ≤ 75% No live load patterning required

Area Loads

Type R1 - Stacking Units & Corridors				
LVL	SIDL	SL	LL	LL _R
Roof	16	25	0	0
2nd	35	0	40	32
1st	30	0	40	32
Σ =	81	25	80	64

-typical floor DL = 22 psf + 10 psf for wall DL

-SIDL at 1st floor = 20 psf + 10 psf for wall DL

Type D1 - Stacking Units and Decks				
LVL	SIDL	SL	LL	LL _R
Roof	16	25	0	0
2nd	50	0	60	0
1st	70	0	60	0
Σ =	136	25	120	0

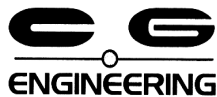
-typical floor DL = 22 psf + 10 psf for wall DL

-SIDL at 1st floor = 20 psf + 10 psf for wall DL

Type C1 - Stacking Units and Commercial Space				
LVL	SIDL	SL	LL	LL _R
Roof	16	25	0	0
2nd	35	0	40	0
1st	30	0	100	0
Σ =	81	25	140	0

-typical floor DL = 22 psf + 10 psf for wall DL

-SIDL at 1st floor = 20 psf + 10 psf for wall DL



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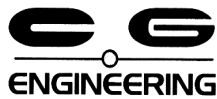
Description	PT Slab Design Loads	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

4.6

Area Loads

Type R2 - Roof Deck				
LVL	SIDL	SL	LL	LL _R
Roof Deck	40	25	100	0
2nd	35	0	40	0
1st	30	0	40	0
Σ =	105	25	180	0

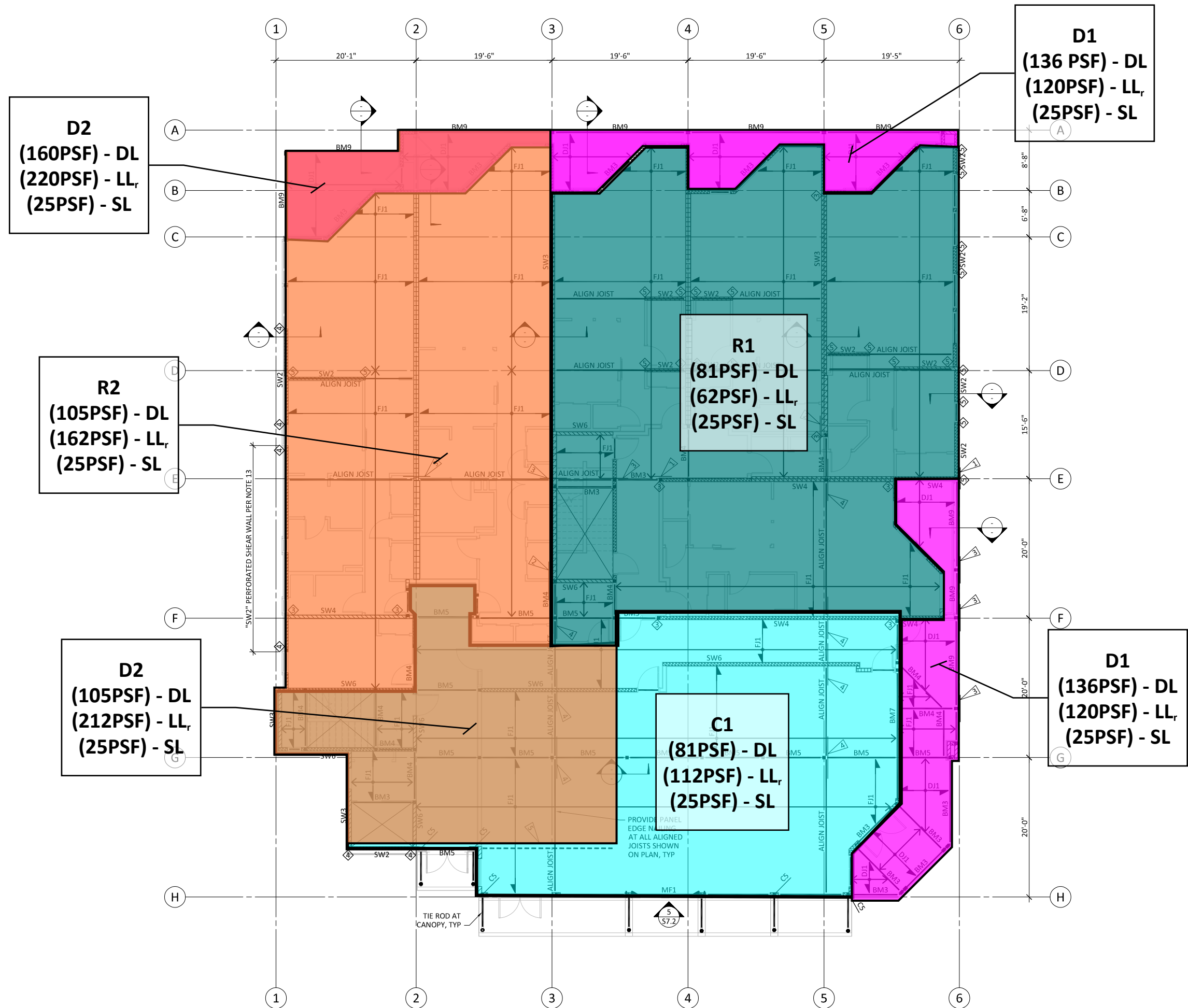
Type D2 - Roof Deck w/ Stacking Unit Decks				
LVL	SIDL	SL	LL	LL _R
Roof Deck	40	25	100	0
2nd	50	0	40	0
1st	70	0	40	0
Σ =	160	25	180	0



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Description	PT Slab Design Loads	By	CGP	Date	01/26/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.			
			22332.10		

4.7

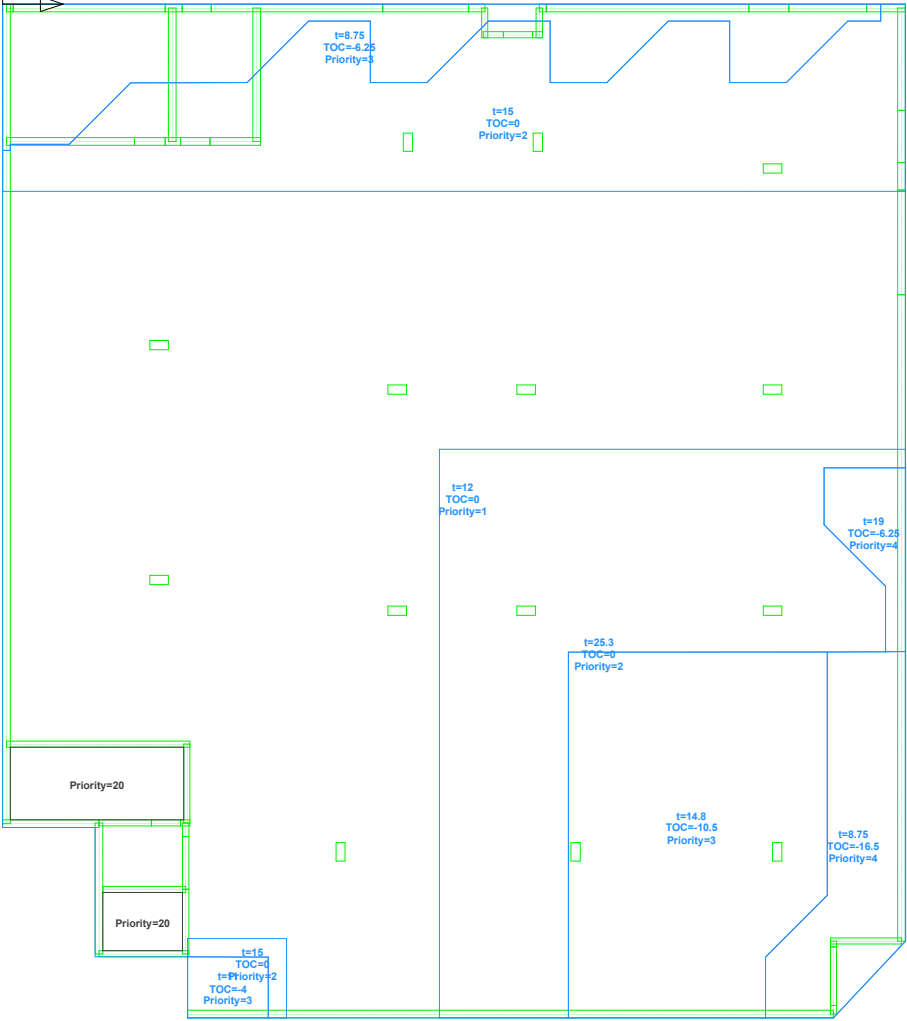


P/T SLAB AREA LOADS KEY PLAN

Mesh Input: Standard Plan

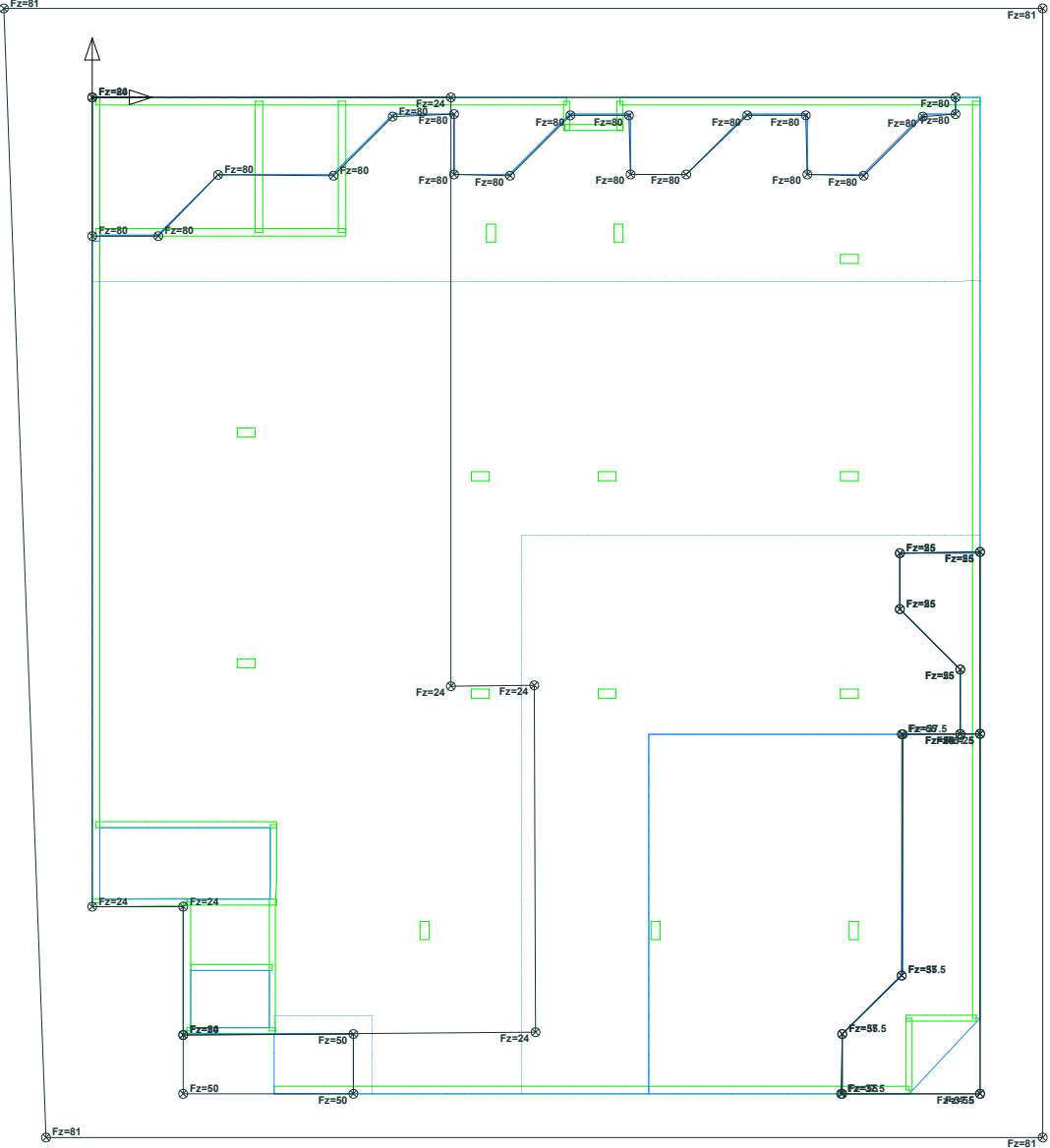
Mesh Input: Beams; Beam Priorities; Slab Areas; Slab Area Thicknesses; Slab Area Elevations; Slab Area Priorities; Slab Openings; Slab Opening Priorities; Point Supports; Point Support Icons; Line Supports; Line Support Icons; Walls Above; Walls Below; Columns Above; Columns Below; Point Springs; Point Spr

Scale = 1:250



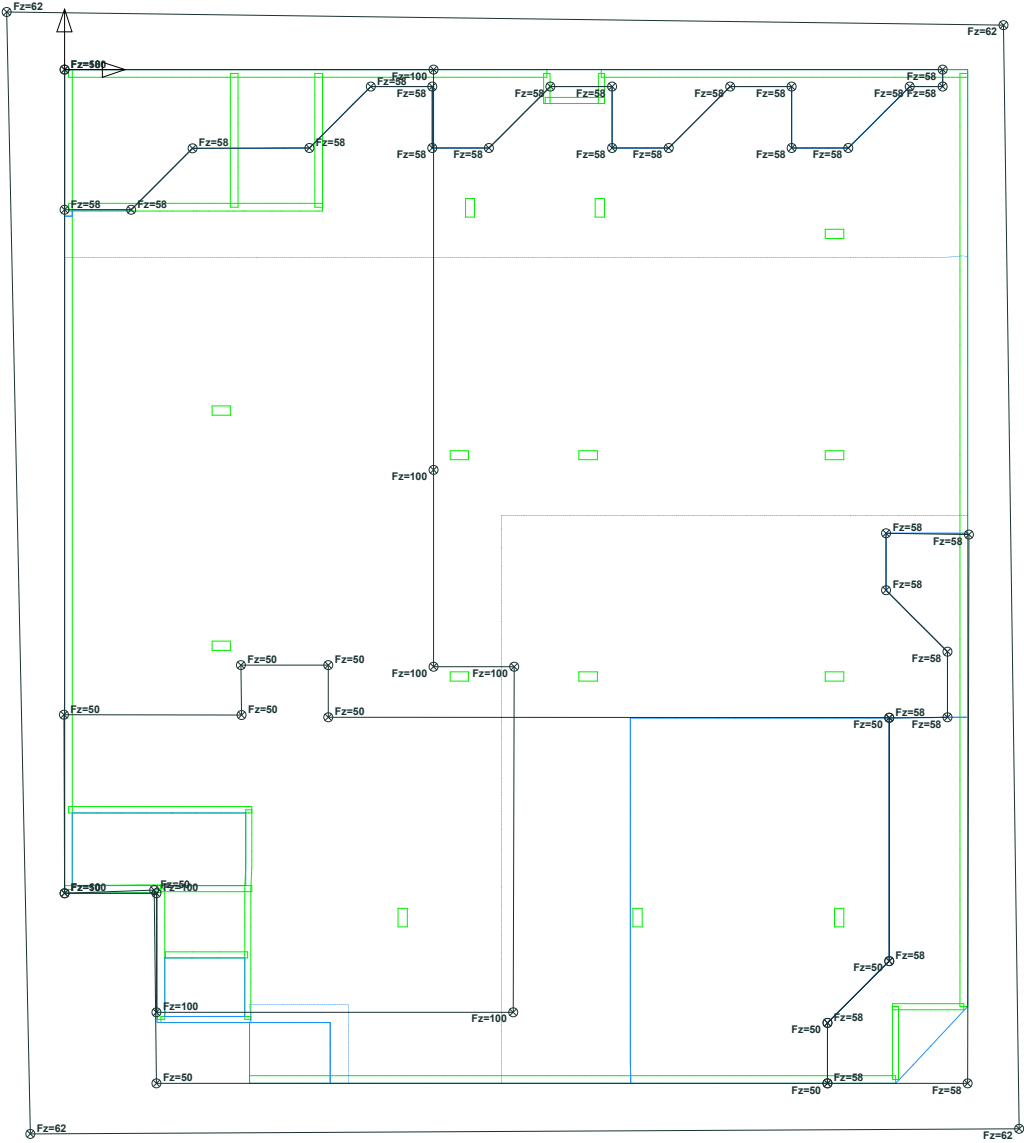
Other Dead Loading: All Loads Plan

Other Dead Loading: Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; User Notes; User Lines; User Dimensions;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Scale = 1:250



