



civil & structural
engineering & planning



Town of
La Conner
Planning Department

Received
3-8-2024

STRUCTURAL CALCULATIONS

The Talmon

306 Centre Street
La Conner, WA 98257



250 4th Ave S Ste 200
Edmonds, WA 98020
Phone: (425) 778-8500
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CG Project No.: 23154.10

Project Description and Scope

This project involves the design of a new 3 level, 19-unit apartment building. The usable area of the proposed structure will be 28,000 sf. The upper levels will be dwelling units and the ground level will consist of common spaces, utility rooms, 8 dwelling units, and parking stalls. The framing will consist of TJI joists with PSL and common lumber beams with steel beams and/or columns where required. The foundation system will be conventional concrete spread footings. The lateral system will consist of plywood sheathed shear walls at upper levels and steel moment frames at the ground level.

Gravity Summary

Design Per 2018 IBC
Conventional Spread Footing Foundation
11 7/8" Plywood Web Joist Roof System
11 7/8" Plywood Web Joist Floor System
Wood or PSL Beams & Wood Columns

Floor Gravity Design Loads

Roof DL	17 PSF
Roof LL (Snow)	25 PSF
Floor DL	23 PSF
Floor LL	40 PSF
Common Space LL	100 PSF

Lateral Summary

Design Per 2018 IBC & ASCE 7-16
Flexible Diaphragm Analysis
Light-Framed Wood Shear Walls

Seismic Parameters

Soil Site Class D (Per Geotech)
 $S_{DS} = 0.960$ (Lat. -122.493, Long. 48.393)
 $R = 6.5$ (Light-Framed Wood Shear Walls)
 $I = 1.0$ (Non-Essential Facility)

Wind Parameters

98 MPH Wind Speed, 3-Sec Gust
Exposure Category C
 $I_w = 1.0$ (Non-Essential Facility)
Mean Roof Height = 35' Above Grade Elevation



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

Description	Project Summary	By	CCC	Date	07/21/23
		Checked		Date	
		Scale		Sheet No.	
Project	The Talmon	Job No.	23154.10		0.001

GENERAL INDEX

The Talmon
Centre Street
La Conner, WA 98257

SECTION TITLE / DESCRIPTION	SECTION NO.
GRAVITY DESIGN	1
LATERAL DESIGN	2

SECTION NO. 1

The Talmon
Centre Street
La Conner, WA 98257

SECTION TITLE: Main Building Gravity Design

SECTION INDEX

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Gravity Design Loads

Roof DL

Roofing Material	2.5	psf
5/8 Sheathing	1.8	psf
Insulation	1.0	psf
(2) 5/8 Gypsum	5.6	psf
2x12 @ 24" oc	2.2	psf
M/E	1.0	psf
Sprinklers	1.5	psf
Misc	1.0	psf
	16.6	psf
USE	17.0	psf

Floor DL

1" gypcrete	8.8	psf	(105 pcf concrete)
3/4 Sheathing	2.3	psf	
Insulation	1.0	psf	
(2) 5/8 Gypsum	5.6	psf	
2x10 @ 16" oc	1.8	psf	
M/E	1.0	psf	
Sprinklers	1.5	psf	
Misc	1.0	psf	
	23.0	psf	
USE	23.0	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
Deck LL	60.0	psf

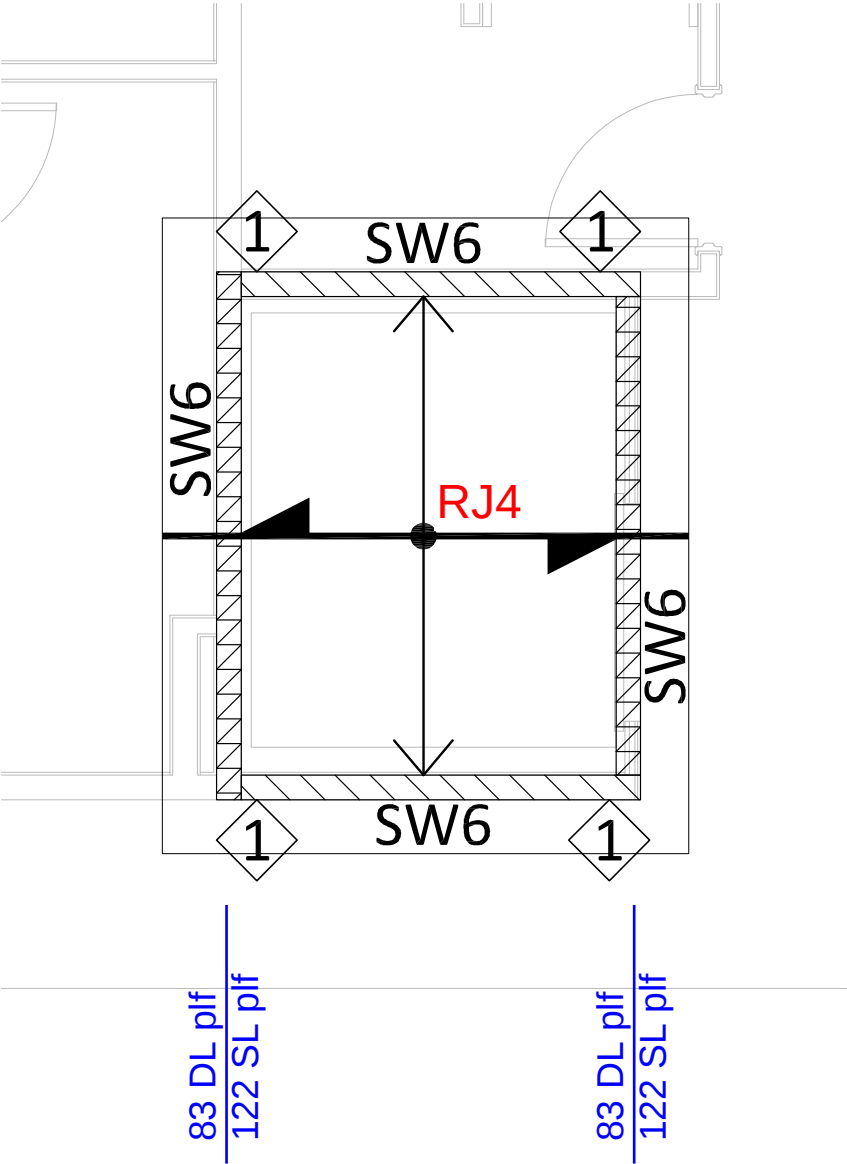


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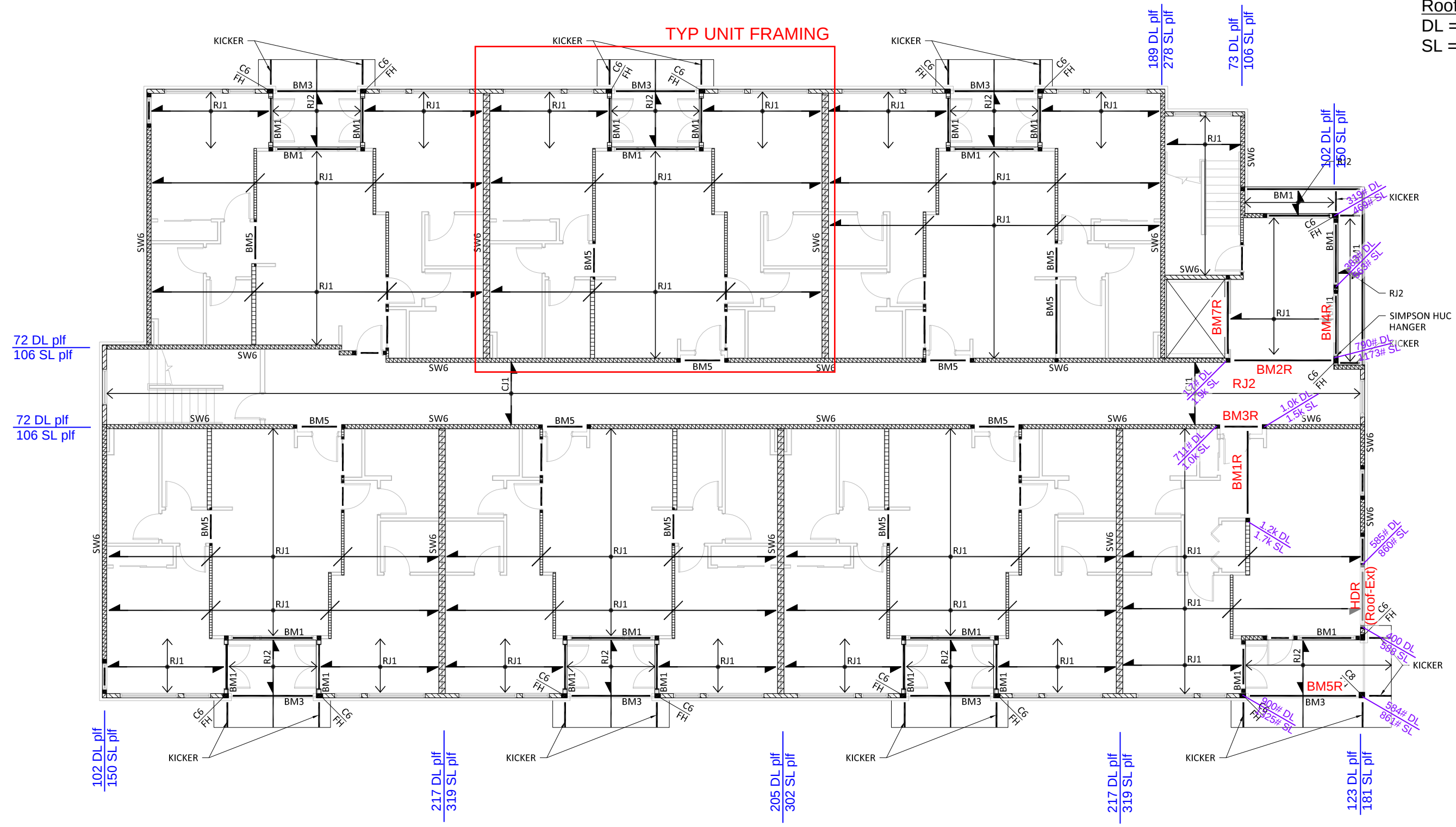
UPPER ROOF FRAMING KEY PLAN

Roof Loading:
DL = 17 PSF
SL = 25 PSF



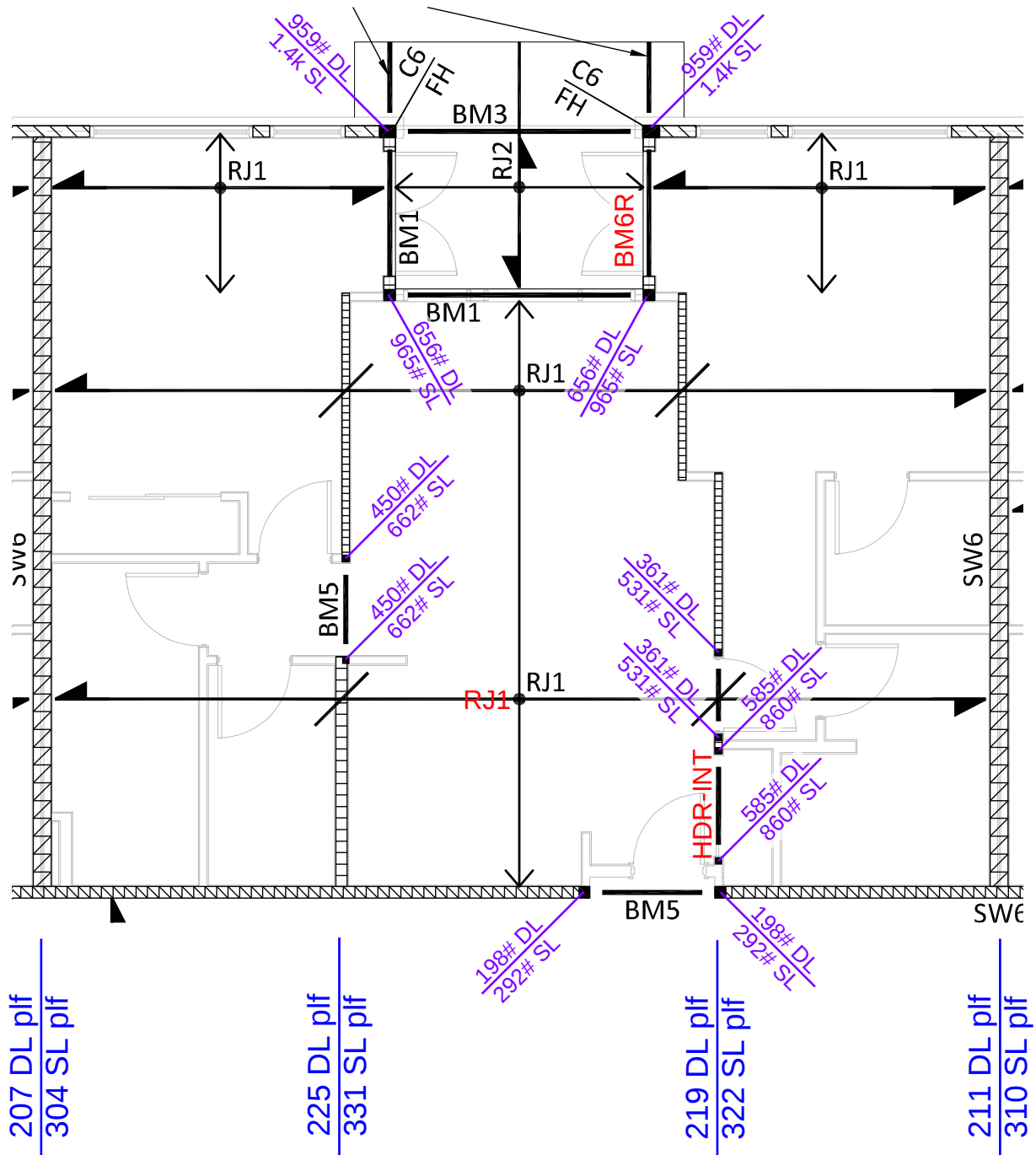
ROOF FRAMING KEY PLAN

Roof Loading:
DL = 17 PSF
SL = 25 PSF



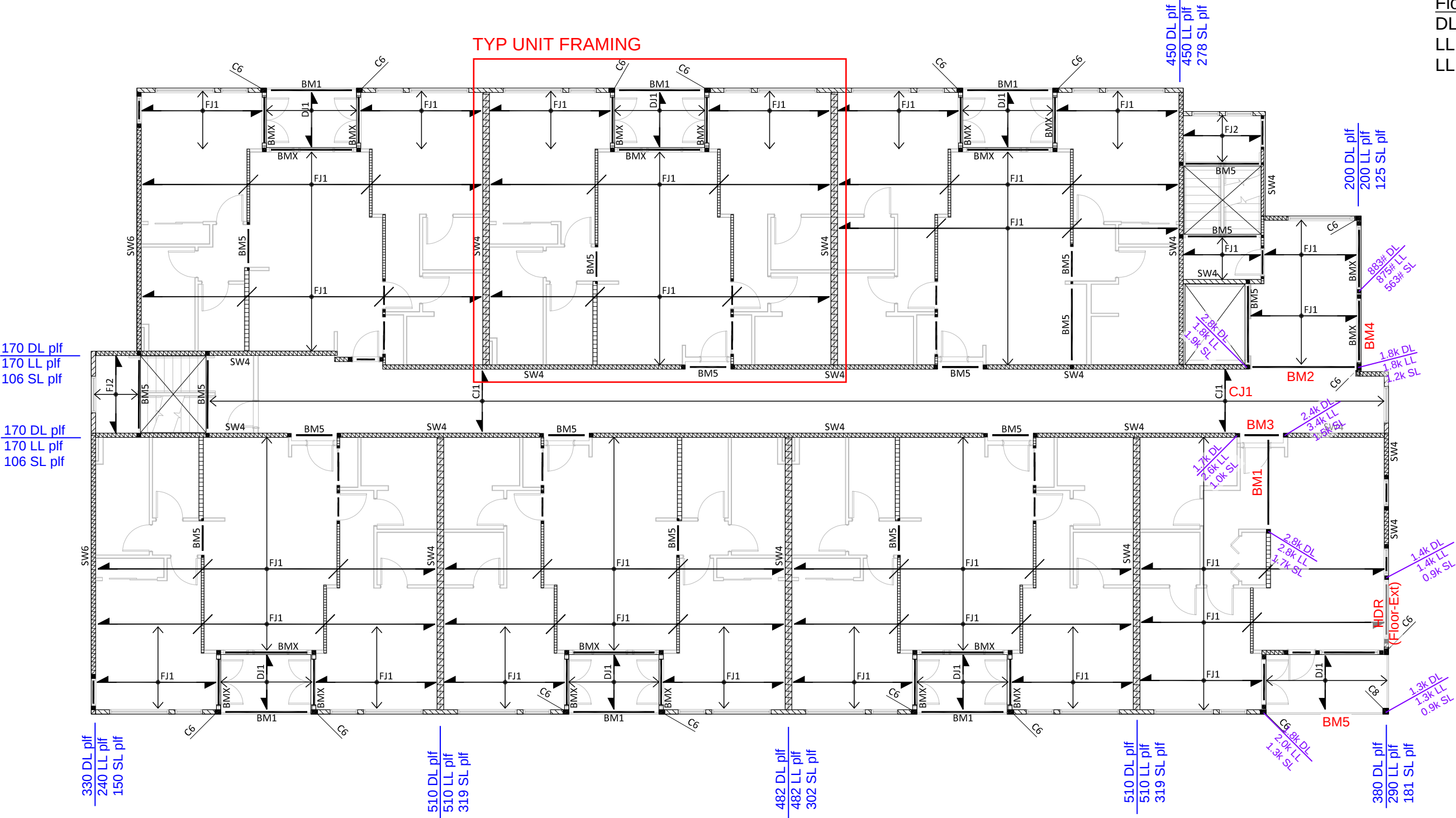
*Non-typical framing indicated on this overall key plan

TYP UNIT FRAMING KEY PLAN (ROOF)



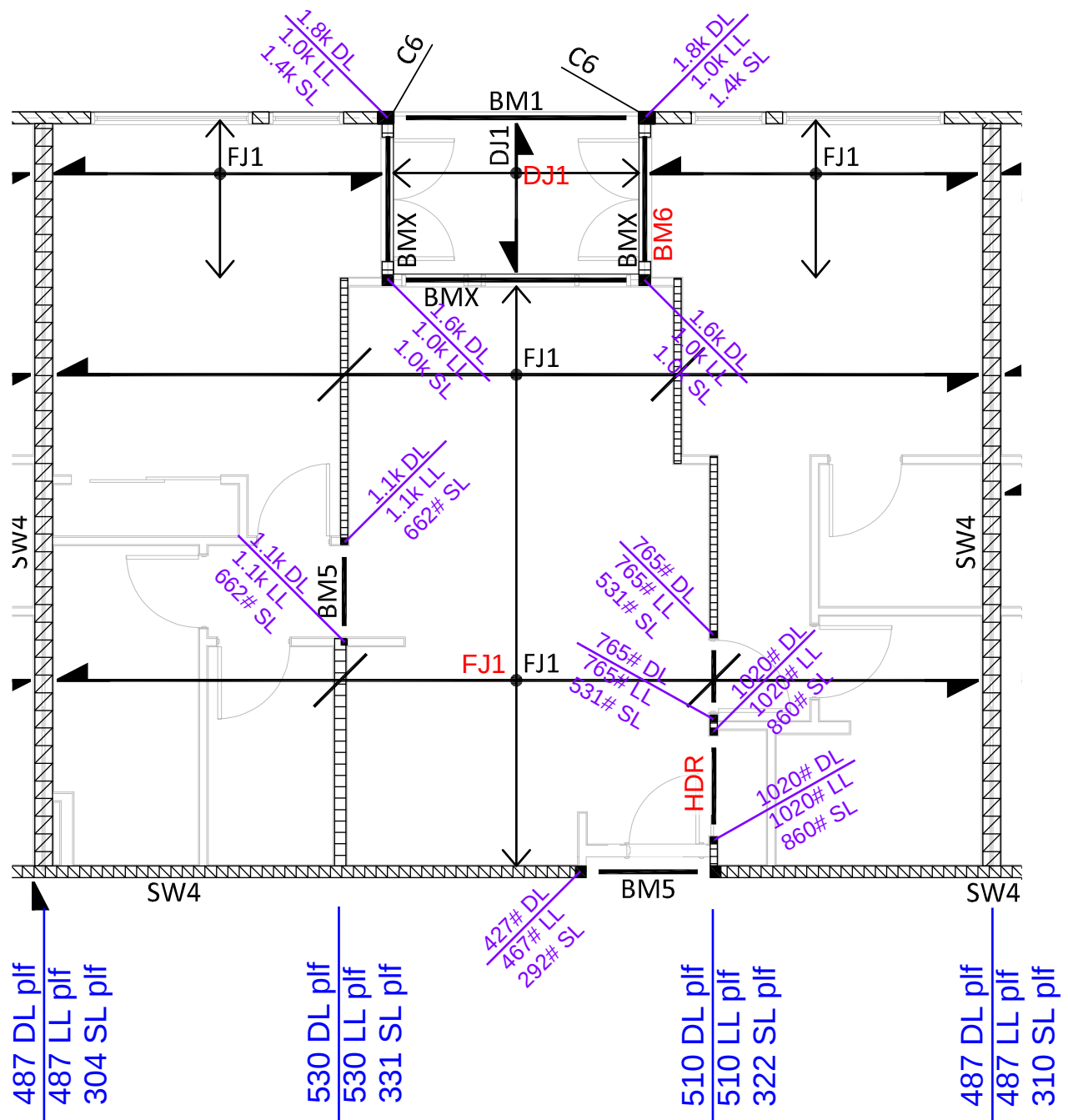
THIRD FLOOR FRAMING KEY PLAN

Floor Loading:
DL = 23 PSF
LL = 40 PSF
LL (Deck) = 60 PSF

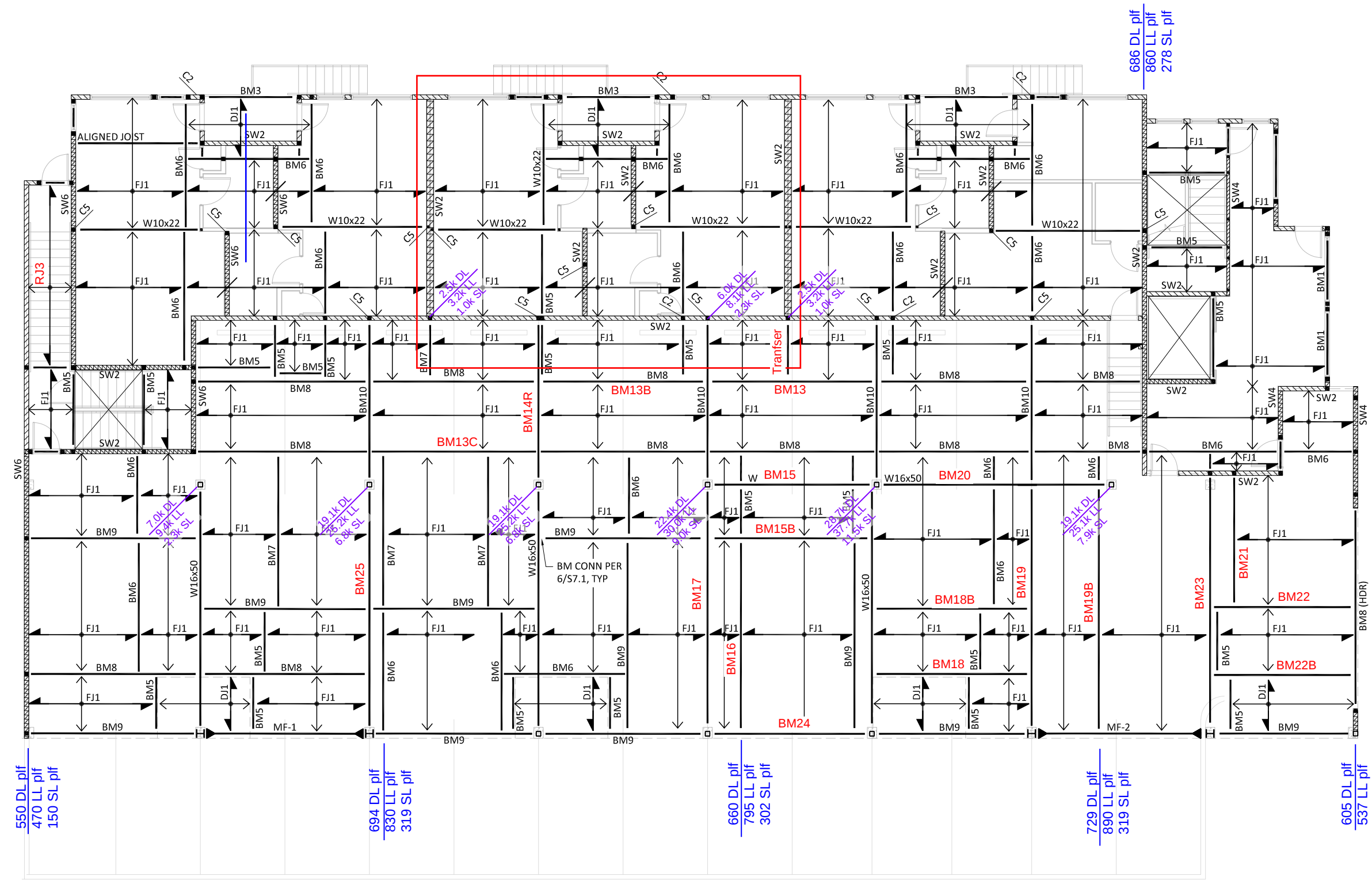


*Non-typical framing indicated on this overall key plan

TYP UNIT FRAMING KEY PLAN (3rd FLOOR)



SECOND FLOOR FRAMING KEY PLAN



Floor Loading:
DL = 23 PSF
LL = 40 PSF
LL (Corridor) = 100 PSF
LL (Deck) = 60 PSF

170 DL plf
170 LL plf
106 SL plf

170 DL plf
170 LL plf
106 SL plf

550 DL plf
470 LL plf
150 SL plf

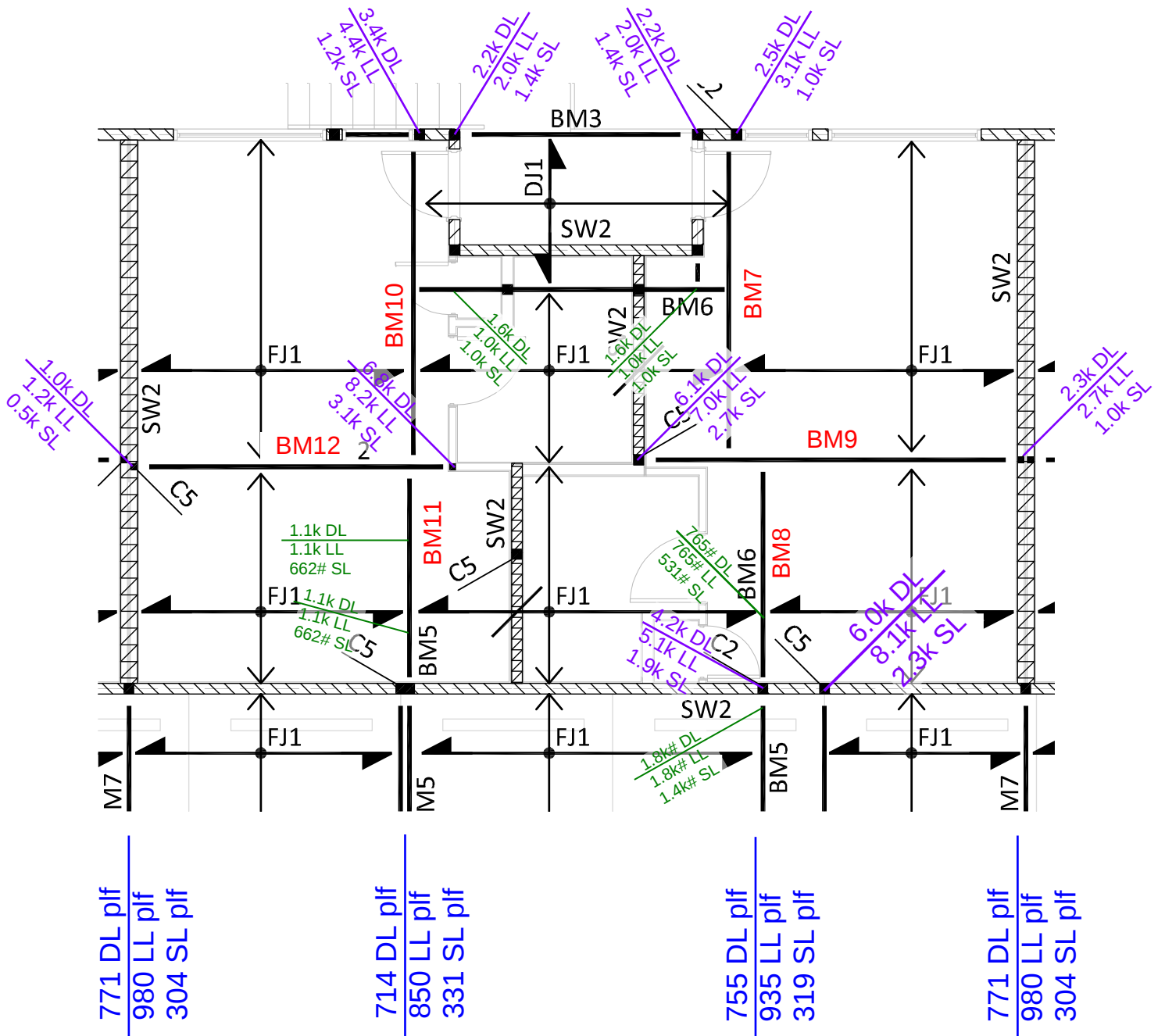
694 DL plf
830 LL plf
319 SL plf

660 DL plf
795 LL plf
302 SL plf

729 DL plf
890 LL plf
319 SL plf

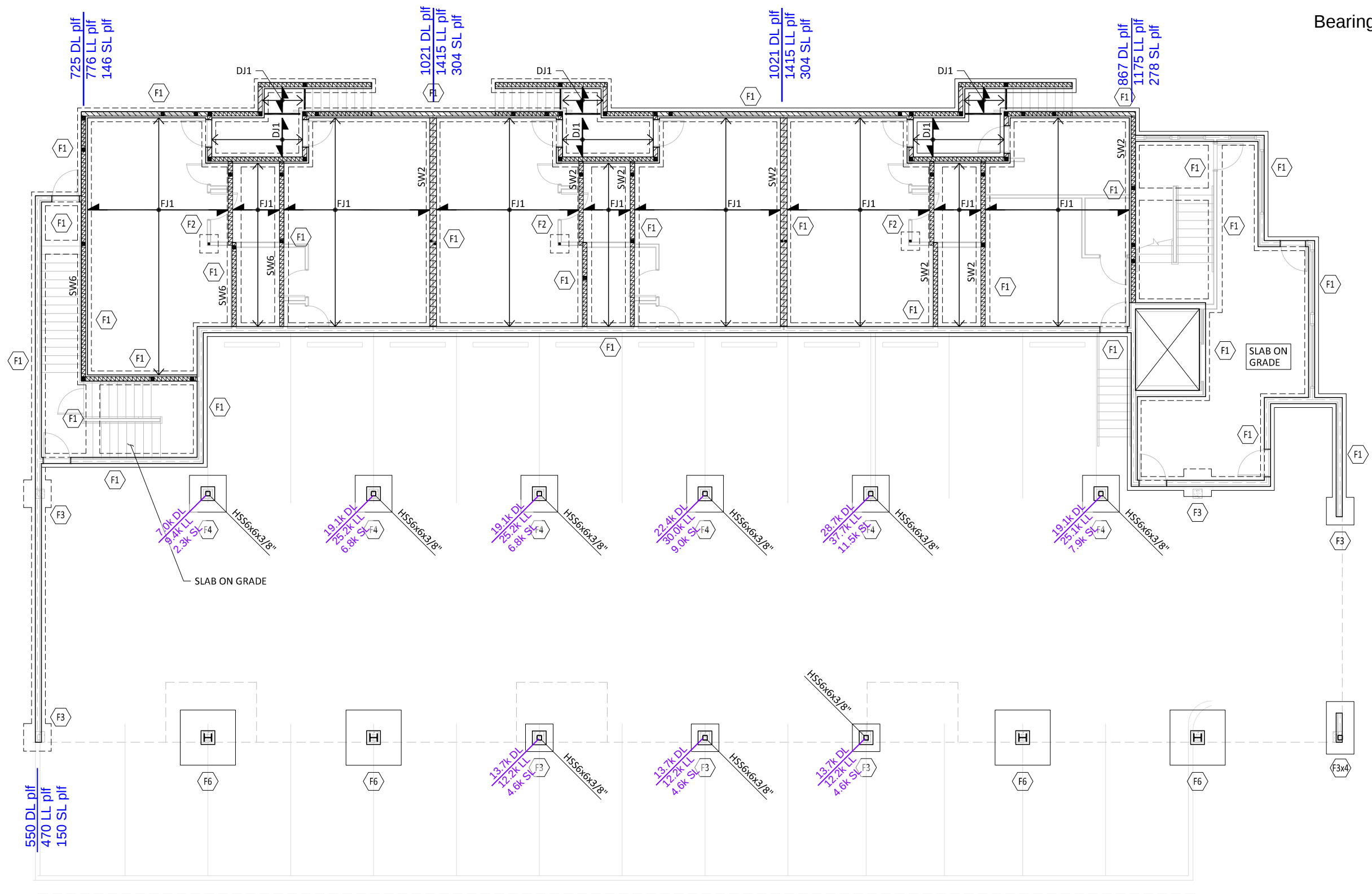
605 DL plf
537 LL plf
168 SL plf

TYP UNIT FRAMING KEY PLAN (2nd FLOOR)



FOUNDATION KEY PLAN

Bearing Capacity: 4000 psf



Beam Span Table - Roof Beams

Allowable Uniform Distributed Load in Pounds Per Lineal Foot (PLF)																	
	Span Length in Feet																
Beam	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)



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		Checked		Date	
		Scale		Sheet No.	
Project	The Talmon	Job No.	23154.10		

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)



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Suite 200
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Description	Beam Span Table	By	CCC	Date	06/19/23
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BEARING WALL TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

	ALLOWABLE LOAD (PLF)										
STUD	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	07/21/23
Bearing Wall Capacity Table	Checked		Date	
Project	Scale		Sheet No.	
The Talmon	Job No.			
	23154.10			

DF Column & DF Sill Plate Capacity TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

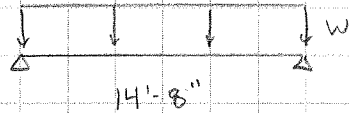
	6	7	8	9	10	11	12	13	14	15	16
(2) 2x4 DF stud	5,781	4,680	3,786	3,092	2,559	2,146	1,823	1,565	1,358	1,188	1,048
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
(3) 2x4 DF Stud	10,202	8,671	7,239	6,029	5,050	4,270	3,645	3,143	2,734	2,398	2,119
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
(4) 2x4 DF Sud	13,603	11,561	9,652	8,039	6,734	5,693	4,860	4,190	3,645	3,197	2,825
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	-	-	-	-	-	-	-	-	-	-
(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
P _{SILL}	-	-	-	-	-	-	-	-	-	-	-
(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
P _{SILL}	16,758	-	-	-	-	-	-	-	-	-	-
(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
P _{SILL}	20,625	20,625	20,625	20,625	-	-	-	-	-	-	-
4x6 HF #1	14,555	11,399	9,047	7,309	6,007	5,015	4,245	3,638	3,150	2,753	2,426
P _{SILL}	12,852	-	-	-	-	-	-	-	-	-	-
4x8 HF #1	18,948	14,909	11,863	9,597	7,896	6,596	5,586	4,788	4,147	3,625	3,195
P _{SILL}	15,859	-	-	-	-	-	-	-	-	-	-
4x10 HF #1	23,837	18,854	15,046	12,193	10,041	8,395	7,113	6,098	5,283	4,620	4,073
P _{SILL}	20,234	-	-	-	-	-	-	-	-	-	-
6x6 DF #1	26,785	25,169	23,176	20,923	18,605	16,403	14,421	12,692	11,207	9,939	8,857
P _{SILL}	20,195	20,195	20,195	20,195	-	-	-	-	-	-	-
6x8 DF #1	35,307	33,177	30,550	27,580	24,525	21,622	19,010	16,730	14,772	13,101	11,675
P _{SILL}	24,922	24,922	24,922	24,922	-	-	-	-	-	-	-
6x10 DF #1	42,159	39,888	37,057	33,783	30,314	26,924	23,802	21,032	18,626	16,554	14,776
P _{SILL}	31,797	31,797	31,797	31,797	-	-	-	-	-	-	-



Description	By	INT	Date
Wood Column Capacity Table	Checked		07/21/23
Project	Scale		Sheet No.
Project Name	Job No.		
	15xxx.10		

Joist Design:

RJ1:



Use 11 $\frac{1}{8}$ " TJI 110 @ 24" OC

$$W = 17 \text{ psf DL} + 25 \text{ psf SL}$$

$$M = 65\% \quad 2.4 \text{ k-ft}$$

$$V = 35\% \quad 623 \text{ lbs}$$

$$\Delta_T = L/449 \quad 0.401"$$

RJ2 (corridor):



Use 2x8 HF #2 @ 24" OC

$$W = 17 \text{ psf DL} + 25 \text{ psf SL}$$

$$M = 78\% \quad 1158 \text{ lbs-ft}$$

$$V = 31\% \quad 383 \text{ lbs}$$

$$\Delta_T = L/340 \quad 0.221"$$

FJ1:



Use 11 $\frac{1}{8}$ " TJI 110 @ 16" OC

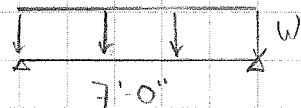
$$W = 23 \text{ psf DL} + 40 \text{ psf LL}$$

$$M = 73\% \quad 2.3 \text{ k-ft}$$

$$\Delta_T = L/567 \quad 0.314"$$

$$\text{TJI Rating} = 47$$

CJ1:



Use 2x6 HF #2 @ 16" OC

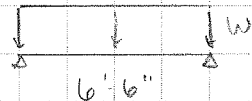
$$W = 23 \text{ psf DL} + 40 \text{ psf LL}$$

$$M = 64\% \quad 514 \text{ ft-lbs}$$

$$V = 31\% \quad 250 \text{ lbs}$$

$$\Delta_T = L/423 \quad 0.20"$$

DS1:



Use 2x6 HF #2 @ 16" OC

$$W = 23 \text{ psf DL} + 60 \text{ psf LL}$$

$$M = 77\% \quad 615 \text{ lbs-ft}$$

$$V = 37\% \quad 309 \text{ lbs}$$

$$\Delta_L = L/608 \quad 0.132"$$



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Description Joist Design

Project The Talmon

By CCC

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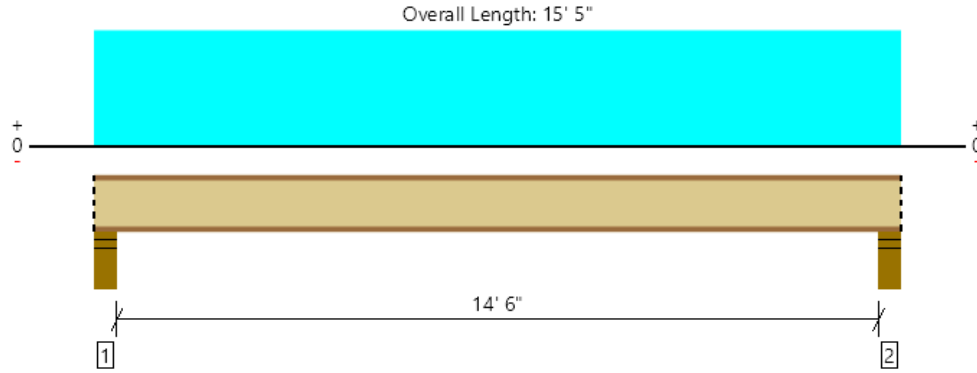
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Date 6/20/23

Date

Sheet No.

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)	648 @ 4 1/2"	1679 (3.50")	Passed (39%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	609 @ 5 1/2"	1903	Passed (32%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2259 @ 7' 8 1/2"	4364	Passed (52%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.189 @ 7' 8 1/2"	0.489	Passed (L/929)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.318 @ 7' 8 1/2"	0.733	Passed (L/553)	--	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"	1.75"	262	385	648	Blocking
2 - Stud wall - HF	5.50"	5.50"	1.75"	262	385	648	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)	4' 11" o/c	
Bottom Edge (Lu)	15' 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 15' 5"	24"	17.0	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

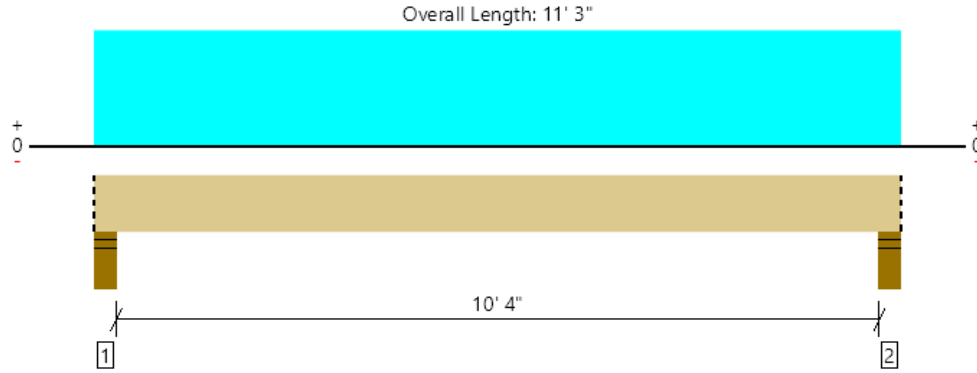
ForteWEB Software Operator	Job Notes
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Level, RJ2
1 piece(s) 2 x 8 HF No.2 @ 24" OC



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)	473 @ 4' 1/2"	3341 (5.50")	Passed (14%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	383 @ 1' 3/4"	1251	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1158 @ 5' 7 1/2"	1477	Passed (78%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.221 @ 5' 7 1/2"	0.350	Passed (L/571)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.371 @ 5' 7 1/2"	0.525	Passed (L/340)	--	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.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.

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"	1.50"	191	281	473	Blocking
2 - Stud wall - HF	5.50"	5.50"	1.50"	191	281	473	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)	6' 1" o/c	
Bottom Edge (Lu)	11' 3" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 11' 3"	24"	17.0	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

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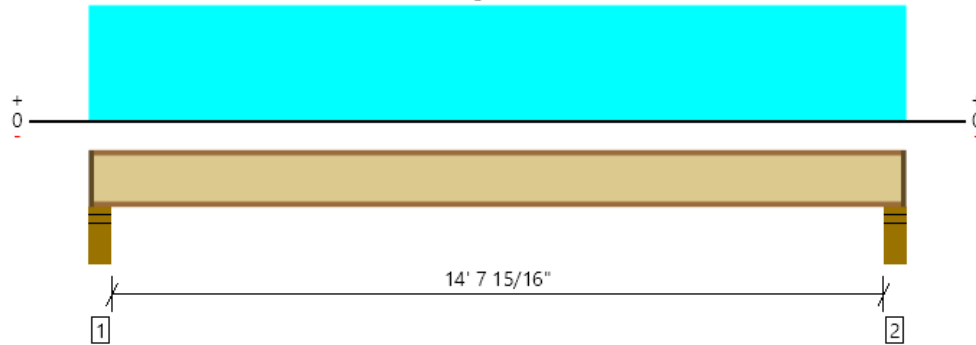
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Level, FJ1

1 piece(s) 11 7/8" TJI® 110 @ 16" OC

Overall Length: 15' 6 15/16"



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)	645 @ 4 1/2"	1375 (3.50")	Passed (47%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	616 @ 5 1/2"	1560	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2308 @ 7' 9 7/16"	3160	Passed (73%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.199 @ 7' 9 7/16"	0.371	Passed (L/894)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.313 @ 7' 9 7/16"	0.741	Passed (L/568)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	47	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 - HF	5.50"	4.25"	1.75"	239	415	654	1 1/4" Rim Board
2 - Stud wall - HF	5.50"	4.25"	1.75"	239	415	654	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' 8" o/c	
Bottom Edge (Lu)	15' 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)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 6 15/16"	16"	23.0	40.0	Default Load

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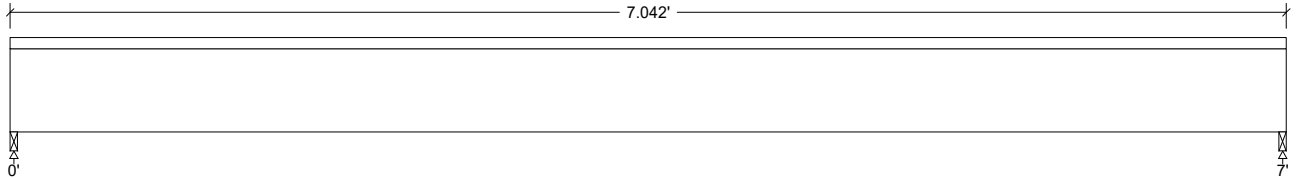
CJ1

Design Check Calculation Sheet
WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Full Area			23.00 (16.0")	psf
LL	Live	Full Area			40.00 (16.0")	psf

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



Unfactored:			
Dead	108		108
Live	188		188
Factored:			
Total	296		296
Bearing:			
Capacity			
Joist	304		304
Support	586		586
Des ratio			
Joist	0.97		0.97
Support	0.50		0.50
Load comb	#2		#2
Length	0.50*		0.50*
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.25		1.25
Fcp sup	625		625

*Minimum bearing length setting used: 1/2" for end supports

Lumber-soft, Hem-Fir, No.2, 2x6 (1-1/2"x5-1/2")

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

Floor joist spaced at 16.0" c/c; Total length: 7.06'; Clear span: 6.938'; Volume = 0.4 cu.ft.

Lateral support: top = continuous, bottom = at supports; Repetitive factor: applied where permitted (refer to online help);

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 = 46	Fv' = 150	psi	fv/Fv' = 0.31
Bending(+)	fb = 816	Fb' = 1271	psi	fb/Fb' = 0.64
Live Defl'n	0.11 = L/788	0.23 = L/360	in	0.46
Total Defl'n	0.20 = L/423	0.35 = L/240	in	0.57

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	LC#
Fv'	150	1.00	1.00	1.00	-	-	-	-	1.00	1.00	2
Fb'+	850	1.00	1.00	1.00	1.000	1.300	-	1.15	1.00	1.00	2
Fcp'	405	-	1.00	1.00	-	-	-	-	1.00	1.00	-
E'	1.3 million	1.00	1.00	-	-	-	-	-	1.00	1.00	2
Emin'	0.47 million	1.00	1.00	-	-	-	-	-	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 = 294, V design = 254 (NDS 3.4.3.1(a)) lbs; M(+) = 514 lbs-ft

EI = 27.04e06 lb-in^2

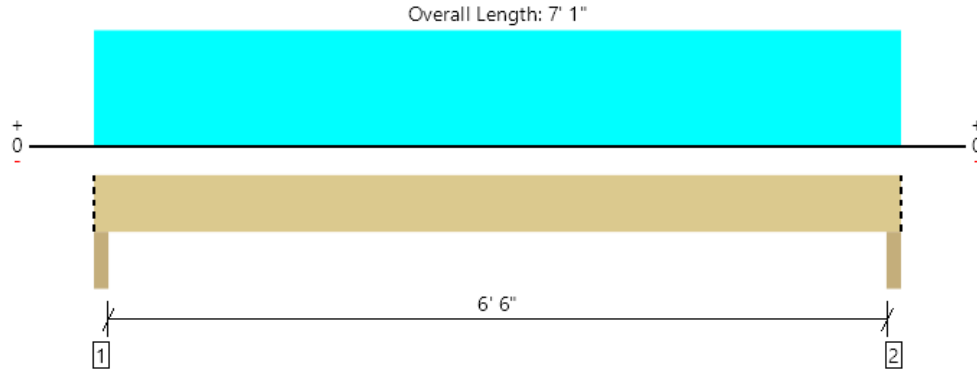
"Live" deflection is due to all non-dead loads (live, wind, snow...)

Total deflection = 1.50 permanent + "live"

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.

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)	392 @ 2 1/2"	2126 (3.50")	Passed (18%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	309 @ 9"	825	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	615 @ 3' 6 1/2"	801	Passed (77%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.132 @ 3' 6 1/2"	0.167	Passed (L/608)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.182 @ 3' 6 1/2"	0.333	Passed (L/440)	--	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 - Beam - PSL	3.50"	3.50"	1.50"	109	283	392	Blocking
2 - Beam - PSL	3.50"	3.50"	1.50"	109	283	392	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)	7' 1" o/c	
Bottom Edge (Lu)	7' 1" 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' 1"	16"	23.0	60.0	Default Load

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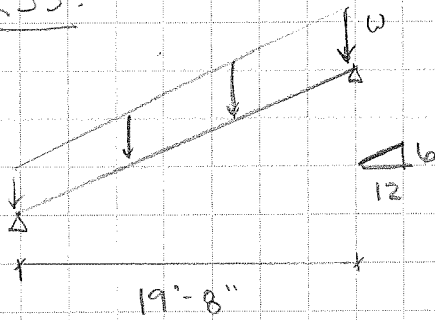
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RJ3:

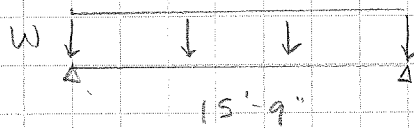


$$W = 17 \text{ psf DL} + 25 \text{ psf SL}$$

$$\begin{aligned} M &= 94\% & 4.1 \text{ K-ft} \\ V &= 44\% & 828 \text{ lbs} \\ \Delta_T &= L/217 & 1.23" \end{aligned}$$

Use 1 1/8" TJI 210 @ 24" OC

FJ1 (@ Crawlspace):



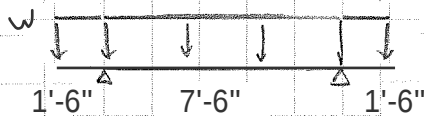
$$W = 17 \text{ psf DL} + 40 \text{ psf LL}$$

$$\begin{aligned} M &= 84\% & 2.66 \text{ K-ft} \\ V &= 42\% & 662 \text{ lbs} \\ \Delta_T &= L/467 & 0.409 \end{aligned}$$

Use 1 1/8" TJI 110 @ 16" OC

TJI Rating = 44

RJ4:



$$w = 17 \text{ psf DL} + 25 \text{ psf SL}$$

$$\begin{aligned} M &= 24\% \\ V &= 16\% \\ \Delta_T &= L/632 \end{aligned}$$

USE 2x10 HF #2 @ 24" OC



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www.cgeengineering.com

Description Jorst Design

Project The Talmon

By CCL

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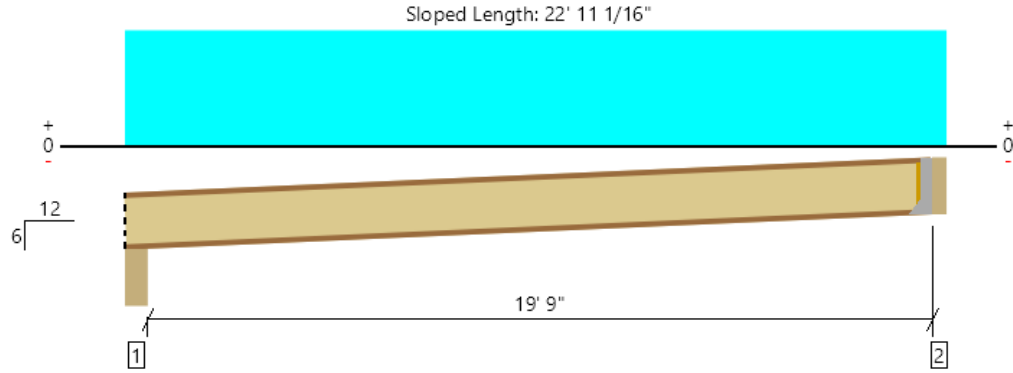
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Date 7/18/23

Date

Sheet No.

Level, RJ3
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.

Member Length : 23' 1 1/16"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	860 @ 4 1/2"	1156 (1.75")	Passed (74%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	828 @ 20' 2 1/2"	1903	Passed (44%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4108 @ 10' 3 1/2"	4364	Passed (94%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.735 @ 10' 3 1/2"	1.109	Passed (L/362)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	1.228 @ 10' 3 1/2"	1.478	Passed (L/217)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- 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 : 6/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - HF	5.50"	5.50"	1.75"	345	515	860	Blocking
2 - Hanger on 11 7/8" HF beam	3.50"	Hanger ¹	1.75" / - ²	341	510	852	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	22' 7" o/c	

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

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LSSR2.1Z	1.88"	N/A	14-10dx2.5	12-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20' 6"	24"	15.0	25.0	Default Load

Weyerhaeuser Notes

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

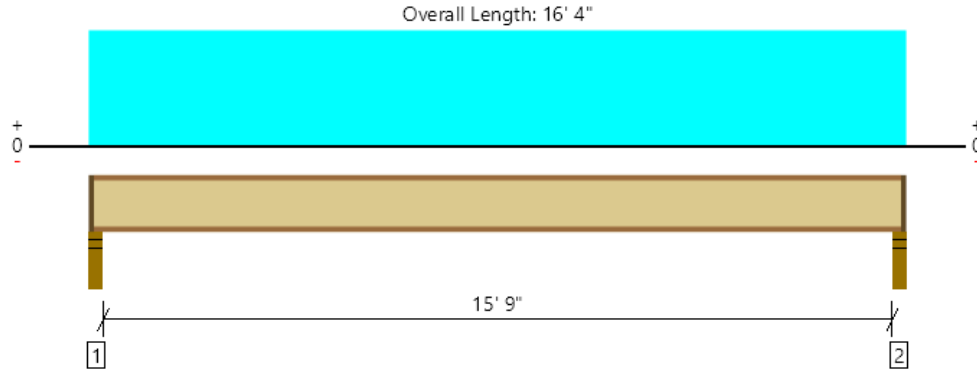
ForteWEB Software Operator	Job Notes
Colin Chase CG Engineering (425) 778-8500 colinc@cgengineering.com	



7/10/2023 6:10:42 PM UTC
ForteWEB v3.6, Engine: V8.3.0.43, Data: V8.1.4.1

File Name: 1022
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Level, FJ1 crawlspace
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)	677 @ 2 1/2"	1041 (2.25")	Passed (65%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	662 @ 3 1/2"	1560	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2660 @ 8' 2"	3160	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.260 @ 8' 2"	0.398	Passed (L/736)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.409 @ 8' 2"	0.796	Passed (L/467)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	44	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"	250	436	686	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	250	436	686	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' 4" o/c	
Bottom Edge (Lu)	16' 2" 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 16' 4"	16"	23.0	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

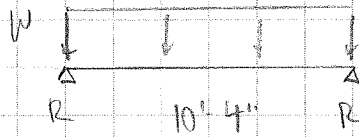
ForteWEB Software Operator	Job Notes
Colin Chase CG Engineering (425) 778-8500 colinc@cgengineering.com	



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BM1R:



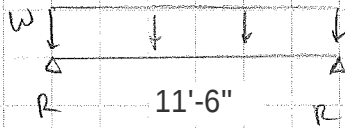
$$W = (227 \text{ DL} + 334 \text{ SL}) \text{ plf}$$

$$R = 1.2 \text{ k DL} + 1.7 \text{ k SL}$$

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

* From Roof Beam Table

BM2R:



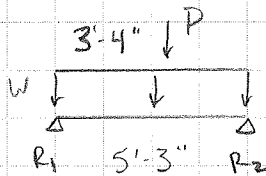
$$W = (72 \text{ DL} + 106 \text{ SL}) \text{ plf}$$

$$R = 415 \text{ \# DL} + 610 \text{ \# SL}$$

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

* From Roof Beam Table

BM3R:



$$P = 1.2 \text{ k DL} + 1.7 \text{ k SL}$$

$$W = (89 \text{ DL} + 131 \text{ SL}) \text{ plf}$$

$$R_1 = 711 \text{ \# DL} + 1022 \text{ \# SL}$$

$$R_2 = 1032 \text{ \# DL} + 1477 \text{ \# SL}$$

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

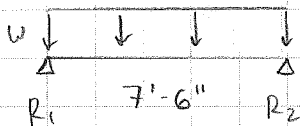
$$M = 19 \%$$

$$4.2 \text{ k-ft}$$

$$V = 24 \%$$

$$2.4 \text{ kips}$$

BM4R:



$$W = (102 \text{ DL} + 150 \text{ SL}) \text{ plf}$$

$$R_1 = 383 \text{ \# DL} + 563 \text{ \# SL}$$

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

* From Roof Beam Table



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BM3R

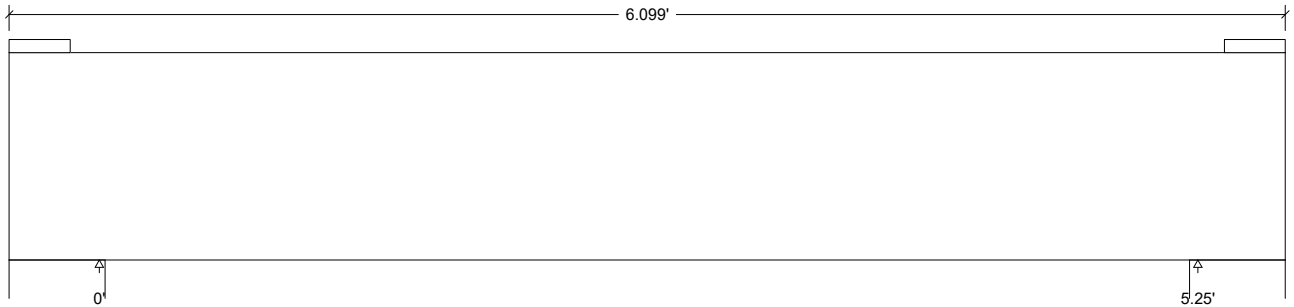
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLw	Dead	Full UDL			89.0	plf
SLw	Snow	Full UDL			131.0	plf
Dlp	Dead	Point		3.76	1200	lbs
SLp	Snow	Point		3.76	1700	lbs

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



Unfactored:			
Dead	711		1032
Snow	1022		1477
Factored:			
Total	1733		2509
Bearing:			
Capacity			
Beam	14438		14438
Support	17046		17046
Des ratio			
Beam	0.12		0.17
Support	0.10		0.15
Load comb	#2		#2
Length	5.50		5.50
Min req'd	0.66		0.96
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fc sup	700		700

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

Supports: All - Timber-soft Column, D.Fir-L No.2
Total length: 6.13'; Clear span: 5.188'; Volume = 1.8 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	$f_v = 79$	$F_v' = 334$	psi	$f_v/F_v' = 0.24$
Bending(+)	$f_b = 618$	$F_b' = 3268$	psi	$f_b/F_b' = 0.19$
Live Defl'n	$0.01 = < L/999$	$0.26 = L/240$	in	0.04
Total Defl'n	$0.02 = < L/999$	$0.35 = L/180$	in	0.06

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.15	-	1.00	0.979	1.001	-	1.00	1.00	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	2
Eminy'	1.14 million	-	-	1.00	-	-	-	-	1.00	-	2

CRITICAL LOAD COMBINATIONS:

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

D=dead 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

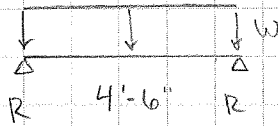
CALCULATIONS:

V max = 2417, V design = 2191 (NDS 3.4.3.1(a)) lbs; M(+) = 4235 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(+): Lu = 5.25' Le = 10.81' RB = 11.2

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.

HDR (Roof - Interior):



$$W = (219 \text{ DL} + 332 \text{ SL}) \text{ plf}$$

$$R = 493 \text{ \# DL} + 747 \text{ \# SL}$$

Use 4x6 HF #2 * From Beam Table

Max Span = 5'-0"

HDR (Roof - Exterior):



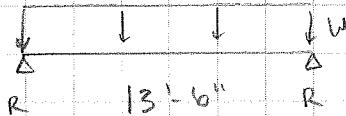
$$W = (123 \text{ DL} + 181 \text{ SL}) \text{ plf}$$

$$R = 400 \text{ \# DL} + 588 \text{ \# SL}$$

Use 4x6 HF #2 * From Beam Table

Max Span = 7'-0"

BM5R:

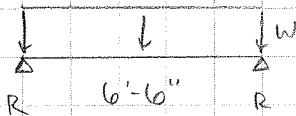


$$W = (114 \text{ DL} + 168 \text{ SL}) \text{ plf}$$

$$R = 770 \text{ \# DL} + 1134 \text{ \# SL}$$

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

BM6R:

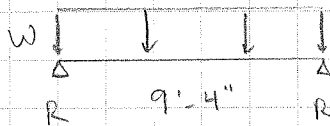


$$W = (115 \text{ DL} + 169 \text{ SL}) \text{ plf}$$

$$R = 375 \text{ \# DL} + 550 \text{ \# SL}$$

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

BM7R:



$$W = (275 \text{ DL} + 272 \text{ SL}) \text{ plf}$$

$$R = 1.3 \text{ k DL} + 1.3 \text{ k SL}$$

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



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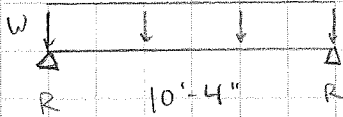
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Date 6/20/23

Date

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BM1:



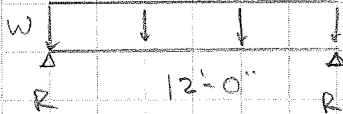
$$w = (307 \text{ DL} + 534 \text{ LL}) \text{ plf}$$

$$R = 1.6 \text{ k DL} + 2.8 \text{ k LL}$$

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

* From Beam Table

BM2:



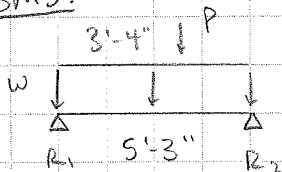
$$w = (86 \text{ DL} + 150 \text{ LL}) \text{ plf}$$

$$R = 518 \# \text{ DL} + 900 \# \text{ LL}$$

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

* From Beam Table

BM3:



$$P = 1.6 \text{ k DL} + 2.8 \text{ k LL}$$

$$w = (121 \text{ DL} + 525 \text{ LL}) \text{ plf}$$

$$R_1 = 951 \# \text{ DL} + 2.6 \text{ k LL}$$

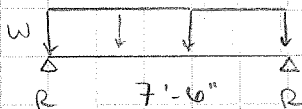
$$R_2 = 1.4 \text{ k DL} + 3.4 \text{ k LL}$$

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

$$M = 38\% \quad 7.4 \text{ k-ft}$$

$$V = 47\% \quad 4.5 \text{ kips}$$

BM4:

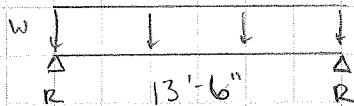


$$w = (133 \text{ DL} + 230 \text{ LL}) \text{ plf}$$

$$R = 500 \# \text{ DL} + 875 \# \text{ LL}$$

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

BM5:



$$w = (75 \text{ DL} + 195 \text{ LL}) \text{ plf}$$

$$R = 506 \# \text{ DL} + 1.3 \text{ k LL}$$

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

* From Beam Table



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BM3

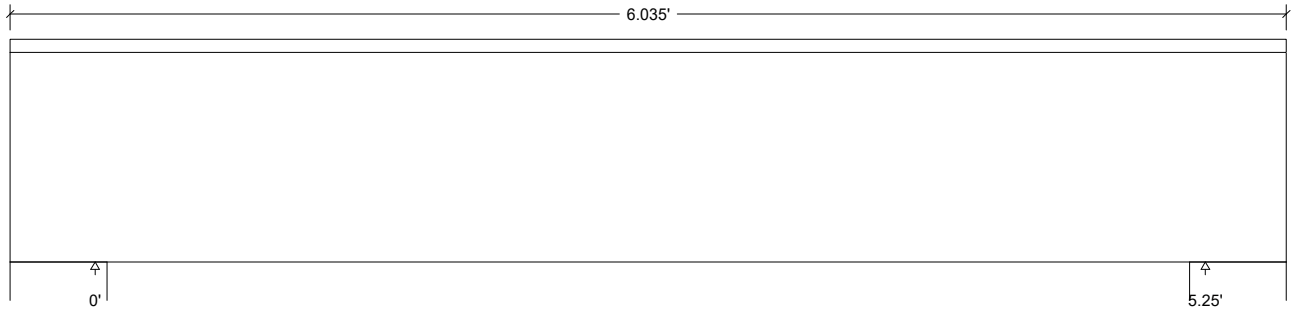
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLw	Dead	Full UDL			121.0	plf
LLw	Live	Full UDL			525.0	plf
Dlp	Dead	Point		3.73	1600	lbs
LLp	Live	Point		3.73	2800	lbs

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



Unfactored:			
Dead	951		1379
Live	2613		3355
Factored:			
Total	3564		4734
Bearing:			
Capacity			
Beam	14438		14438
Support	14822		14822
Des ratio			
Beam	0.25		0.33
Support	0.24		0.32
Load comb	#2		#2
Length	5.50		5.50
Min req'd	1.36		1.80
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fc sup	700		700

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

Supports: All - Timber-soft Column, D.Fir-L No.2
Total length: 6.06'; Clear span: 5.125'; Volume = 1.7 cu.ft.
Lateral support: top = continuous, 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 = 137	Fv' = 290	psi	fv/Fv' = 0.47
Bending(+)	fb = 1083	Fb' = 2903	psi	fb/Fb' = 0.37
Live Defl'n	0.02 = < L/999	0.17 = L/360	in	0.12
Total Defl'n	0.03 = < L/999	0.26 = L/240	in	0.13

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	1.001	-	1.00	1.00	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	2
Eminy'	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

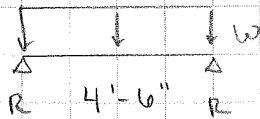
CALCULATIONS:

V max = 4487, V design = 3801 (NDS 3.4.3.1(a)) lbs; M(+) = 7424 lbs-ft
EI = 1074.51e06 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 = 1.50 permanent + "live"

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.