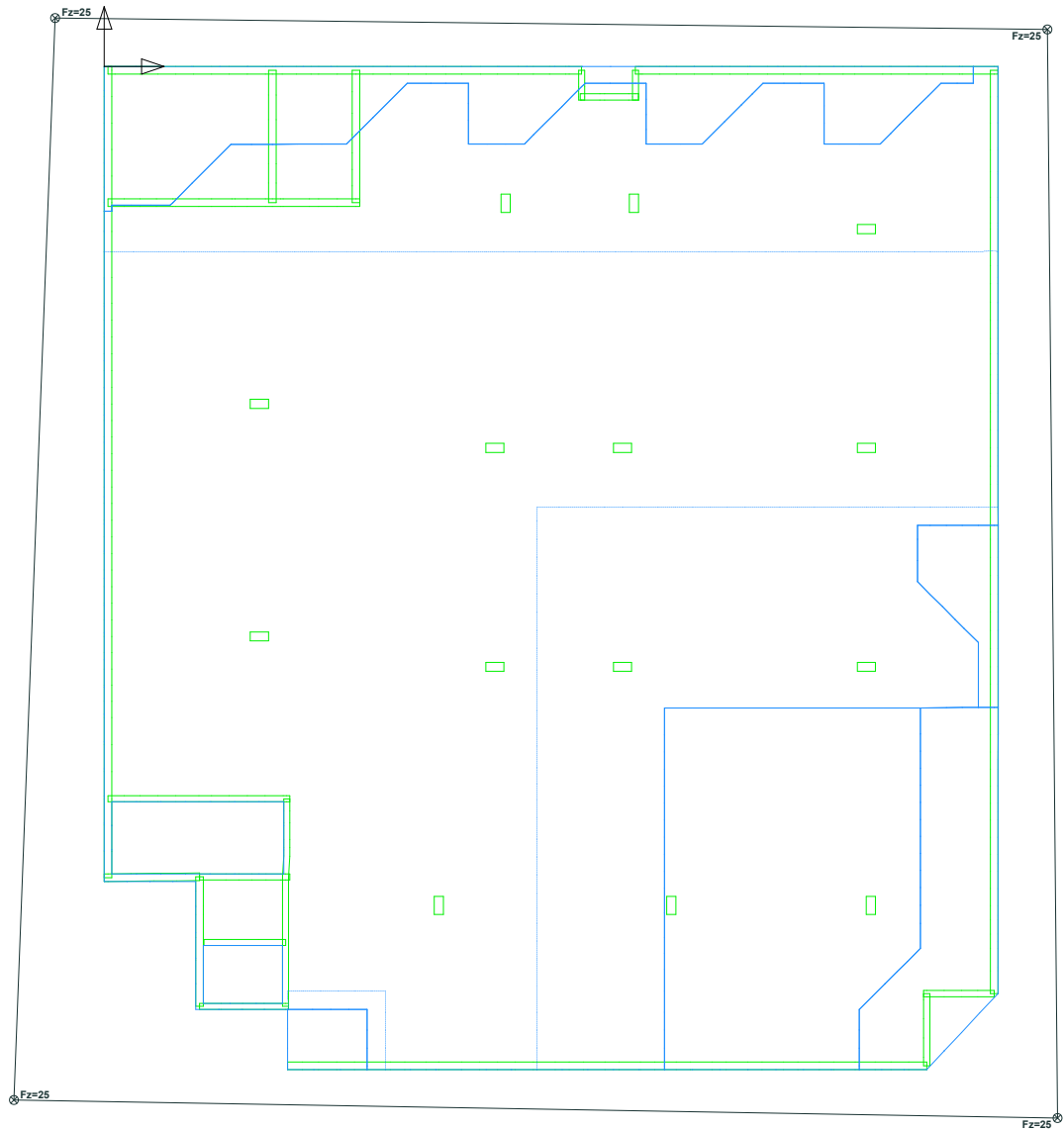
Live (Unreducible) Loading: All Loads Plan

Live (Unreducible) Loading: Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; User Notes; User Lines; User Dimensions;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Scale = 1:250



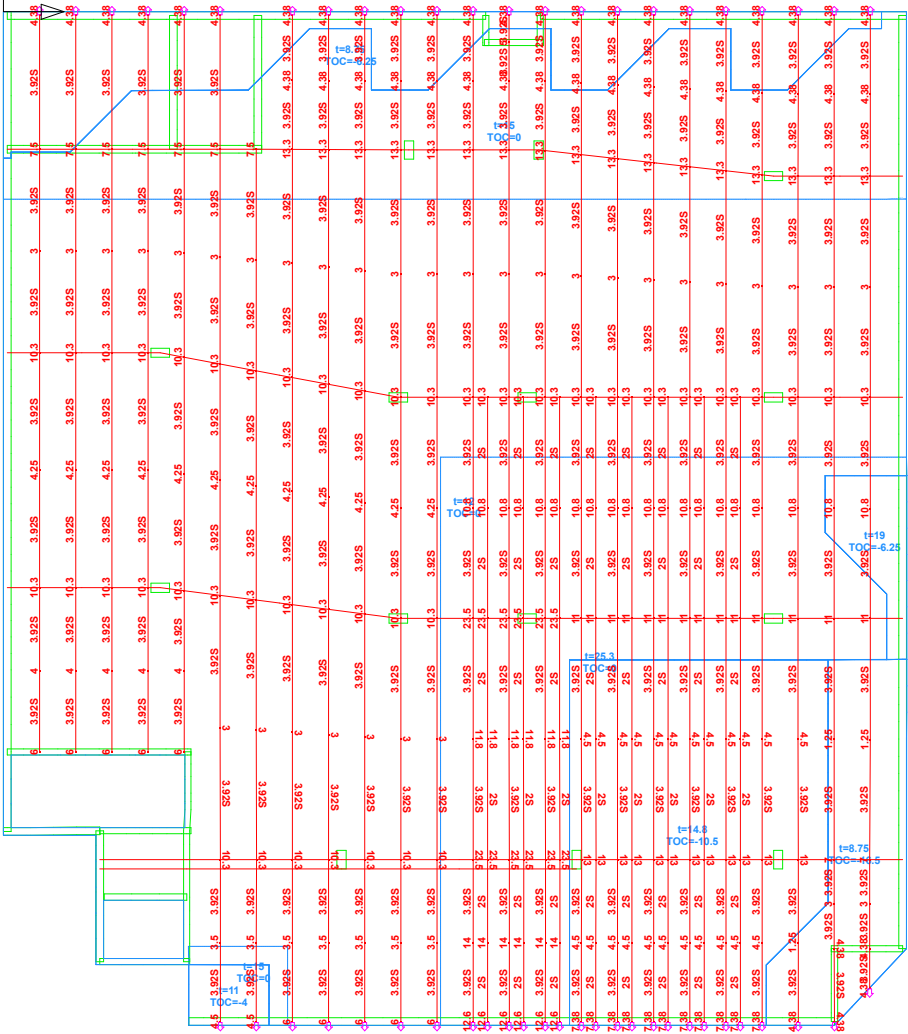
Snow Loading: All Loads Plan

Snow Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250

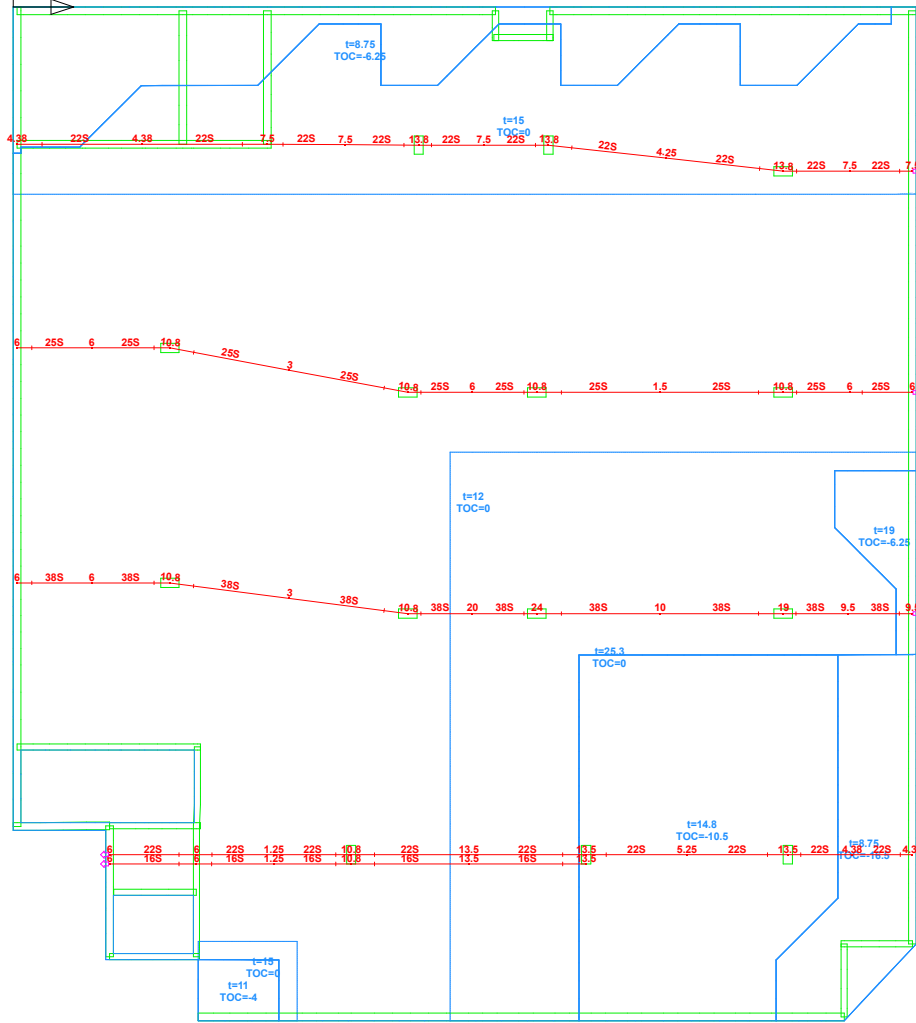


Manual Latitude Tendon: Standard Plan

Manual Latitude Tendon: Tendons; Num Strands; Jacks; Tendon Points; Profile Values;
Mesh Input: Slab Areas; Slab Area Thicknesses; Slab Area Elevations;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Latitude Tendon Parameters: Banded Tendons; Banded Tendon Description; Distributed Tendon Quadrilateral; Distributed Tendon Description; Distributed Tendon Overlap Areas; Profile Points; Profile Elevation Values; Profile Nodes; Jack Region;
Manual Longitude Tendon: Tendons;
Scale = 1:250

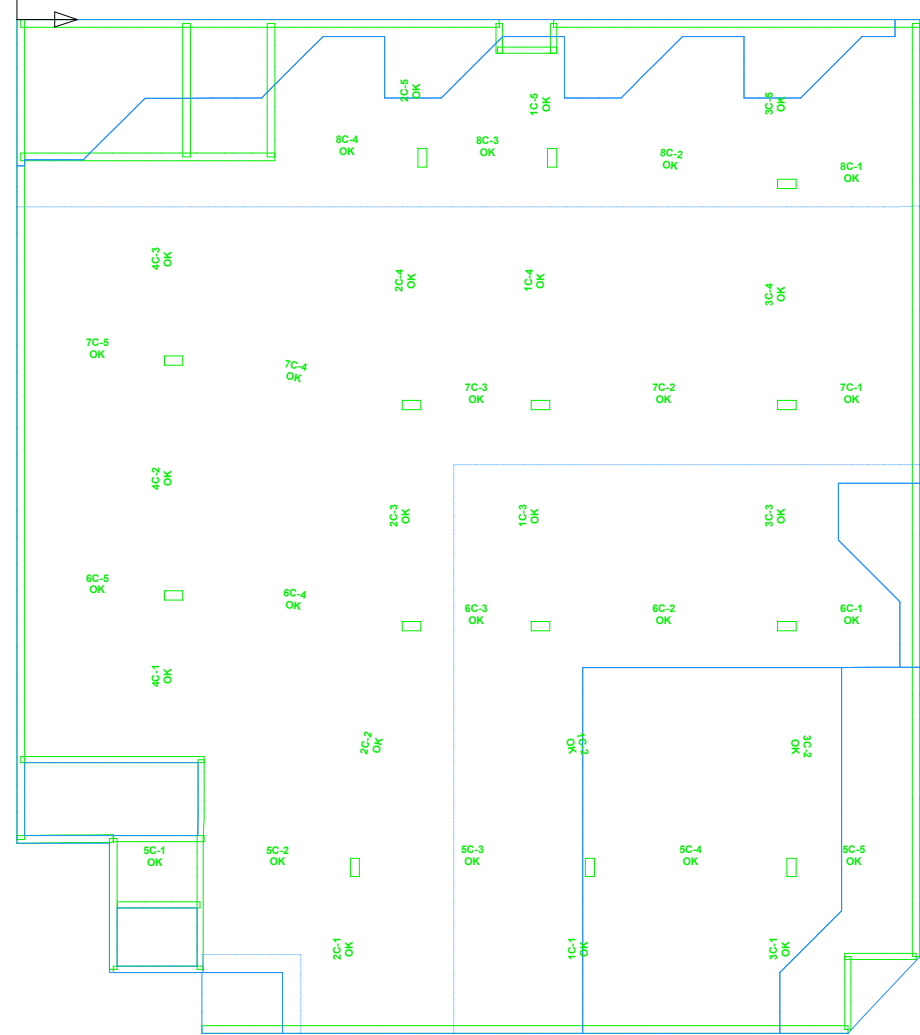


Scale = 1:250



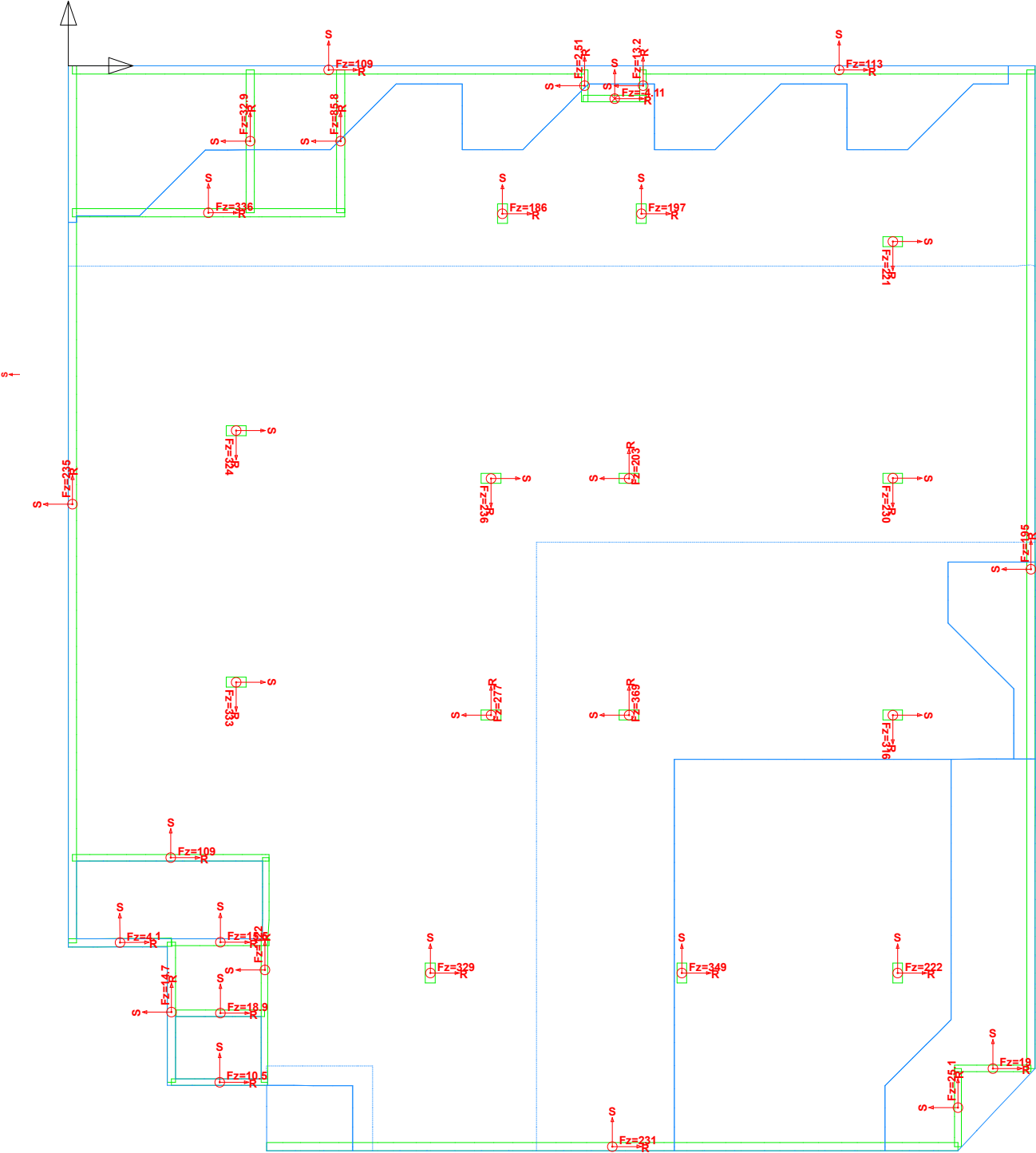
Design Status: Status Plan

Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; Latitude Span Segment D
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250