HDR (Floor - Interior):



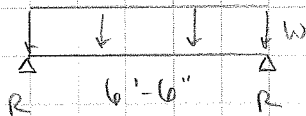
$$w = (293 \text{ DL} + 510 \text{ LL}) \text{ plf}$$

$$R = 660 \# \text{ DL} + 1150 \# \text{ LL}$$

Use 4x8 HF #2 * From Beam Table

Max Span = 5'-0"

HDR (Floor - Exterior):



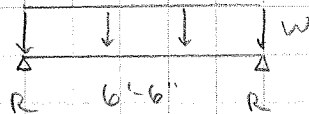
$$w = (167 \text{ DL} + 290 \text{ LL}) \text{ plf}$$

$$R = 543 \# \text{ DL} + 943 \# \text{ LL}$$

Use 4x8 HF #2 * From Beam Table

Max Span = 7'-0"

BM6:



$$w = (155 \text{ DL} + 270 \text{ LL}) \text{ plf}$$

$$R = 504 \# \text{ DL} + 878 \# \text{ LL}$$

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



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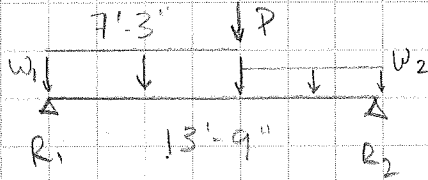
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BM7:



$$P = 1066 \text{ DL} + 667 \text{ LL} + 667 \text{ SL}$$

$$W_1 = (694 \text{ DL} + 830 \text{ LL} + 319 \text{ SL}) \text{ plf}$$

$$W_2 = (132 \text{ DL} + 230 \text{ LL}) \text{ plf}$$

$$R_1 = 4.3 \text{ k DL} + 5.0 \text{ k LL} + 2.0 \text{ k SL}$$

$$R_2 = 2.5 \text{ k DL} + 3.1 \text{ k LL} + 1.0 \text{ k SL}$$

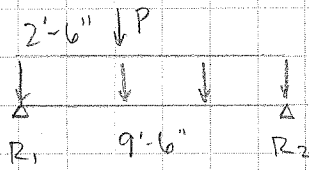
Use 7" x 11 7/8" PSL

$$M = 75\% \quad 29.7 \text{ k-ft}$$

$$V = 49\% \quad 7.9 \text{ kips}$$

$$\Delta_T = L/293 \quad 0.56"$$

BM8:



$$P = 765 \text{ DL} + 765 \text{ LL} + 531 \text{ SL}$$

$$W = (755 \text{ DL} + 935 \text{ LL} + 319 \text{ SL}) \text{ plf}$$

$$R_1 = 4.2 \text{ k DL} + 5.1 \text{ k LL} + 1.9 \text{ k SL}$$

$$R_2 = 3.9 \text{ k DL} + 4.7 \text{ k LL} + 1.7 \text{ k SL}$$

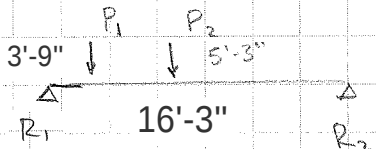
Use 5 1/4" x 11 7/8" PSL

$$M = 71\% \quad 21.0 \text{ k-ft}$$

$$V = 61\% \quad 9.2 \text{ kips}$$

$$\Delta_T = L/430 \quad 0.26"$$

BM9:



$$P_1 = 4.3 \text{ k DL} + 5.0 \text{ k LL} + 2.0 \text{ k SL}$$

$$P_2 = 3.9 \text{ k DL} + 4.7 \text{ k LL} + 1.7 \text{ k SL}$$

$$R_1 = 6.1 \text{ k DL} + 7.0 \text{ k LL} + 2.7 \text{ k SL}$$

$$R_2 = 2.3 \text{ k DL} + 2.7 \text{ k LL} + 1.0 \text{ k SL}$$

Use W10x22

$$M = 86\% \quad 55.7 \text{ k-ft}$$

$$V = 27\% \quad 13.6 \text{ kips}$$

$$\Delta_T = L/313 \quad 0.62"$$



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Description	By CCC	Date 7/11/23
	Checked	Date
	Scale	Sheet No.
Project The Talmon	Job No.	
	23157.10	



COMPANY

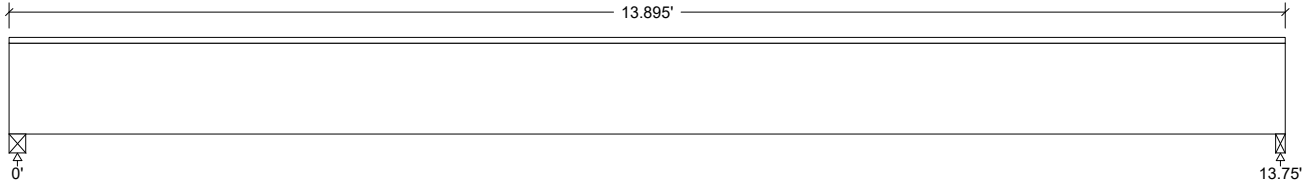
PROJECT

July 21, 2023 10:37

BM7

Design Check Calculation Sheet
WoodWorks Sizer 2023**Loads:**

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
Dlp	Dead	Point		7.34	1066	lbs
LLp	Live	Point		7.34	667	lbs
SLp	Snow	Point		7.34	667	lbs
DL1	Dead	Partial UDL		0.20 7.34	694.0 694.0	plf
LL1	Live	Partial UDL		0.20 7.34	830.0 830.0	plf
SL1	Snow	Partial UDL		0.20 7.34	319.0 319.0	plf
DL2	Dead	Partial UDL		7.34 13.84	132.0 132.0	plf
LL2	Live	Partial UDL		7.34 13.84	230.0 230.0	plf

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

Unfactored:			
Dead	4336		2543
Live	5009		3079
Snow	1983		961
Factored:			
Total	9580		5623
Bearing:			
Capacity			
Beam	11496		6748
Support	9580		5623
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#2
Length	2.19		1.29
Min req'd	2.19**		1.29**
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"x11-7/8"Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 13.88'; Clear span: 13.625'; Volume = 8.0 cu.ft.
Lateral support: top = continuous, 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	$f_v = 143$	$F_v' = 290$	psi	$f_v/F_v' = 0.49$
Bending(+)	$f_b = 2165$	$F_b' = 2903$	psi	$f_b/F_b' = 0.75$
Live Defl'n	$0.25 = L/665$	$0.46 = L/360$	in	0.54
Total Defl'n	$0.56 = L/293$	$0.69 = L/240$	in	0.82

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	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 #2 = D + L

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 = 9345, V design = 7915 (NDS 3.4.3.1(a)) lbs; M(+) = 29676 lbs-ft
EI = 2149.02e06 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"**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:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

CG ENGINEERING

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DESCRIPTION: BM9

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Load(s) for Span Number 1

Point Load : D = 4.50, L = 5.0, S = 2.0 k @ 3.750 ft

Point Load : D = 3.90, L = 4.70, S = 1.70 k @ 5.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.858 : 1	Maximum Shear Stress Ratio =	0.273 : 1
Section used for this span	W10x22	Section used for this span	W10x22
Ma : Applied	55.655 k-ft	Va : Applied	13.389 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+0.750L+0.750S
Span # where maximum occurs	Span # 1	Location of maximum on span	0.000 ft
		Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.328 in Ratio = 594 >=360	Span: 1 : L Only	
Max Upward Transient Deflection	0 in Ratio = 0 <360	n/a	
Max Downward Total Deflection	0.622 in Ratio = 313 >=240	Span: 1 : +D+0.750L+0.750S	
Max Upward Total Deflection	0 in Ratio = 0 <240.0	n/a	

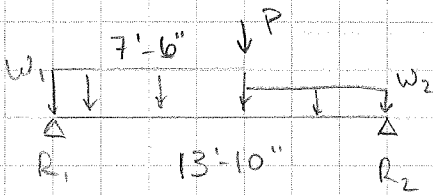
Vertical Reactions

Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	13.389	5.061
Max Upward from Load Combinations	13.389	5.061
Max Upward from Load Cases	7.028	2.672
D Only	6.102	2.298
+D+L	13.129	4.971
+D+S	8.791	3.309
+D+0.750L	11.372	4.303
+D+0.750L+0.750S	13.389	5.061
+0.60D	3.661	1.379
L Only	7.028	2.672
S Only	2.689	1.011

BM10:



$$P = 800 \text{ DL} + 500 \text{ LL} + 500 \text{ SL}$$

$$w_1 = (714 \text{ DL} + 850 \text{ LL} + 331 \text{ SL}) \text{ plf}$$

$$w_2 = (138 \text{ DL} + 240 \text{ LL}) \text{ plf}$$

$$R_1 = 4.5 \text{ k DL} + 5.2 \text{ k LL} + 2.0 \text{ k SL}$$

$$R_2 = 2.6 \text{ k DL} + 3.2 \text{ k LL} + 1.0 \text{ k SL}$$

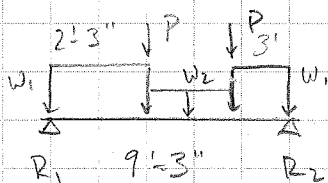
Use 7" x 11 7/8" PSL

$$M = 50\%$$

$$V = 76\%$$

$$\Delta_T = L/288$$

BM11:



$$P = 1.1 \text{ k DL} + 1.1 \text{ k LL} + 662 \text{ SL}$$

$$w_1 = (714 \text{ DL} + 850 \text{ LL} + 331 \text{ SL}) \text{ plf}$$

$$w_2 = (181 \text{ DL} + 315 \text{ LL}) \text{ plf}$$

$$R_1 = 4.7 \text{ k DL} + 2.8 \text{ k LL} + 1.6 \text{ k SL}$$

$$R_2 = 4.4 \text{ k DL} + 3.0 \text{ k LL} + 1.6 \text{ k SL}$$

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

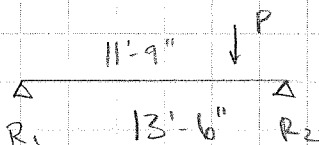
$$M = 80\%$$

$$V = 71\%$$

$$15.8 \text{ k-ft}$$

$$5.7 \text{ kips}$$

BM12:



$$P = 8.9 \text{ k DL} + 8.2 \text{ k LL} + 3.6 \text{ k SL}$$

$$R_1 = 1.1 \text{ k DL} + 1.0 \text{ k LL} + 0.5 \text{ k SL}$$

$$R_2 = 7.8 \text{ k DL} + 7.2 \text{ k LL} + 3.1 \text{ k SL}$$

Use 7" x 11 7/8" PSL

$$M = 64\%$$

$$V = 93\%$$

$$\Delta_T = L/470$$

$$25.5 \text{ k-ft}$$

$$15 \text{ kips}$$

$$0.34"$$



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Description Beam Design

Project The Talmon

By CCC

Checked

Scale

Job No.

23157.10

Date 7/11/23

Date

Sheet No.



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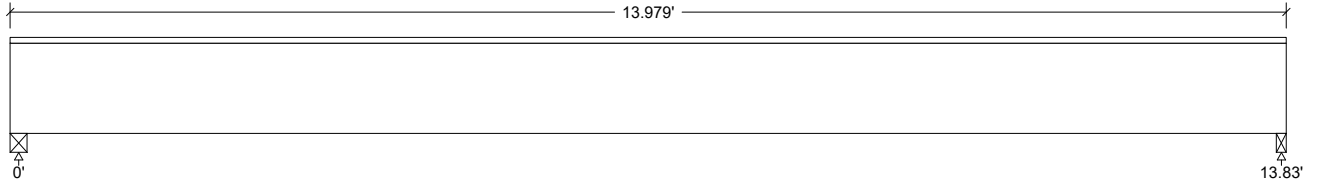
PROJECT

July 21, 2023 15:09

BM10

Design Check Calculation Sheet
WoodWorks Sizer 2023**Loads:**

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLw1	Dead	Partial UDL		0.09 7.59	714.0 714.0	plf
LLw1	Live	Partial UDL		0.09 7.59	850.0 850.0	plf
SLw1	Snow	Partial UDL		0.09 7.59	331.0 331.0	plf
DLw2	Dead	Partial UDL		7.59 13.92	138.0 138.0	plf
LLw2	Live	Partial UDL		7.59 13.92	240.0 240.0	plf
DLp	Dead	Point		7.59	800	lbs
LLp	Live	Point		7.59	500	lbs
SLp	Snow	Point		7.59	500	lbs

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

Unfactored:			
Dead	4469		2559
Live	5223		3171
Snow	2038		944
Factored:			
Total	9915		5731
Bearing:			
Capacity			
Beam	11898		6877
Support	9915		5731
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#2
Length	2.27		1.31
Min req'd	2.27**		1.31**
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"x11-7/8"

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

Total length: 14.0'; Clear span: 13.688'; Volume = 8.1 cu.ft.

Lateral support: top = continuous, 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	$f_v = 145$	$F_v' = 290$	psi	$f_v/F_v' = 0.50$
Bending(+)	$f_b = 2190$	$F_b' = 2903$	psi	$f_b/F_b' = 0.75$
Live Defl'n	$0.26 = L/650$	$0.46 = L/360$	in	0.55
Total Defl'n	$0.57 = L/288$	$0.69 = L/240$	in	0.83

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	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 #2 = D + L

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 = 9692, V design = 8048 (NDS 3.4.3.1(a)) lbs; M(+) = 30030 lbs-ft
EI = 2149.02e06 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"

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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PROJECT

July 21, 2023 11:27

BM11

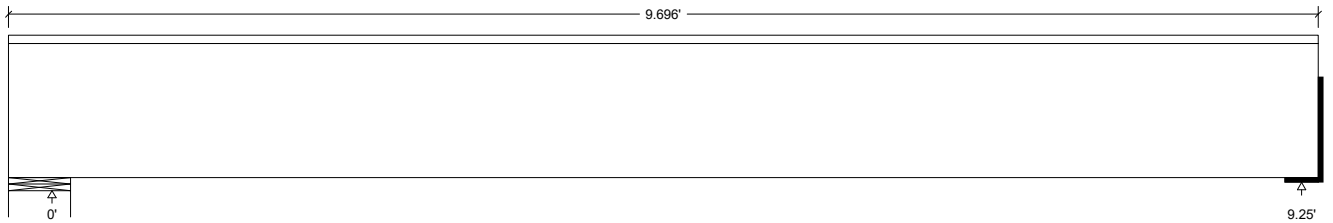
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft]	Magnitude	Unit
				Start End	Start End	
DLp1	Dead	Point		2.57	1100	lbs
LLp1	Dead	Point		2.57	1100	lbs
SLp1	Snow	Point		2.57	662	lbs
DLw1	Dead	Partial UDL		0.32 2.57	714.0 714.0	plf
LLw1	Live	Partial UDL		0.32 2.57	850.0 850.0	plf
SLw1	Snow	Partial UDL		0.32 2.57	331.0 331.0	plf
DLw2	Dead	Partial UDL		2.57 6.32	181.0 181.0	plf
LLw2	Live	Partial UDL		2.57 6.32	315.0 315.0	plf
DLp2	Dead	Point		6.32	1100	lbs
LLp2	Dead	Point		6.32	1100	lbs
SLp2	Snow	Point		6.32	662	lbs
DLw3	Dead	Partial UDL		6.32 9.57	714.0 714.0	plf
LLw3	Live	Partial UDL		6.32 9.57	850.0 850.0	plf
SLw3	Snow	Partial UDL		6.32 9.57	331.0 331.0	plf

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



Unfactored:			
Dead	4633		4373
Live	2820		3037
Snow	1577		1568
Factored:			
Total	7930		7826
Bearing:			
Capacity			
Beam	14438		7826
Support	13320		-
Des ratio			
Beam	0.55		1.00
Support	0.60		-
Load comb	#3		#3
Length	5.50		2.98
Min req'd	3.27**		2.98
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fcp_sup	625		-

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

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

Supports: 1 - Lumber Stud Wall, D Fir-L Stud; 2 - Hanger;

Total length: 9.69'; Clear span: 9'; Volume = 2.8 cu.ft.

Lateral support: top = continuous, 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	$f_v = 205$	$F_v' = 290$	psi	$f_v/F_v' = 0.71$
Bending(+)	$f_b = 2310$	$F_b' = 2903$	psi	$f_b/F_b' = 0.80$
Live Defl'n	$0.10 = < L/999$	$0.31 = L/360$	in	0.32
Total Defl'n	$0.32 = L/341$	$0.46 = L/240$	in	0.70

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'	2900	1.00	-	1.00	1.000	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 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 = 7452, V design = 5694 (NDS 3.4.3.1(a)) lbs; M(+) = 15838 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"

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.



COMPANY

July 21, 2023 11:33

PROJECT

BM12

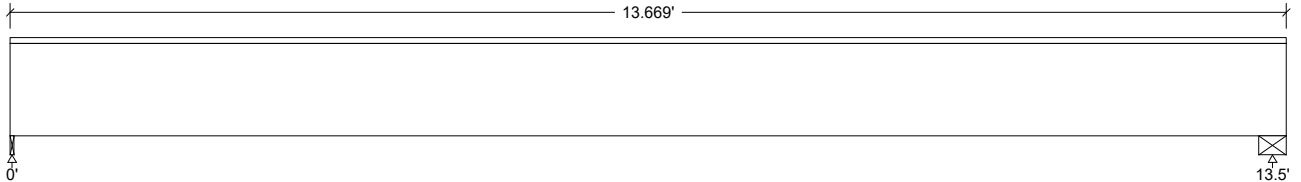
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Point		11.81	8900	lbs
LL	Live	Point		11.81	8200	lbs
SL	Snow	Point		11.77	3600	lbs

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



Unfactored:		
Dead	1127	7773
Live	1039	7161
Snow	467	3133
Factored:		
Total	2256	15494
Bearing:		
Capacity		
Beam	2708	18593
Support	2256	15494
Des ratio		
Beam	0.83	0.83
Support	1.00	1.00
Load comb	#3	#3
Length	0.52	3.54
Min req'd	0.52**	3.54**
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"x11-7/8"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 13.69'; Clear span: 13.313'; Volume = 7.9 cu.ft.
Lateral support: top = continuous, 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 = 269	Fv' = 290	psi	fv/Fv' = 0.93
Bending(+)	fb = 1863	Fb' = 2903	psi	fb/Fb' = 0.64
Live Defl'n	0.14 = < L/999	0.45 = L/360	in	0.31
Total Defl'n	0.35 = L/463	0.68 = L/240	in	0.52

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	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 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

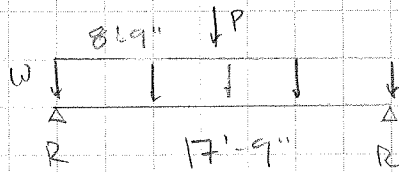
CALCULATIONS:

V max = 14934, V design = 14934 (NDS 3.4.3.1(a)) lbs; M(+) = 25537 lbs-ft
EI = 2149.02e06 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 = 1.50 permanent + "live"

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.

BM13:



$$P = 2.4 \text{ k DL} + 3.0 \text{ k LL} + 1.0 \text{ k SL}$$

$$W = (170 \text{ DL} + 170 \text{ LL} + 106 \text{ SL}) \text{ plf}$$

$$R = 7.8 \text{ k DL} + 3.1 \text{ k LL} + 1.5 \text{ k SL}$$

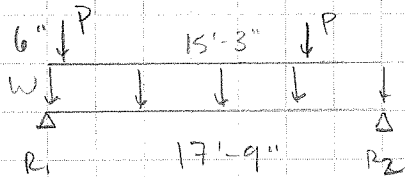
$$M = 73\% \quad 36.9 \text{ k-ft}$$

$$V = 29\% \quad 5.4 \text{ kips}$$

$$\Delta_T = L/242 \quad 0.87"$$

Use 5 1/4" x 14" PSL

BM13B:



$$P = 2.4 \text{ k DL} + 2.9 \text{ k LL} + 1.0 \text{ k SL}$$

$$W = (170 \text{ DL} + 170 \text{ LL} + 106 \text{ SL}) \text{ plf}$$

$$R_1 = 4.2 \text{ k DL} + 4.8 \text{ k LL} + 2.0 \text{ k SL}$$

$$R_2 = 3.7 \text{ k DL} + 4.1 \text{ k LL} + 1.8 \text{ k SL}$$

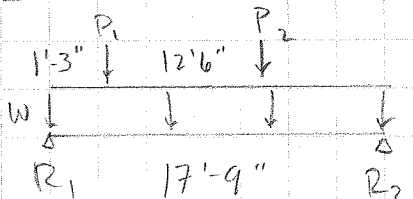
$$M = 55\% \quad 21.9 \text{ k-ft}$$

$$V = 51\% \quad 7.2 \text{ kips}$$

$$\Delta_T = L/334 \quad 0.64"$$

Use 5 1/4" x 14" PSL

BM13C:



$$P_1 = 5.7 \text{ k DL} + 6.0 \text{ k LL} + 2.6 \text{ k SL}$$

$$P_2 = 5.8 \text{ k DL} + 6.9 \text{ k LL} + 2.6 \text{ k SL}$$

$$W = (170 \text{ DL} + 170 \text{ LL} + 106 \text{ SL}) \text{ plf}$$

$$R_1 = 8.6 \text{ k DL} + 7.9 \text{ k LL} + 4.1 \text{ k SL}$$

$$R_2 = 6.0 \text{ k DL} + 6.9 \text{ k LL} + 3.0 \text{ k SL}$$

$$M = 96\% \quad 62.7 \text{ k-ft}$$

$$V = 78\% \quad 14.3 \text{ kips}$$

$$\Delta_T = L/235 \quad 0.75"$$

Use 5 1/4" x 18" PSL



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Description

Project The Talmon

By CCC

Checked

Scale

Job No.

23154.10

Date 7/10/23

Date

Sheet No.



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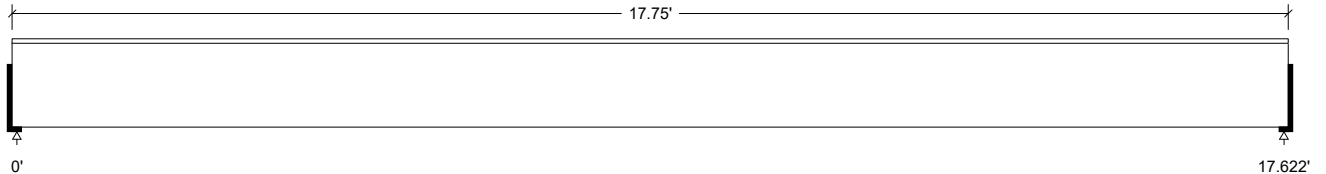
PROJECT

July 21, 2023 13:15

BM13

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			170.0	plf
LL	Live	Full UDL			170.0	plf
SL	Snow	Full UDL			106.0	plf
LLp	Live	Point		8.50	3000	lbs
SLp	Snow	Point		8.50	1000	lbs
DLP	Dead	Point		8.50	2400	lbs

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

Unfactored:			
Dead	2760		2657
Live	3073		2944
Snow	1462		1419
Factored:			
Total	6162		5930
Bearing:			
Capacity			
Beam	6162		5930
Des ratio			
Beam	1.00		1.00
Load comb	#3		#3
Length	1.56		1.51
Min req'd	1.56		1.51
Cb	1.00		1.00
Cb min	1.00		1.00

PSL, PSL, 2.2E, 5-1/4"x14"

Supports: All - Hanger

Total length: 17.75'; Clear span: 17.5'; Volume = 9.1 cu.ft.

Lateral support: top = continuous, 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	$f_v = 110$	$F_v' = 290$	psi	$f_v/F_v' = 0.38$
Bending(+)	$f_b = 2583$	$F_b' = 2851$	psi	$f_b/F_b' = 0.91$
Live Defl'n	$0.39 = L/537$	$0.59 = L/360$	in	0.67
Total Defl'n	$0.87 = L/242$	$0.88 = L/240$	in	0.99

Additional Data:

FACTORS:	F/E(ksi)	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	1.000	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 = 5811, V design = 5392 (NDS 3.4.3.1(a)) lbs; M(+) = 36920 lbs-ft
EI = 2641.10e06 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"

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.



COMPANY

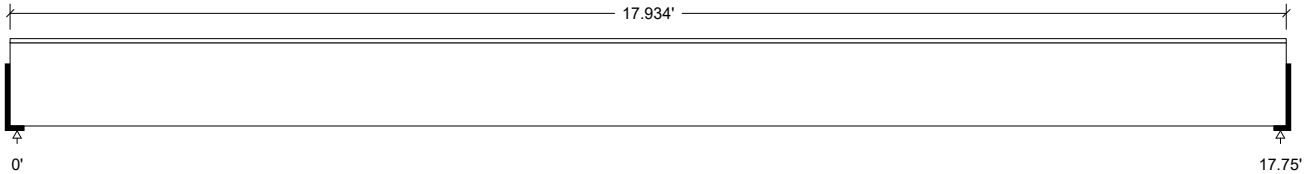
PROJECT

July 21, 2023 13:41

BM13B

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			170.0	plf
LL	Live	Full UDL			170.0	plf
SL	Snow	Full UDL			106.0	plf
DLp1	Dead	Point		0.60	2400	lbs
LLp1	Live	Point		0.60	2900	lbs
DLp2	Dead	Point		15.35	2400	lbs
LLp2	Live	Point		15.35	2900	lbs
SLp1	Snow	Point		0.60	1000	lbs
SLp2	Snow	Point		15.35	1000	lbs

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

Unfactored:			
Dead	4196		3653
Live	4752		4097
Snow	2064		1837
Factored:			
Total	9308		8103
Bearing:			
Capacity			
Beam	9308		8103
Des ratio			
Beam	1.00		1.00
Load comb	#3		#3
Length	2.36		2.06
Min req'd	2.36		2.06
Cb	1.00		1.00
Cb min	1.00		1.00

PSL, PSL, 2.2E, 5-1/4"x14"

Supports: All - Hanger

Total length: 17.94'; Clear span: 17.563'; Volume = 9.2 cu.ft.

Lateral support: top = continuous, 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	$f_v = 147$	$F_v' = 290$	psi	$f_v/F_v' = 0.51$
Bending(+)	$f_b = 1530$	$F_b' = 2851$	psi	$f_b/F_b' = 0.54$
Live Defl'n	0.29 = L/745	0.59 = L/360	in	0.48
Total Defl'n	0.64 = L/334	0.89 = L/240	in	0.72

Additional Data:

FACTORS:	F/E(ksi)	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	1.000	0.983	-	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 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 = 7217, V design = 7217 (NDS 3.4.3.1(a)) lbs; M(+) = 21865 lbs-ft
EI = 2641.10e06 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"

Design Notes:

1. 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.
2. Please verify that the default deflection limits are appropriate for your application.
3. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.
4. SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches
5. SCL: Deflection is calculated using an apparent modulus of elasticity E that incorporates the effect of shear deflection.



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PROJECT

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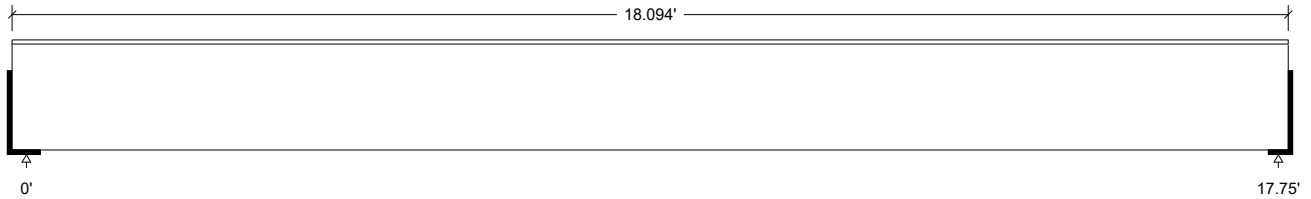
BM13C

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			170.0	plf
LL	Live	Full UDL			170.0	plf
SL	Snow	Full UDL			106.0	plf
DLp1	Dead	Point		1.45	5700	lbs
LLp1	Live	Point		1.45	6800	lbs
DLp2	Dead	Point		12.70	5800	lbs
LLp2	Live	Point		12.70	6900	lbs
SLp1	Snow	Point		1.45	2600	lbs
SLp2	Snow	Point		12.70	2600	lbs

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

Unfactored:		
Dead	8557	6019
Live	9905	6871
Snow	4148	2970
Factored:		
Total	19097	13399
Bearing:		
Capacity		
Beam	19097	13399
Des ratio		
Beam	1.00	1.00
Load comb	#3	#3
Length	4.85	3.40
Min req'd	4.85	3.40
Cb	1.00	1.00
Cb min	1.00	1.00

PSL, PSL, 2.2E, 5-1/4"x18"

Supports: All - Hanger

Total length: 18.13'; Clear span: 17.375'; Volume = 11.9 cu.ft.

Lateral support: top = continuous, 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	$f_v^* = 227$	$F_v' = 290$	psi	$f_v^*/F_v' = 0.78$
Bending(+)	$f_b = 2655$	$F_b' = 2772$	psi	$f_b/F_b' = 0.96$
Live Defl'n	$0.34 = L/634$	$0.59 = L/360$	in	0.57
Total Defl'n	$0.75 = L/285$	$0.89 = L/240$	in	0.84

*The effect of point loads within a distance d of the support has been included as per NDS 3.4.3.1

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.956	-	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 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

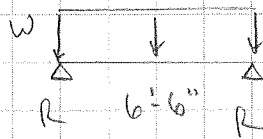
CALCULATIONS:

V max = 18394, V design* = 14315 (NDS 3.4.3.1(a)) lbs; M(+) = 62732 lbs-ft
EI = 5613.30e06 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"

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.

Transfer BM:



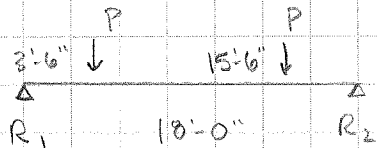
$$W = (738 \text{ DL} + 923 \text{ LL} + 304 \text{ SL}) \text{ p/f}$$

$$\text{ASD Total} = 1661 \text{ p/f}$$

$$R = 2.4 \text{ k DL} + 3.0 \text{ k LL} + 1.0 \text{ k SL}$$

Use $3\frac{1}{2}" \times 11\frac{7}{8}"$ PSL * Max Span = 9'-0"

BM15:



$$P = 3.0 \text{ k DL} + 3.7 \text{ k LL} + 1.3 \text{ k SL}$$

$$R_1 = 2.8 \text{ k DL} + 3.5 \text{ k LL} + 1.2 \text{ k SL}$$

$$R_2 = 3.2 \text{ k DL} + 3.9 \text{ k LL} + 1.4 \text{ k SL}$$

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

$$M = 56\%$$

$$22.1 \text{ k-ft}$$

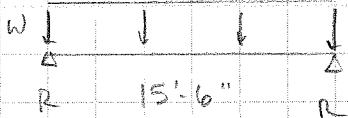
$$V = 44\%$$

$$7.1 \text{ kips}$$

$$\Delta_T = L/279$$

$$0.77"$$

BM16:



$$W = (658 \text{ DL} + 790 \text{ LL} + 300 \text{ SL}) \text{ p/f}$$

$$\text{ASD} = 1476 \text{ p/f}$$

$$R = 5.1 \text{ k DL} + 6.2 \text{ k LL} + 2.4 \text{ k SL}$$

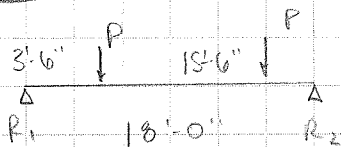
$$M = 66\%$$

$$V = 48\%$$

$$\Delta_T = L/445$$

Use $5\frac{1}{4}" \times 18"$ PSL

BM15B:



$$P = 5.1 \text{ k DL} + 6.1 \text{ k LL} + 2.3 \text{ k SL}$$

$$R_1 = 4.8 \text{ k DL} + 5.8 \text{ k LL} + 2.2 \text{ k SL}$$

$$R_2 = 5.4 \text{ k DL} + 6.4 \text{ k LL} + 2.4 \text{ k SL}$$

$$M = 58\%$$

$$V = 65\%$$

$$\Delta_T = L/431$$

Use $5\frac{1}{4}" \times 18"$ PSL



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Description Beam Design

By CCC

Date 7/11/23

Checked

Date

Scale

Sheet No.

Project The Talmon

Job No.

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PROJECT

July 21, 2023 14:17

BM15

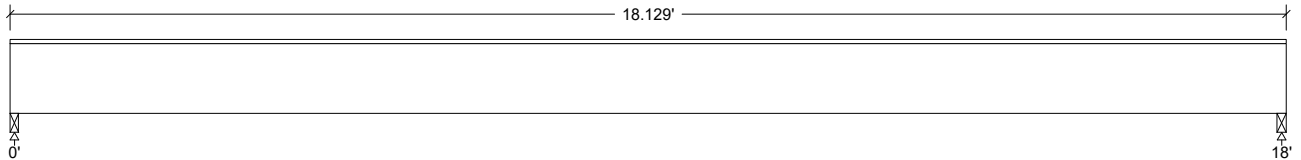
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL1	Dead	Point		3.56	3000	lbs
DL2	Dead	Point		15.56	3000	lbs
LL1	Live	Point		3.56	3700	lbs
LL2	Live	Point		15.56	3700	lbs
SL1	Snow	Point		3.56	1300	lbs
SL2	Snow	Point		15.56	1300	lbs

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



Unfactored:			
Dead	2833		3167
Live	3494		3906
Snow	1228		1372
Factored:			
Total	6375		7125
Bearing:			
Capacity			
Beam	7650		8550
Support	6375		7125
Des ratio			
Beam	0.83		0.83
Support	1.00		1.00
Load comb	#3		#3
Length	1.46		1.63
Min req'd	1.46**		1.63**
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"x11-7/8"

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

Total length: 18.13'; Clear span: 17.875'; Volume = 10.5 cu.ft.

Lateral support: top = continuous, 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 = 128	Fv' = 290	psi	fv/Fv' = 0.44
Bending(+)	fb = 1615	Fb' = 2903	psi	fb/Fb' = 0.56
Live Defl'n	0.35 = L/614	0.60 = L/360	in	0.59
Total Defl'n	0.77 = L/279	0.90 = L/240	in	0.86

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	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 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 = 7072, V design = 7072 (NDS 3.4.3.1(a)) lbs; M(+) = 22147 lbs-ft
 EI = 2149.02e06 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"

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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July 21, 2023 14:18

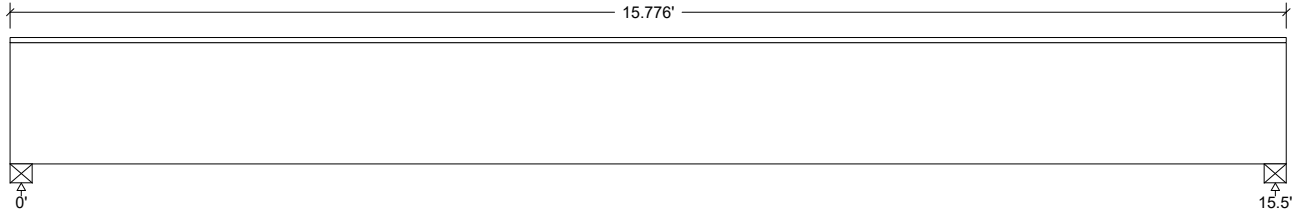
BM16

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			658.0	plf
LL	Live	Full UDL			790.0	plf
SL	Snow	Full UDL			300.0	plf

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

Unfactored:			
Dead	5190		5190
Live	6231		6231
Snow	2366		2366
Factored:			
Total	11639		11639
Bearing:			
Capacity			
Beam	13036		13036
Support	11639		11639
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	3.31		3.31
Min req'd	3.31**		3.31**
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"x18"

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

Total length: 15.75'; Clear span: 15.25'; Volume = 10.4 cu.ft.

Lateral support: top = continuous, 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	$f_v = 140$	$F_v' = 290$	psi	$f_v/F_v' = 0.48$
Bending(+)	$f_b = 1841$	$F_b' = 2772$	psi	$f_b/F_b' = 0.66$
Live Defl'n	$0.19 = L/983$	$0.52 = L/360$	in	0.37
Total Defl'n	$0.42 = L/445$	$0.77 = L/240$	in	0.54

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.956	-	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 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: $V_{max} = 11222$, $V_{design} = 8850$ (NDS 3.4.3.1(a)) lbs; $M(+) = 43485$ lbs-ft $EI = 5613.30e06$ 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"

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.



COMPANY

PROJECT

July 21, 2023 14:20

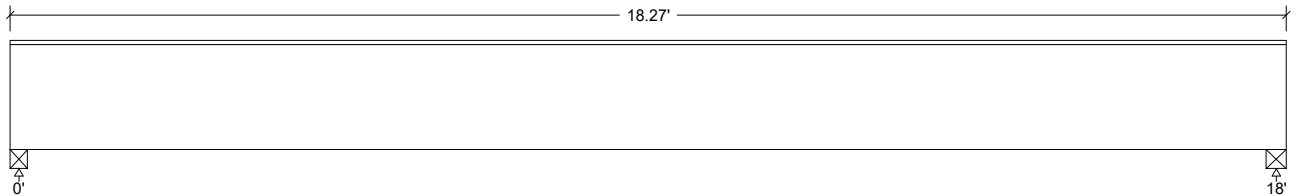
BM15B

Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DL1	Dead	Point		3.74	5100	lbs
DL2	Dead	Point		15.74	5100	lbs
LL1	Live	Point		3.74	6100	lbs
LL2	Live	Point		15.74	6100	lbs
SL1	Snow	Point		3.74	2300	lbs
SL2	Snow	Point		15.74	2300	lbs

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

Unfactored:			
Dead	4754		5446
Live	5687		6513
Snow	2144		2456
Factored:			
Total	10627		12173
Bearing:			
Capacity			
Beam	11903		13634
Support	10627		12173
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	3.02		3.46
Min req'd	3.02**		3.46**
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"x18"

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

Total length: 18.25'; Clear span: 17.75'; Volume = 12.0 cu.ft.

Lateral support: top = continuous, 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 = 190	Fv' = 290	psi	fv/Fv' = 0.65
Bending(+)	fb = 1595	Fb' = 2772	psi	fb/Fb' = 0.58
Live Defl'n	0.23 = L/956	0.60 = L/360	in	0.38
Total Defl'n	0.50 = L/431	0.90 = L/240	in	0.56

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cftr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.956	-	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 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 = 11959, V design = 11959 (NDS 3.4.3.1(a)) lbs; M(+) = 37692 lbs-ft

EI = 5613.30e06 lb-in^2 Apparent E approximates the effect of shear deflection.

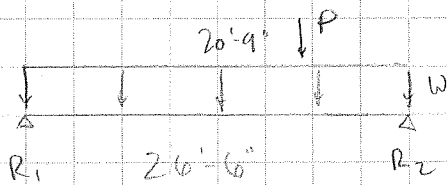
"Live" deflection is due to all non-dead loads (live, wind, snow...)

Total deflection = 1.50 permanent + "live"

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.

BM17:



$$P = 10.2 \text{ k DL} + 12.2 \text{ k LL} + 4.6 \text{ k SL}$$

$$W = (138 \text{ DL} + 240 \text{ LL}) \text{ plf}$$

$$R_1 = 4.0 \text{ k DL} + 5.6 \text{ k LL} + 1.0 \text{ k SL}$$

$$R_2 = 9.8 \text{ k DL} + 12.7 \text{ k LL} + 3.6 \text{ k SL}$$

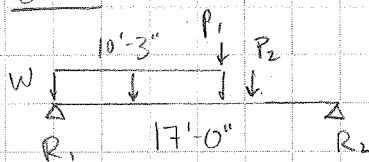
$$M = 68\%$$

$$V = 23\%$$

$$\Delta_T = L/353$$

Use W16 x 40

BM18:



$$P_1 = 1.9 \text{ k DL} + 1.4 \text{ k LL} + 1.0 \text{ k SL}$$

$$P_2 = 2.5 \text{ k DL} + 3.0 \text{ k LL} + 1.1 \text{ k SL}$$

$$W = (75 \text{ DL} + 195 \text{ LL}) \text{ plf}$$

$$R_1 = 2.1 \text{ k DL} + 2.9 \text{ k LL} + 0.7 \text{ k SL}$$

$$R_2 = 3.1 \text{ k DL} + 3.5 \text{ k LL} + 1.4 \text{ k SL}$$

$$M = 92\%$$

$$V = 46\%$$

$$\Delta_T = L/257$$

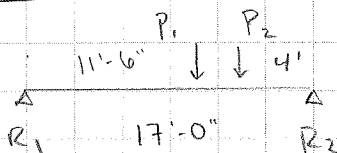
$$36.8 \text{ k-ft}$$

$$6.6 \text{ kips}$$

$$0.79"$$

Use 5'4" x 14" PSL

BM18B:



$$P_1 = 2.5 \text{ k DL} + 3.0 \text{ k LL} + 1.1 \text{ k SL}$$

$$P_2 = 4.6 \text{ k DL} + 5.6 \text{ k LL} + 2.1 \text{ k SL}$$

$$R_1 = 1.9 \text{ k DL} + 2.3 \text{ k LL} + 0.8 \text{ k SL}$$

$$R_2 = 5.2 \text{ k DL} + 6.3 \text{ k LL} + 2.4 \text{ k SL}$$

$$M = 74\%$$

$$V = 63\%$$

$$\Delta_T = L/460$$

$$48.1 \text{ k-ft}$$

$$11.6 \text{ kips}$$

$$0.44"$$

Use 5'4" x 18" PSL



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www.cgeengineering.com

Description Beam Design

Project The Talmon

By CCC

Checked

Scale

Job No.

23154.10

Date 7/12/23

Date

Sheet No.

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

CG ENGINEERING

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DESCRIPTION: **BM17**

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

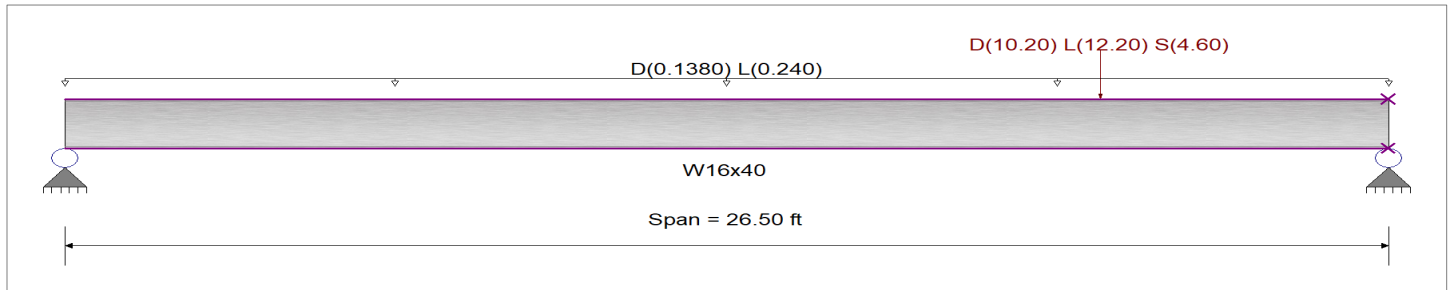
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.1380, L = 0.240 k/ft, Tributary Width = 1.0 ft

Point Load : D = 10.20, L = 12.20, S = 4.60 k @ 20.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.677 : 1	Maximum Shear Stress Ratio =	0.231 : 1
Section used for this span	W16x40	Section used for this span	W16x40
Ma : Applied	123.394 k-ft	Va : Applied	22.548 k
Mn / Omega : Allowable	182.136 k-ft	Vn/Omega : Allowable	97.60 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Location of maximum on span	26.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.515 in Ratio = 617	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in Ratio = 0	<360	n/a
Max Downward Total Deflection	0.900 in Ratio = 353	>=240	Span: 1 : +D+L
Max Upward Total Deflection	0 in Ratio = 0	<240.0	n/a

Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	9.869	22.548
Max Upward from Load Combinations	9.869	22.548
Max Upward from Load Cases	5.827	12.733
D Only	4.042	9.815
+D+L	9.869	22.548
+D+S	5.040	13.417
+D+0.750L	8.412	19.365
+D+0.750L+0.750S	9.161	22.066
+0.60D	2.425	5.889
L Only	5.827	12.733
S Only	0.998	3.602



COMPANY

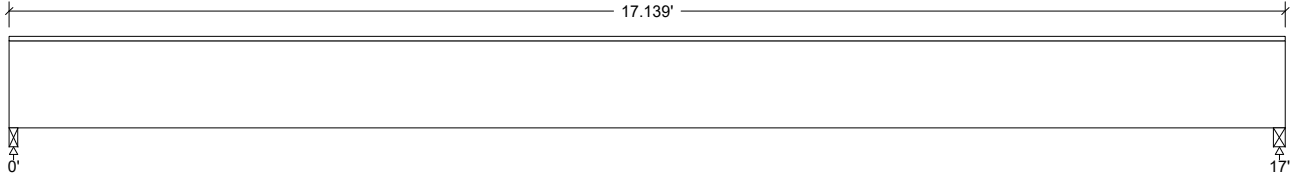
PROJECT

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BM18

Design Check Calculation Sheet
WoodWorks Sizer 2023**Loads:**

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLw	Dead	Partial UDL		0.06 10.31	75.0 75.0	plf
LLw	Live	Partial UDL		0.06 10.31	195.0 195.0	plf
DLp1	Dead	Point		10.31	1900	lbs
LLp1	Live	Point		10.31	1400	lbs
SLp1	Snow	Point		10.31	1000	lbs
DLp2	Dead	Point		11.72	2500	lbs
LLp2	Live	Point		11.72	3000	lbs
SLp2	Snow	Point		11.72	1100	lbs

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

Unfactored:			
Dead	2077		3092
Live	2894		3504
Snow	743		1357
Factored:			
Total	4971		6738
Bearing:			
Capacity			
Beam	5568		7547
Support	4971		6738
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#2		#3
Length	1.41		1.92
Min req'd	1.41**		1.92**
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"x14"

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

Total length: 17.13'; Clear span: 16.875'; Volume = 8.7 cu.ft.

Lateral support: top = continuous, 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	$f_v = 135$	$F_v' = 290$	psi	$f_v/F_v' = 0.46$
Bending(+)	$f_b = 2573$	$F_b' = 2851$	psi	$f_b/F_b' = 0.90$
Live Defl'n	$0.35 = L/579$	$0.57 = L/360$	in	0.62
Total Defl'n	$0.79 = L/257$	$0.85 = L/240$	in	0.93

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.983	-	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 #2 = D + L
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 = 6596, V design = 6596 (NDS 3.4.3.1(a)) lbs; M(+) = 36771 lbs-ft
EI = 2641.10e06 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"

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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PROJECT

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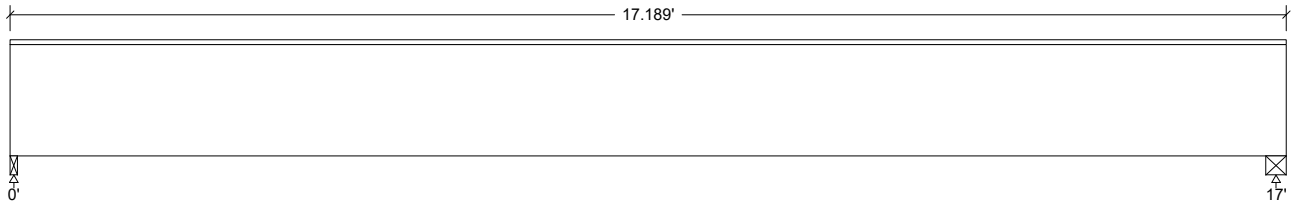
BM18B

Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLp1	Dead	Point		11.71	2500	lbs
LLp1	Live	Point		11.71	3000	lbs
SLp1	Snow	Point		11.71	1100	lbs
DLp2	Dead	Point		13.05	4600	lbs
LLp2	Live	Point		13.05	5600	lbs
SLp2	Snow	Point		13.05	2100	lbs

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

Unfactored:			
Dead	1868		5232
Live	2260		6340
Snow	840		2360
Factored:			
Total	4192		11758
Bearing:			
Capacity			
Beam	4696		13169
Support	4192		11758
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	1.19		3.34
Min req'd	1.19**		3.34**
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"x18"

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

Total length: 17.19'; Clear span: 16.813'; Volume = 11.3 cu.ft.

Lateral support: top = continuous, 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	$f_v = 184$	$F_v' = 290$	psi	$f_v/F_v' = 0.63$
Bending(+)	$f_b = 2037$	$F_b' = 2772$	psi	$f_b/F_b' = 0.73$
Live Defl'n	$0.20 < L/999$	$0.57 = L/360$	in	0.35
Total Defl'n	$0.44 = L/460$	$0.85 = L/240$	in	0.52

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrr	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.956	-	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 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

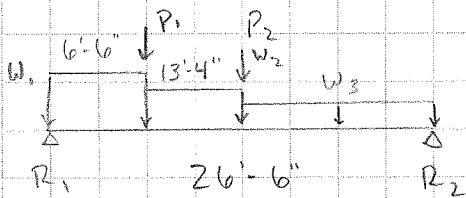
CALCULATIONS:

V max = 11572, V design = 11572 (NDS 3.4.3.1(a)) lbs; M(+) = 48128 lbs-ft
EI = 5613.30e06 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"

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.

BM19:



$$W_1 = (158 \text{ DL} + 275 \text{ LL}) \text{ plf}$$

$$P_1 = 3.1 \text{ k DL} + 3.5 \text{ k LL} + 1.4 \text{ k SL}$$

$$W_2 = (144 \text{ DL} + 250 \text{ LL}) \text{ plf}$$

$$P_2 = 5.2 \text{ k DL} + 6.3 \text{ k LL} + 2.4 \text{ k SL}$$

$$W_3 = (129 \text{ DL} + 225 \text{ LL}) \text{ plf}$$

$$R_1 = 6.9 \text{ k DL} + 9.2 \text{ k LL} + 2.3 \text{ k SL}$$

$$R_2 = 5.2 \text{ k DL} + 7.1 \text{ k LL} + 1.6 \text{ k SL}$$

Use W16 x 40

$$M = 72\%$$

$$114.2 \text{ k-ft}$$

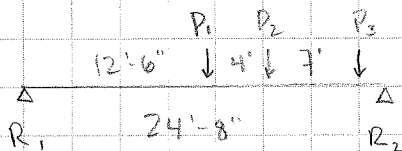
$$V = 16\%$$

$$16 \text{ kips}$$

$$\Delta_T = L/319$$

$$1.0''$$

BM20:



$$P_1 = 5.0 \text{ k DL} + 7.1 \text{ k LL} + 2.6 \text{ k SL}$$

$$P_2 = 10.4 \text{ k DL} + 15.4 \text{ k LL} + 3.8 \text{ k SL}$$

$$P_3 = 9.7 \text{ k DL} + 11.8 \text{ k LL} + 4.2 \text{ k SL}$$

$$R_1 = 6.8 \text{ k DL} + 9.2 \text{ k LL} + 2.7 \text{ k SL}$$

$$R_2 = 19.1 \text{ k DL} + 25.1 \text{ k LL} + 7.9 \text{ k SL}$$

Use W16 x 50

$$M = 92\%$$

$$220.3 \text{ k-ft}$$

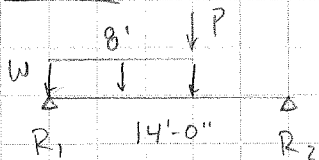
$$V = 36\%$$

$$44.2 \text{ kips}$$

$$\Delta_T = L/275$$

$$1.05''$$

BM21:



$$P = 2.8 \text{ k DL} + 2.8 \text{ k LL} + 1.7 \text{ k SL}$$

$$W = (706 \text{ DL} + 843 \text{ LL} + 333 \text{ SL}) \text{ plf}$$

$$R_1 = 5.2 \text{ k DL} + 6.0 \text{ k LL} + 2.6 \text{ k SL}$$

$$R_2 = 3.0 \text{ k DL} + 3.3 \text{ k LL} + 1.6 \text{ k SL}$$

Use 5 1/4" x 14" PSL

$$M = 99\%$$

$$40.2 \text{ k-ft}$$

$$V = 65\%$$

$$9.2 \text{ kips}$$

$$\Delta_T = L/260$$

$$0.65''$$



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Suite 200
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425.778.8500
www.cgengineering.com

Description Room Design

By CCC

Date 7/12/23

Checked

Date

Scale

Sheet No.

Project The Talmon

Job No.

23154.10

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: BM19

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

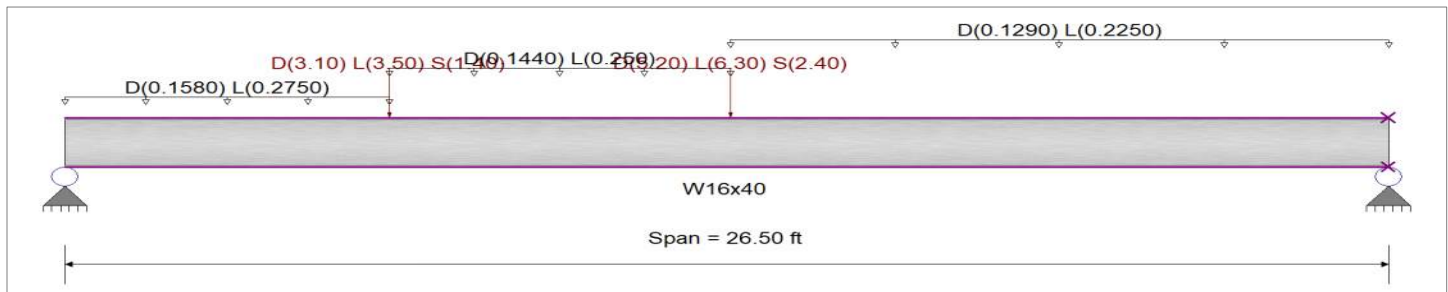
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



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 = 0.1580, L = 0.2750 k/ft, Extent = 0.0 --> 6.50 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.1440, L = 0.250 k/ft, Extent = 6.50 --> 13.330 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.1290, L = 0.2250 k/ft, Extent = 13.330 --> 26.50 ft, Tributary Width = 1.0 ft

Point Load : D = 3.10, L = 3.50, S = 1.40 k @ 6.50 ft

Point Load : D = 5.20, L = 6.30, S = 2.40 k @ 13.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.718 : 1		Maximum Shear Stress Ratio =		0.164 : 1	
Section used for this span		W16x40		Section used for this span		W16x40	
Ma : Applied		130.738 k-ft		Va : Applied		16.008 k	
Mn / Omega : Allowable		182.136 k-ft		Vn/Omega : Allowable		97.60 k	
Load Combination		+D+L		Load Combination		+D+L	
Span # where maximum occurs		Span # 1		Location of maximum on span		0.000 ft	
				Span # where maximum occurs		Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.567 in Ratio =		560 >=360		Span: 1 : L Only	
Max Upward Transient Deflection		0 in Ratio =		0 <360		n/a	
Max Downward Total Deflection		0.998 in Ratio =		319 >=240.		Span: 1 : +D+L	
Max Upward Total Deflection		0 in Ratio =		0 <240.0		n/a	

Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	16.008	12.259
Max Upward from Load Combinations	16.008	12.259
Max Upward from Load Cases	9.146	7.113
D Only	6.863	5.147
+D+L	16.008	12.259
+D+S	9.112	6.697
+D+0.750L	13.722	10.481
+D+0.750L+0.750S	15.409	11.644
+0.60D	4.118	3.088
L Only	9.146	7.113
S Only	2.249	1.551

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

CG ENGINEERING

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DESCRIPTION: BM20

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

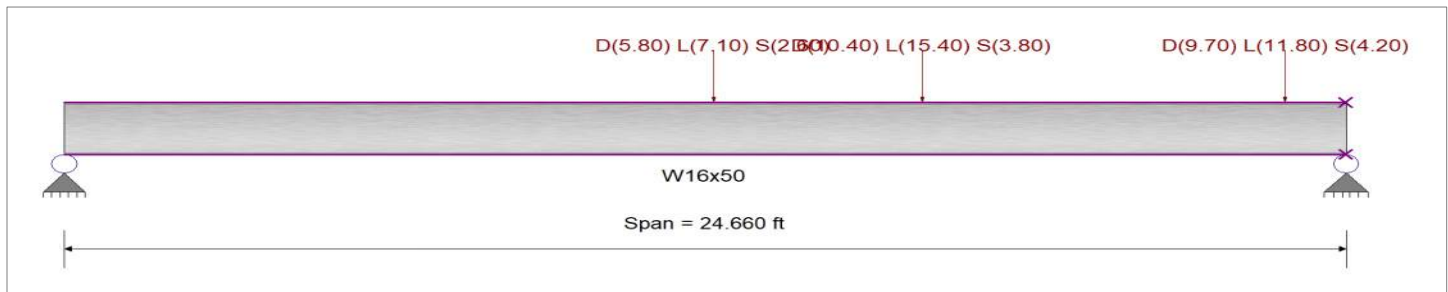
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Load(s) for Span Number 1

Point Load : D = 5.80, L = 7.10, S = 2.60 k @ 12.50 ft

Point Load : D = 10.40, L = 15.40, S = 3.80 k @ 16.50 ft

Point Load : D = 9.70, L = 11.80, S = 4.20 k @ 23.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.919 : 1	Maximum Shear Stress Ratio =	0.358 : 1
Section used for this span	W16x50	Section used for this span	W16x50
Ma : Applied	210.870 k-ft	Va : Applied	44.290 k
Mn / Omega : Allowable	229.541 k-ft	Vn/Omega : Allowable	123.880 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Location of maximum on span	23.533 ft
		Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.622 in Ratio =	475 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in Ratio =	0 <360	n/a
Max Downward Total Deflection	1.077 in Ratio =	275 >=240.	Span: 1 : +D+L
Max Upward Total Deflection	0 in Ratio =	0 <240.0	n/a

Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	15.910	44.290
Max Upward from Load Combinations	15.910	44.290
Max Upward from Load Cases	9.152	25.148
D Only	6.758	19.142
+D+L	15.910	44.290
+D+S	9.495	27.005
+D+0.750L	13.622	38.003
+D+0.750L+0.750S	15.674	43.901
+0.60D	4.055	11.485
L Only	9.152	25.148
S Only	2.737	7.863



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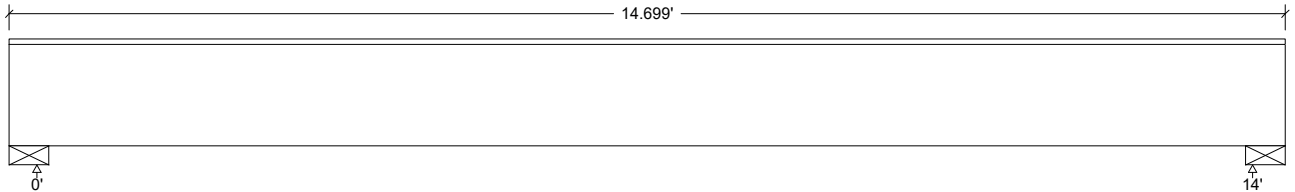
BM21

Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLw	Dead	Partial UDL		0.32 8.00	705.0 705.0	plf
LLw	Live	Partial UDL		0.32 8.00	843.0 843.0	plf
SLw	Snow	Partial UDL		0.32 8.00	333.0 333.0	plf
DLP	Dead	Point		8.00	2800	lbs
Llp	Live	Point		8.00	2800	lbs
SLp	Snow	Point		8.00	1700	lbs

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

Unfactored:			
Dead	5194		3020
Live	5964		3311
Snow	2624		1633
Factored:			
Total	11635		6728
Bearing:			
Capacity			
Beam	21656		21656
Support	19336		19336
Des ratio			
Beam	0.54		0.31
Support	0.60		0.35
Load comb	#3		#3
Length	5.50		5.50
Min req'd	3.31**		1.91**
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"x14"

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

Total length: 14.69'; Clear span: 13.813'; Volume = 7.5 cu.ft.

Lateral support: top = continuous, 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 = 188	Fv' = 290	psi	fv/Fv' = 0.65
Bending(+)	fb = 2812	Fb' = 2851	psi	fb/Fb' = 0.99
Live Defl'n	0.29 = L/577	0.47 = L/360	in	0.62
Total Defl'n	0.65 = L/260	0.70 = L/240	in	0.92

Additional Data:

FACTORS:	F/E (psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.983	-	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 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

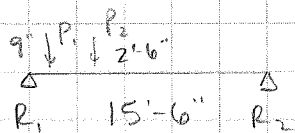
CALCULATIONS:

V max = 11155, V design = 9230 (NDS 3.4.3.1(a)) lbs; M(+) = 40193 lbs-ft
EI = 2641.10e06 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 = 1.50 permanent + "live"

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.

BM 22:



$$P_1 = 2.6 \text{ k DL} + 3.0 \text{ k LL} + 1.2 \text{ k SL}$$

$$P_2 = 5.2 \text{ k DL} + 6.0 \text{ k LL} + 2.6 \text{ k SL}$$

$$R_1 = 6.8 \text{ k DL} + 7.9 \text{ k LL} + 3.3 \text{ k SL}$$

$$R_2 = 1.0 \text{ k DL} + 1.1 \text{ k LL} + 0.5 \text{ k SL}$$

Use 5'1/4" x 14" PSL

$$V = 84\%$$

$$M = 67\%$$

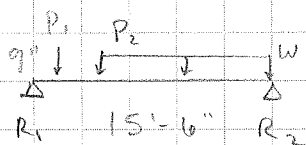
$$\Delta_T = L/466$$

$$12 \text{ kips}$$

$$27 \text{ k-ft}$$

$$0.40"$$

BM 22B:



$$P_1 = 2.6 \text{ k DL} + 3.0 \text{ k LL} + 1.2 \text{ k SL}$$

$$P_2 = 879 \text{ DL} + 879 \text{ LL} + 550 \text{ SL}$$

$$W = (196 \text{ DL} + 375 \text{ LL} + 78 \text{ SL}) \text{ p/f}$$

$$R_1 = 4.3 \text{ k DL} + 5.6 \text{ k LL} + 2.0 \text{ k SL}$$

$$R_2 = 1.7 \text{ k DL} + 3.1 \text{ k LL} + 0.7 \text{ k SL}$$

Use 5'1/4" x 11'3/8" PSL

$$M = 71\%$$

$$V = 66\%$$

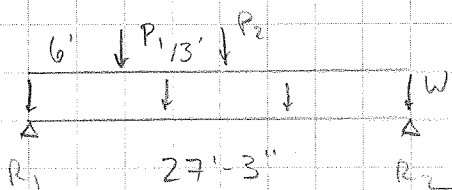
$$\Delta_T = L/275$$

$$20.7 \text{ k-ft}$$

$$8.0 \text{ kips}$$

$$0.67"$$

BM 23:



$$W = (138 \text{ DL} + 240 \text{ LL}) \text{ p/f}$$

$$P_1 = 4.3 \text{ k DL} + 5.6 \text{ k LL} + 2.0 \text{ k SL}$$

$$P_2 = 6.8 \text{ k DL} + 7.9 \text{ k LL} + 3.3 \text{ k SL}$$

$$R_1 = 8.7 \text{ k DL} + 11.7 \text{ k LL} + 3.3 \text{ k SL}$$

$$R_2 = 6.2 \text{ k DL} + 8.4 \text{ k LL} + 2.1 \text{ k SL}$$

Use W16 x 40

$$M = 91\%$$

$$V = 21\%$$

$$\Delta_T = L/246$$



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Description Beam Design

By CCC

Date 7/12/23

Checked

Date

Scale

Sheet No.

Project The Talman

Job No.