Factored LC: 1.2D + 1.6L + 0.5Lr: Std Reactions Plan

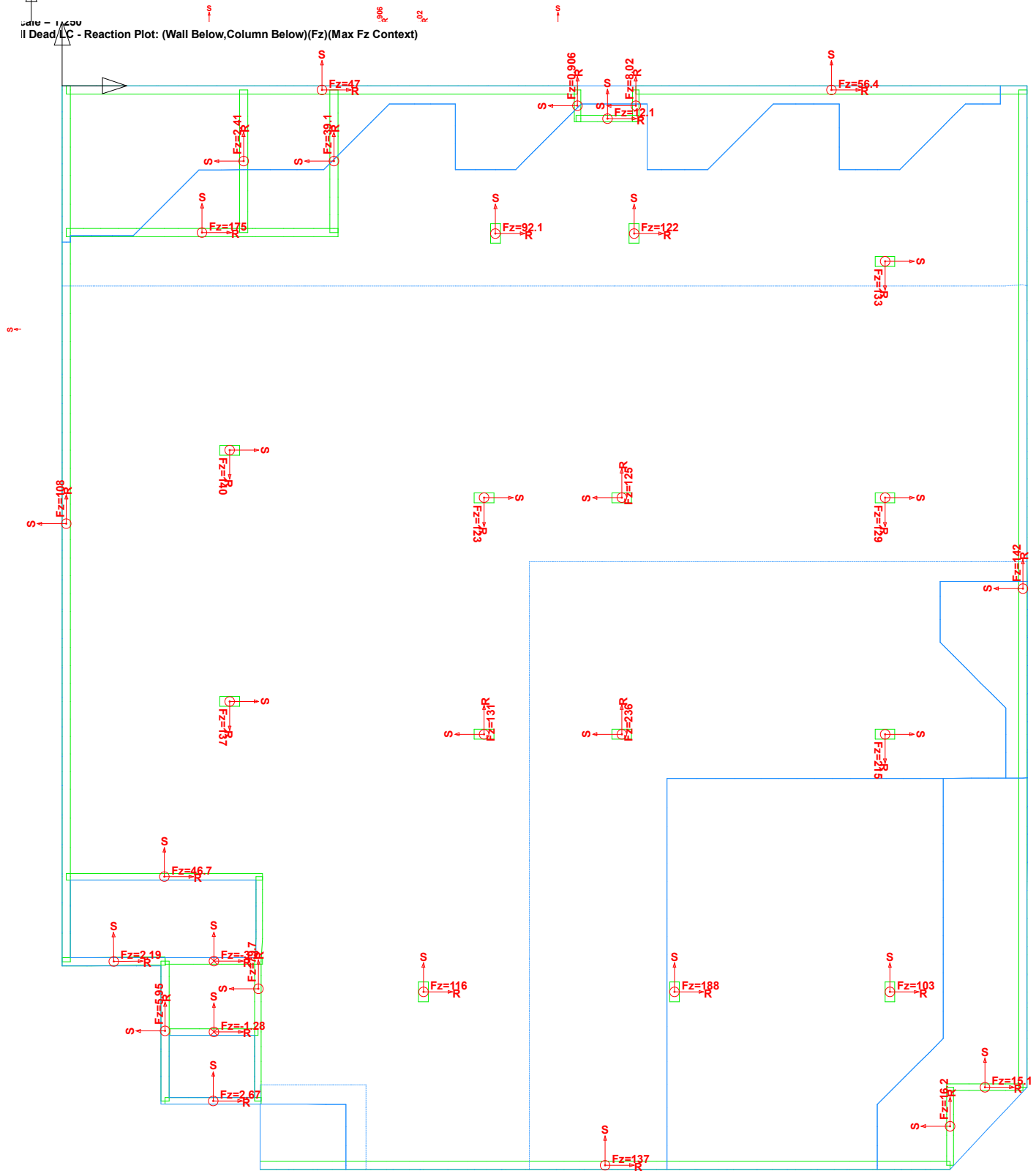
Factored LC: 1.2D + 1.6L + 0.5Lr: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250
Factored LC: 1.2D + 1.6L + 0.5Lr - Reaction Plot: (Wall Below,Column Below,Point Spring,Line Spring,Point Support,Line Support)(Fz)(Standard Context)



WORST CASE FACTORED LOADS PLANS

4.16

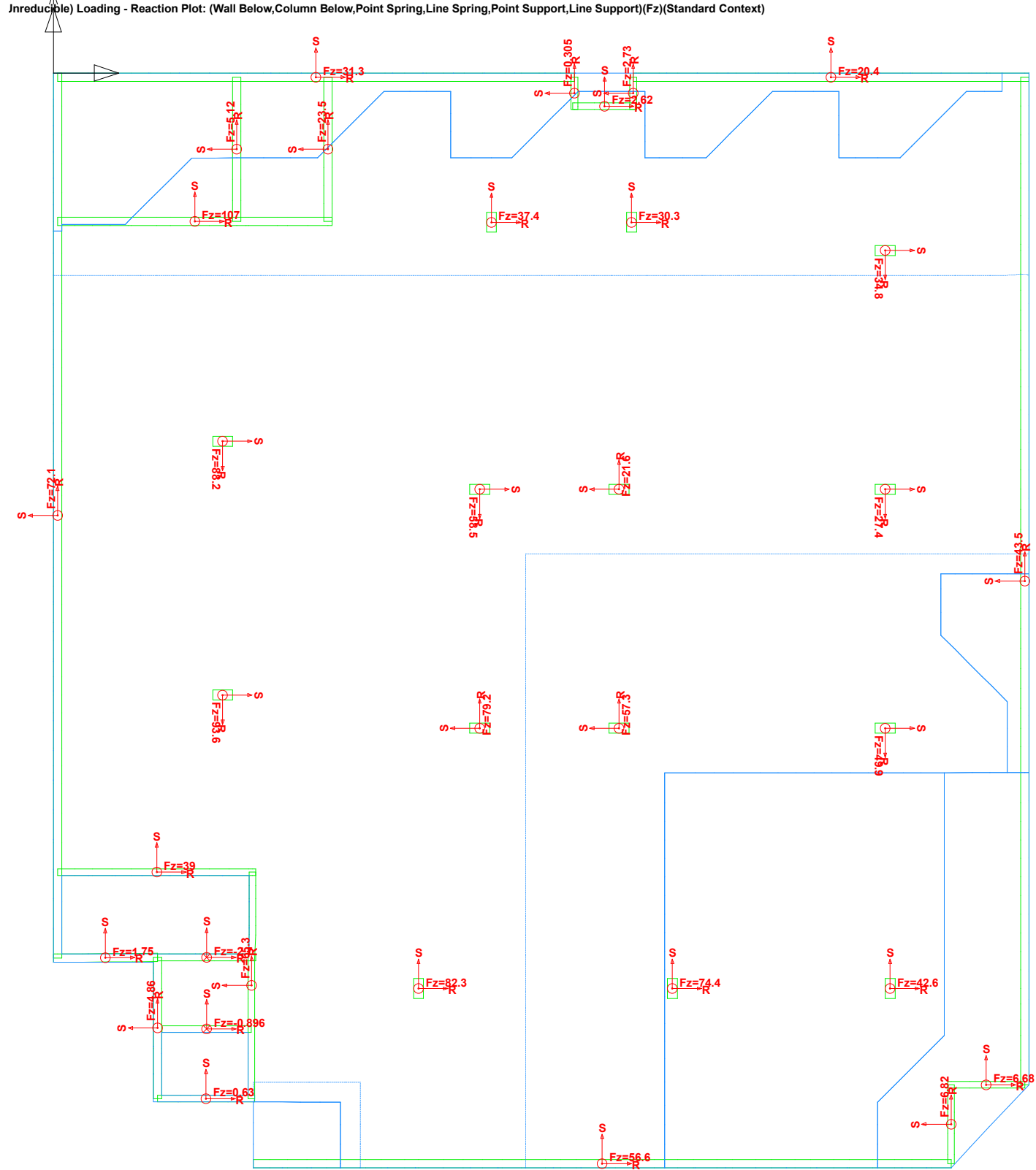
II Dead LC - Reaction Plot: (Wall Below,Column Below)(Fz)(Max Fz Context)



4.17

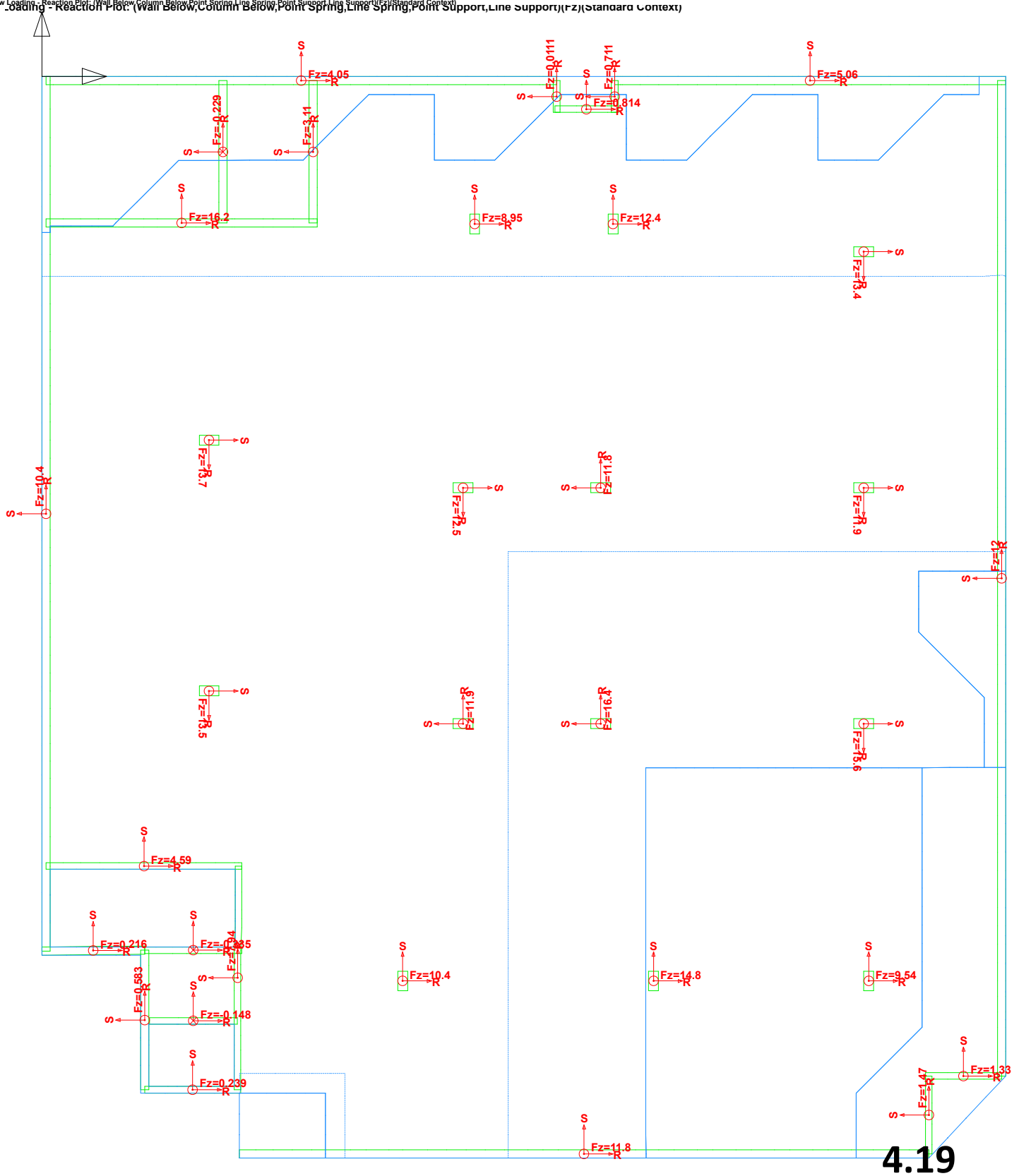
Live (Unreducible) Loading: Std Reactions Plan

Live (Unreducible) Loading: Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Load Icons; Area Load Values; User Notes; User Lines; User Dimensions;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Scale = 1:250
Live (Unreducible) Loading - Reaction Plot: (Wall Below,Column Below,Point Spring,Line Spring,Point Support,Line Support)(Fz)(Standard Context)

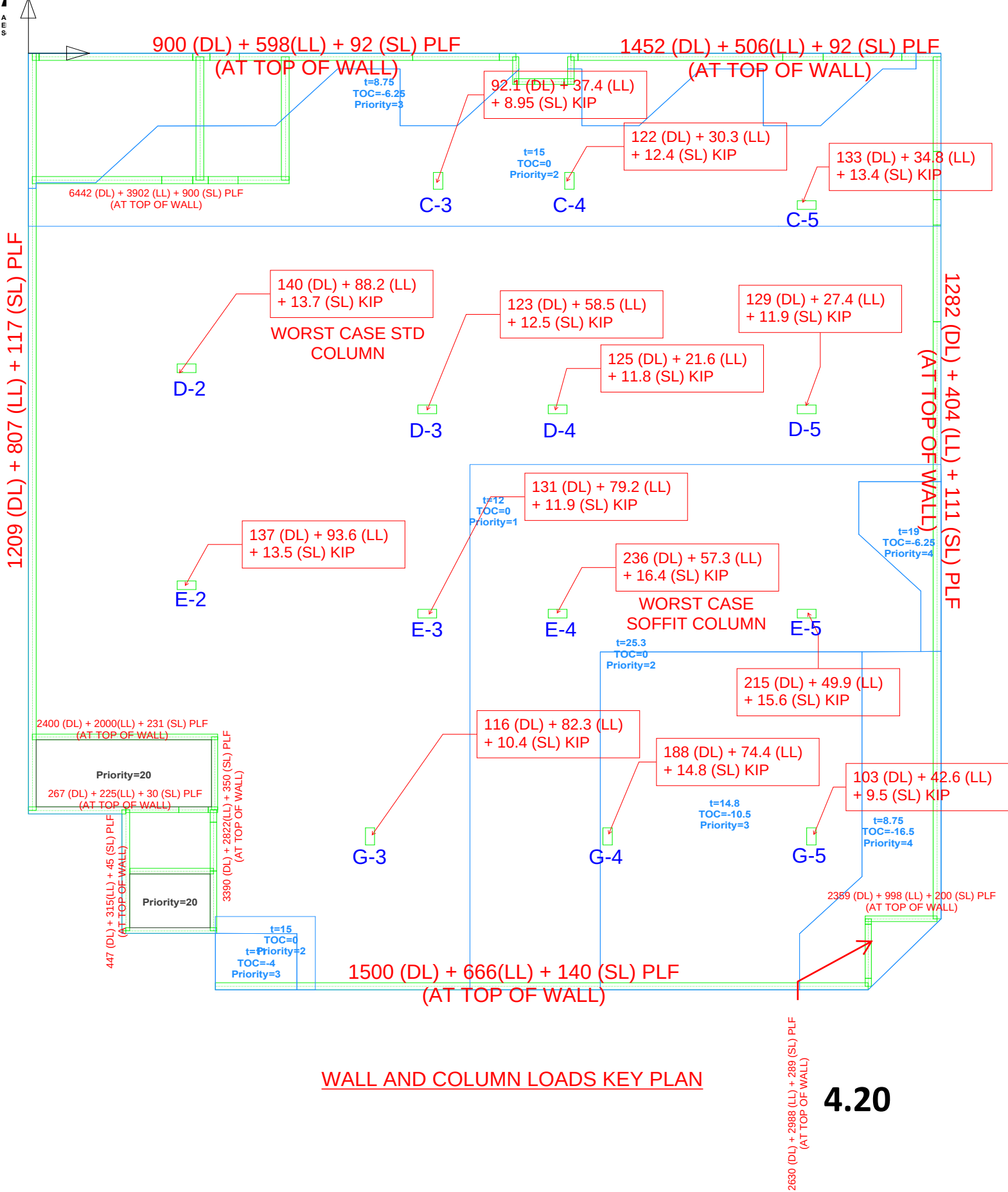


4.18

Scale = 1:250
Snow Loading - Reaction Plot: (Wall Below, Column Below, Point Spring, Line Spring, Point Support, Line Support)(Fz)(Standard Context)



All Dead LC: Std Reactions Plan



Flat Slab Shear Stud Design - Interior Column

Project: **Rose Hill Mukilteo**
Project #: **22332**

Description: **12 x 24 column at LVL 1 (12" slab)**

Engineer: **CGP**
Date: **1/26/23**

Max nominal shear strength at d/2 w/ stud reinforcement, $v_n =$ **8** $\sqrt{f_c}$
Max nominal shear strength at d/2 w/ stud reinforcement, $v_c =$ **3** $\sqrt{f_c}$

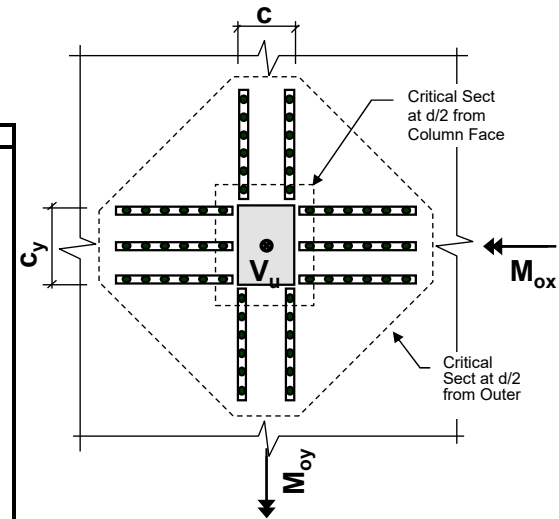
Summary	v_u/ϕ	v_n
d/2 from col	5.86	7.59 $\sqrt{f_c}$
d/2 from outer studs	1.41	2.00 $\sqrt{f_c}$