23154.10



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BM22

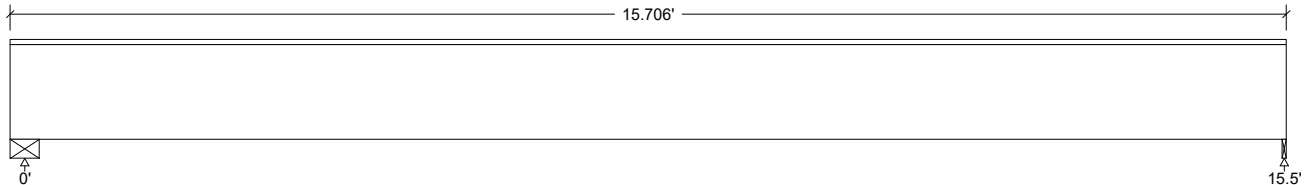
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLp1	Dead	Point		0.93	2600	lbs
LLp1	Live	Point		0.93	3000	lbs
SLp1	Snow	Point		0.93	1200	lbs
DLp2	Dead	Point		2.68	5200	lbs
LLp2	Live	Point		2.68	6000	lbs
SLp2	Snow	Point		2.68	2600	lbs

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



Unfactored:			
Dead	6835		965
Live	7887		1113
Snow	3323		477
Factored:			
Total	15243		2157
Bearing:			
Capacity			
Beam	17072		2416
Support	15243		2157
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	4.34		0.61
Min req'd	4.34**		0.61**
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"x14"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 15.69'; Clear span: 15.313'; Volume = 8.0 cu.ft.
Lateral support: top = continuous, 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	$f_v^* = 245$	$F_v' = 290$	psi	$f_v^*/F_v' = 0.84$
Bending(+)	$f_b = 1890$	$F_b' = 2851$	psi	$f_b/F_b' = 0.66$
Live Defl'n	$0.18 = < L/999$	$0.52 = L/360$	in	0.35
Total Defl'n	$0.40 = L/466$	$0.77 = L/240$	in	0.51

*The effect of point loads within a distance d of the support has been included as per NDS 3.4.3.1

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	1.000	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:

$V_{max} = 14723$, $V_{design}^* = 11994$ (NDS 3.4.3.1(a)) lbs; $M(+)$ = 27006 lbs-ft
 $EI = 2641.10e06$ 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"

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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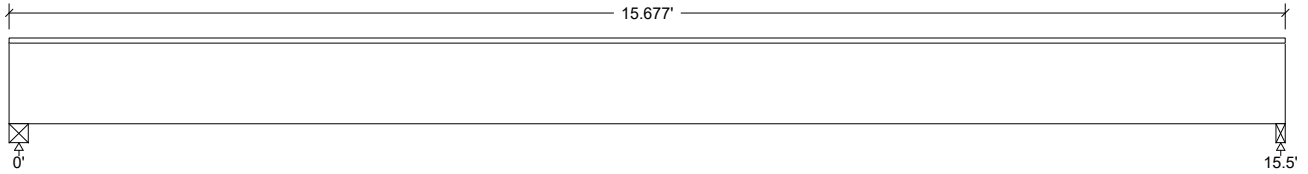
BM22B

Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
DLp1	Dead	Point		0.87	2600	lbs
LLp1	Live	Point		0.87	3000	lbs
SLp1	Snow	Point		0.87	1200	lbs
DLp2	Dead	Point		2.62	879	lbs
LLp2	Live	Point		2.62	879	lbs
SLp2	Snow	Point		2.62	550	lbs
DLw	Dead	Partial UDL		2.62 15.62	196.0 196.0	plf
LLw	Live	Partial UDL		2.62 15.62	375.0 375.0	plf
SLw	Snow	Partial UDL		2.62 15.62	78.0 78.0	plf

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

Unfactored:			
Dead	4280		1747
Live	5636		3118
Snow	2028		736
Factored:			
Total	10029		4865
Bearing:			
Capacity			
Beam	11232		5448
Support	10029		4865
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#2
Length	2.85		1.38
Min req'd	2.85**		1.38**
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: 15.69'; Clear span: 15.313'; Volume = 6.8 cu.ft.

Lateral support: top = continuous, 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	$f_v^* = 192$	$F_v' = 290$	psi	$f_v^*/F_v' = 0.66$
Bending(+)	$f_b = 2015$	$F_b' = 2903$	psi	$f_b/F_b' = 0.69$
Live Defl'n	$0.35 = L/525$	$0.52 = L/360$	in	0.68
Total Defl'n	$0.67 = L/275$	$0.77 = L/240$	in	0.87

*The effect of point loads within a distance d of the support has been included as per NDS 3.4.3.1

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	1.000	1.001	-	1.00	1.00	-	2
F_{cp}'	750	-	-	1.00	-	-	-	-	1.00	-	-
E'	2.2 million	-	-	1.00	-	-	-	-	1.00	-	2
E_{miny}'	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 #3 = D + 0.75(L + S)

Support 2 - LC #2 = D + L

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: $V_{max} = 9916$, $V_{design}^* = 7966$ (NDS 3.4.3.1(a)) lbs; $M(+)$ = 20722 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"

Design Notes:

1. 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.

2. Please verify that the default deflection limits are appropriate for your application.

3. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

4. SCL: Structural composite lumber design has assumed: - dry service conditions - no preservative or fire-retardant treatment - no notches

5. 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:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

CG ENGINEERING

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DESCRIPTION: BM23

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

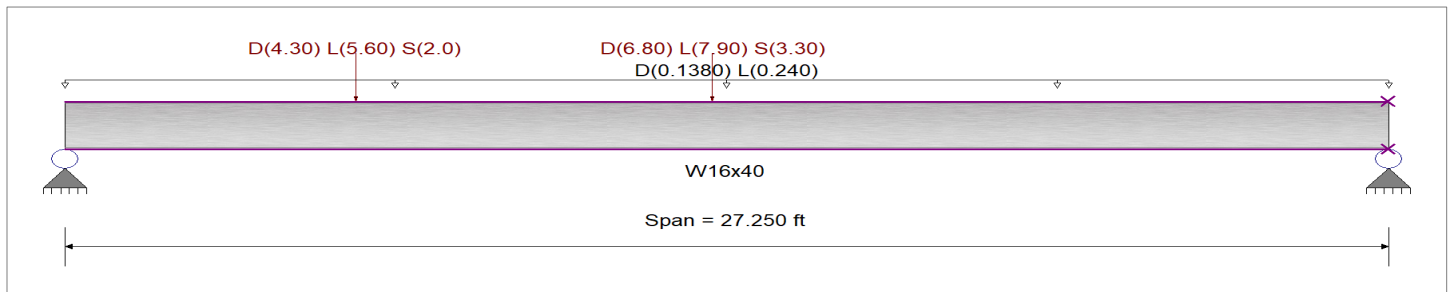
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.1380, L = 0.240 k/ft, Tributary Width = 1.0 ft

Point Load : D = 4.30, L = 5.60, S = 2.0 k @ 6.0 ft

Point Load : D = 6.80, L = 7.90, S = 3.30 k @ 13.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.908 : 1	Maximum Shear Stress Ratio =		0.209 : 1
Section used for this span		W16x40	Section used for this span		W16x40
Ma : Applied		165.420 k-ft	Va : Applied		20.380 k
Mn / Omega : Allowable		182.136 k-ft	Vn/Omega : Allowable		97.60 k
Load Combination		+D+L	Load Combination		+D+L
Span # where maximum occurs		Span # 1	Location of maximum on span		0.000 ft
			Span # where maximum occurs		Span # 1
Maximum Deflection					
Max Downward Transient Deflection		0.752 in Ratio =	434 >=360	Span: 1 : L Only	
Max Upward Transient Deflection		0 in Ratio =	0 <360	n/a	
Max Downward Total Deflection		1.329 in Ratio =	246 >=240.	Span: 1 : +D+L	
Max Upward Total Deflection		0 in Ratio =	0 <240.0	n/a	

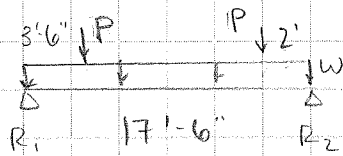
Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	20.380	14.521
Max Upward from Load Combinations	20.380	14.521
Max Upward from Load Cases	11.672	8.368
D Only	8.707	6.153
+D+L	20.380	14.521
+D+S	11.952	8.208
+D+0.750L	17.461	12.429
+D+0.750L+0.750S	19.895	13.970
+0.60D	5.224	3.692
L Only	11.672	8.368
S Only	3.245	2.055

BM24:



$$P = 5.1 \text{ k DL} + 6.1 \text{ k LL} + 2.3 \text{ k SL}$$

$$W = 200 \text{ DL plf}$$

$$R_1 = 6.4 \text{ k DL} + 5.6 \text{ k LL} + 2.1 \text{ k SL}$$

$$R_2 = 7.3 \text{ k DL} + 6.6 \text{ k LL} + 2.5 \text{ k SL}$$

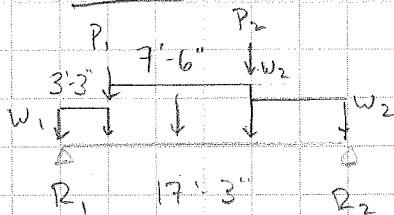
Use 5 1/4" x 18" PSL

$$M = 62\%$$

$$V = 74\%$$

$$\Delta_T = L/384$$

BM14R:



$$P_1 = 8.6 \text{ k DL} + 10.4 \text{ k LL} + 3.9 \text{ k SL}$$

$$P_2 = 7 \text{ k DL} + 7.7 \text{ k LL} + 3.5 \text{ k SL}$$

$$W_1 = 173 \text{ DL} + 300 \text{ LL}$$

$$W_2 = 414 \text{ DL} + 720 \text{ LL}$$

$$W_3 = 135 \text{ DL} + 235 \text{ LL}$$

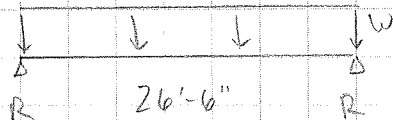
Use W12 x 35

$$R_1 = 12.1 \text{ k DL} + 15.8 \text{ k LL} + 4.5 \text{ k SL}$$

$$R_2 = 8 \text{ k DL} + 10.4 \text{ k LL} + 2.9 \text{ k SL}$$

$$M = 88\% \quad \Delta_T = L/277$$

BM19B:



$$W = (729 \text{ DL} + 890 \text{ LL} + 319 \text{ SL}) \text{ plf}$$

$$R = 9.7 \text{ k DL} + 11.8 \text{ k LL} + 4.2 \text{ k SL}$$

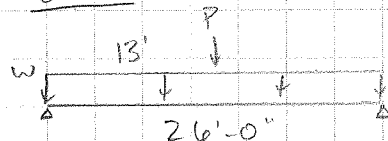
$$M = 79\%$$

$$\Delta_T = L/262$$

$$V = 22\%$$

Use W16 x 40

BM25:



$$P = 10.2 \text{ k DL} + 12.2 \text{ k LL} + 4.6 \text{ k SL}$$

$$W = (114 \text{ DL} + 250 \text{ LL})$$

$$R = 7.0 \text{ k DL} + 9.4 \text{ k LL} + 2.3 \text{ k SL}$$

Use W16 x 40

$$M = 98\% \quad \Delta_T = L/256$$



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Edmonds, WA 98020
425.778.8500
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Description	Beam Design		By	CLL	Date	7/20/23
			Checked		Date	
			Scale		Sheet No.	
			Project	The Talmon		
				Job No.		
				23154-10		



COMPANY

PROJECT

July 21, 2023 14:47

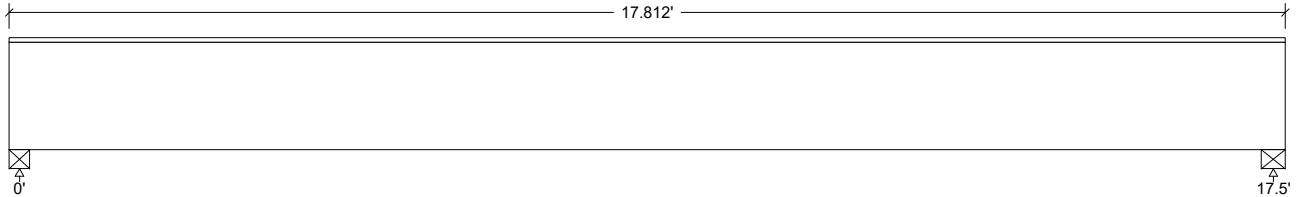
BM24

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			200.0	plf
DLp1	Dead	Point		3.64	5100	lbs
DLp2	Dead	Point		15.64	5100	lbs
LLp1	Live	Point		3.64	6100	lbs
LLp2	Live	Point		15.64	6100	lbs
SLp1	Snow	Point		3.64	2300	lbs
SLp2	Snow	Point		15.64	2300	lbs

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

Unfactored:			
Dead	6442		7321
Live	5577		6623
Snow	2103		2497
Factored:			
Total	12202		14161
Bearing:			
Capacity			
Beam	13666		15860
Support	12202		14161
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	3.47		4.03
Min req'd	3.47**		4.03**
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"x18"

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

Total length: 17.81'; Clear span: 17.188'; Volume = 11.7 cu.ft.

Lateral support: top = continuous, 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 = 216	Fv' = 290	psi	fv/Fv' = 0.74
Bending(+)	fb = 1725	Fb' = 2772	psi	fb/Fb' = 0.62
Live Defl'n	0.20 = < L/999	0.58 = L/360	in	0.34
Total Defl'n	0.55 = L/384	0.88 = L/240	in	0.62

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	2
Fb'+	2900	1.00	-	1.00	1.000	0.956	-	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 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 = 13910, v design = 13577 (NDS 3.4.3.1(a)) lbs; M(+) = 40760 lbs-ft
EI = 5613.30e06 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 = 1.50 permanent + "live"

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:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: BM14R

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

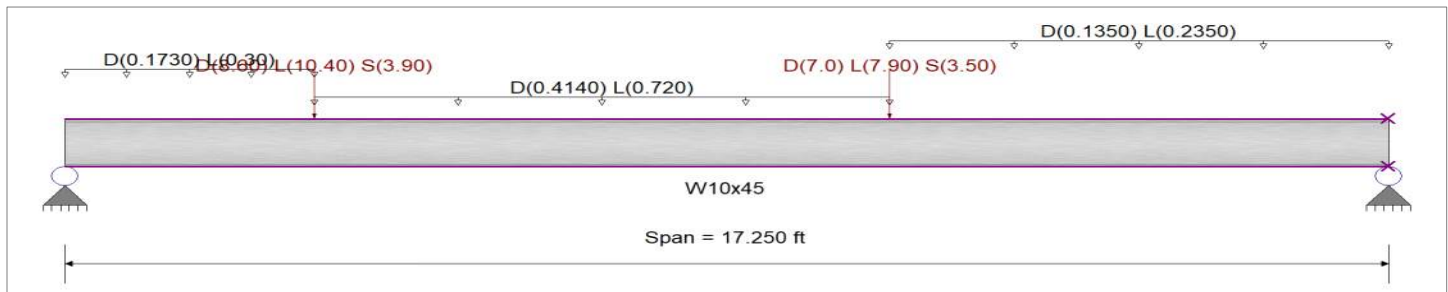
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



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 = 0.4140, L = 0.720 k/ft, Extent = 3.250 --> 10.750 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.1730, L = 0.30 k/ft, Extent = 0.0 --> 3.250 ft, Tributary Width = 1.0 ft

Uniform Load : D = 0.1350, L = 0.2350 k/ft, Extent = 10.750 --> 17.250 ft, Tributary Width = 1.0 ft

Point Load : D = 7.0, L = 7.90, S = 3.50 k @ 10.750 ft

Point Load : D = 8.60, L = 10.40, S = 3.90 k @ 3.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.821 : 1	Maximum Shear Stress Ratio =	0.395 : 1
Section used for this span	W10x45	Section used for this span	W10x45
Ma : Applied	112.411 k-ft	Va : Applied	27.934 k
Mn / Omega : Allowable	136.976 k-ft	Vn/Omega : Allowable	70.70 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Location of maximum on span	0.000 ft
		Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.480 in Ratio = 430 >=360	Span: 1 : L Only	
Max Upward Transient Deflection	0 in Ratio = 0 <360	n/a	
Max Downward Total Deflection	0.852 in Ratio = 243 >=240.	Span: 1 : +D+L	
Max Upward Total Deflection	0 in Ratio = 0 <240.0	n/a	

Vertical Reactions

Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	27.934	18.413
Max Upward from Load Combinations	27.934	18.413
Max Upward from Load Cases	15.797	10.405
D Only	12.137	8.008
+D+L	27.934	18.413
+D+S	16.621	10.924
+D+0.750L	23.985	15.812
+D+0.750L+0.750S	27.348	17.999
+0.60D	7.282	4.805
L Only	15.797	10.405
S Only	4.484	2.916

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: BM19B

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

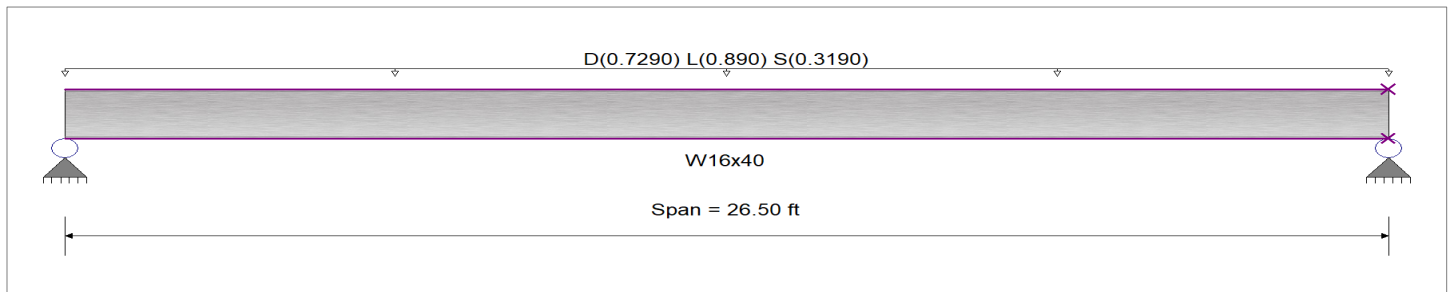
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.7290, L = 0.890, S = 0.3190 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		0.788 : 1		Maximum Shear Stress Ratio =		0.222 : 1	
Section used for this span		W16x40		Section used for this span		W16x40	
Ma : Applied		143.588 k-ft		Va : Applied		21.674 k	
Mn / Omega : Allowable		182.136 k-ft		Vn/Omega : Allowable		97.60 k	
Load Combination		+D+0.750L+0.750S		Load Combination		+D+0.750L+0.750S	
				Location of maximum on span		0.000 ft	
Span # where maximum occurs		Span # 1		Span # where maximum occurs		Span # 1	
Maximum Deflection							
Max Downward Transient Deflection		0.660 in Ratio =		481		>=360 Span: 1 : L Only	
Max Upward Transient Deflection		0 in Ratio =		0		<360 n/a	
Max Downward Total Deflection		1.214 in Ratio =		262		>=240. Span: 1 : +D+0.750L+0.750S	
Max Upward Total Deflection		0 in Ratio =		0		<240.0 n/a	

Vertical Reactions

Support notation : Far left is #

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	21.674	21.674
Max Upward from Load Combinations	21.674	21.674
Max Upward from Load Cases	11.793	11.793
D Only	9.659	9.659
+D+L	21.452	21.452
+D+S	13.886	13.886
+D+0.750L	18.504	18.504
+D+0.750L+0.750S	21.674	21.674
+0.60D	5.796	5.796
L Only	11.793	11.793
S Only	4.227	4.227

Project Title:
 Engineer:
 Project ID:
 Project Descr:

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: **BM25**

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

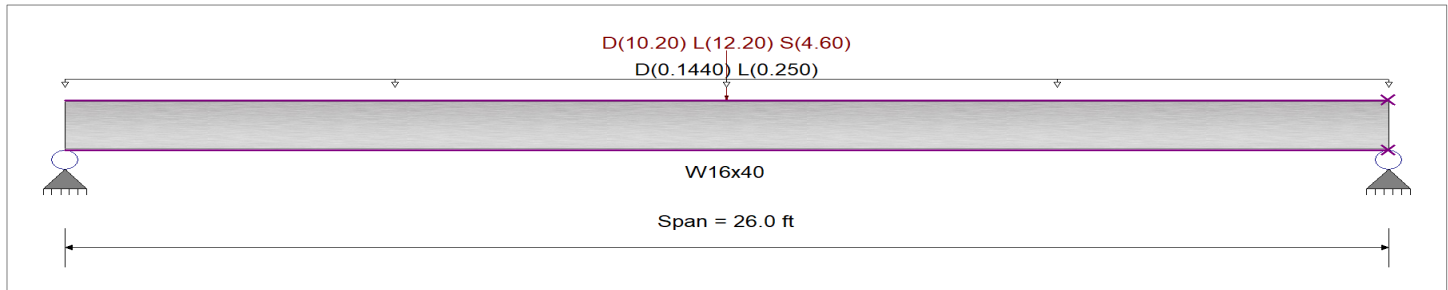
Analysis Method : Allowable Strength Design

Fy : Steel Yield : 50.0 ksi

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

E: Modulus : 29,000.0 ksi

Bending Axis : Major Axis Bending



Applied Loads

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

Beam self weight NOT internally calculated and added

Load(s) for Span Number 1

Point Load : D = 10.20, L = 12.20, S = 4.60 k @ 13.0 ft

Uniform Load : D = 0.1440, L = 0.250 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.982 : 1	Maximum Shear Stress Ratio =	0.167 : 1
Section used for this span	W16x40	Section used for this span	W16x40
Ma : Applied	178.893 k-ft	Va : Applied	16.322 k
Mn / Omega : Allowable	182.136 k-ft	Vn/Omega : Allowable	97.60 k
Load Combination	+D+L	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Maximum Deflection			
Max Downward Transient Deflection	0.687 in Ratio = 453	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in Ratio = 0	<360	n/a
Max Downward Total Deflection	1.218 in Ratio = 256	>=240.	Span: 1 : +D+L
Max Upward Total Deflection	0 in Ratio = 0	<240.0	n/a

Vertical Reactions

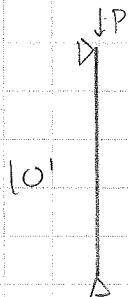
Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	16.322	16.322
Max Upward from Load Combinations	16.322	16.322
Max Upward from Load Cases	9.350	9.350
D Only	6.972	6.972
+D+L	16.322	16.322
+D+S	9.272	9.272
+D+0.750L	13.985	13.985
+D+0.750L+0.750S	15.710	15.710
+0.60D	4.183	4.183
L Only	9.350	9.350
S Only	2.300	2.300

Typ Steel Column (Non-Moment Frame):

Use HSS 6x6x3/8



Determine Max P:

$P = 140 \text{ kips}$ ← Max Column Loading

Axial = 99%

Foundation Design:

Bearing Pressure = 4000 psf

Max Wall Line Load = $(1021 \text{ DL} + 1415 \text{ LL} + 304 \text{ SL}) \text{ p/f}$
ASD = 2436 p/f

Typ Cont FTG = 1'-6" width → 6000 p/f > 2436 p/f

Typ 1'-6" wide Cont FTG Adequate

Pad FTG Design:

Max Point Load on Cont FTG = 12 Kips

If Point Load greater than 12K, use pad footing...

<u>Size</u>	<u>Capacity</u>
2'x2'	16 K
3'x3'	36 K
4'x4'	64 K



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Description

By CCC

Date 7/21/23

Checked

Date

Scale

Sheet No.

Project The Talmon

Job No.

23154.10

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Column

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: Typ Column

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS6x6x3/8**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0
Unbraced Length for buckling ABOUT Y-Y Axis = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 274.058 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 140.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =
Load Combination
Location of max.above base
At maximum location values are . . .
Pa : Axial 140.274 k
Pn / Omega : Allowable 141.228 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 28.383 k-ft
Ma-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 28.383 k-ft

0.9932 : 1
D Only
0.0 ft
140.274 k
141.228 k
0.0 k-ft
28.383 k-ft
0.0 k-ft
28.383 k-ft

Maximum Load Reactions . .

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

Maximum Load Deflections . . .

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

PASS Maximum Shear Stress Ratic

Load Combination
Location of max.above base
At maximum location values are . . .
Va : Applied
Vn / Omega : Allowable

0.0 : 1
0.0
0.0 ft
0.0 k
0.0 k

Sketches



SECTION NO. 2

The Talmon
Centre Street
La Conner, WA 98257

SECTION TITLE: Main Building Lateral Design

SECTION INDEX

DESCRIPTION	PAGE
Lateral Force Analysis	2.001-2.011
Elevation Key Plan	2.012
Lateral Key Plan	2.013-2.015
3 rd Floor Lateral Analysis	2.016-2.017
2 nd Floor Lateral Analysis	2.018-2.019
1 st Floor Lateral Analysis	2.020-2.026
Diaphragm Analysis	2.027-2.030
Moment Frame Design	2.031-2.052

Seismic Analysis

Design Per 2018 IBC & ASCE 7-16

Seismic Coefficients

Soil Site Class	E (per Geotech)
Occupancy Category	II (non-essential facility)
Seismic Design Category	D

$S_S =$	1.200	Lat. =	48.393
$S_1 =$	0.427	Long. =	-122.493

$S_{MS} = F_a S_S =$	1.440	$F_a =$	1.200
$S_{M1} = F_v S_1 =$	0.811	$F_v =$	1.900

$S_{DS} = (2/3) S_{MS} =$	0.960		
$S_{D1} = (2/3) S_{M1} =$	0.541		

$T_a = C_t h_n^x =$	0.27	$C_t =$	0.02
		$h_n =$	32
		$x =$	0.75

R Factor =	6.5	(Wood 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

$V = 0.044 S_{DS} I =$	0.042 W	(Minimum Force)	ASCE 7-16 (Eq. 12.8-5)
------------------------	---------	-----------------	------------------------

$V = (S_{DS} I W) / R =$	0.148 W	(Governing Force)	ASCE 7-16 (Eq. 12.8-2)
--------------------------	---------	-------------------	------------------------

$V = (S_{D1} I W) / R T_a =$	0.309 W	(Maximum Force)	ASCE 7-16 (Eq. 12.8-3)
------------------------------	---------	-----------------	------------------------



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

Description	Seismic Base Shear	By	ERH	Date	07/07/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	The Talmon	Job No.	23154.10		

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= **34.75**
 Mean Roof Height (feet) h_{mean}= **34.75**
 Building Length (feet) L= **68.5**
 Building Width (feet) W= **142**
 End Zone Width, a (feet) a= 6.85

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

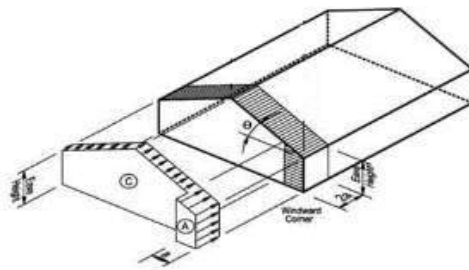
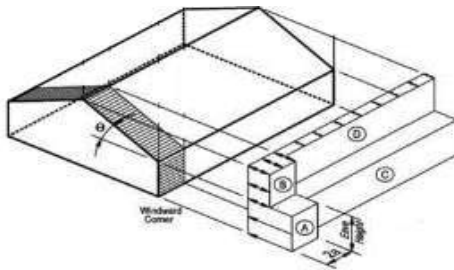
Figure 28.6-1

Figure 28.6-1

Height/Exposure Adjustm λ_{max}= 1.40
 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} =	21.4	12.8
p _{s30B} =	-11.0	-6.6
p _{s30C} =	14.1	8.5
p _{s30D} =	-6.6	-3.9

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

The Talmon

By

ERH

Checked

Scale

NTS

Job No.

23154.10

Date

7/6/2023

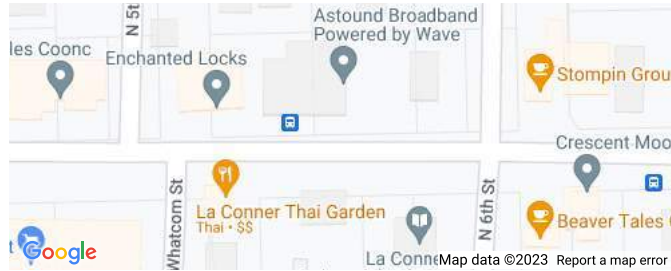
Date

Sheet No.

ATC Hazards by Location

Search Information

Address: Centre St, La Conner, WA 98257, USA
Coordinates: 48.3929132, -122.492883
Elevation: 5 ft
Timestamp: 2023-05-23T17:34:08.915Z
Hazard Type: Wind



ASCE 7-16

MRI 10-Year 67 mph
MRI 25-Year 73 mph
MRI 50-Year 78 mph
MRI 100-Year 83 mph
Risk Category I 92 mph
Risk Category II 98 mph
Risk Category III 105 mph
Risk Category IV 109 mph

ASCE 7-10

MRI 10-Year 72 mph
MRI 25-Year 79 mph
MRI 50-Year 85 mph
MRI 100-Year 91 mph
Risk Category I 100 mph
Risk Category II 110 mph
Risk Category III-IV 115 mph

ASCE 7-05

ASCE 7-05 Wind Speed 85 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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ATC Hazards by Location

Search Information

Address: Centre St, La Conner, WA 98257, USA
Coordinates: 48.3929132, -122.492883
Elevation: 5 ft
Timestamp: 2023-05-23T17:41:02.328Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: II
Site Class: E



Basic Parameters

Name	Value	Description
S_S	1.199	MCE_R ground motion (period=0.2s)
S_1	0.427	MCE_R ground motion (period=1.0s)
S_{MS}	** null	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	** null	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

** See Section 11.4.8

▼Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	** null	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.909	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.512	MCE_G peak ground acceleration
F_{PGA}	1.188	Site amplification factor at PGA
PGA_M	0.608	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
S_{sRT}	1.199	Probabilistic risk-targeted ground motion (0.2s)
S_{sUH}	1.32	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S_{sD}	1.5	Factored deterministic acceleration value (0.2s)
S_{1RT}	0.427	Probabilistic risk-targeted ground motion (1.0s)
S_{1UH}	0.477	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S_{1D}	0.6	Factored deterministic acceleration value (1.0s)
$PGAd$	0.539	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

** See Section 11.4.8

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

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

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Seismic Hazard

The overall subsurface profile corresponds to a Site Class *E* as defined by Table 1613.5.2 of the International Building Code (IBC).

We referenced the U.S. Geological Survey (USGS) Earthquake Hazards Program Website to obtain values for S_s , S_i , F_a , and F_v . The USGS website includes the most updated published data on seismic conditions. The following tables provide seismic parameters from the USGS web site with referenced parameters from ASCE 7-16.

Seismic Design Parameters (ASCE 7-16)

Site Class	Spectral Acceleration at 0.2 sec. (g)	Spectral Acceleration at 1.0 sec. (g)	Site Coefficients		Design Spectral Response Parameters		Design PGA
			F_a	F_v	S_{DS}	S_{D1}	
E	1.2	0.427	Null	Null	Null	Null	0.512

For items listed as “Null” see Section 11.4.8 of the ASCE.

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft/loose soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table.

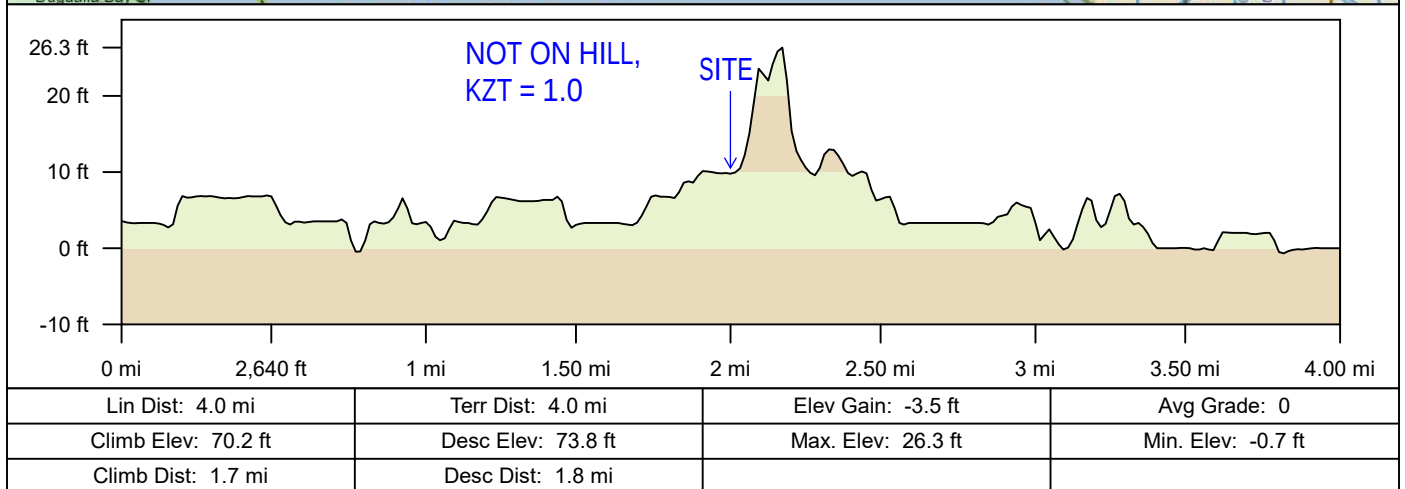
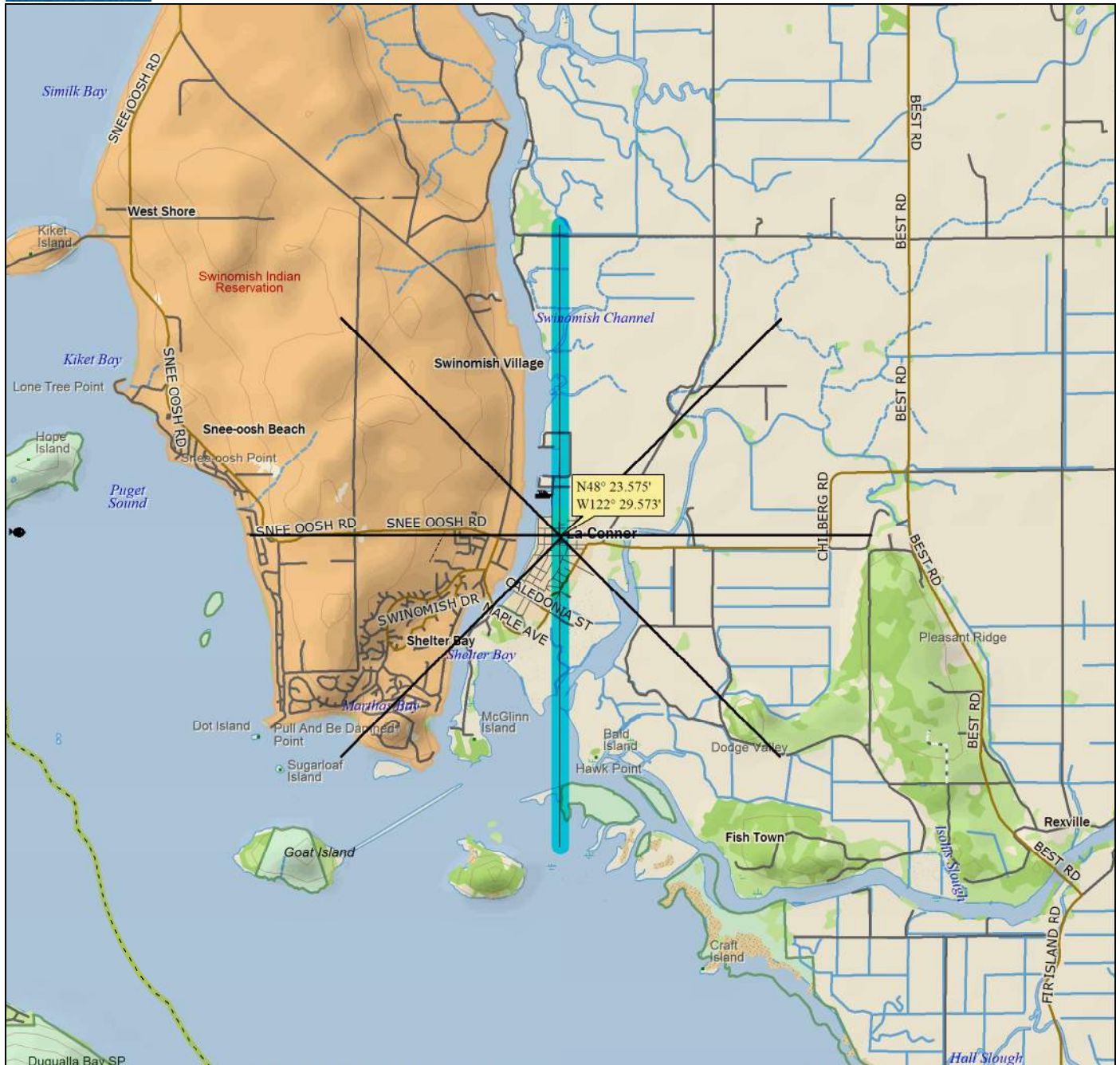
Soil liquefaction is a state where soil particles lose contact with each other and become suspended in a viscous fluid. This suspension of the soil grains results in a complete loss of strength as the effective stress drops to zero as a result of increased pore pressures. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. However, liquefaction has occurred in soils other than clean sand, such as low plasticity silt. Liquefaction usually occurs under vibratory conditions such as those induced by seismic events.

To evaluate the liquefaction potential of the site, we analyzed the following factors:

- 1) Soil type and plasticity
- 2) Groundwater depth
- 3) Relative soil density
- 4) Initial confining pressure
- 5) Maximum anticipated intensity and duration of ground shaking

The commercially available liquefaction analysis software, LiqSVS was used to evaluate the liquefaction potential and the possible liquefaction induced settlement for the existing site soil conditions. Maximum Considered Earthquake (MCE) was selected in accordance with the ASCE, *International Building Code* and the U.S. Geological Survey (USGS) Earthquake Hazards Program website.

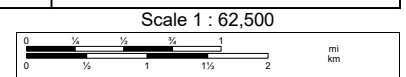
For this site, we used a peak ground acceleration of 0.512g and a 7.0M earthquake in the liquefaction analyses.



Data use subject to license.

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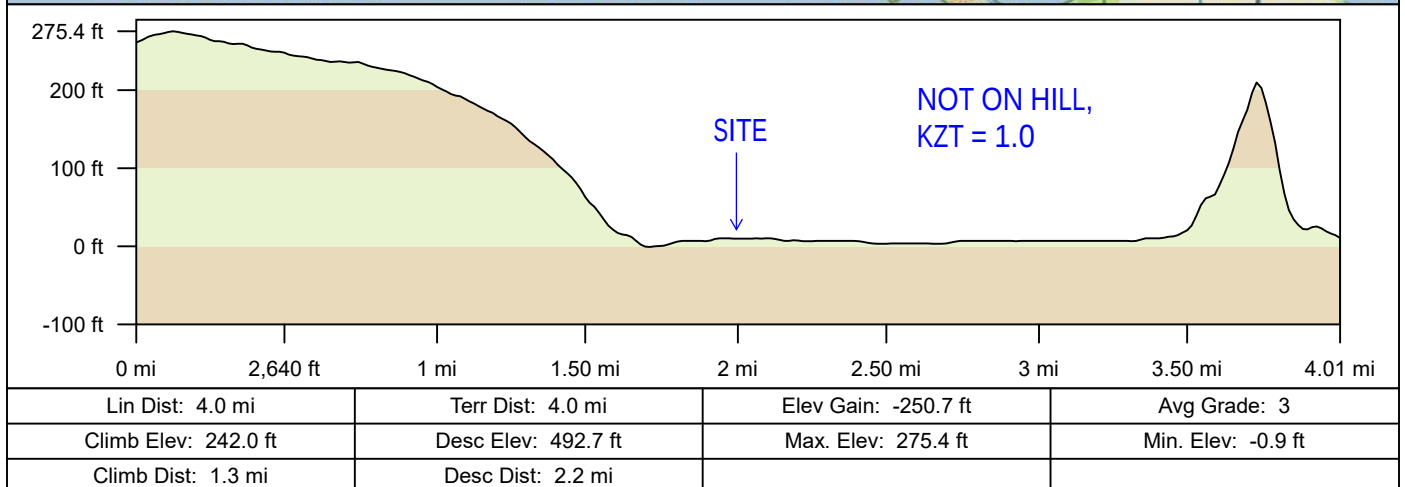
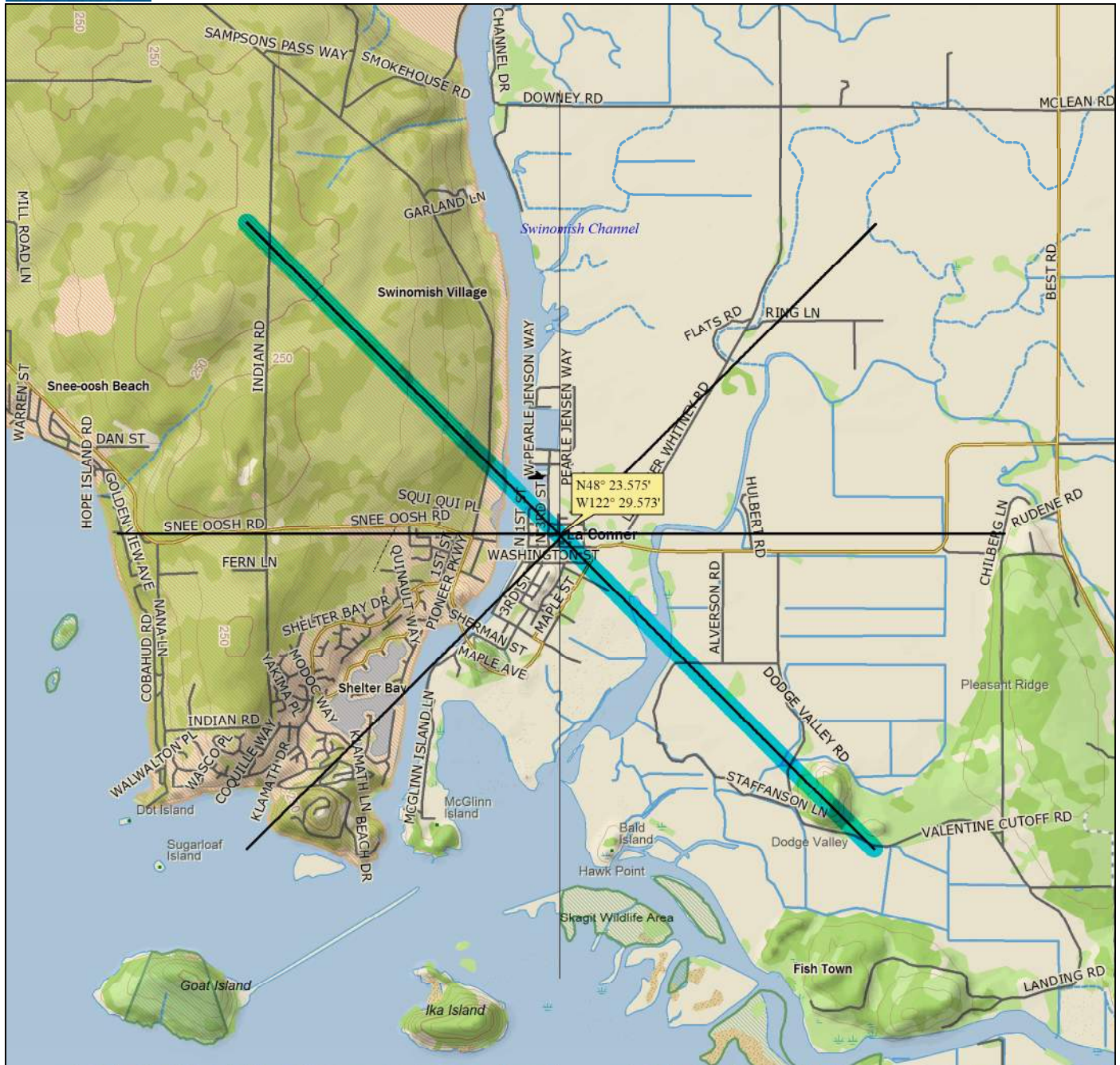
www.delorme.com



Scale 1 : 62,500

1" = 5,208.3 ft Data Zoom 11-6

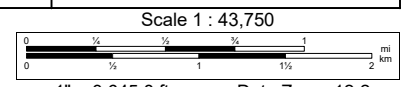
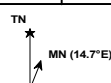
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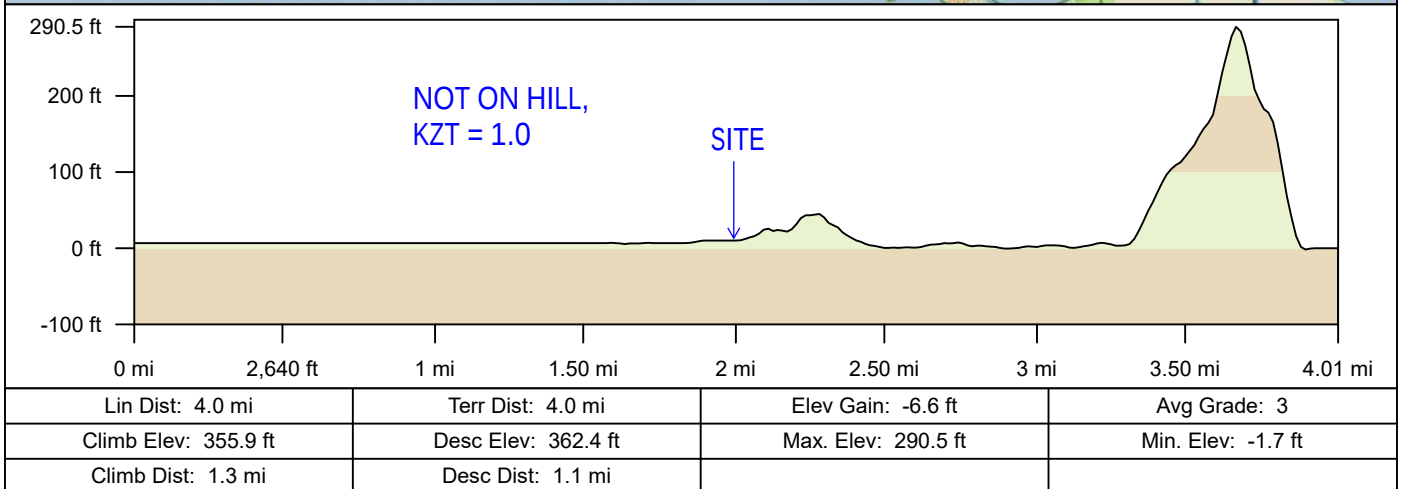
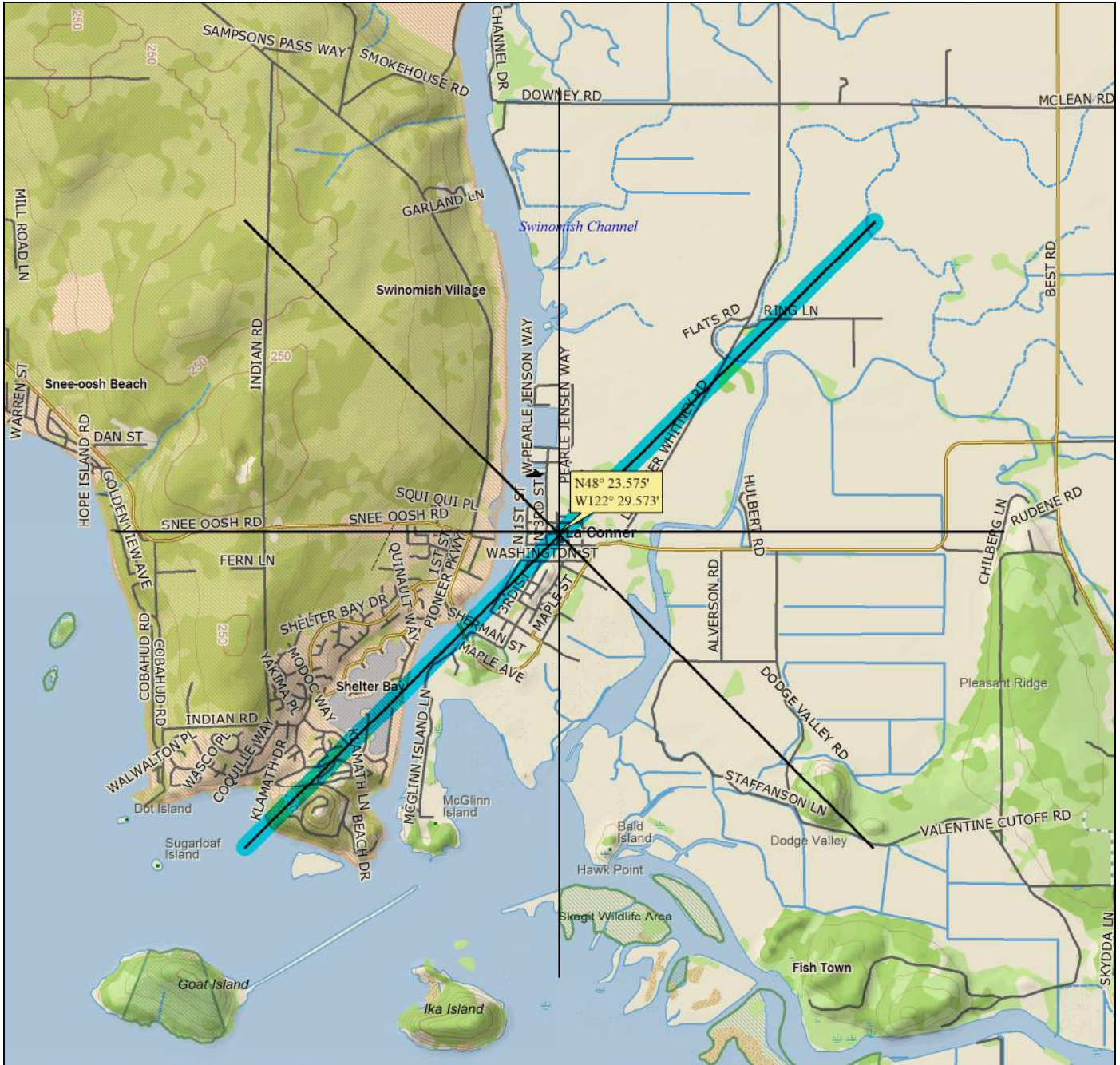
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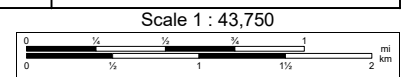
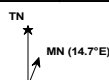
2.007



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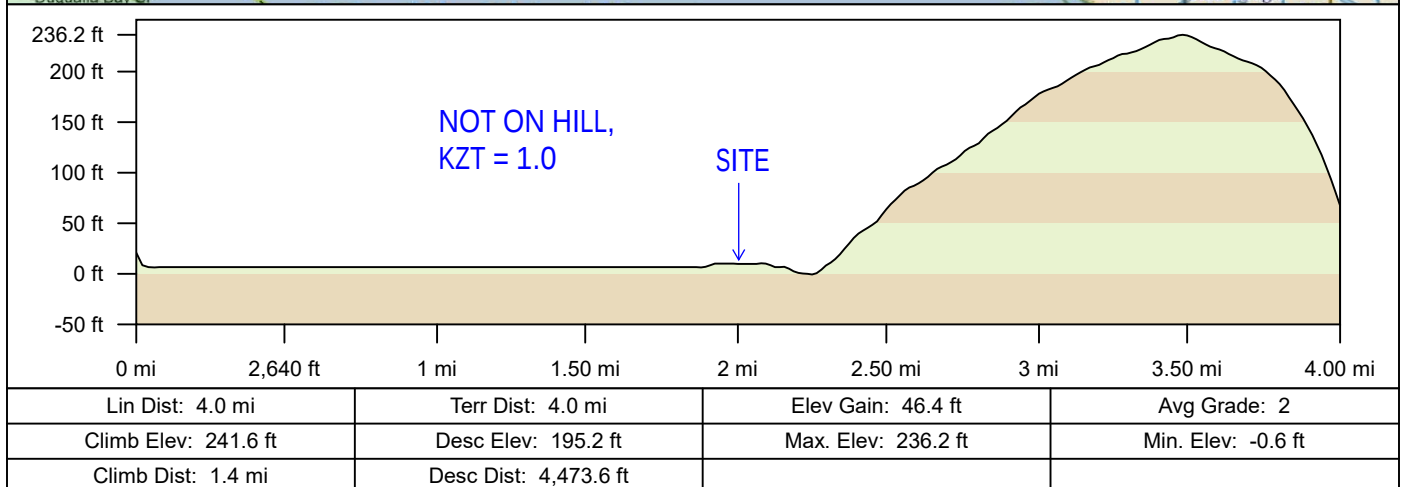


Scale 1 : 43,750

1" = 3,645.8 ft

Data Zoom 12-2

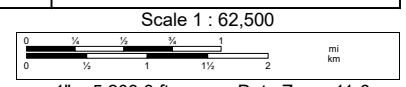
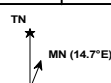
2.008



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Data Zoom 11-6
2.009

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
Roof Framing	22	9851	216.7	33.59	7280	0.38	33.7	33.7
3rd Framing	33	9317	307.5	23.92	7354	0.38	34.1	67.8
2nd Framing	33	9591	316.5	14.25	4510	0.24	20.9	88.7
				4.58				
		Σ =	840.7	-	19144	1.00	88.7	-

Base Shear (ULT) 124.2 kips

Base Shear (ASD) **88.7 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
Roof Framing	1.0	33.7	33.7
3rd Framing	1.0	34.1	67.8
2nd Framing	1.0	20.9	88.7
	1.0	0.0	88.7
	1.0	0.0	88.7
		Σ =	88.7 -

Diaphragm Forces - Vertical Distribution

Refer to ASCE 7-16 Section 12.10.1.1

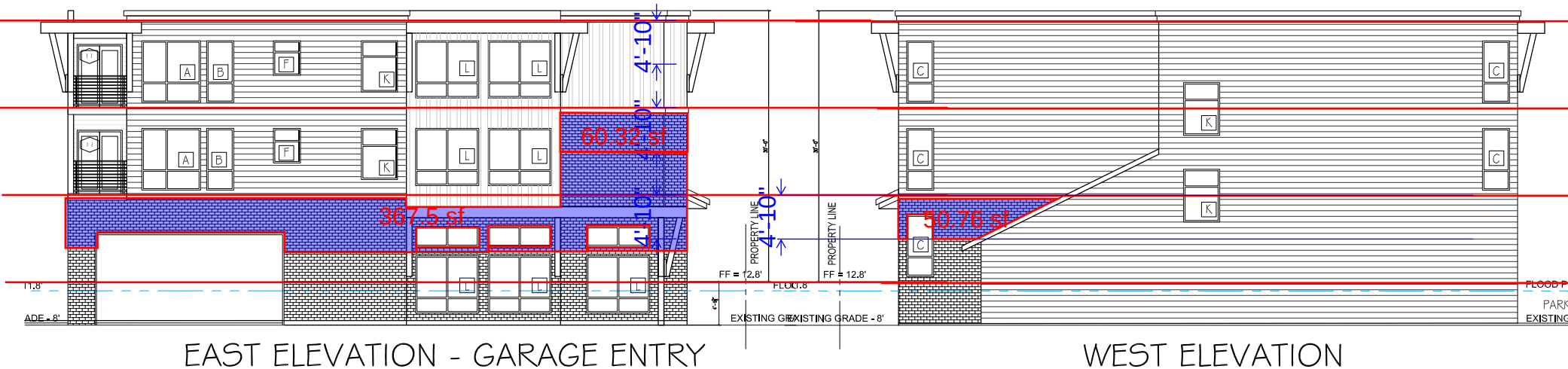
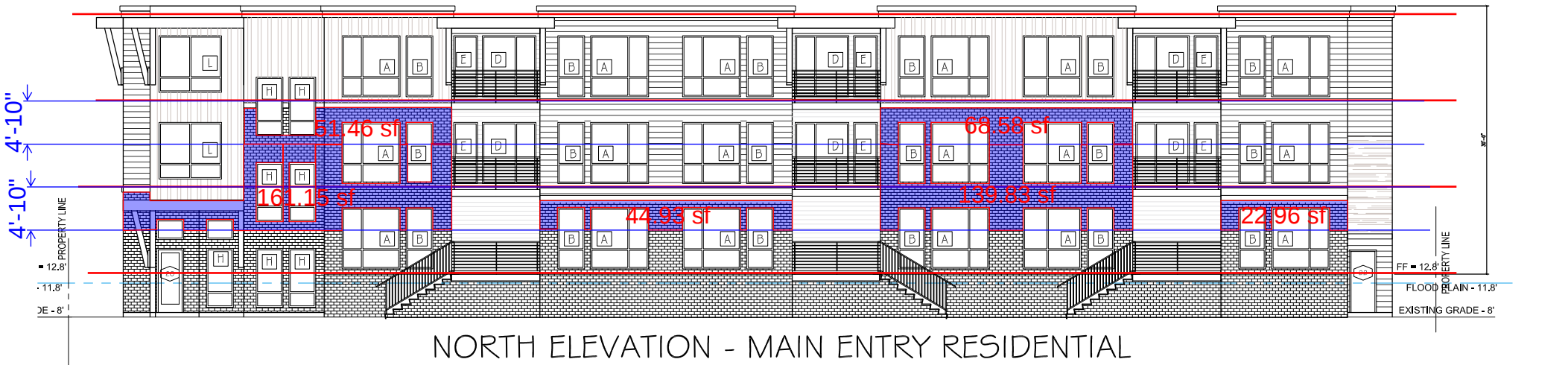
Diaphragm Level	w _i (kips)	Σ w _i (kips)	F _i (kips)	Σ F _i (kips)	Σ F _i + w _{px} Σ w _i	F _{px} (Min) 0.2S _{DS} lw _{px}	F _{px} (Max) 0.4S _{DS} lw _{px}	F _{px} Govern
Roof Framing	216.7	216.7	33.7	33.7	33.7	29.7	59.4	33.7
3rd Framing	307.5	524.2	34.1	67.8	39.8	42.2	84.3	42.2
2nd Framing	316.5	840.7	20.9	88.7	33.4	43.4	86.8	43.4
	0.0	840.7	0.0	88.7	0.0	0.0	0.0	0.0
	0.0	840.7	0.0	88.7	0.0	0.0	0.0	0.0

 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Seismic & Diaphragm Force Distribution	By	ERH	Date	07/17/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	The Talmon	Job No.	23154.10		

Wind Forces

					X-Dir	Y-Dir
Diaphragm	Story Height	Elevation	B	L	p,asd	p,asd
Level					(ft)	(ft)
Roof Framing	9.67	33.59	68.5	142	3.6	7.0
3rd Framing	9.67	23.92	68.5	142	7.2	14.0
2nd Framing	9.67	14.25	68.5	142	7.2	14.0
	4.58	4.58			0.6	0.6
	0	0			0.8	0.2

 ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description	Wind Design	By	ERH	Date	07/17/23
			Checked		Date	
			Scale	NTS	Sheet No.	
	Project	The Talmon	Job No.			
				23154.10		

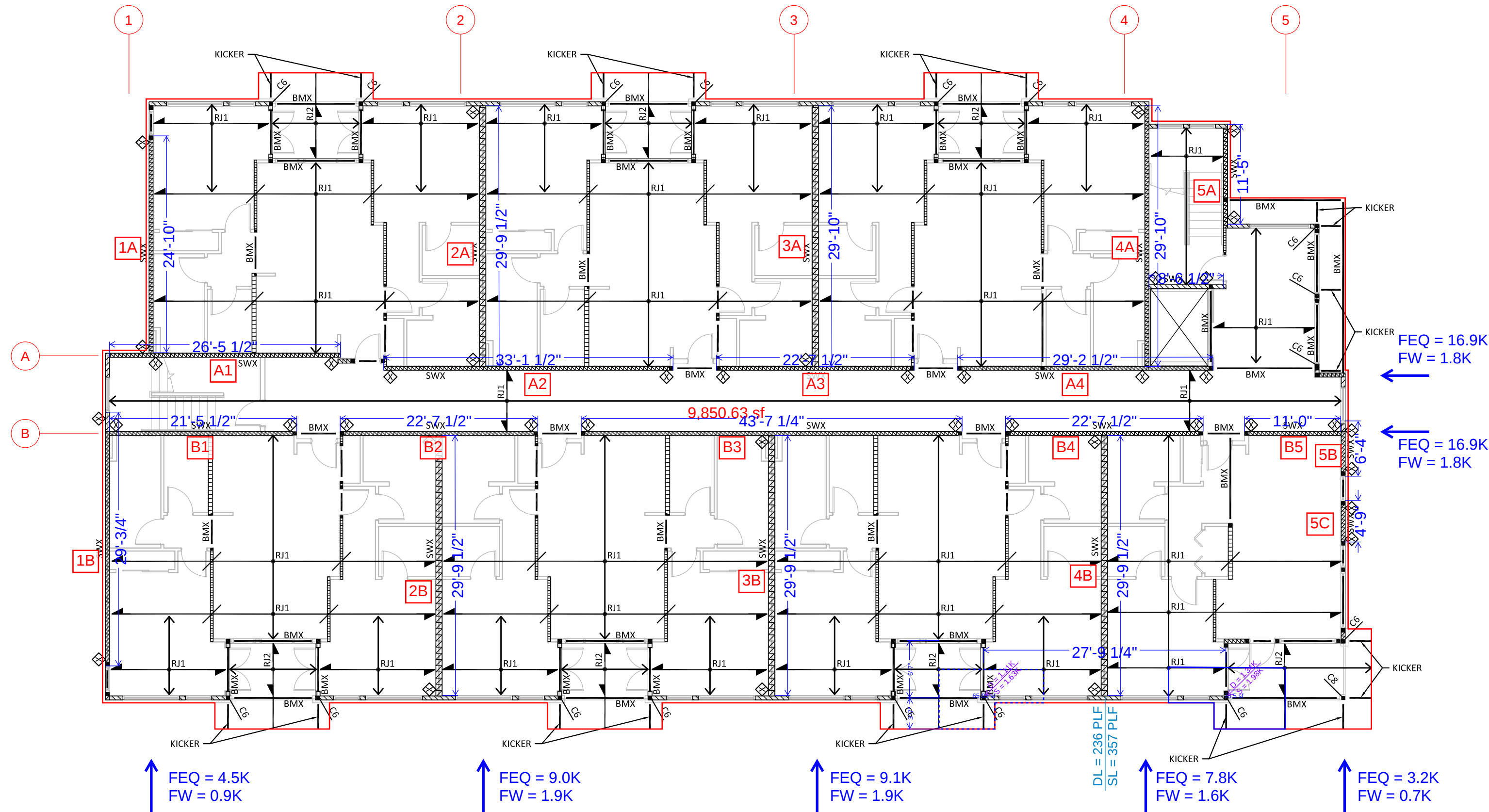


BRICK AREA KEY PLAN

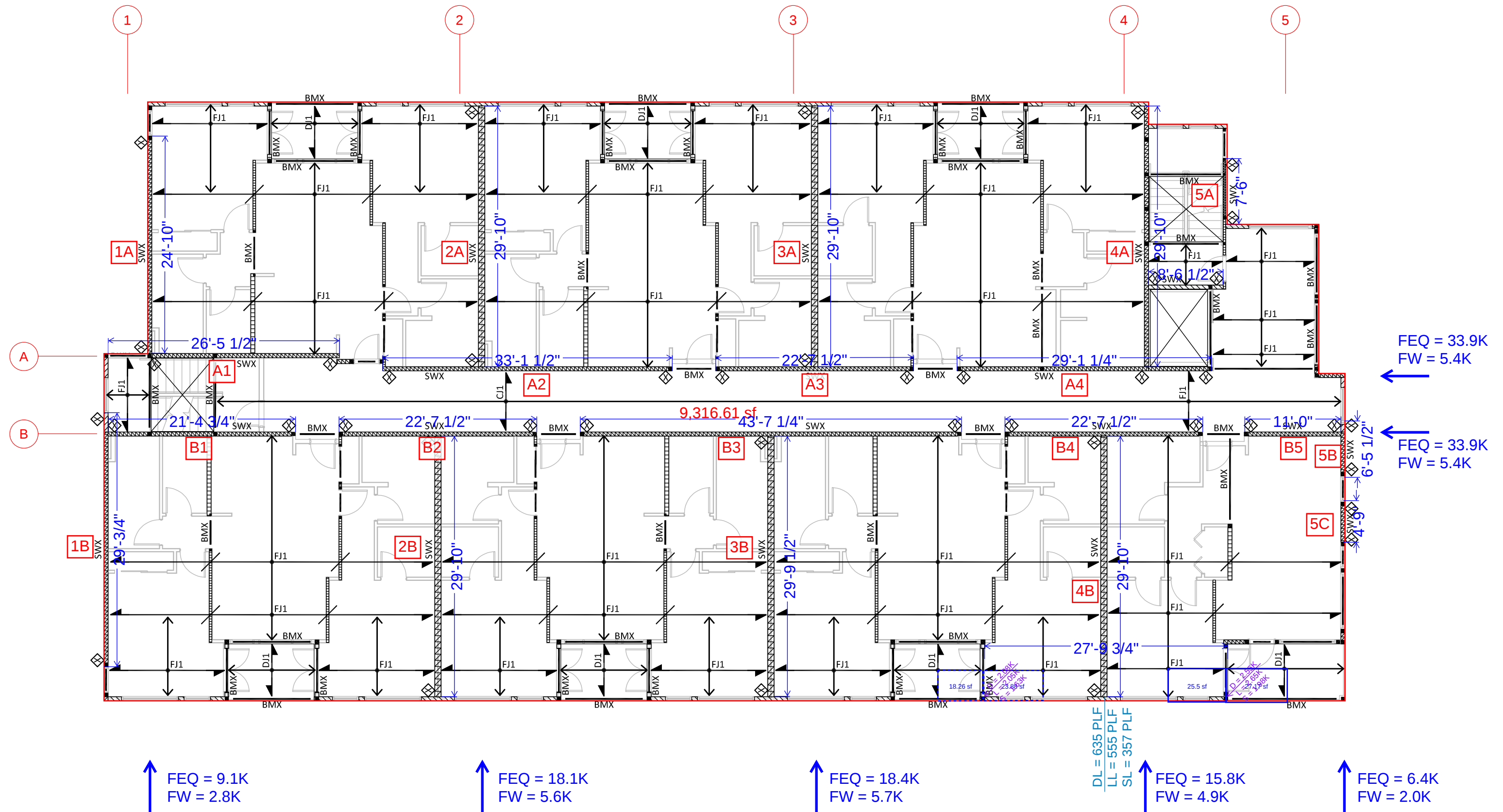
BRICK AREA = 967 SF
 BRICK DENSITY = $4.5\text{LB}/65.25\text{CI} = 0.069\text{PCI} = 119\text{ PCF}$
 2" BRICK VENEER = $119\text{ PCF} \times (2"/12\text{IN}/\text{FT}) = 19.83\text{ PSF}$
 BRICK WEIGHT = $967\text{ SF} \times 19.83\text{ PSF} = 19.18\text{ K}$