Input		
$f_c =$	5	ksi
avg prestress =	288	psi
$V_u =$	331	k
$M_{ox} =$	49.5	k-ft
$M_{oy} =$	17.4	k-ft
$c_x =$	12	in
$c_y =$	24	in
$h =$	12	in
$d =$	10.375	in
<u>Shear Studs</u>		
stud $f_y =$	50	ksi
min # rails =	10	
provided # rails =	12	
# studs per rail =	12	
stud diameter =	0.625	in
stud area =	0.31	in ²
total stud area =	3.68	in ²
first stud spacing, $s_o =$	4.00	in
stud spacing, $s =$	5.00	in
rail length =	55.00	in

Crit Sect @ d/2 from col		
$b_o =$	113.50	in
$L_x =$	22.38	in
$L_y =$	34.38	in
$A_c =$	1,178	in ²
$I_x =$	207,390	in ⁴
$I_y =$	108,644	in ⁴
$Y_{vx} =$	0.45	
$Y_{vy} =$	0.35	
$X =$	11.19	in
$Y =$	17.19	in
$v_u =$	311	psi
$v_u/\phi =$	415	psi
=	5.86	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	5.66	$\sqrt{f_c}$
$v_n (pt) =$	4.72	$\sqrt{f_c}$
$v_n =$	334	psi
=	4.72	$\sqrt{f_c}$

Crit Sect @ d/2 w/ Reinf		
$v_c =$	3	$\sqrt{f_c}$
=	212	psi
$v_s =$	324	psi
$v_n =$	536	psi
=	7.59	$\sqrt{f_c}$
Critical Section @ Outer Studs		
$Y_{vx} =$	0.47	
$Y_{vy} =$	0.34	

Crit sect @ d/2 beyond studs		
$L_{x1} =$	16.30	in
$L_{x2} =$	140.38	in
$L_{y1} =$	28.30	in
$L_{y2} =$	152.38	in
$L =$	87.74	in
$X_a =$	70.19	in
$Y_a =$	14.15	in
$X_b =$	8.15	in
$Y_b =$	76.19	in
$I_x =$	#####	in ⁴
$I_y =$	9,653,527	in ⁴
$b_o =$	440.13	in
$A_c =$	4,566	in ²
$v_{ua} =$	73	psi
$v_{ua}/\phi =$	98	psi
=	1.38	$\sqrt{f_c}$
$v_{ub} =$	75	psi
$v_{ub}/\phi =$	99	psi
=	1.41	$\sqrt{f_c}$



NOTES:

1 Slab shear design in accordance with ACI 11.11.5.

Flat Slab Shear Stud Design - Interior Column

Project: **Rose Hill Mukilteo**
Project #: **22332**

Description: **12 x 24 column at LVL 1 (15" slab)**

Engineer: **CGP**
Date: **1/26/23**

Max nominal shear strength at d/2 w/ stud reinforcement, $v_n = 8 \sqrt{f_c}$
Max nominal shear strength at d/2 w/ stud reinforcement, $v_c = 3 \sqrt{f_c}$

Input
8
3

Summary	v_u/ϕ	v_n
d/2 from col	2.75	6.46
d/2 from outer studs	0.84	2.00

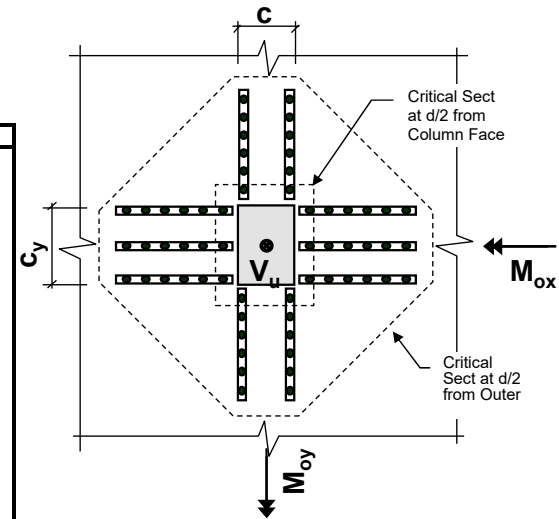
Input		
$f_c =$	5	ksi
avg prestress =	288	psi
$V_u =$	228	k
$M_{ox} =$	33	k-ft
$M_{oy} =$	1.94	k-ft
$c_x =$	12	in
$c_y =$	24	in
$h =$	15	in
$d =$	13.375	in
<u>Shear Studs</u>		
stud $f_y =$	50	ksi
min # rails =	8	
provided # rails =	8	
# studs per rail =	12	
stud diameter =	0.625	in
stud area =	0.31	in ²
total stud area =	2.45	in ²
first stud spacing, $s_o =$	5.00	in
stud spacing, $s =$	4.00	in
rail length =	44.00	in

Crit Sect @ d/2 from col		
$b_o =$	125.50	in
$L_x =$	25.38	in
$L_y =$	37.38	in
$A_c =$	1,679	in ²
$I_x =$	353,428	in ⁴
$I_y =$	197,359	in ⁴
$Y_{vx} =$	0.45	
$Y_{vy} =$	0.35	
$X =$	12.69	in
$Y =$	18.69	in
$v_u =$	146	psi
$v_u/\phi =$	194	psi
=	2.75	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	6.26	$\sqrt{f_c}$
$v_n (pt) =$	4.72	$\sqrt{f_c}$
$v_n =$	334	psi
=	4.72	$\sqrt{f_c}$

Crit Sect @ d/2 w/ Reinf		
$v_c =$	3	$\sqrt{f_c}$
=	212	psi
$v_s =$	244	psi
$v_n =$	457	psi
=	6.46	$\sqrt{f_c}$

Critical Section @ Outer Studs		
$Y_{vx} =$	0.46	
$Y_{vy} =$	0.34	

Crit sect @ d/2 beyond studs		
$L_{x1} =$	17.54	in
$L_{x2} =$	123.38	in
$L_{y1} =$	29.54	in
$L_{y2} =$	135.38	in
$L =$	74.84	in
$X_a =$	61.69	in
$Y_a =$	14.77	in
$X_b =$	8.77	in
$Y_b =$	67.69	in
$I_x =$	9,946,723	in ⁴
$I_y =$	8,921,943	in ⁴
$b_o =$	393.50	in
$A_c =$	5,263	in ²
$v_{ua} =$	44	psi
$v_{ua}/\phi =$	58	psi
=	0.82	$\sqrt{f_c}$
$v_{ub} =$	45	psi
$v_{ub}/\phi =$	59	psi
=	0.84	$\sqrt{f_c}$



NOTES:

1 Slab shear design in accordance with ACI 11.11.5.

Flat Slab Shear Stud Design - Interior Column

Project: **Rose Hill Mukilteo**
Project #: **22332**

Description: **12 x 24 column at LVL 1 (25.25" slab)**

Engineer: **CGP**
Date: **1/26/23**

Max nominal shear strength at d/2 w/ stud reinforcement, $v_n = 8 \sqrt{f_c}$
Max nominal shear strength at d/2 w/ stud reinforcement, $v_c = 3 \sqrt{f_c}$

Input
8
3

Summary	v_u/ϕ	v_n
d/2 from col	1.85	5.61 $\sqrt{f_c}$
d/2 from outer studs	0.68	2.00 $\sqrt{f_c}$

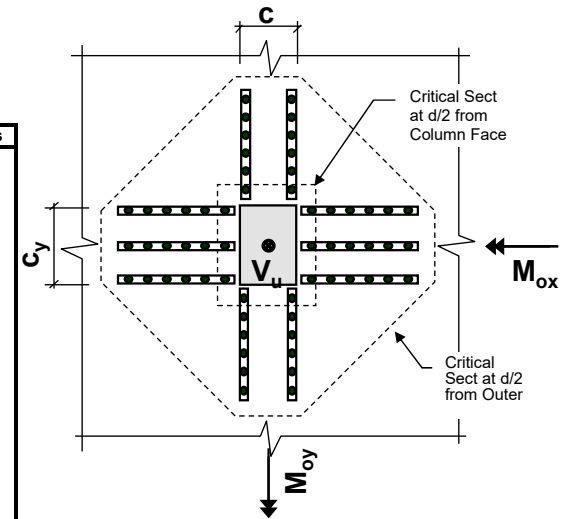
Input		
$f_c =$	5	ksi
avg prestress =	288	psi
$V_u =$	378	k
$M_{ox} =$	18.4	k-ft
$M_{oy} =$	6.42	k-ft
$c_x =$	12	in
$c_y =$	24	in
$h =$	25.25	in
$d =$	23.625	in
<u>Shear Studs</u>		
stud $f_y =$	50	ksi
min # rails =	8	
provided # rails =	8	
# studs per rail =	12	
stud diameter =	0.625	in
stud area =	0.31	in ²
total stud area =	2.45	in ²
first stud spacing, $s_o =$	8.50	in
stud spacing, $s =$	4.00	in
rail length =	44.00	in

Crit Sect @ d/2 from col		
$b_o =$	166.50	in
$L_x =$	35.63	in
$L_y =$	47.63	in
$A_c =$	3,934	in ²
$I_x =$	1,379,809	in ⁴
$I_y =$	892,008	in ⁴
$Y_{vx} =$	0.44	
$Y_{vy} =$	0.37	
$X =$	17.81	in
$Y =$	23.81	in
$v_u =$	98	psi
$v_u/\phi =$	131	psi
$=$	1.85	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	7.68	$\sqrt{f_c}$
$v_n (pt) =$	4.72	$\sqrt{f_c}$
$v_n =$	334	psi
$=$	4.72	$\sqrt{f_c}$

Crit Sect @ d/2 w/ Reinf		
$v_c =$	3	$\sqrt{f_c}$
$=$	212	psi
$v_s =$	184	psi
$v_n =$	396	psi
$=$	5.61	$\sqrt{f_c}$

Critical Section @ Outer Studs		
$Y_{vx} =$	0.45	
$Y_{vy} =$	0.35	

Crit sect @ d/2 beyond studs		
$L_{x1} =$	21.78	in
$L_{x2} =$	140.63	in
$L_{y1} =$	33.78	in
$L_{y2} =$	152.63	in
$L =$	84.04	in
$X_a =$	70.31	in
$Y_a =$	16.89	in
$X_b =$	10.89	in
$Y_b =$	76.31	in
$I_x =$	#####	in ⁴
$I_y =$	#####	in ⁴
$b_o =$	447.27	in
$A_c =$	10,567	in ²
$v_{ua} =$	36	psi
$v_{ua}/\phi =$	48	psi
$=$	0.68	$\sqrt{f_c}$
$v_{ub} =$	36	psi
$v_{ub}/\phi =$	48	psi
$=$	0.68	$\sqrt{f_c}$



NOTES:

1 Slab shear design in accordance with ACI 11.11.5.

Flat Slab Shear Stud Design - Interior Column

Project: **Rose Hill Mukilteo**
Project #: **22332**

Description: **12 x 24 column at LVL 1 (14.75" slab)**

Engineer: **CGP**
Date: **1/26/23**

Max nominal shear strength at d/2 w/ stud reinforcement, $v_n = 8 \sqrt{f_c}$
Max nominal shear strength at d/2 w/ stud reinforcement, $v_c = 3 \sqrt{f_c}$

Input
8
3

Summary	v_u/ϕ	v_n
d/2 from col	4.29	6.48
d/2 from outer studs	1.32	2.00

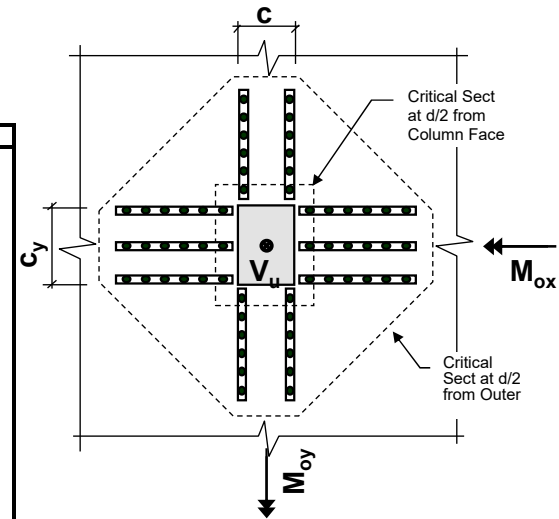
Input		
$f_c =$	5	ksi
avg prestress =	288	psi
$V_u =$	356	k
$M_{ox} =$	24.4	k-ft
$M_{oy} =$	8.83	k-ft
$c_x =$	12	in
$c_y =$	24	in
$h =$	14.75	in
$d =$	13.125	in
<u>Shear Studs</u>		
stud $f_y =$	50	ksi
min # rails =	8	
provided # rails =	8	
# studs per rail =	12	
stud diameter =	0.625	in
stud area =	0.31	in ²
total stud area =	2.45	in ²
first stud spacing, $s_o =$	5.00	in
stud spacing, $s =$	4.00	in
rail length =	44.00	in

Crit Sect @ d/2 from col		
$b_o =$	124.50	in
$L_x =$	25.13	in
$L_y =$	37.13	in
$A_c =$	1,634	in ²
$I_x =$	339,183	in ⁴
$I_y =$	188,492	in ⁴
$Y_{vx} =$	0.45	
$Y_{vy} =$	0.35	
$X =$	12.56	in
$Y =$	18.56	in
$v_u =$	228	psi
$v_u/\phi =$	303	psi
=	4.29	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	4.00	$\sqrt{f_c}$
$v_n =$	6.22	$\sqrt{f_c}$
$v_n (pt) =$	4.72	$\sqrt{f_c}$
$v_n =$	334	psi
=	4.72	$\sqrt{f_c}$

Crit Sect @ d/2 w/ Reinf		
$v_c =$	3	$\sqrt{f_c}$
=	212	psi
$v_s =$	246	psi
$v_n =$	459	psi
=	6.48	$\sqrt{f_c}$

Critical Section @ Outer Studs		
$Y_{vx} =$	0.46	
$Y_{vy} =$	0.34	

Crit sect @ d/2 beyond studs		
$L_{x1} =$	17.43	in
$L_{x2} =$	123.13	in
$L_{y1} =$	29.43	in
$L_{y2} =$	135.13	in
$L =$	74.73	in
$X_a =$	61.56	in
$Y_a =$	14.72	in
$X_b =$	8.72	in
$Y_b =$	67.56	in
$I_x =$	9,698,421	in ⁴
$I_y =$	8,697,776	in ⁴
$b_o =$	392.67	in
$A_c =$	5,154	in ²
$v_{ua} =$	70	psi
$v_{ua}/\phi =$	93	psi
=	1.31	$\sqrt{f_c}$
$v_{ub} =$	70	psi
$v_{ub}/\phi =$	93	psi
=	1.32	$\sqrt{f_c}$



NOTES:

1 Slab shear design in accordance with ACI 11.11.5.

SECTION NO. 5

Rose Hill Mukilteo
3rd & Park Ave
Mukilteo, WA 98275

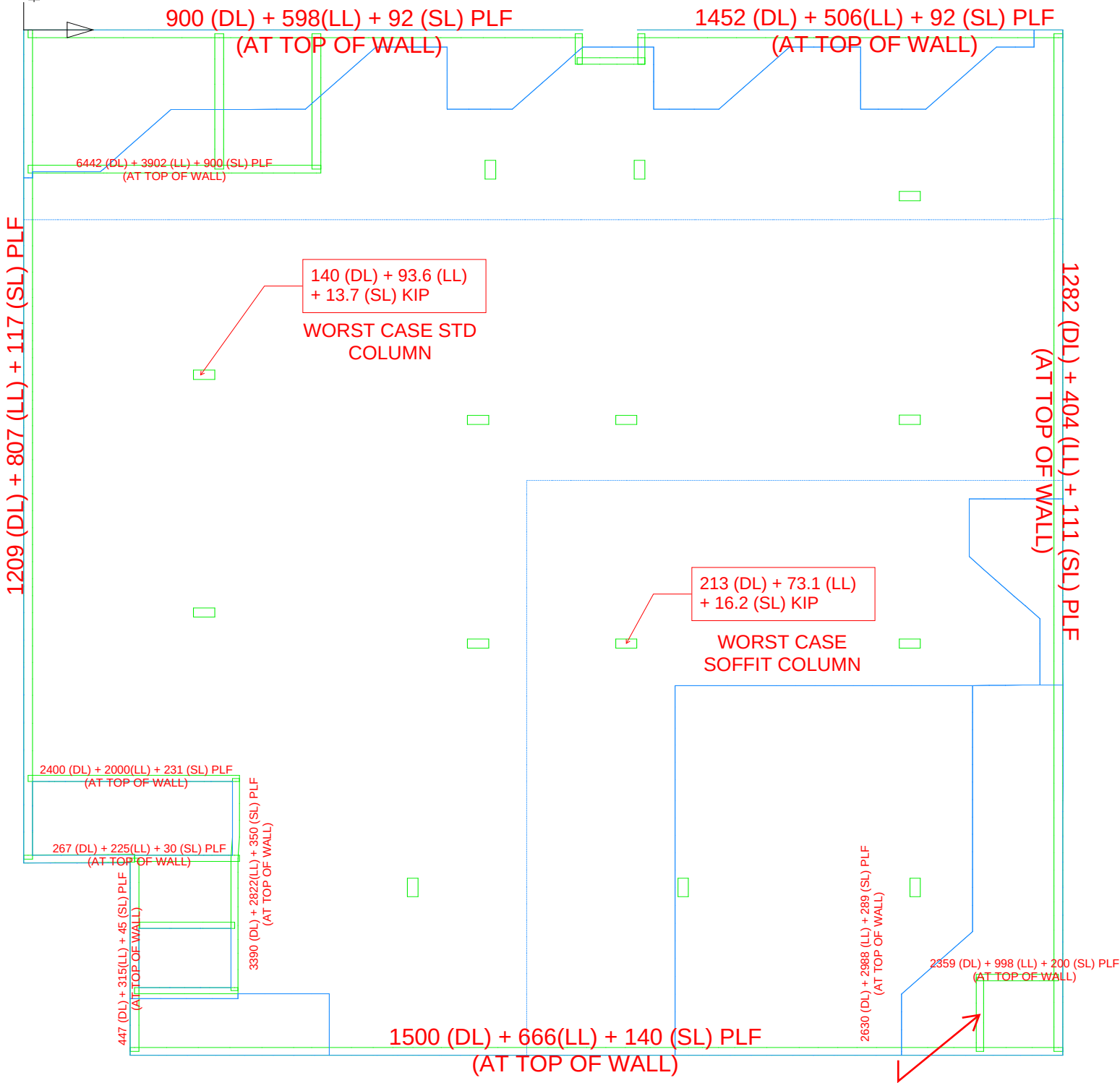
SECTION TITLE: Concrete Design

SECTION INDEX

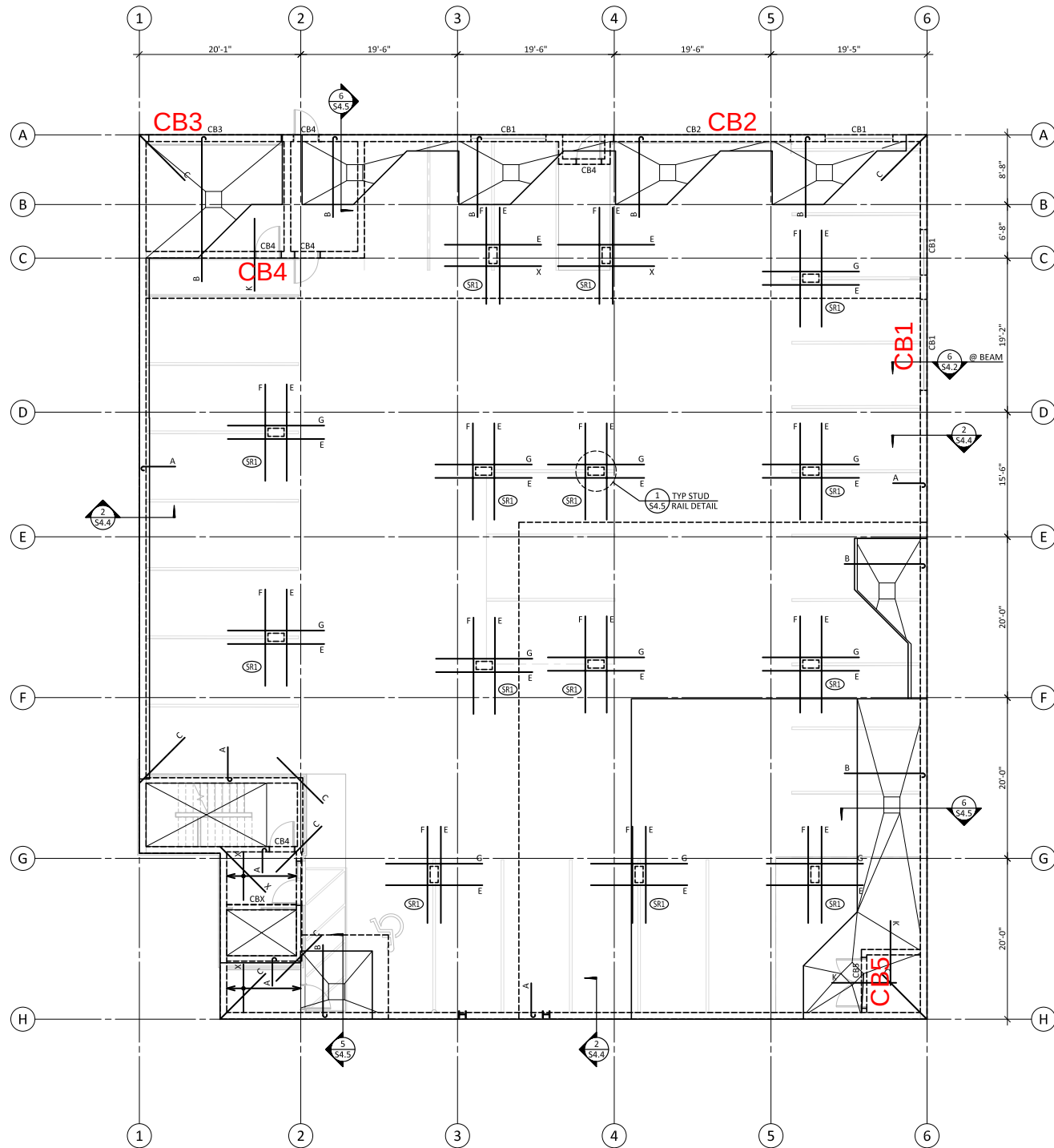
DESCRIPTION	PAGE
Load Map	5.01
Concrete Beam Key Plan	5.02
Concrete Beam and Column Design	5.03-5.12
Retaining Wall Design	5.13-5.16
Foundation Design	5.17-5.24

All Dead LC: Std Reactions Plan

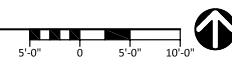
All Dead LC: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1/250



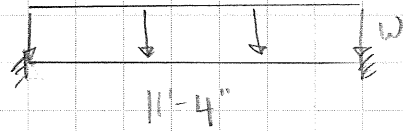
WALL AND COLUMN LOADS KEY PLAN



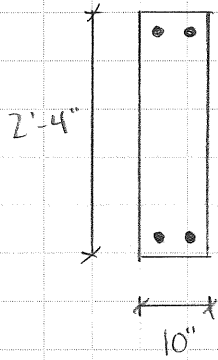
1 SECOND FLOOR MILD STEEL PLAN
SCALE: 1/8" = 1'-0"



CB1:



$$W = 1452 \text{ DL} + 506 \text{ LL} + 92 \text{ SL}$$



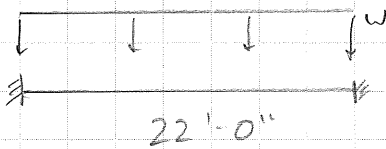
Reinforcement:

Top = (2) #4 Bars
Bottom = (2) #4 Bars

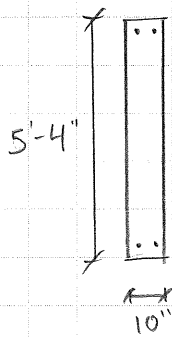
M = 58%
OK

Stirrup Design: Use min Stirrup spacing, #4 @ 12" OC

CB2:



$$W = 1452 \text{ DL} + 506 \text{ LL} + 92 \text{ SL}$$



Reinforcement:

Top = (2) #5 Bars
Bottom = (2) #5 Bars

M = 61%
OK

Stirrup Design: Stirrups not required



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Description	Concrete Beams	By	CCC	Date	1/25/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.	22332.10		5.03

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

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DESCRIPTION: CB1

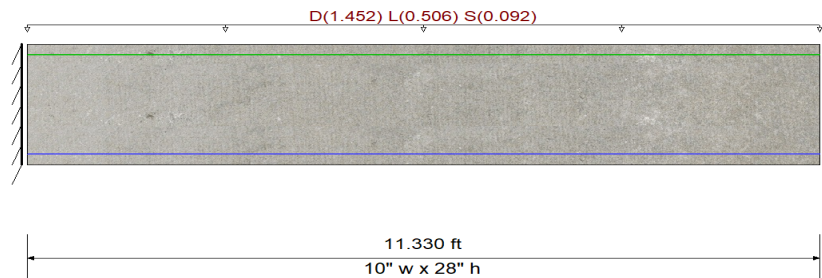
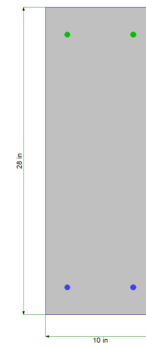
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

General Information

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	4
			Number of Resisting Legs Per Stirrup	=	1.0



Cross Section & Reinforcing Details

Rectangular Section, Width = 10.0 in, Height = 28.0 in

Span #1 Reinforcing....

2-#4 at 2.50 in from Bottom, from 0.0 to 11.330 ft in this span 2-#4 at 2.50 in from Top, from 0.0 to 11.330 ft in this sp:

Load for Span Number 1

Uniform Load : D = 1.452, L = 0.5060, S = 0.0920 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.584 : 1
Section used for this span		Typical Section
Mu : Applied		-27.792 k-ft
Mn * Phi : Allowable		47.607 k-ft
Location of maximum on span		11.330 ft
Span # where maximum occurs		Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	L Only
Max Downward Total Deflection	0.003 in	Ratio =	53507 >=240.0	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0	Span: 1 : +D+L

Shear Stirrup Requirements

Entire Beam Span Length : $\Phi V_c / 2 < V_u \leq \Phi V_c$, Req'd Vs = Min per 9.6.3.1, use #4 stirrups spaced at 12.000 in

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

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DESCRIPTION: CB2

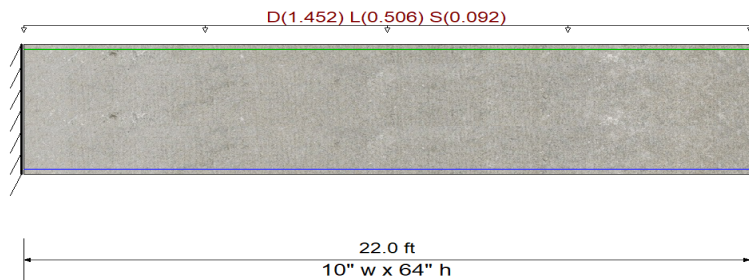
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

General Information

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	4
			Number of Resisting Legs Per Stirrup	=	1.0



Cross Section & Reinforcing Details

Rectangular Section, Width = 10.0 in, Height = 64.0 in

Span #1 Reinforcing....

2-#5 at 2.50 in from Bottom, from 0.0 to 22.0 ft in this span

2-#5 at 2.50 in from Top, from 0.0 to 22.0 ft in this span

Load for Span Number 1

Uniform Load : D = 1.452, L = 0.5060, S = 0.0920 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.611 : 1
Section used for this span		Typical Section
Mu : Applied		-104.781 k-ft
Mn * Phi : Allowable		171.561 k-ft
Location of maximum on span		0.000 ft
Span # where maximum occurs		Span # 1

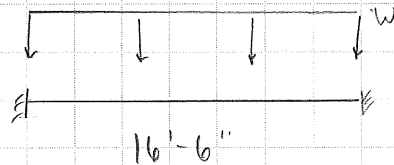
Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360.0	L Only
Max Downward Total Deflection	0.003 in	Ratio =	87261 >=240.0	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0 <240.0	Span: 1 : +D+L

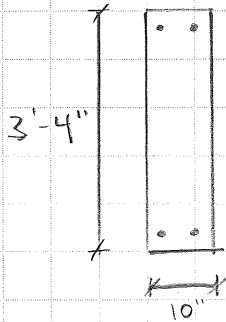
Shear Stirrup Requirements

Entire Beam Span Length : $V_u < \phi V_c / 2$, Req'd Vs = Not Req'd per 9.6.3.1, use #4 stirrups spaced at 24.000 in

CB3:



$$W = 900 \text{ DL} + 598 \text{ LL} + 92 \text{ SL}$$



Reinforcement:

Top = (2) #5 Bars

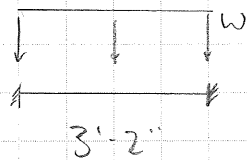
Bottom = (2) #5 Bars

$$M = 416\%$$

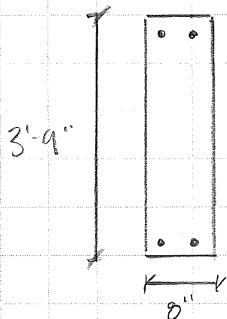
OK

Stirrup Design: Use #4 @ 18" OC stirrups

CB4:



$$W = 6442 \text{ DL} + 3902 \text{ LL} + 900 \text{ SL}$$



Reinforcement:

Top = (2) #4 Bars

Bottom = (2) #4 Bars

$$M = 16\%$$

OK

Stirrup Design: Use #4 Stirrups @ 18" OC



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Description Concrete Beams

By CCC

Date 1/25/23

Checked

Date

Scale

Sheet No.

Project Rose Hill Mukilteo

Job No.

5.06

22332.10

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

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DESCRIPTION: CB3

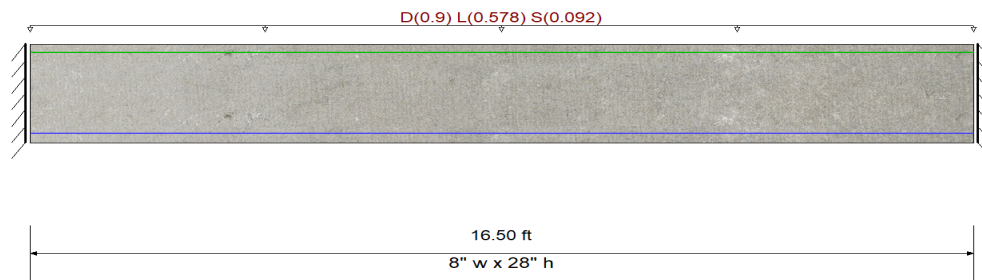
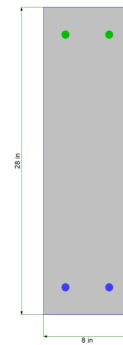
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

General Information

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	4
			Number of Resisting Legs Per Stirrup	=	1.0



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 28.0 in

Span #1 Reinforcing....

2-#6 at 2.50 in from Bottom, from 0.0 to 16.50 ft in this span

2-#6 at 2.50 in from Top, from 0.0 to 16.50 ft in this span

Load for Span Number 1

Uniform Load : D = 0.90, L = 0.5780, S = 0.0920 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.478 : 1
Section used for this span		Typical Section
Mu : Applied		-46.527 k-ft
Mn * Phi : Allowable		97.246 k-ft
Location of maximum on span		0.000 ft
Span # where maximum occurs		Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.004 in	Ratio = 46949	>=360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio = 0	<360.0	L Only
Max Downward Total Deflection	0.011 in	Ratio = 18360	>=180.0	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio = 0	<180.0	Span: 1 : +D+L

Shear Stirrup Requirements

Entire Beam Span Length : $\Phi V_c / 2 < V_u \leq \Phi V_c$, Req'd Vs = Min per 9.6.3.1, use #4 stirrups spaced at 12.000 in

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: CB4

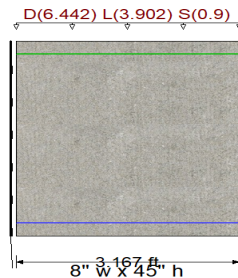
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

General Information

f_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	4
			Number of Resisting Legs Per Stirrup	=	1.0



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 45.0 in

Span #1 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 3.167 ft in this span

2-#4 at 3.0 in from Top, from 0.0 to 3.167 ft in this span

Load for Span Number 1

Uniform Load : D = 6.442, L = 3.902, S = 0.90 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.156 : 1
Section used for this span		Typical Section
Mu : Applied		-12.054 k-ft
Mn * Phi : Allowable		77.434 k-ft
Location of maximum on span		0.000 ft
Span # where maximum occurs		Span # 1

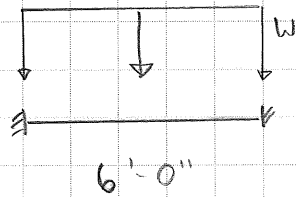
Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 < 360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 < 360.0	L Only
Max Downward Total Deflection	0.000 in	Ratio =	0 < 180.0	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0 < 180.0	Span: 1 : +D+L

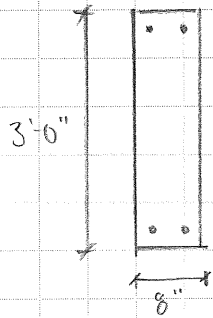
Shear Stirrup Requirements

Entire Beam Span Length : $\phi V_c / 2 < V_u \leq \phi V_c$, Req'd Vs = Min per 9.6.3.1, use #4 stirrups spaced at 21.000 in

CBS:



$$W = 2630 \text{ DL} + 2990 \text{ LL} + 289 \text{ SL}$$



Reinforcement:

Top = (2) #5 Bars

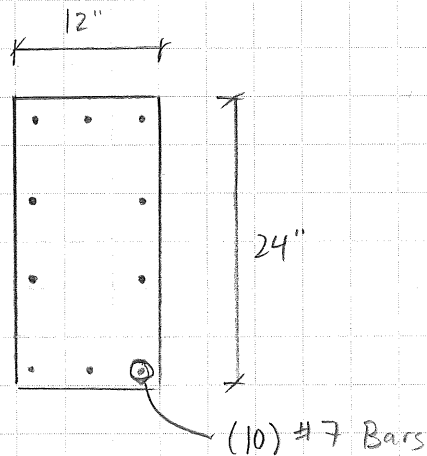
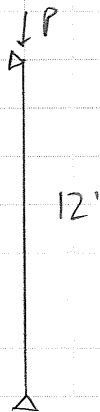
Bottom = (2) #4 Bars

M = 60%

OK

Stirrup Design: Use #4 Stirrups @ 16" OC

CC1: Concrete Column



Max Capacity = 696 kips > 305 kips Max Applied OK



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Description	Concrete Beams	By	CCC	Date	1/25/23
		Checked		Date	
		Scale		Sheet No.	
Project	Rose Hill Mukilteo	Job No.	22332-10		5.09

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Concrete Beam

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

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DESCRIPTION: CB5

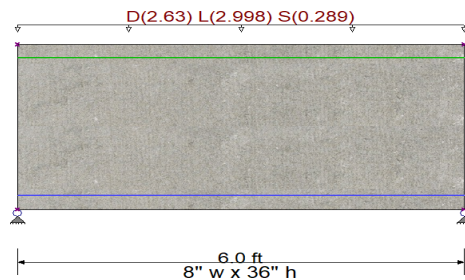
CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

General Information

f'_c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
$f_r = f'_c^{1/2} \cdot 7.50$	=	474.342 psi		Shear :	0.750
ψ Density	=	150.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups	=	60.0 ksi
f_y - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #	=	4
			Number of Resisting Legs Per Stirrup	=	1.0



Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 36.0 in

Span #1 Reinforcing....