SEISMIC FORCE = $0.148 \times 19.18\text{K} = 2.84\text{K}$
 --> APPLY AT LOWER LEVEL

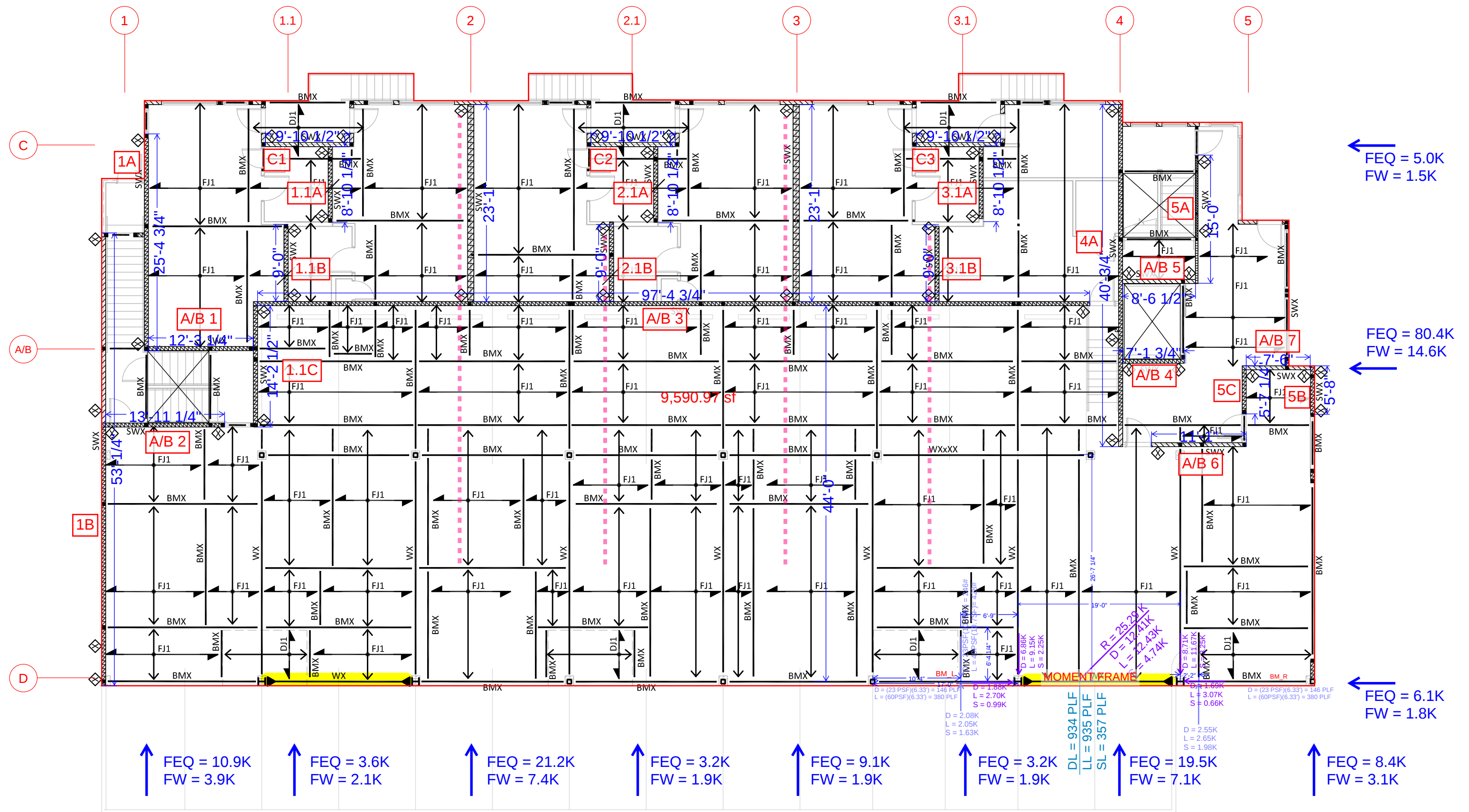
UPPER FLOOR LATERAL KEY PLAN



MAIN FLOOR LATERAL KEY PLAN



LOWER FLOOR LATERAL KEY PLAN



X - Direction Walls

F_x (EQ) =	33.7 kips (Story Shear)
F_x (wind) =	3.6 kips (Story Shear)

Story HT =	9.67
Wall HT =	8.67
Max h/w	3.5
S_{DS} =	0.96

Wx =	16861.9 PLF	seismic
Wx =	1800.0 PLF	wind

[illegible]

Shearwalls: 1/2" sheathing w/ HF studs			
Nil	-	0	plf
SW6	8d@6"o.c.	242	plf
SW4	8d@4"o.c.	350	plf
SW3	8d@3"o.c.	455	plf
SW2	8d@2"o.c.	595	plf
2SW4	8d@4"o.c.	706	plf
2SW3	8d@3"o.c.	910	plf
2SW2	8d@2"o.c.	1190	plf
Re-Calc	-	1200	plf

Holdown Table (Floor Clear Span = 16')			
Nil	-	0	kips
None	-	0.5	kips
MST37	(2)-2x HF	2.345	kips
MST48	(2)-2x HF	3.640	kips
MST60	(2)-2x HF	5.405	kips
MST72	(2)-2x HF	6.475	kips
			kips
			kips
			kips
			kips
Re-Calc	-	6.5	kips

- Input Cell
- Input Cell w/ Formula



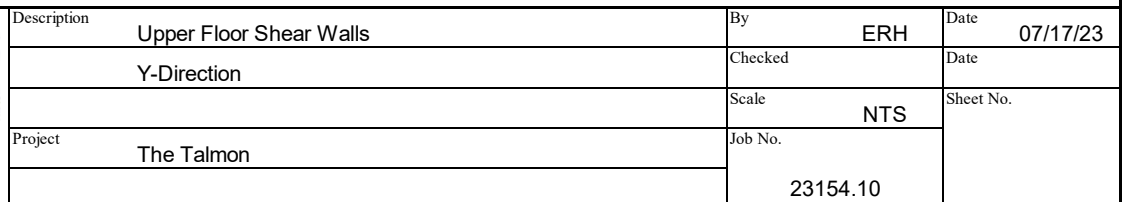
250 4th Ave. South
Suite 200
Edmonds, WA 98020

Description	Upper Floor Shear Walls	By	ERH	Date	07/17/23
	X-Direction	Checked		Date	
		Scale	NTS	Sheet No.	
Project	The Talmon	Job No.			
		23154.10			

Y - Direction Walls

Story HT =	9.67
Wall HT =	8.67
Max h/w	3.5
S_{DS} =	0.96

- Input Cell
- Input Cell w/ Formula



Main Floor Shear Walls - Walls Below the Upper Floor Framing

X - Direction Walls

Fx (EQ) = 34.1 kips (Story Shear)
 Fx (wind) = 7.2 kips (Story Shear)

Story HT = 9.67
 Wall HT = 8.67
 Max h/w = 3.5
 $S_{DS} = 0.96$

Wx = 17035 PLF seismic
 Wx = 3600 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	0.6 * DL	From Above		Net Uplift		End-down						
				EQ	Wind										End i	End j	End i	End j	End i	End j	End i	End j			
A	1	26.46	1	16.9	1.8	1.0	283	32	SW4	26.0	2.8	0.4	1.0	1.0	1.3	1.3	0.5	0.5	2.2	2.2	MST37	MST37	33.9	5.4	3.5
	2	33.125	-	-	-	1.0	283	32	SW4	32.6	2.8	0.4	1.3	1.3	1.7	1.7	0.3	0.3	1.7	1.7	MST37	MST37	-	-	3.5
	3	22.625	-	-	-	1.0	283	32	SW4	22.1	2.8	0.4	0.9	0.9	1.1	1.1	0.6	0.6	2.5	2.5	MST48	MST48	-	-	3.5
	4	29.08	-	-	-	1.0	283	32	SW4	28.6	2.8	0.4	1.1	1.1	1.5	1.5	0.4	0.4	2.0	2.0	MST37	MST37	-	-	3.5
	5	8.54	-	-	-	1.0	283	32	SW4	8.0	2.9	0.5	0.2	0.2	0.2	0.2	1.3	1.3	4.0	4.0	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
B	1	21.4	1	16.9	1.8	1.0	280	32	SW4	20.9	2.8	0.4	0.8	0.8	1.1	1.1	0.6	0.6	2.6	2.6	MST48	MST48	33.9	5.4	3.5
	2	22.625	-	-	-	1.0	280	32	SW4	22.1	2.8	0.4	0.9	0.9	1.1	1.1	0.6	0.6	2.5	2.5	MST48	MST48	-	-	3.5
	3	43.58	-	-	-	1.0	280	32	SW4	43.1	2.7	0.4	1.7	1.7	2.2	2.2	0.0	0.0	1.0	1.0	MST37	MST37	-	-	3.5
	4	22.625	-	-	-	1.0	280	32	SW4	22.1	2.8	0.4	0.9	0.9	1.1	1.1	0.6	0.6	2.5	2.5	MST48	MST48	-	-	3.5
	5	11	-	-	-	1.0	280	32	SW4	10.5	2.8	0.5	0.2	0.2	0.3	0.3	1.0	1.0	3.6	3.6	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
										-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
										-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
										-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				2.0	33.7	3.6																		67.8	10.8

Shearwalls: 1/2" sheathing w/ HF studs			
Nil	-	0	plf
SW6	8d@6"o.c.	242	plf
SW4	8d@4"o.c.	350	plf
SW3	8d@3"o.c.	455	plf
SW2	8d@2"o.c.	595	plf
2SW4	8d@4"o.c.	706	plf
2SW3	8d@3"o.c.	910	plf
2SW2	8d@2"o.c.	1190	plf
Re-Calc	-	1200	plf

Holdown Table (Floor Clear Span = 16")			
Nil	-	0	kips
None	-	0.5	kips
MST37	(2)-2x HF	2.345	kips
MST48	(2)-2x HF	3.640	kips
MST60	(2)-2x HF	5.405	kips
MST72	(2)-2x HF	6.475	kips
			kips
			kips
			kips
Re-Calc	-	6.5	kips

Input Cell
 Input Cell w/ Formula



Description	Main Floor Shear Walls	By	ERH	Date	07/17/23
	X-Direction	Checked		Date	
	Project	Scale	NTS	Sheet No.	
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Main Floor Shear Walls - Walls Below the Upper Floor Framing

Y - Direction Walls

Fy (EQ) = 34.1 kips (Story Shear)
 Fy (wind) = 14.0 kips (Story Shear)

Story HT = 9.67
 Wall HT = 8.67
 Max h/w = 3.5
 S_{DS} = 0.96

Wy = 240 PLF seismic
 Wy = 99 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	0.6 * DL	From Above	Net Uplift	End i	End j	End i	End j					
				EQ	Wind																End i	End j			End i
1	A	24.83	19	4.5	0.9	1.0	169	37	SW6	24.3	1.7	0.5	1.2	1.2	1.5	1.5	0.0	0.0	0.5	0.5	None	None	9.1	2.8	5.0
		29	-	-	-	1.0	169	37	SW6	28.5	1.7	0.5	1.4	1.4	1.8	1.8	0.0	0.0	0.3	0.3	None	None	-	-	5.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
										-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
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									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
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									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
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									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
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									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
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									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
									-0.5			0.0	0.0	0.0	0.0	0.00	0.00							0.0	
	</																								

Input Cell
 Input Cell w/ Formula



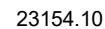
Description	Main Floor Shear Walls	By	ERH	Date	07/17/23
	Y-Direction	Checked		Date	
		Scale	NTS	Sheet No.	
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X - Direction Walls

$W_x =$	346 PLF	seismic
$W_x =$	105 PLF	wind

Σ	68.5	101.7	16.2
----------	-------------	--------------	-------------

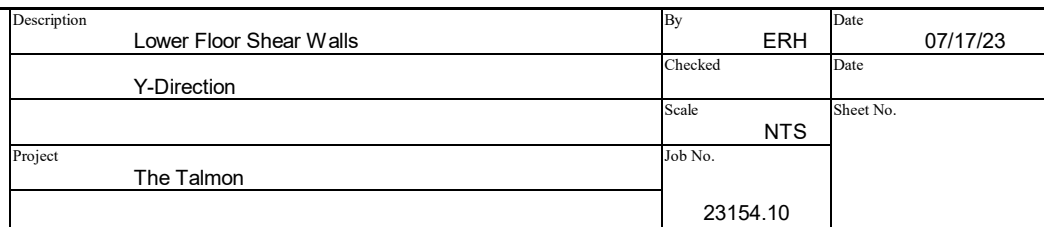
- Input Cell
- Input Cell w/ Formula



Y - Direction Walls

Wy =	167 PLF	seismic
Wy =	99 PLF	wind

Input Cell
Input Cell w/ Formula



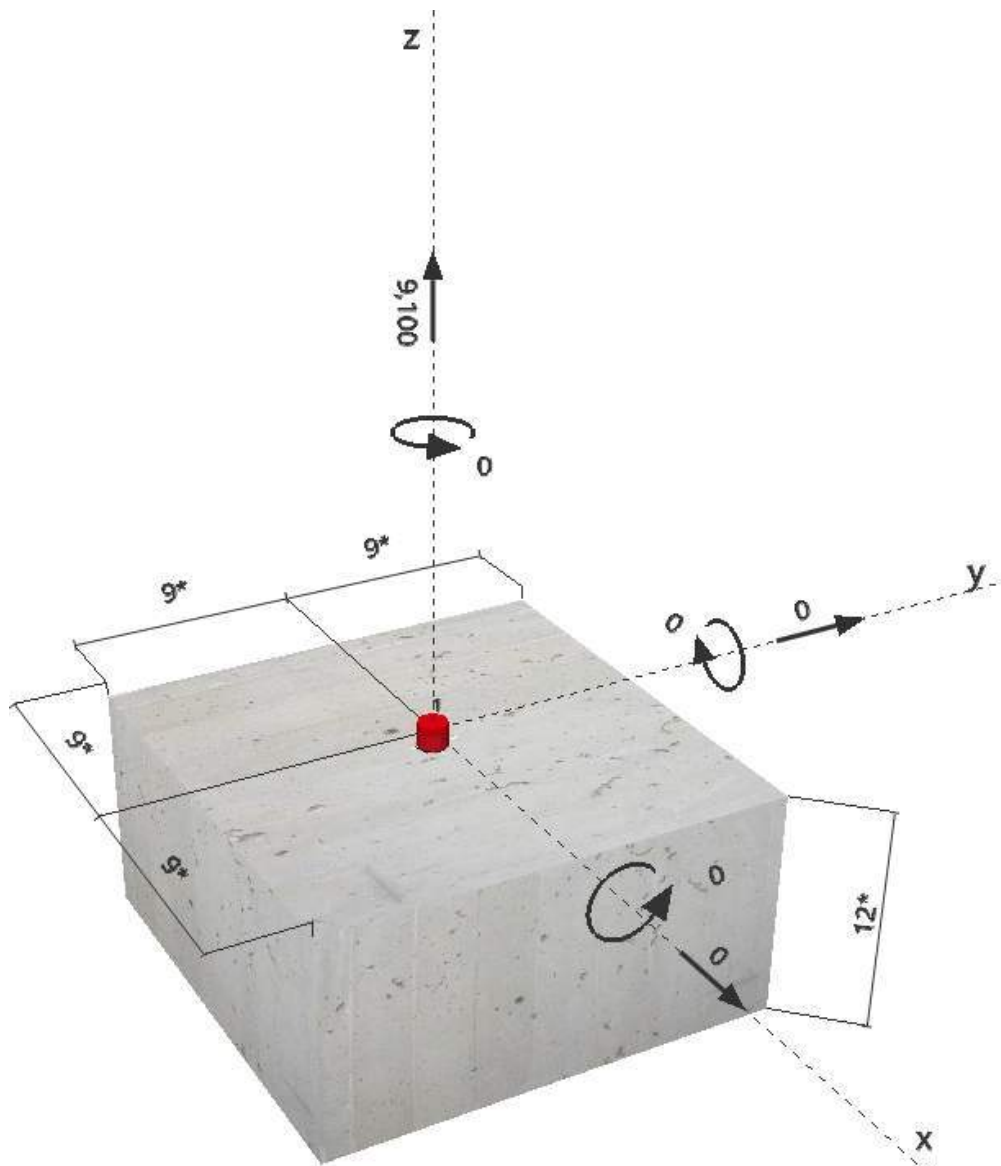
Specifier's comments:

1 Input data

Anchor type and diameter:	Hex Head ASTM F 1554 GR. 36 1
Effective embedment depth:	$h_{ef} = 9.000$ in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	- (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, $f_c' = 2500$ psi; $h = 12.000$ in.
Reinforcement:	tension: condition B, shear: condition B; edge reinforcement: none or < 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: CG Engineering
 Specifier:
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 Phone I Fax: |
 E-Mail:

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2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	9100	0	0	0
max. concrete compressive strain:		- [%]		
max. concrete compressive stress:		- [psi]		
resulting tension force in (x/y)=(0.000/0.000):	0 [lb]			
resulting compression force in (x/y)=(0.000/0.000):	0 [lb]			

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	9100	26361	35	OK
Pullout Strength*	9100	12211	75	OK
Concrete Breakout Strength**	9100	9259	99	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$$N_{sa} = A_{se,N} f_{uta} \quad \text{ACI 318-14 Eq. (17.4.1.2)}$$

$$\phi N_{sa} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$A_{se,N}$ [in. ²]	f_{uta} [psi]
0.61	58000

Calculations

N_{sa} [lb]
35148

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
35148	0.750	26361	9100

3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-14 Eq. (17.4.3.1)}$$

$$N_p = 8 A_{brg} f'_c \quad \text{ACI 318-14 Eq. (17.4.3.4)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$\psi_{c,p}$	A_{brg} [in. ²]	λ_a	f'_c [psi]
1.000	1.16	1.000	2500

Calculations

N_p [lb]
23260

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{pn} [lb]	N_{ua} [lb]
23260	0.700	0.750	1.000	12211	9100

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3.3 Concrete Breakout Strength

$$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-14 Eq. (17.4.2.1a)}$$

$$\phi N_{cb} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

$$A_{Nc} \text{ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-14 Eq. (17.4.2.1c)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)}$$

$$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} \quad \text{ACI 318-14 Eq. (17.4.2.2a)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	9.000	1.000
c_{ac} [in.]	k_c	λ_a	f'_c [psi]	
-	24	1.000	2500	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
324.00	324.00	1.000	1.000	1.000	1.000	17636

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{cb} [lb]	N_{ua} [lb]
17636	0.700	0.750	1.000	9259	9100

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 E-Mail:

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

5 Warnings

- Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading! Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by Ω_0 .

Fastening meets the design criteria!

Company: CG Engineering
 Specifier:
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 Phone | Fax: |
 E-Mail:

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6 Installation data

Anchor plate, steel: -
 Profile: -
 Hole diameter in the fixture: -
 Plate thickness (input): -
 Recommended plate thickness: -
 Drilling method: Hammer drilled
 Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1
 Installation torque: -
 Hole diameter in the base material: - in.
 Hole depth in the base material: 9.000 in.
 Minimum thickness of the base material: 10.172 in.

Coordinates Anchor in.

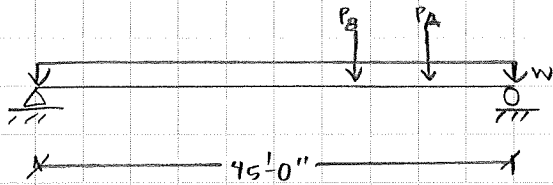
Anchor	x	y	C-x	C+x	C-y	C+y
1	0.000	0.000	9.000	9.000	9.000	9.000

7 Remarks; Your Cooperation Duties

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LATERAL DESIGN

CHECK DIAPHRAGM



$$P_A = P_B = 33.9 \text{ k (ASD EQ)}$$

$$W = 23.5 \text{ k} / 68.5' = 343 \text{ PLF (ASD EQ)}$$

$$V_{\max} = 59.321 \quad (\text{SEE ENERGY CALC})$$

$$\frac{V_{\max}}{d} = \frac{59.321}{101'} = 587 \text{ PLF}$$

- Blocked Diaphragm Required -

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms^{1,2,3,4,5}

					A SEISMIC																		
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)																		
					6			4			2-1/2			2									
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)																		
Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	6			6			4			3									
					v_s (plf)	G_s (kips/in.)	PLY	v_s (plf)	G_s (kips/in.)	PLY	v_s (plf)	G_s (kips/in.)	PLY	v_s (plf)	G_s (kips/in.)	PLY							
Structural I	6d	1-1/4	5/16	2	370	OSB	PLY	12	500	OSB	PLY	8.5	7.5	750	OSB	PLY	12	10	840	OSB	PLY	20	15
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13							
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15							
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13							
Sheathing and Single-Floor	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21							
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18							
	6d	1-1/4	5/16	2	340	15	10	450	9.0	7.0	670	13	9.5	760	21	13							
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12							
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12							
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10							
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13							
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12							
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13							
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12							
	10d	1-1/2	15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13							
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11							
			19/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18							
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16							
					2	640	21	14	850	13	9.5	1280	18	12	1460	28	17						
					3	720	17	12	960	10	8.0	1440	14	11	1640	24	15						



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Description

Lateral Design

Project

The Talmon

By

SRH

Checked

Scale

NTS

Job No.

23154.10

Date

7/13/13

Date

Sheet No.

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

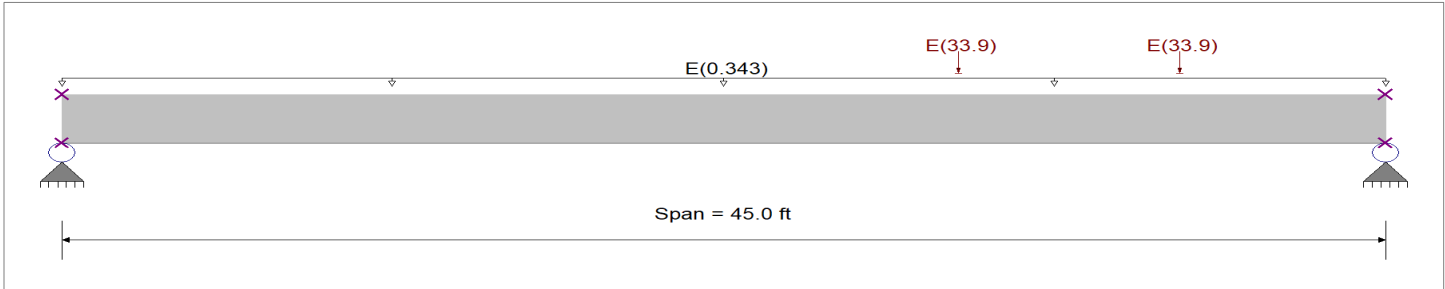
CG ENGINEERING

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DESCRIPTION: Lower Level Diaphragm Analysis

General Beam Properties

Elastic Modulus 29,000.0 ksi
Span #1 Span Length = 45.0 ft Area = 10.0 in² Moment of Inertia = 100.0 in⁴



Applied Loads

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

Uniform Load : E = 0.3430 k/ft, Tributary Width = 1.0 ft

Point Load : E = 33.90 k @ 30.50 ft

Point Load : E = 33.90 k @ 38.0 ft

DESIGN SUMMARY

Maximum Bending =		568.160 k-ft	Maximum Shear =		59.321 k
Load Combination	E Only		Load Combination	E Only	
Span # where maximum occurs	Span # 1		Span # where maximum occurs	Span # 1	
Location of maximum on span	30.375 ft		Location of maximum on span	45.000 ft	
Maximum Deflection					
Max Downward Transient Deflection	61.164 in		8		
Max Upward Transient Deflection	0.850 in		635		
Max Downward Total Deflection	42.815 in		12		
Max Upward Total Deflection	0.446 in		1209		

Vertical Reactions

Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	23.914	59.321
Overall MINimum		
E Only * 0.70	16.740	41.525
E Only * 0.5250	12.555	31.143
E Only	23.914	59.321

General Beam Analysis

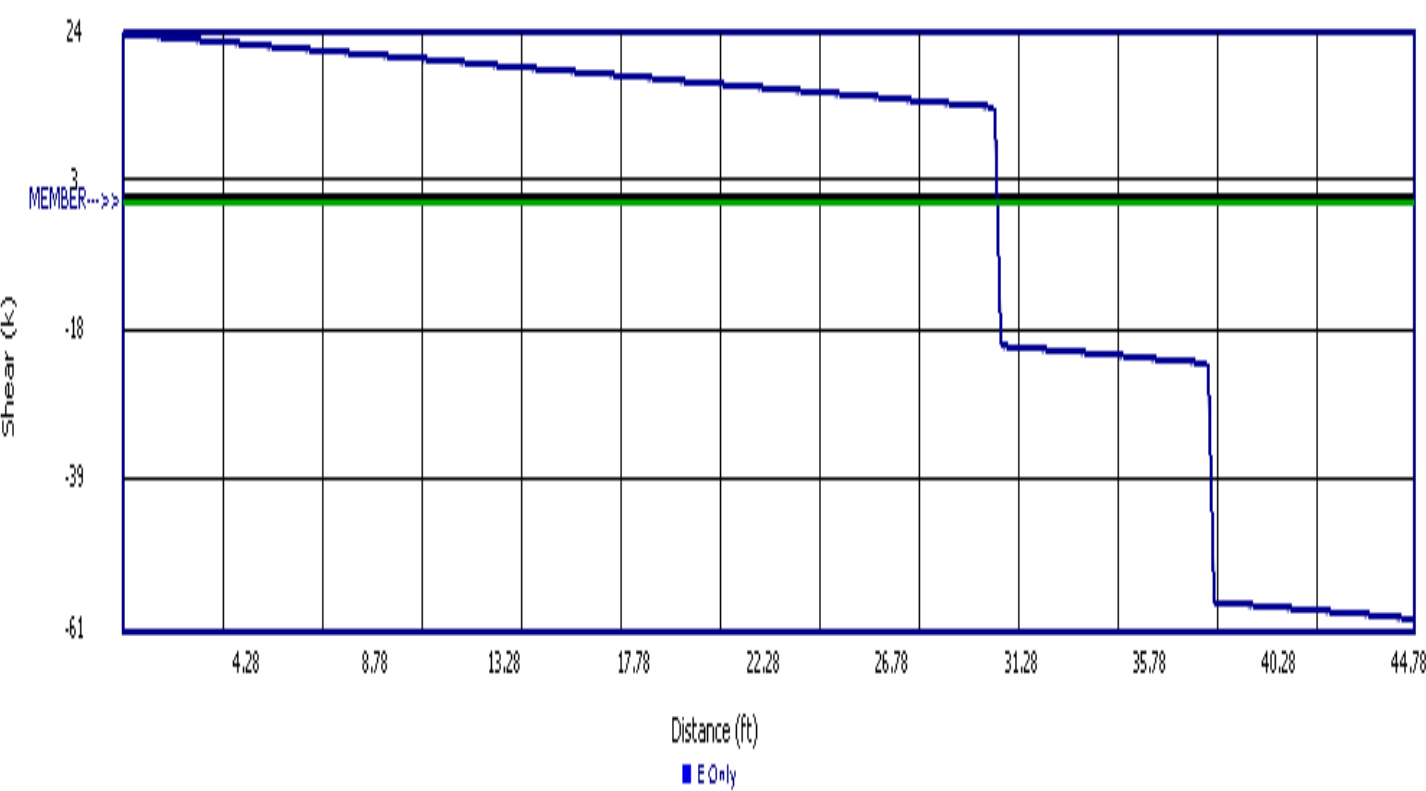
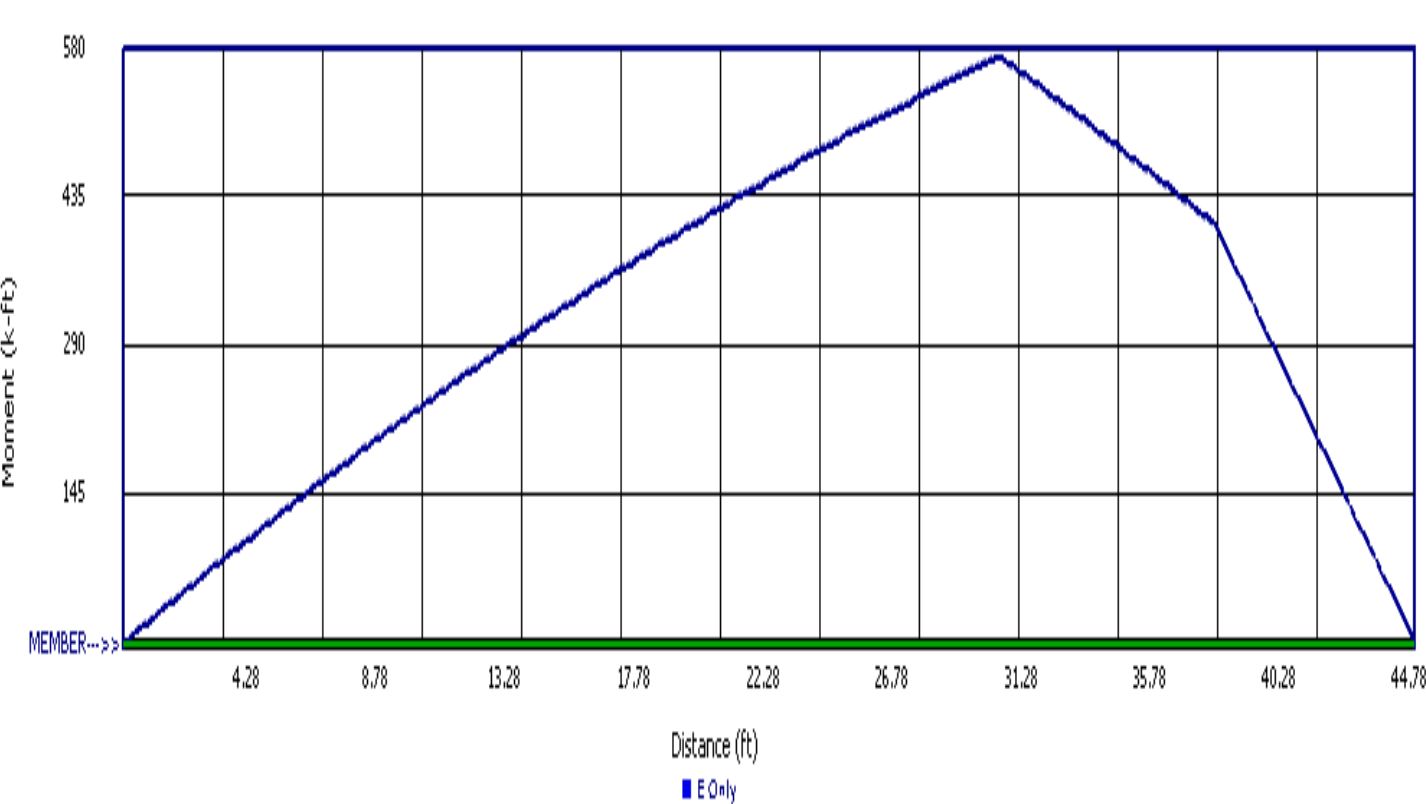
Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