2-#4 at 3.0 in from Bottom, from 0.0 to 6.0 ft in this span

2-#4 at 3.0 in from Top, from 0.0 to 6.0 ft in this span

Load for Span Number 1

Uniform Load : D = 2.630, L = 2.998, S = 0.2890 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.595 : 1
Section used for this span		Typical Section
Mu : Applied		36.438 k-ft
Mn * Phi : Allowable		61.234 k-ft
Location of maximum on span		2.995 ft
Span # where maximum occurs		Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 < 360.0	S Only
Max Upward Transient Deflection	0.000 in	Ratio =	0 < 360.0	L Only
Max Downward Total Deflection	0.002 in	Ratio =	42616 >= 240.0	Span: 1 : +D+L
Max Upward Total Deflection	0.000 in	Ratio =	0 < 240.0	Span: 1 : +D+L

Shear Stirrup Requirements

Entire Beam Span Length : $\Phi V_c / 2 < V_u \leq \Phi V_c$, Req'd Vs = Min per 9.6.3.1, use #4 stirrups spaced at 16.000 in

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Column

Project File: Rose Hill.ec6

LIC# : KW-06015244, Build:20.22.12.28

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DESCRIPTION: CC1

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

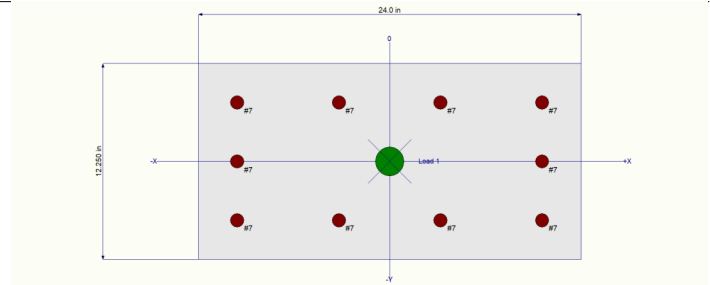
f'_c : Concrete 28 day streng = 4.0 ksi
E = 3,122.0 ksi
Density = 150.0 pcf
 β = 0.850
 f_y - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %

Overall Column Height = 12.0 ft
End Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along colum
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 12.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 12.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 12.250in high x 24.0in Wide,
Column Edge to Rebar Edge Cover = 2.0in

Column Reinforcing : 4 - #7 bars @ corners,, 2 - #7 bars
top & bottom between corner bars, 1
- #7 bars left & right between corner



Entered loads are factored per load combinations specified by user.

Applied Loads

Column self weight included : 3,675.0 lbs * Dead Load Factor

AXIAL LOADS . . .

Axial Load at 12.0 ft above base, D = 213.0, L = 73.10, S = 16.20 k

DESIGN SUMMARY

Load Combination +1.20D+1.60L+0.50S
Location of max. above base 11.919 ft
Maximum Stress Ratio **0.546 : 1**
Ratio = $(P_u^2 + M_u^2)^{0.5} / (\phi P_n^2 + \phi M_n^2)^{0.5}$
 $P_u = 385.070$ k $\phi * P_n = 696.38$ k
 $M_{u-x} = 0.0$ k-ft $\phi * M_{n-x} = 0.0$ k-ft
 $M_{u-y} = 42.358$ k-ft $\phi * M_{n-y} = 76.377$ k-ft
Mu Angle = 90.0 deg
 M_u at Angle = 42.358 k-ft ϕM_n at Angle = 78.194 k-ft
P_n & M_n values located at P_u-M_u vector intersection with capacity curve

Column Capacities . .

P_nmax : Nominal Max. Compressive Axial Capacity 1,339.20 k
P_nmin : Nominal Min. Tension Axial Capacity k
 ϕP_n , max : Usable Compressive Axial Capacity 696.38 k
 ϕP_n , min : Usable Tension Axial Capacity k

Maximum SERVICE Load Reactions .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Deflections . .

Along Y-Y	0.0 in	at	0.0 ft	above base
for load combination :				
Along X-X	0.0 in	at	0.0 ft	above base
for load combination :				

General Section Information $\phi = 0.650$ $\beta = 0.850$ $\theta = 0.80$

ρ : % Reinforcing	2.041 %	Rebar %	Ok
Reinforcing Area	6.0 in ²		
Concrete Area	294.0 in ²		

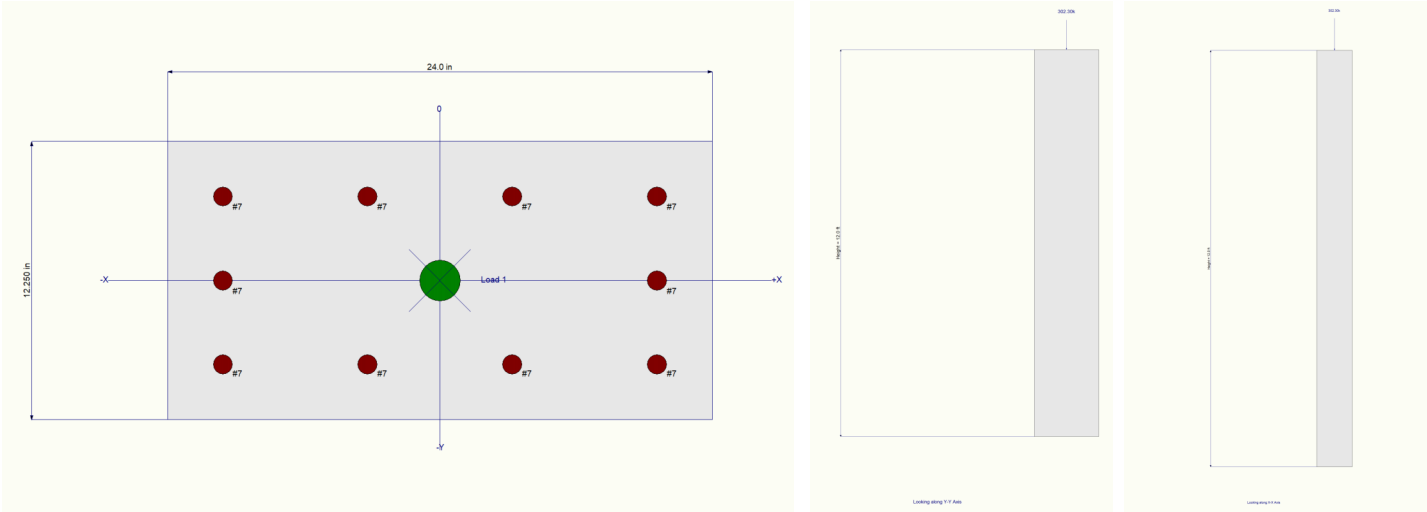
Concrete Column

Project File: Rose Hill.ec6
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LIC# : KW-06015244, Build:20.22.12.28
CG ENGINEERING

DESCRIPTION: CC1

Sketches



ROSE HILL APARTMENTS

FOUNDATION DESIGN

$$Q_{MAX} = \underline{3000 \text{ PSF}} \text{ (PER GEOTECH)}$$

EXTERIOR WALL CONTINUOUS FOOTING

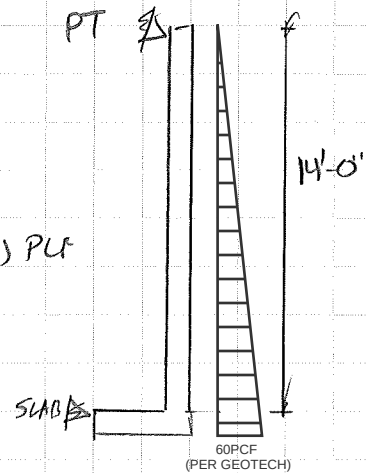
$$\text{WALL HT} = 14'$$

$$\text{ADDED DL} = (14')(10"/12)(150 \text{ PSF}) = \underline{1750 \text{ PLF}}$$

$$\begin{aligned} \text{WORST CASE LINE LOAD} &= 1455(\text{DU}) + 1750(\text{DL}) \\ &+ 1000(\text{LL}) + 190(\text{SL}) \text{ PLF} \end{aligned}$$

$$\text{LINE LOAD} = \underline{4205 \text{ PLF}}$$

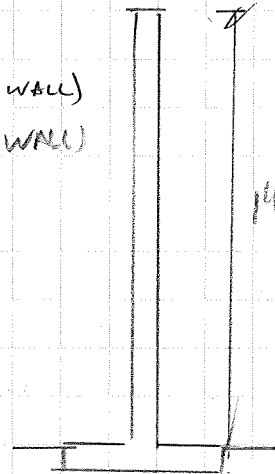
PROVIDE 3'-0" WIDE L-SHAPED FTG
W/ #5 @ 12" OC TOP + BOT W/
10" CONC STEM W/ #5 @ 6" OC VERT
EA FACE



INTERIOR WALL

$$\begin{aligned} \text{WALL WT} &= (10'')(150 \text{ PCF})(14') = 1750 \text{ PLF} \text{ (10" WALL)} \\ &+ (8'')(150 \text{ PCF})(14') = 1400 \text{ PLF} \text{ (8" WALL)} \end{aligned}$$

MARK	SIZE	CAPACITY
F1C	1'-6" X 1'-0" DEEP	4500 PLF
F2C	2'-0" X 1'-0" DEEP	6000 PLF
F3C	3'-0" X 1'-0" DEEP	9000 PLF
F4C	4'-0" X 1'-2" DEEP	12000 PLF
F5C	5'-0" X 1'-2" DEEP	15000 PLF



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Description

FOUNDATION LOADS

Project

ROSE HILL APARTMENTS

By

Checked

Scale

Job No.

Date

Date

Sheet No.

5.13

Project Title:
Engineer:
Project ID:
Project Descr:

Restrained Retaining Wall

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

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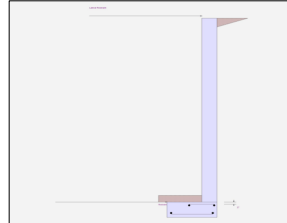
(c) ENERCALC INC 1983-2022

DESCRIPTION: Building Exterior Footing Wall and Footing

Code Reference:

Calculations per IBC 2018 1807.3, CBC 2019, ASCE 7-16

Criteria		Soil Data	
Retained Height	= 14.0 ft	Allow Soil Bearing	= 3,000.0 psf
Wall height above soil	= ft	Equivalent Fluid Pressure Method	= 60.0 psf/ft
Total Wall Height	= 14.0 ft	At-Rest Heel Pressure	= 0.0 psf/ft
Top Support Height	= 14.0 ft	Passive Pressure	= 200.0 psf/ft
Slope Behind Wall	= 0	Soil Density	= 110 pcf
Height of Soil over Toe	= 6 in	Footing Soil Frictior	= 0.4 psf
		Soil height to ignore for passive pressure	= 12 in



Surcharge Loads	
Surcharge Over Heel	= psf
>>>Used To Resist Sliding & Overturning	
Surcharge Over Toe	= psf
Used for Sliding & Overturning	
Axial Load Applied to Stem	
Axial Dead Load	= 1,455.0 lbs
Axial Live Load	= 1,000.0 lbs
Axial Load Eccentricity	= in
Earth Pressure Seismic Load	

Uniform Lateral Load Applied to Stem	
Lateral Load	= #/ft
...Height to Top	= ft
...Height to Bottom	= ft
Load Type	= Wind (W)
	(Strength Level)
Wind on Exposed Stem	= 0.00 psf
	(Strength Level)
Wind acts left-to-right toward retention side.	
K _h Soil Density Multiplier	= 0.2 g
Added seismic per unit area	= 0.0 psf

Adjacent Footing Load	
Adjacent Footing Load	= lbs
Footing Width	= ft
Eccentricity	= in
Wall to Ftg CL Dist	= ft
Footing Type	= Line Load
Base Above/Below Soil at Back of Wall	= ft
Poisson's Ratio	= 0.3
Added seismic per unit area	= 0.0 psf

Design Summary

Total Bearing Load	= 4,849.24 lbs
...resultant ecc.	= 0.0 in
Soil Pressure @ Toe	= 1,616.24 psf OK
Soil Pressure @ Heel	= 1,616.24 psf OK
Allowable	= psf
Soil Pressure Less Than Allowable	
ACI Factored @ Toe	= 2,072.80 psf
ACI Factored @ Heel	= 2,072.80 psf
Footing Shear @ Toe	= 18.424 psi OK
Footing Shear @ Heel	= 0.0 psi OK
Allowable	= 82.158 psi
Reaction at Top	= 1,174.89 lbs
Reaction at Bottom	= 5,724.85 lbs

Sliding Calcs	
Lateral Sliding Force	= 5,724.85 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors	
Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Concrete Stem Construction

Thickness	= 10.00 in
Wall Weight	= 125.0 psf
Stem is FIXED to top of footing	

	@ Top Support		Mmax Between Top & Base		@ Base of Wall	
	Stem OK		Stem OK		Stem OK	
Design Height Above Ftg	= 14.0 ft		0.05623 ft		0.00 ft	
Rebar Size	= # 5		# 5		# 5	
Rebar Spacing	= 12.00 in		6.00 in		6.00 in	
Rebar Placed at	= Edge		Edge		Edge	
Rebar Depth 'd'	= 7.50 in		8.0 in		7.50 in	
Design Data						
f _{cr} /F _B + f _a /F _a	=		0.835		0.894	
Moment.....Actual	=	0.0 ft-#	17,562.0 ft-#		17,562.0 ft-#	
Moment.....Allowable	=	10,143.6 ft-#	21,044.3 ft-#		19,649.3 ft-#	
Shear Force @ this height	=	1,881.58 lbs			7,526.43 lbs	
Shear.....Actual	=	20.906 psi			83.627 psi	
Shear.....Allowable	=	94.868 psi			94.868 psi	

Project Title:
Engineer:
Project ID:
Project Descr:

Restrained Retaining Wall

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

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DESCRIPTION: Building Exterior Footing Wall and Footing

Footing Strengths & Dimensions

Toe Width	= 2.167 ft
Heel Width	= 0.8333
Total Footing Width	= 3.0
Footing Thickness	= 14.0 in
Key Width	= in
Key Depth	= in
Key Distance from Toe	= ft
F _c = 3,000 psi	F _y = 60000 psi
Footing Concrete Density	= 150 pcf
Min. As %	= 0.0018
Cover @ Top	= 2 in @ Btm.= 3 in

Footing Design Results

	Toe	Heel
Factored Pressure	= 2,072.80	2,072.80 psf
Mu' : Upward	= 4,866.82	ft-#
Mu' : Downward	= 648.03	ft-#
Mu: Design	= 4,219	0 ft-#
Actual 1-Way Shear	= 18.424	psi
Allow 1-Way Shear	= 82.158	0.0 psi

Other Acceptable Sizes & Spacings:

Toe: # 5 @ 12.30 in	-or-	#4 @ 7.93 in, #5 @ 12.30 in, #6 @ 17.46 in, #7 @ 23.1
Heel: # 5 @ 12.30 in	-or-	#4 @ 7.93 in, #5 @ 12.30 in, #6 @ 17.46 in, #7 @ 23.1
Key: # 0 @ 0.00 in	-or-	No key defined
Min footing T&S reinf Area		0.91 in2
Min footing T&S reinf Area per foot		0.30 in2 /ft
If one layer of horizontal bars:		
If two layers of horizontal bars:		
#4 @ 7.94 in		#4 @ 15.87 in
#5 @ 12.30 in		#5 @ 24.60 in
#6 @ 17.46 in		#6 @ 34.92 in

Summary of Forces on Footing : Slab RESISTS sliding, stem is FIXED at footing

Forces acting on footing for soil pressure

>>> Sliding Forces are restrained by the adjacent slab

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem	=	-10,976.2 ft-#
Surcharge Over Heel	= 0.0 lbs	0.0 ft
Adjacent Footing Load	= 0.0 lbs	0.0 ft
Axial Dead Load on Stem	= 2,455.0 lbs	2.584 ft
Soil Over Toe	= 119.185 lbs	1.084 ft
Surcharge Over Toe	= 0.0 lbs	0.0 ft
Stem Weight	= 1,750.0 lbs	2.584 ft
Soil Over Heel	= 0.0 lbs	3.0 ft
Footing Weight	= 525.06 lbs	1.50 ft
Total Vertical Force	= 4,849.24 lbs	Base Moment = 804.91 ft-#

Stem is specified to be fixed to footing, and top restraint is assumed to react out any tendency for moment at the footing/soil interface, so uniform soil pressure is assumed.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Project Title:
Engineer:
Project ID:
Project Descr:

Restrained Retaining Wall

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Building Exterior Footing Wall and Footing

Rebar Lap & Embedment Lengths Information

Project Title:
Engineer:
Project ID:
Project Descr:

Restrained Retaining Wall

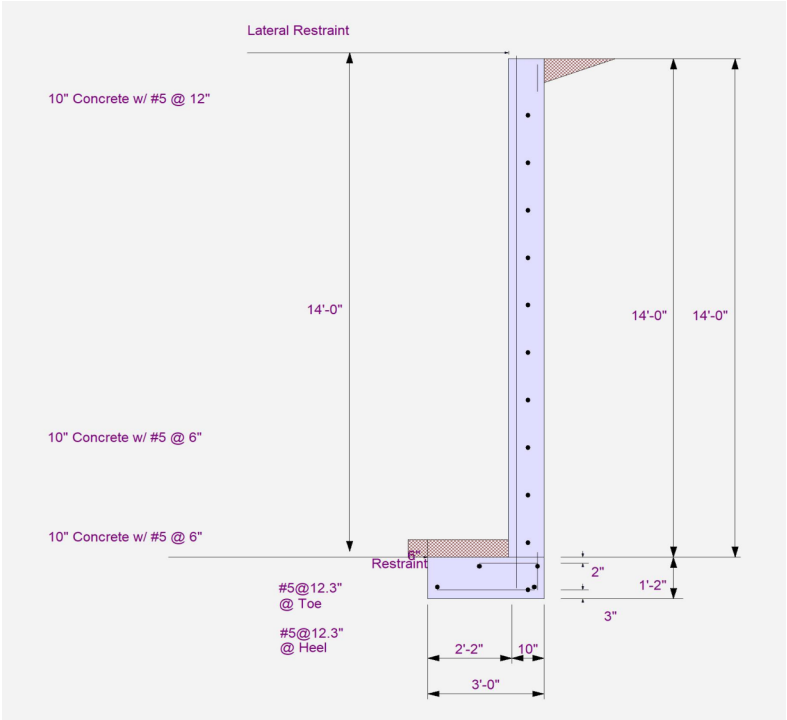
Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

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DESCRIPTION: Building Exterior Footing Wall and Footing



Project Title:
Engineer:
Project ID:
Project Descr:

Restrained Retaining Wall

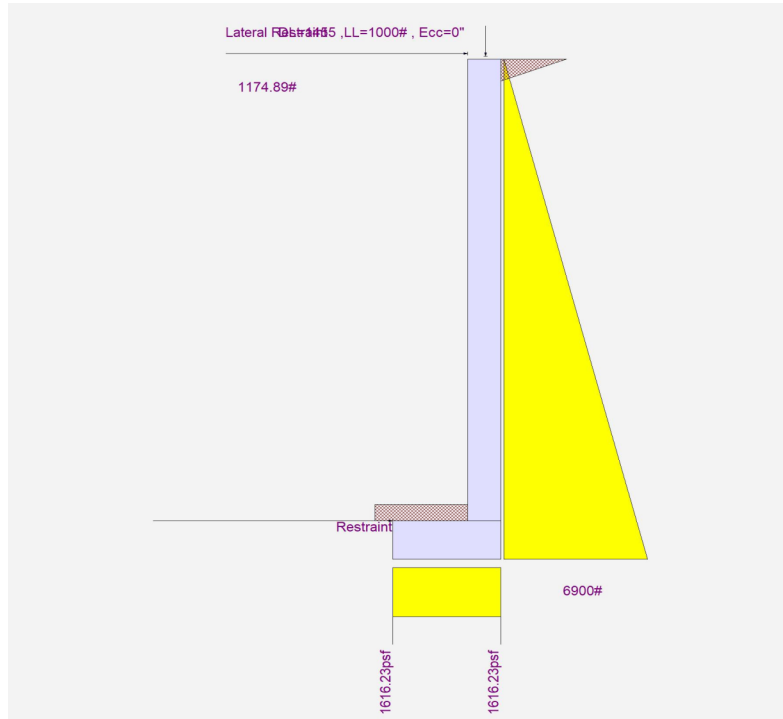
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DESCRIPTION: Building Exterior Footing Wall and Footing



ROSE HILL APARTMENTS

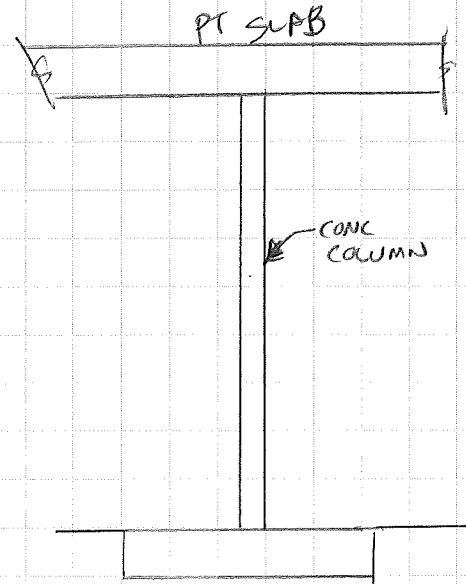
FOUNDATION DESIGN * SEE SPREAD SHEET

SOFFIT COLUMN FOUNDATION

$$P_{MAX} = \frac{213^k (DL) + 673^k (LL) + 16.4^k (SL)}{\text{WORST CASE (E-4)}}$$

$$Q_{MAX} = 3000 \text{ PSF}$$

PROVIDE A 10' SQ X 26" DEEP FTG
W/ (13) #6 BAR EA WAY BOT



SLAB COLUMN FOUNDATION

$$P_{MAX} = \frac{140^k (DL) + 882^k (LL) + 13.7^k (SL)}{\text{WORST CASE (D-2)}}$$

$$Q_{MAX} = 3000 \text{ PSF}$$

PROVIDE A 9'-0" SQ X 24" DEEP FTG
W/ (11) #6 BAR EA WAY BOT

$$P_{MAX} = \frac{133^k (DL) + 34.8^k (LL) + 13.4^k (SL)}{\text{TYPICAL (C-5)}}$$

PROVIDE A 8'-0" SQ X 20" DEEP FTG
W/ (8) #6 BAR EA WAY BOT



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Suite 200
Edmonds, WA 98020
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Description

PAD FOOTINGS

Project

ROSE HILL APARTMENTS

By CGP

Checked

Scale

Job No.

Date

Date

Sheet No.

5.17

Rose Hill Apartments Concrete Column Pad Footing Design

CC1 12"x24" w/ (8) #7 Bars $\phi P_n =$ 721

Allowable Bearing Pressure = 3000 psf

	Footing Size	Capacity (k)
1	1'-0" SQ	3
2	2'-0" SQ	12
3	3'-0" SQ	27
4	4'-0" SQ	48
5	5'-0" SQ	75
6	6'-0" SQ	108
7	7'-0" SQ	147
8	8'-0" SQ	192
9	9'-0" SQ	243
10	10'-0" SQ	300
11	11'-0" SQ	363
12	12'-0" SQ	432

ID	DEMAND (K) per RAM			LRFD LOAD COMBINATIONS			COLUMN SIZE		ASD LOAD COMBINATIONS				FOOTING SIZE	
	Dead	Live	Snow	1.4D	1.2D+1.6L+0.5S	MAX	Size	DCR	D+L	D+S	D+0.75(L+S)	MAX	Size	DCR
C-3	92.1	27.4	8.95	128.9	158.8	158.8	CC1	22%	119.5	101.1	119.4	119.5	7	81%
C-4	122	30.3	12.4	170.8	201.1	201.1	CC1	28%	152.3	134.4	154.0	154.025	8	80%
C-5	133	34.8	13.4	186.2	222.0	222.0	CC1	31%	167.8	146.4	169.2	169.15	8	88%
D-2	140	88.2	13.7	196.0	316.0	316.0	CC1	44%	228.2	153.7	216.4	228.2	9	94%
D-3	123	58.5	12.5	172.2	247.5	247.5	CC1	34%	181.5	135.5	176.3	181.5	8	95%
D-4	125	21.6	11.8	175.0	190.5	190.5	CC1	26%	146.6	136.8	150.1	150.05	8	78%
D-5	129	27.4	11.9	180.6	204.6	204.6	CC1	28%	156.4	140.9	158.5	158.475	8	83%
F-2	137	93.6	13.5	191.8	320.9	320.9	CC1	45%	230.6	150.5	217.3	230.6	9	95%
F-3	131	79.2	11.9	183.4	289.9	289.9	CC1	40%	210.2	142.9	199.3	210.2	9	87%
F-4	236	57.3	16.4	330.4	383.1	383.1	CC1	53%	293.3	252.4	291.3	293.3	10	98%
F-5	215	49.9	15.6	301.0	345.6	345.6	CC1	48%	264.9	230.6	264.1	264.9	10	88%
G-3	116	82.3	10.4	162.4	276.1	276.1	CC1	38%	198.3	126.4	185.5	198.3	9	82%
G-4	188	74.4	14.8	263.2	352.0	352.0	CC1	49%	262.4	202.8	254.9	262.4	10	87%
G-5	103	42.6	9.5	144.2	196.5	196.5	CC1	27%	145.6	112.5	142.1	145.6	7	99%

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Slab Column Footing - Typical

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

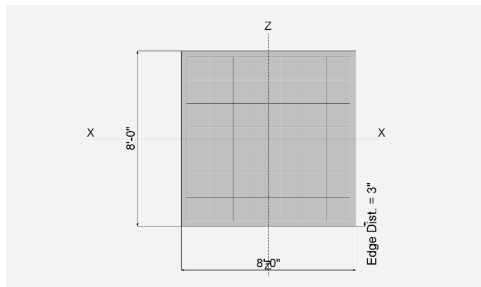
General Information

Material Properties				Soil Design Values			
f _c : Concrete 28 day strength	=	4.0	ksi	Allowable Soil Bearing	=	3.0	ksf
f _y : Rebar Yield	=	60.0	ksi	Soil Density	=	110.0	pcf
E _c : Concrete Elastic Modulus	=	3,122.0	ksi	Increase Bearing By Footing Weight	=	Yes	
Concrete Density	=	150.0	pcf	Soil Passive Resistance (for Sliding)	=	250.0	pcf
φ Values	=	0.90		Soil/Concrete Friction Coeff.	=	0.30	
Flexure	=						
Shear	=	0.750					
Analysis Settings				Increases based on footing Depth			
Min Steel % Bending Reinf.	=			Footing base depth below soil surface	=		ft
Min Allow % Temp Reinf.	=	0.00180		Allow press. increase per foot of depth	=		ksf
Min. Overturning Safety Factor	=	1.0 : 1		when footing base is below	=		ft
Min. Sliding Safety Factor	=	1.0 : 1					
Add Ftg Wt for Soil Pressure	:	Yes		Increases based on footing plan dimension			
Use ftg wt for stability, moments & shears	:	Yes		Allowable pressure increase per foot of depth			
Add Pedestal Wt for Soil Pressure	:	No					
Use Pedestal wt for stability, mom & shear	:	No		when max. length or width is greater than	=		ksf
						</	

Dimensions

Width parallel to X-X Axis	=	8.0 ft
Length parallel to Z-Z Axis	=	8.0 ft
Footing Thickness	=	20.0 in

Pedestal dimensions...		
px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0 in



Reinforcing

Bars parallel to X-X Axis		
Number of Bars	=	8
Reinforcing Bar Size	=	# 6
Bars parallel to Z-Z Axis		
Number of Bars	=	8
Reinforcing Bar Size	=	# 6

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation

# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

	D	Lr	L	S	W	E	H
P : Column Load	=	133.0		34.80	13.40		k
OB : Overburden	=						ksf
M-xx	=						k-ft
M-zz	=						k-ft
V-x	=						k
V-z	=						k

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

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DESCRIPTION: Slab Column Footing - Typical

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8902	Soil Bearing	2.893 ksf	3.250 ksf	+D+0.750L+0.750S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.8403	Z Flexure (+X)	27.748 k-ft/ft	33.019 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8403	Z Flexure (-X)	27.748 k-ft/ft	33.019 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8403	X Flexure (+Z)	27.748 k-ft/ft	33.019 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8403	X Flexure (-Z)	27.748 k-ft/ft	33.019 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.4588	1-way Shear (+X)	43.525 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4588	1-way Shear (-X)	43.525 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4588	1-way Shear (+Z)	43.525 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4588	1-way Shear (-Z)	43.525 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.9793	2-way Punching	185.803 psi	189.737 psi	+1.20D+1.60L+0.50S

Detailed Results

Soil Bearing		Xecc		Zecc		Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
Rotation Axis & Load Combination...	Gross Allowable		(in)	Bottom, -Z	Top, +Z	Left, -X	Right, +X			
X-X, D Only	3.250	n/a	0.0	2.328	2.328	n/a	n/a			0.716
X-X, +D+L	3.250	n/a	0.0	2.872	2.872	n/a	n/a			0.884
X-X, +D+S	3.250	n/a	0.0	2.538	2.538	n/a	n/a			0.781
X-X, +D+0.750L	3.250	n/a	0.0	2.736	2.736	n/a	n/a			0.842
X-X, +D+0.750L+0.750S	3.250	n/a	0.0	2.893	2.893	n/a	n/a			0.890
X-X, +0.60D	3.250	n/a	0.0	1.397	1.397	n/a	n/a			0.430
Z-Z, D Only	3.250	0.0	n/a	n/a	n/a	2.328	2.328			0.716
Z-Z, +D+L	3.250	0.0	n/a	n/a	n/a	2.872	2.872			0.884
Z-Z, +D+S	3.250	0.0	n/a	n/a	n/a	2.538	2.538			0.781
Z-Z, +D+0.750L	3.250	0.0	n/a	n/a	n/a	2.736	2.736			0.842
Z-Z, +D+0.750L+0.750S	3.250	0.0	n/a	n/a	n/a	2.893	2.893			0.890
Z-Z, +0.60D	3.250	0.0	n/a	n/a	n/a	1.397	1.397			0.430

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvm. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	23.275	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.40D	23.275	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+1.60L	26.910	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+1.60L	26.910	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+1.60L+0.50S	27.748	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+1.60L+0.50S	27.748	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L	22.125	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L	22.125	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D	19.950	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D	19.950	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+1.60S	24.805	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+1.60S	24.805	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+1.60S	22.630	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Slab Column Footing - Typical

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvm. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.20D+1.60S	22.630	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+0.50S	22.963	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+0.50S	22.963	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+0.70S	23.298	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +1.20D+0.50L+0.70S	23.298	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +0.90D	14.963	+Z	Bottom	0.4320	AsMin	0.440	33.019	OK
X-X, +0.90D	14.963	-Z	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.40D	23.275	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.40D	23.275	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60L	26.910	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60L	26.910	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60L+0.50S	27.748	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60L+0.50S	27.748	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L	22.125	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L	22.125	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D	19.950	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D	19.950	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+1.60S	24.805	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+1.60S	24.805	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60S	22.630	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+1.60S	22.630	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+0.50S	22.963	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+0.50S	22.963	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+0.70S	23.298	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +1.20D+0.50L+0.70S	23.298	+X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +0.90D	14.963	-X	Bottom	0.4320	AsMin	0.440	33.019	OK
Z-Z, +0.90D	14.963	+X	Bottom	0.4320	AsMin	0.440	33.019	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	36.51 psi	36.51 psi	36.51 psi	36.51 psi	36.51 psi	94.87 psi	0.38	OK
+1.20D+1.60L	42.21 psi	42.21 psi	42.21 psi	42.21 psi	42.21 psi	94.87 psi	0.45	OK
+1.20D+1.60L+0.50S	43.53 psi	43.53 psi	43.53 psi	43.53 psi	43.53 psi	94.87 psi	0.46	OK
+1.20D+0.50L	34.71 psi	34.71 psi	34.71 psi	34.71 psi	34.71 psi	94.87 psi	0.37	OK
+1.20D	31.29 psi	31.29 psi	31.29 psi	31.29 psi	31.29 psi	94.87 psi	0.33	OK
+1.20D+0.50L+1.60S	38.91 psi	38.91 psi	38.91 psi	38.91 psi	38.91 psi	94.87 psi	0.41	OK
+1.20D+1.60S	35.50 psi	35.50 psi	35.50 psi	35.50 psi	35.50 psi	94.87 psi	0.37	OK
+1.20D+0.50L+0.50S	36.02 psi	36.02 psi	36.02 psi	36.02 psi	36.02 psi	94.87 psi	0.38	OK
+1.20D+0.50L+0.70S	36.55 psi	36.55 psi	36.55 psi	36.55 psi	36.55 psi	94.87 psi	0.39	OK
+0.90D	23.47 psi	23.47 psi	23.47 psi	23.47 psi	23.47 psi	94.87 psi	0.25	OK

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	155.85 psi	189.74psi	0.8214	OK
+1.20D+1.60L	180.20 psi	189.74psi	0.9497	OK
+1.20D+1.60L+0.50S	185.80 psi	189.74psi	0.9793	OK
+1.20D+0.50L	148.15 psi	189.74psi	0.7808	OK
+1.20D	133.59 psi	189.74psi	0.7041	OK
+1.20D+0.50L+1.60S	166.10 psi	189.74psi	0.8754	OK
+1.20D+1.60S	151.54 psi	189.74psi	0.7987	OK
+1.20D+0.50L+0.50S	153.76 psi	189.74psi	0.8104	OK
+1.20D+0.50L+0.70S	156.01 psi	189.74psi	0.8222	OK
+0.90D	100.19 psi	189.74psi	0.5281	OK

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Slab Column Footing - Worst Case

Code References

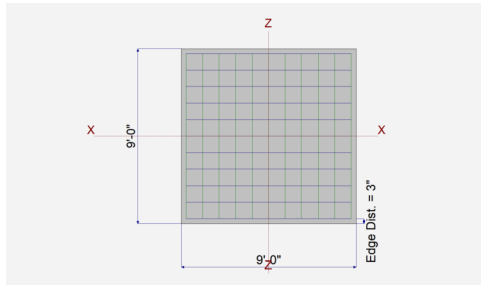
Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties		Soil Design Values	
f _c : Concrete 28 day strength	=	4.0 ksi	Allowable Soil Bearing
f _y : Rebar Yield	=	60.0 ksi	Soil Density
E _c : Concrete Elastic Modulus	=	3,122.0 ksi	Increase Bearing By Footing Weight
Concrete Density	=	150.0 pcf	Soil Passive Resistance (for Sliding)
φ Values	=	0.90	Soil/Concrete Friction Coeff.
Flexure	=	0.750	
Shear	=		
Analysis Settings		Increases based on footing Depth	
Min Steel % Bending Reinf.	=		Footing base depth below soil surface
Min Allow % Temp Reinf.	=	0.00180	Allow press. increase per foot of depth
Min. Overturning Safety Factor	=	1.0 : 1	when footing base is below
Min. Sliding Safety Factor	=	1.0 : 1	
Add Ftg Wt for Soil Pressure	: Yes		Increases based on footing plan dimension
Use ftg wt for stability, moments & shears	: Yes		Allowable pressure increase per foot of depth
Add Pedestal Wt for Soil Pressure	: No		when max. length or width is greater than
Use Pedestal wt for stability, mom & shear	: No		

Dimensions

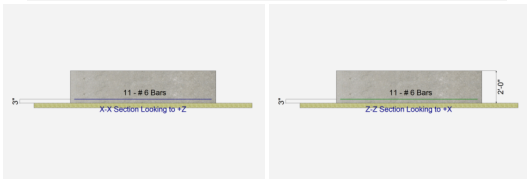
Width parallel to X-X Axis	=	9.0 ft
Length parallel to Z-Z Axis	=	9.0 ft
Footing Thickness	=	24.0 in



Pedestal dimensions...		
px : parallel to X-X Axis	=	in
pz : parallel to Z-Z Axis	=	in
Height	=	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0 in

Reinforcing

Bars parallel to X-X Axis	=	11.0
Number of Bars	=	# 6
Reinforcing Bar Size	=	# 6
Bars parallel to Z-Z Axis	=	11.0
Number of Bars	=	# 6
Reinforcing Bar Size	=	# 6
Bandwidth Distribution Check (ACI 15.4.4.2)	=	n/a
Direction Requiring Closer Separation	=	n/a
# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a



Applied Loads

	D	Lr	L	S	W	E	H
P : Column Load	=	140.0		88.20	13.70		k
OB : Overburden	=						ksf
M-xx	=						k-ft
M-zz	=						k-ft
V-x	=						k
V-z	=						k

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Slab Column Footing - Worst Case

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9445	Soil Bearing	3.117 ksf	3.30 ksf	+D+L about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.7921	Z Flexure (+X)	39.496 k-ft/ft	49.863 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.7921	Z Flexure (-X)	39.496 k-ft/ft	49.863 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.7921	X Flexure (+Z)	39.496 k-ft/ft	49.863 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.7921	X Flexure (-Z)	39.496 k-ft/ft	49.863 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.4552	1-way Shear (+X)	43.188 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4552	1-way Shear (-X)	43.188 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4552	1-way Shear (+Z)	43.188 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4552	1-way Shear (-Z)	43.188 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.9063	2-way Punching	171.956 psi	189.737 psi	+1.20D+1.60L+0.50S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
				Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	3.30	n/a	0.0	2.028	2.028	n/a	n/a	0.615
X-X, +D+L	3.30	n/a	0.0	3.117	3.117	n/a	n/a	0.945
X-X, +D+S	3.30	n/a	0.0	2.198	2.198	n/a	n/a	0.666
X-X, +D+0.750L	3.30	n/a	0.0	2.845	2.845	n/a	n/a	0.862
X-X, +D+0.750L+0.750S	3.30	n/a	0.0	2.972	2.972	n/a	n/a	0.901
X-X, +0.60D	3.30	n/a	0.0	1.217	1.217	n/a	n/a	0.369
Z-Z, D Only	3.30	0.0	n/a	n/a	n/a	2.028	2.028	0.615
Z-Z, +D+L	3.30	0.0	n/a	n/a	n/a	3.117	3.117	0.945
Z-Z, +D+S	3.30	0.0	n/a	n/a	n/a	2.198	2.198	0.666
Z-Z, +D+0.750L	3.30	0.0	n/a	n/a	n/a	2.845	2.845	0.862
Z-Z, +D+0.750L+0.750S	3.30	0.0	n/a	n/a	n/a	2.972	2.972	0.901
Z-Z, +0.60D	3.30	0.0	n/a	n/a	n/a	1.217	1.217	0.369

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvm. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	24.50	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.40D	24.50	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+1.60L	38.640	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+1.60L	38.640	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+1.60L+0.50S	39.496	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+1.60L+0.50S	39.496	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L	26.513	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L	26.513	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D	21.0	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D	21.0	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+1.60S	29.253	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+1.60S	29.253	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+1.60S	23.740	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Slab Column Footing - Worst Case

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in²	Gvm. As in²	Actual As in²	Phi*Mn k-ft	Status
X-X, +1.20D+1.60S	23.740	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+0.50S	27.369	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+0.50S	27.369	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+0.70S	27.711	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +1.20D+0.50L+0.70S	27.711	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +0.90D	15.750	+Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
X-X, +0.90D	15.750	-Z	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.40D	24.50	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.40D	24.50	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60L	38.640	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60L	38.640	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60L+0.50S	39.496	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60L+0.50S	39.496	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L	26.513	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L	26.513	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D	21.0	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D	21.0	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+1.60S	29.253	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+1.60S	29.253	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60S	23.740	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+1.60S	23.740	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+0.50S	27.369	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+0.50S	27.369	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+0.70S	27.711	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +1.20D+0.50L+0.70S	27.711	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +0.90D	15.750	-X	Bottom	0.5184	AsMin	0.5378	49.863	OK
Z-Z, +0.90D	15.750	+X	Bottom	0.5184	AsMin	0.5378	49.863	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	26.79 psi	26.79 psi	26.79 psi	26.79 psi	26.79 psi	94.87 psi	0.28	OK
+1.20D+1.60L	42.25 psi	42.25 psi	42.25 psi	42.25 psi	42.25 psi	94.87 psi	0.45	OK
+1.20D+1.60L+0.50S	43.19 psi	43.19 psi	43.19 psi	43.19 psi	43.19 psi	94.87 psi	0.46	OK
+1.20D+0.50L	28.99 psi	28.99 psi	28.99 psi	28.99 psi	28.99 psi	94.87 psi	0.31	OK
+1.20D	22.96 psi	22.96 psi	22.96 psi	22.96 psi	22.96 psi	94.87 psi	0.24	OK
+1.20D+0.50L+1.60S	31.99 psi	31.99 psi	31.99 psi	31.99 psi	31.99 psi	94.87 psi	0.34	OK
+1.20D+1.60S	25.96 psi	25.96 psi	25.96 psi	25.96 psi	25.96 psi	94.87 psi	0.27	OK
+1.20D+0.50L+0.50S	29.93 psi	29.93 psi	29.93 psi	29.93 psi	29.93 psi	94.87 psi	0.32	OK
+1.20D+0.50L+0.70S	30.30 psi	30.30 psi	30.30 psi	30.30 psi	30.30 psi	94.87 psi	0.32	OK
+0.90D	17.22 psi	17.22 psi	17.22 psi	17.22 psi	17.22 psi	94.87 psi	0.18	OK

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	106.67 psi	189.74psi	0.5622	OK
+1.20D+1.60L	168.23 psi	189.74psi	0.8866	OK
+1.20D+1.60L+0.50S	171.96 psi	189.74psi	0.9063	OK
+1.20D+0.50L	115.43 psi	189.74psi	0.6084	OK
+1.20D	91.43 psi	189.74psi	0.4819	OK
+1.20D+0.50L+1.60S	127.36 psi	189.74psi	0.6712	OK
+1.20D+1.60S	103.36 psi	189.74psi	0.5447	OK
+1.20D+0.50L+0.50S	119.16 psi	189.74psi	0.628	OK
+1.20D+0.50L+0.70S	120.65 psi	189.74psi	0.6359	OK
+0.90D	68.57 psi	189.74psi	0.3614	OK

Project Title:
Engineer:
Project ID:
Project Descr:

General Footing

Project File: Rose Hill Foundation.ec6

LIC#: KW-06015244, Build:20.22.12.28

CG ENGINEERING

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DESCRIPTION: Soffit Column Footing - Worst Case

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

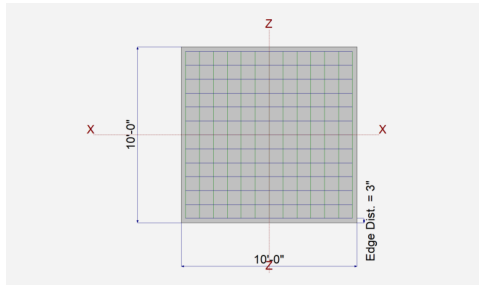
General Information

Material Properties		Soil Design Values	
f _c : Concrete 28 day strength	=	4.0 ksi	Allowable Soil Bearing
f _y : Rebar Yield	=	60.0 ksi	Soil Density
E _c : Concrete Elastic Modulus	=	3,122.0 ksi	Increase Bearing By Footing Weight
Concrete Density	=	150.0 pcf	Soil Passive Resistance (for Sliding)
φ Values	=	0.90	Soil/Concrete Friction Coeff.
Flexure	=	0.750	
Shear	=		
Analysis Settings		Increases based on footing Depth	
Min Steel % Bending Reinf.	=		Footing base depth below soil surface
Min Allow % Temp Reinf.	=	0.00180	Allow press. increase per foot of depth
Min. Overturning Safety Factor	=	1.0 : 1	when footing base is below
Min. Sliding Safety Factor	=	1.0 : 1	
Add Ftg Wt for Soil Pressure	: Yes		Increases based on footing plan dimension
Use ftg wt for stability, moments & shears	: Yes		Allowable pressure increase per foot of depth
Add Pedestal Wt for Soil Pressure	: No		when max. length or width is greater than
Use Pedestal wt for stability, mom & shear	: No		

Dimensions

Width parallel to X-X Axis	=	10.0 ft
Length parallel to Z-Z Axis	=	10.0 ft
Footing Thickness	=	26.0 in

Pedestal dimensions...	
px : parallel to X-X Axis	= in
pz : parallel to Z-Z Axis	= in
Height	= in
Rebar Centerline to Edge of Concrete... at Bottom of footing	= 3.0 in



Reinforcing

Bars parallel to X-X Axis	=	13.0
Number of Bars	=	# 6
Reinforcing Bar Size	=	# 6
Bars parallel to Z-Z Axis	=	13.0
Number of Bars	=	# 6
Reinforcing Bar Size	=	# 6

Bandwidth Distribution Check (ACI 15.4.4.2)
Direction Requiring Closer Separation

# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

	D	L _r	L	S	W	E	H
P : Column Load	=	236.0	57.30	16.40			k
OB : Overburden	=						ksf
M-xx	=						k-ft
M-zz	=						k-ft
V-x	=						k
V-z	=						k

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DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9799	Soil Bearing	3.258 ksf	3.325 ksf	+D+L about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.8239	Z Flexure (+X)	47.885 k-ft/ft	58.119 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8239	Z Flexure (-X)	47.885 k-ft/ft	58.119 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8239	X Flexure (+Z)	47.885 k-ft/ft	58.119 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.8239	X Flexure (-Z)	47.885 k-ft/ft	58.119 k-ft/ft	+1.20D+1.60L+0.50S
PASS	0.4535	1-way Shear (+X)	43.027 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4535	1-way Shear (-X)	43.027 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4535	1-way Shear (+Z)	43.027 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.4535	1-way Shear (-Z)	43.027 psi	94.868 psi	+1.20D+1.60L+0.50S
PASS	0.9160	2-way Punching	173.798 psi	189.737 psi	+1.20D+1.60L+0.50S

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location				Actual / Allow Ratio
				Bottom, -Z	Top, +Z	Left, -X	Right, +X	
X-X, D Only	3.325	n/a	0.0	2.685	2.685	n/a	n/a	0.808
X-X, +D+L	3.325	n/a	0.0	3.258	3.258	n/a	n/a	0.980
X-X, +D+S	3.325	n/a	0.0	2.849	2.849	n/a	n/a	0.857
X-X, +D+0.750L	3.325	n/a	0.0	3.115	3.115	n/a	n/a	0.937
X-X, +D+0.750L+0.750S	3.325	n/a	0.0	3.238	3.238	n/a	n/a	0.974
X-X, +0.60D	3.325	n/a	0.0	1.611	1.611	n/a	n/a	0.485
Z-Z, D Only	3.325	0.0	n/a	n/a	n/a	2.685	2.685	0.808
Z-Z, +D+L	3.325	0.0	n/a	n/a	n/a	3.258	3.258	0.980
Z-Z, +D+S	3.325	0.0	n/a	n/a	n/a	2.849	2.849	0.857
Z-Z, +D+0.750L	3.325	0.0	n/a	n/a	n/a	3.115	3.115	0.937
Z-Z, +D+0.750L+0.750S	3.325	0.0	n/a	n/a	n/a	3.238	3.238	0.974
Z-Z, +0.60D	3.325	0.0	n/a	n/a	n/a	1.611	1.611	0.485

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrm. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	41.30	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.40D	41.30	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+1.60L	46.860	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+1.60L	46.860	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+1.60L+0.50S	47.885	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+1.60L+0.50S	47.885	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L	38.981	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L	38.981	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D	35.40	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D	35.40	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+1.60S	42.261	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+1.60S	42.261	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+1.60S	38.680	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK

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Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in²	Gvm. As in²	Actual As in²	Phi*Mn k-ft	Status
X-X, +1.20D+1.60S	38.680	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+0.50S	40.006	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+0.50S	40.006	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+0.70S	40.416	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +1.20D+0.50L+0.70S	40.416	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +0.90D	26.550	+Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
X-X, +0.90D	26.550	-Z	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.40D	41.30	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.40D	41.30	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60L	46.860	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60L	46.860	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60L+0.50S	47.885	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60L+0.50S	47.885	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L	38.981	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L	38.981	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D	35.40	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D	35.40	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+1.60S	42.261	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+1.60S	42.261	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60S	38.680	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+1.60S	38.680	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+0.50S	40.006	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+0.50S	40.006	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+0.70S	40.416	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +1.20D+0.50L+0.70S	40.416	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +0.90D	26.550	-X	Bottom	0.5616	AsMin	0.5720	58.119	OK
Z-Z, +0.90D	26.550	+X	Bottom	0.5616	AsMin	0.5720	58.119	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	37.11 psi	37.11 psi	37.11 psi	37.11 psi	37.11 psi	94.87 psi	0.39	OK
+1.20D+1.60L	42.11 psi	42.11 psi	42.11 psi	42.11 psi	42.11 psi	94.87 psi	0.44	OK
+1.20D+1.60L+0.50S	43.03 psi	43.03 psi	43.03 psi	43.03 psi	43.03 psi	94.87 psi	0.45	OK
+1.20D+0.50L	35.03 psi	35.03 psi	35.03 psi	35.03 psi	35.03 psi	94.87 psi	0.37	OK
+1.20D	31.81 psi	31.81 psi	31.81 psi	31.81 psi	31.81 psi	94.87 psi	0.34	OK
+1.20D+0.50L+1.60S	37.97 psi	37.97 psi	37.97 psi	37.97 psi	37.97 psi	94.87 psi	0.40	OK
+1.20D+1.60S	34.76 psi	34.76 psi	34.76 psi	34.76 psi	34.76 psi	94.87 psi	0.37	OK
+1.20D+0.50L+0.50S	35.95 psi	35.95 psi	35.95 psi	35.95 psi	35.95 psi	94.87 psi	0.38	OK
+1.20D+0.50L+0.70S	36.32 psi	36.32 psi	36.32 psi	36.32 psi	36.32 psi	94.87 psi	0.38	OK
+0.90D	23.86 psi	23.86 psi	23.86 psi	23.86 psi	23.86 psi	94.87 psi	0.25	OK

All units k

Two-Way "Punching" Shear

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	149.90 psi	189.74 psi	0.79	OK
+1.20D+1.60L	170.08 psi	189.74 psi	0.8964	OK
+1.20D+1.60L+0.50S	173.80 psi	189.74 psi	0.916	OK
+1.20D+0.50L	141.48 psi	189.74 psi	0.7457	OK
+1.20D	128.48 psi	189.74 psi	0.6772	OK
+1.20D+0.50L+1.60S	153.39 psi	189.74 psi	0.8084	OK
+1.20D+1.60S	140.39 psi	189.74 psi	0.7399	OK
+1.20D+0.50L+0.50S	145.20 psi	189.74 psi	0.7653	OK
+1.20D+0.50L+0.70S	146.69 psi	189.74 psi	0.7731	OK
+0.90D	96.36 psi	189.74 psi	0.5079	OK