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DESCRIPTION: Lower Level Diaphragm Analysis

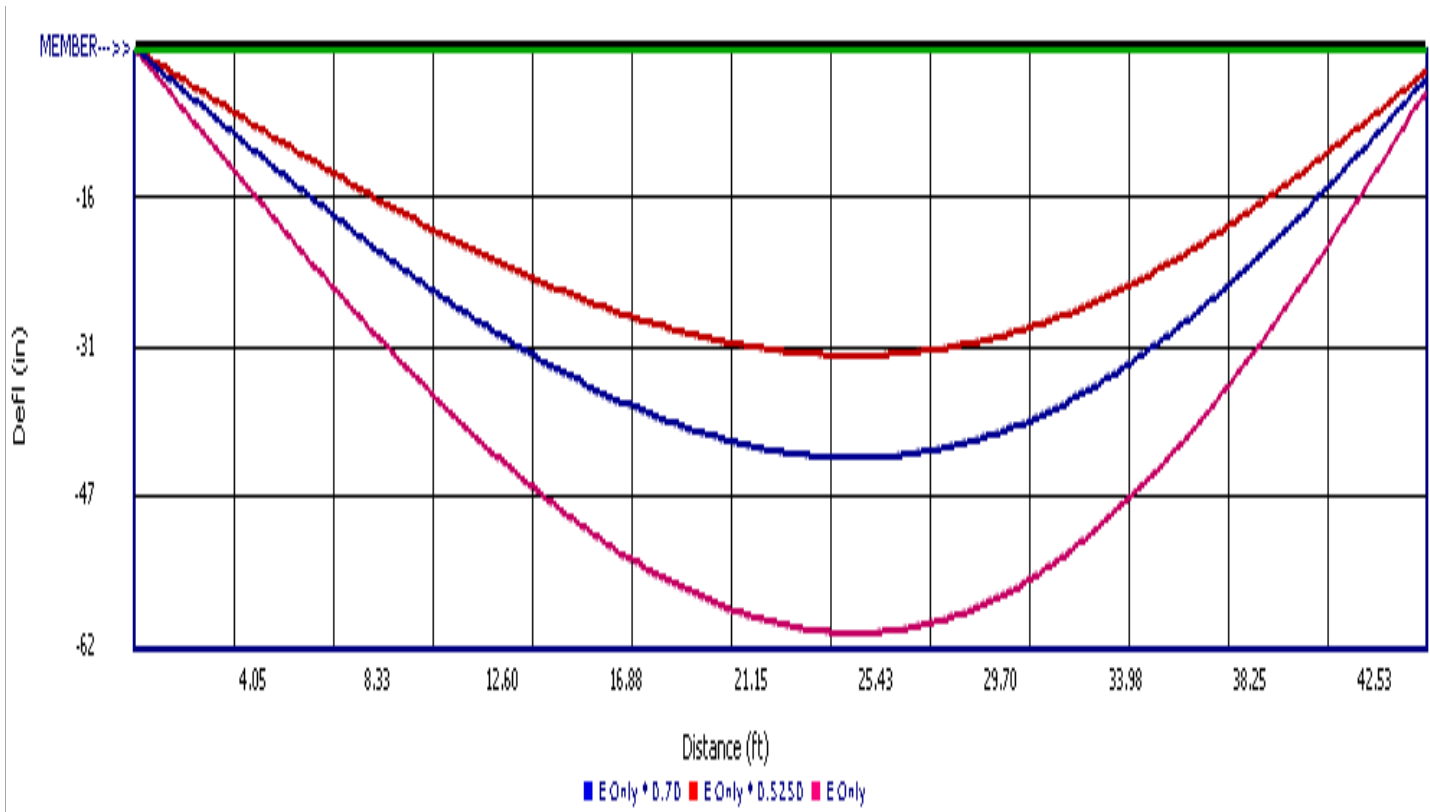


General Beam Analysis

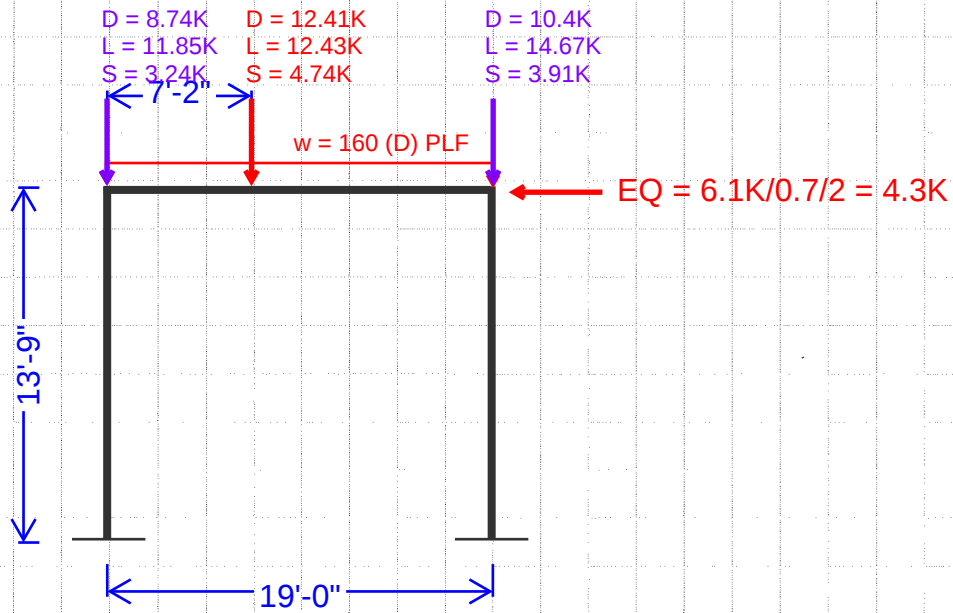
Project File: Talmon.ec6
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LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING

DESCRIPTION: Lower Level Diaphragm Analysis



MOMENT FRAME DESIGN



REFERT TO RAM ELEMENTS MOMENT FRAME MODEL OUTPUT



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www.cgengineering.com

Description

MOMENT FRAME

By ERH

Date 07.07.23

Checked

Date

Scale NTS

Sheet No.

Project

THE TALMON

Job No.

23154.10

Design of Special Moment Frames (Reduced Beam Section)

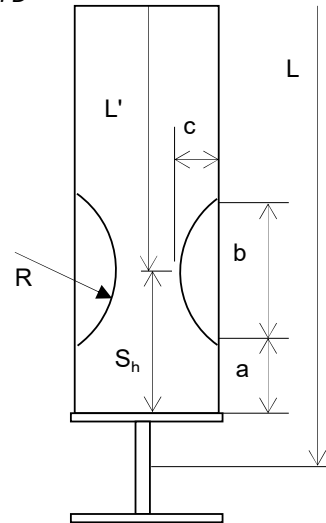
Per AISC Seismic Design Manual 2016 - LRFD

Beam Properties:

Size= W10X68
 L= 228 in (col. CL)
 d= 10.4 in
 t_w= 0.47 in
 b_f= 10.1 in
 t_f= 0.77 in
 Z_x= 85.3 in³
 r_y= 2.59 in
 R_y= 1.1 in²
 F_y= 50 ksi
 F_u= 65 ksi

Column Properties:

Size= W12X136
 H= 165 in
 A_g= 39.9 in²
 d= 13.4 in
 t_w= 0.79 in
 b_f= 12.4 in
 t_f= 1.25 in
 k_{det}= 2.125 in
 Z_x= 214 in³
 r_y= 3.16 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= 15.2 in

a = .6*b_f = 6.1 in (Eq. 5.8-1)
 b = .75*d = 7.8 in (Eq. 5.8-2)
 c = .2*b_f = 2.0 in (Eq. 5.8-3)
 Radius = 4.8 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 = 6.6 ≤ 7.3 **OKAY**
 h/t = 18.9 ≤ 36.1 **OKAY**

Column

b/t = 5.0 ≤ 7.3 **OKAY**
 h/t = 13.8 ≤ 36.1 **OKAY**

Compute Probable Maximum Moment @ Center of RBS

Z_{RBS} = Z_x - 2ct_f(d - t_f) = 55.3 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} = 3500 kip-in (AISC 358 Eq. 5.8-5)



250 4th Ave South
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 Edmonds, WA 98020

Description

Moment Frame Calculation

Project

The Talmon

By

LS

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Scale

Job No.

22332.10

Date 7/17/2023

Date

Sheet No.

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2016 - LRFD

Compute Shear Force @ Center of RBS

$$\begin{aligned}
 S_h &= a + b/2 = & 10.0 \text{ in} \\
 V_{gravity} &= & 21.4 \text{ kips (See RAM model LC: 1.2D + 1.6L)} \\
 V_{pr} &= 2 * M_{pr} / L_h = & 36.0 \text{ kips} \\
 V_{RBS} &= 1.2V_{DL} + 1.0V_{LL} + V_{pr} = & 57.4 \text{ kips}
 \end{aligned}$$

Compute Maximum Moment @ Face of Column

$$M_f = M_{pr} + V_{RBS} S_h = 4071.8 \text{ kip-in} \quad (\text{AISC 358 Eq. 5.8-6})$$

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

$$\begin{aligned}
 M_{pe} &= R_y F_y Z_x = 4691.5 \text{ kip-in} & (\text{AISC 358 Eq. 5.8-7}) \\
 M_f &\leq \phi_d M_{pe} & \text{OKAY (for } \phi_d = 1.0) & (\text{AISC 358 Eq. 5.8-8})
 \end{aligned}$$

Check Column-Beam Relationship Limitations

$$\begin{aligned}
 M_{uv} &= V_{RBS} * (a + b/2 + d_c/2) = 956 \text{ kip-in} \\
 \Sigma M^*_{pb} &= \Sigma (M_{pr} + M_{uv}) = 4456 \text{ kip-in} & (\text{Seismic Provisions Eq. E3-3a}) \\
 & & (\text{assuming single beam-column})
 \end{aligned}$$

$$\begin{aligned}
 P_{uc} &= 46 \text{ kips (See RAM model LC: 1.2D + 1.0E + 1.0L)} \\
 \Sigma M^*_{pc} &= \Sigma Z_{xc} (F_{yc} - P_{uc} / A_g) = 10453 \text{ kip-in} & (\text{Seismic Provisions Eq. E3-2a}) \\
 & & (\text{assuming single beam-column})
 \end{aligned}$$

$$\Sigma M^*_{pc} / \Sigma M^*_{pb} \geq 1.0 = 2.3 \text{ OKAY}$$

Check Shear Strength of the Beam

$$\begin{aligned}
 L_h &= 194.7 \text{ in (distance between plastic hinges)} \\
 V_{gravity} &= 21.4 \text{ kips (See RAM model LC: 1.2D + 1.0L + 0.7S)} \\
 E_{mh} &= 2 * 1.1 * M_{pr} / L_h = 39.6 \text{ kips} & (\text{AISC 341 Eq. E3-6}) \\
 V_u &= 2 * 1.1 * M_{pr} / L_h + V_{gravity} = 61.0 \text{ kips} & (\text{AISC 358 Eq. 5.8-9}) \\
 \phi V_{n(web)} &= \phi * 0.6 * F_y * A_w * C_v = 102.0 \text{ kips} & \text{OKAY (per AISC Sec G Eq. G2-1)}
 \end{aligned}$$

*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 Moment Frame Calculation	By LS	Date 7/17/2023
		Checked	Date
	Project The Talmon	Scale	Sheet No.
		Job No. 22332.10	

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2016 - LRFD

Check Column Panel Zone Shear Strength

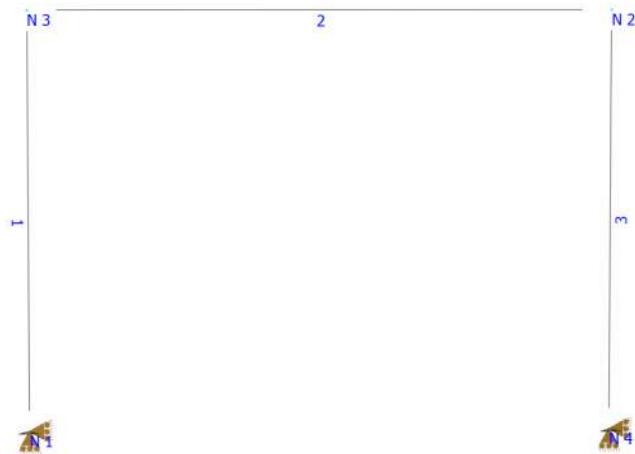
$$\begin{aligned}
 R_u &= \Sigma M_f / (d - t_f) = & 423 \text{ kips} \\
 .75 * F_y * A_g &= & 1496 \text{ kips} \text{ -----> USE J10-11} \\
 R_u \leq \phi R_n &= & 448 \text{ kips} \quad \text{OKAY! DOUBLER PLATE NOT REQ'D}
 \end{aligned}$$

Check Continuity Plate Requirements

$$\begin{aligned}
 t_{cf} &\geq .4 \sqrt{1.8 b_{bf} t_{bf} F_{yb} R_{yb} / (F_{yc} * R_{yc})} = & 1.5 \text{ REQUIRED} \\
 t_{cf} &\geq b_{bf} / 6 = & 1.7 \text{ REQUIRED}
 \end{aligned}$$

If required, continuity plates shall match thickness of beam flanges

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		Checked	Date
		Scale	Sheet No.
	Project The Talmon	Job No. 22332.10	







Current Date: 7/14/2023 4:54 PM
Units system: English
File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx
Load condition: DL=Dead Load

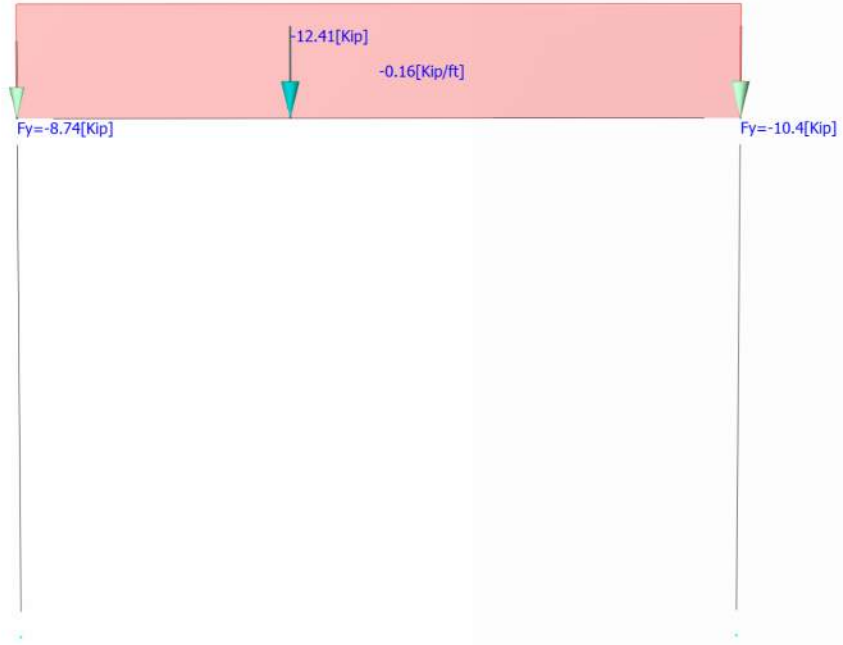
Loads

Axial force

Distributed user loads - Members

Concentrated user loads - Members

Concentrated - Nodes





Current Date: 7/14/2023 4:54 PM

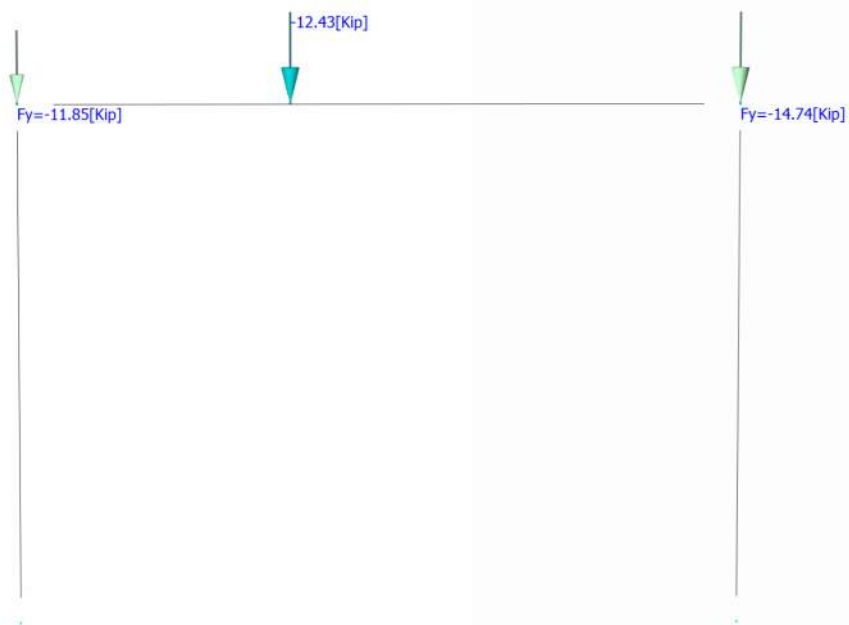
Units system: English

File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx

Load condition: L=Live Load

Loads

- Axial force
- Concentrated user loads - Members
- Concentrated - Nodes





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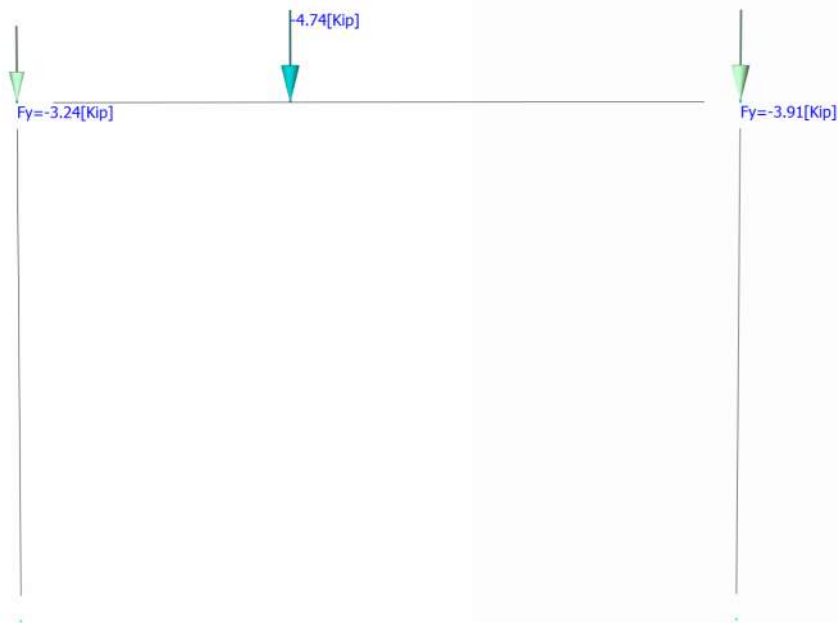
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File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx

Load condition: S=Snow Load

Loads

- Axial force
- Concentrated user loads - Members
- Concentrated - Nodes





Current Date: 7/21/2023 11:17 AM

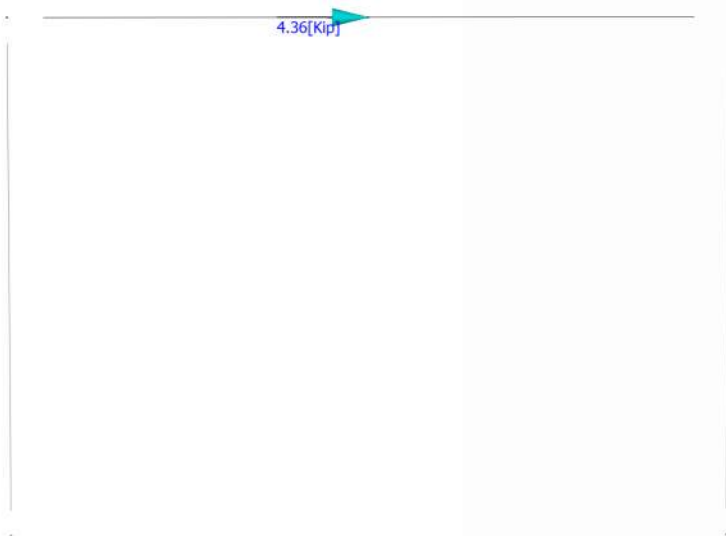
Units system: English

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Load condition: Eq=Seismic Load

Loads

- Axial force
- Concentrated user loads - Members





Current Date: 7/17/2023 9:46 AM

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File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

LC1=1.4DL
LC2=1.2DL+1.6L+0.5S
LC3=1.2DL+L+1.6S
LC4=1.2DL+L+0.5S+W
LC5=1.392DL+L+0.7S+Eq
LC6=0.9DL+W
LC7=0.708DL+Eq
Vu=1.2DL+L+0.7S
omeg1=1.392DL+L+0.7S+2.5Eq
omeg2=0.708DL+2.5Eq
LC5_neg=1.392DL+L+0.7S-Eq
LC7_neg=0.708DL-Eq
omeg1_neg=1.392DL+L+0.7S+2.5Eq
omeg2_neg=0.708DL+2.5Eq

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam	W 10X68	2	omeg1 at 0.00%	0.46	OK	
Column	W 12X136	1	LC5_neg at 100.00%	0.14	OK	
		3	omeg1 at 0.00%	0.20	OK	



Current Date: 7/21/2023 11:19 AM

Units system: English

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Analysis result

Nodal displacements envelope

Note.- **Ic** is the controlling load condition

Nodal displacements envelope for :

LC1=1.4DL

LC2=1.2DL+1.6L+0.5S

LC3=1.2DL+L+1.6S

LC4=1.2DL+L+0.5S+W

LC5=1.392DL+L+0.7S+Eq

LC6=0.9DL+W

LC7=0.708DL+Eq

Vu=1.2DL+L+0.7S

drift1=1.192DL+Eq

drift2=1.192DL+0.75L+0.75S+Eq

drift3=0.408DL+Eq

LC5_neg=1.392DL+L+0.7S-Eq

LC7_neg=0.708DL-Eq

MAXIMUM DEFLECTION < 0.6" --> OK

Node		Translation						Rotation					
		X	Ic	Y	Ic	Z	Ic	Rx	Ic	Ry	Ic	Rz	Ic
		[in]		[in]		[in]		[Rad]		[Rad]		[Rad]	
1	Max	0.000	LC1	0.000	LC1	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00205	LC7_neg
	Min	0.000	LC1	0.000	LC1	0.000	LC1	0.00000	LC1	0.00000	LC1	-0.00224	LC5
2	Max	0.425	LC5	-0.001	LC7_neg	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00185	LC5_neg
	Min	-0.269	LC7_neg	-0.008	LC2	0.000	LC1	0.00000	LC1	0.00000	LC1	-0.00116	drift3
3	Max	0.427	LC5	-0.001	drift3	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00068	LC7_neg
	Min	-0.268	LC7_neg	-0.008	LC2	0.000	LC1	0.00000	LC1	0.00000	LC1	-0.00336	LC5
4	Max	0.000	LC1	0.000	LC1	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00171	LC7_neg
	Min	0.000	LC1	0.000	LC1	0.000	LC1	0.00000	LC1	0.00000	LC1	-0.00344	LC5



Current Date: 7/17/2023 10:01 AM

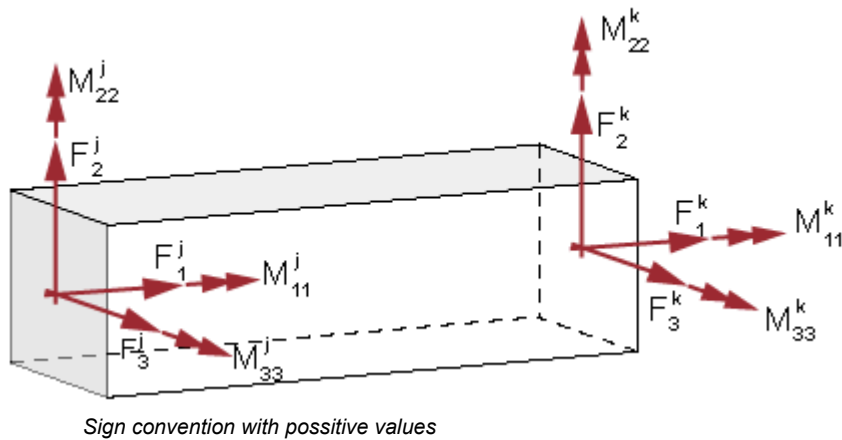
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File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx

Analysis result

Forces at member ends

Notes.- F1: Axial forces
F2: Shear force in 2
F3: Shear force in 3
M11: Torsional moment
M22: Bending moments 2
M33: Bending moments 3



CONDITION: $V_u = 1.2DL + L + 0.7S$

Member	End	F1 [Kip]	F2 [Kip]	F3 [Kip]	M11 [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
2	NJ: 2	4.74205	3.86142	0.00000	0.00000	0.00000	65.20325
2	NK: 3	-4.74205	21.35607	0.00000	0.00000	0.00000	-65.20325

MAX SHEAR @ CENTER OF RBS



Current Date: 7/17/2023 10:04 AM

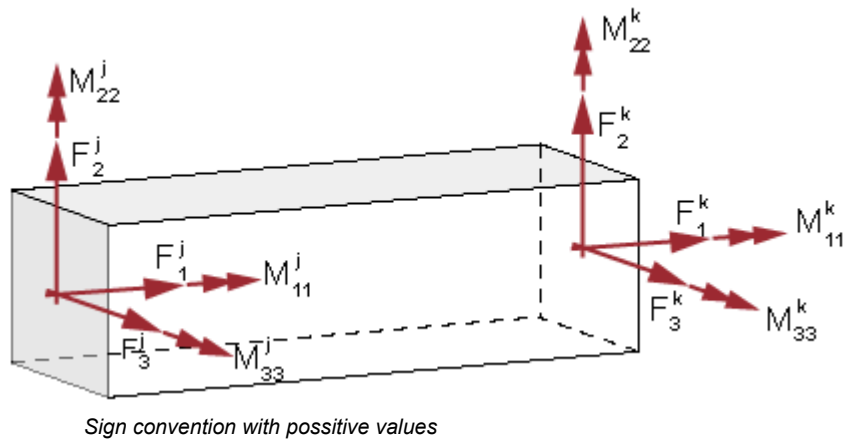
Units system: English

File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx

Analysis result

Forces at member ends

Notes.- F1: Axial forces
F2: Shear force in 2
F3: Shear force in 3
M11: Torsional moment
M22: Bending moments 2
M33: Bending moments 3



CONDITION: $V_u = 1.2DL + L + 0.7S$

Member	End	F1 [Kip]	F2 [Kip]	F3 [Kip]	M11 [Kip*ft]	M22 [Kip*ft]	M33 [Kip*ft]
1	NJ: 1	47.26070	-4.74205	0.00000	0.00000	0.00000	0.00000
1	NK: 3	-45.96207	4.74205	0.00000	0.00000	0.00000	-65.20325
3	NJ: 2	43.78842	4.74205	0.00000	0.00000	0.00000	65.20325
3	NK: 4	-45.08706	-4.74205	0.00000	0.00000	0.00000	0.00000

MAX COLUMN COMPRESSION



Current Date: 7/21/2023 11:22 AM

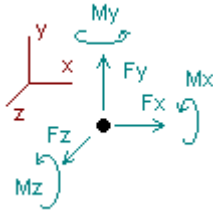
Units system: English

File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\RAM\Moment Frame.retx

Analysis result

Envelope for nodal reactions

Note.- I_c is the controlling load condition



Direction of positive forces and moments

Envelope of nodal reactions for :

LC1=1.4DL

LC2=1.2DL+1.6L+0.5S

LC3=1.2DL+L+1.6S

LC4=1.2DL+L+0.5S+W

LC5=1.392DL+L+0.7S+Eq

LC6=0.9DL+W

LC7=0.708DL+Eq

Vu=1.2DL+L+0.7S

omeg1=1.392DL+L+0.7S+2.5Eq

omeg2=0.708DL+2.5Eq

drift1=1.192DL+Eq

drift2=1.192DL+0.75L+0.75S+Eq

drift3=0.408DL+Eq

LC5_neg=1.392DL+L+0.7S-Eq

LC7_neg=0.708DL-Eq

omeg1_neg=1.392DL+L+0.7S+2.5Eq

omeg2_neg=0.708DL+2.5Eq

Node		Forces						Moments					
		Fx	Ic	Fy	Ic	Fz	Ic	Mx	Ic	My	Ic	Mz	Ic
		[Kip]		[Kip]		[Kip]		[Kip*ft]		[Kip*ft]		[Kip*ft]	
1	Max	7.314	LC5_neg	57.278	LC2	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00000	LC1
	Min	-4.006	omeg2	4.184	drift3	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00000	LC1
4	Max	0.736	LC7_neg	56.163	omeg1	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00000	LC1
	Min	-10.584	omeg1	8.598	LC7_neg	0.000	LC1	0.00000	LC1	0.00000	LC1	0.00000	LC1

MAX SHEAR

NO UPLIFT

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

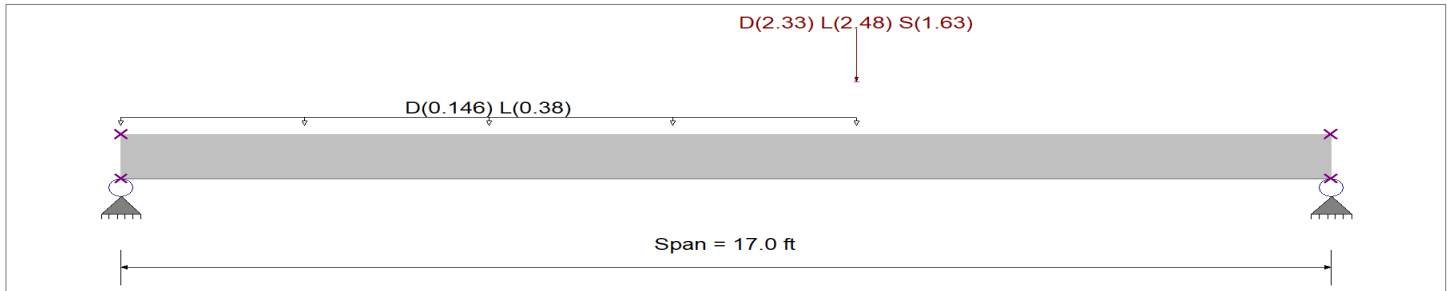
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DESCRIPTION: BM_L (RXN for Beam Left of Moment Frame)

General Beam Properties

Elastic Modulus 29,000.0 ksi
 Span #1 Span Length = 17.0 ft Area = 10.0 in² Moment of Inertia = 100.0 in⁴



Applied Loads

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

Load for Span Number 1

Uniform Load : D = 0.1460, L = 0.380 k/ft, Extent = 0.0 --> 10.333 ft, Tributary Width = 1.0 ft

Point Load : D = 2.330, L = 2.480, S = 1.630 k @ 10.333 ft

DESIGN SUMMARY

Maximum Bending =	30.920 k-ft	Maximum Shear =	5.670 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	10.285 ft	Location of maximum on span	0.000 ft
Maximum Deflection			
Max Downward Transient Deflection	0.309 in		661
Max Upward Transient Deflection	0.001 in		154588
Max Downward Total Deflection	0.506 in		402
Max Upward Total Deflection	0.002 in		114468

Vertical Reactions

Support notation : Far left is #'

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	5.670	4.643
Overall MINimum		
D Only	1.964	1.875
+D+L	5.670	4.575
+D+S	2.603	2.865
+D+0.750L	4.743	3.900
+D+0.750L+0.750S	5.223	4.643
+0.60D	1.178	1.125
L Only	3.706	2.701
S Only	0.639	0.991

Project Title:
 Engineer:
 Project ID:
 Project Descr:

General Beam Analysis

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25

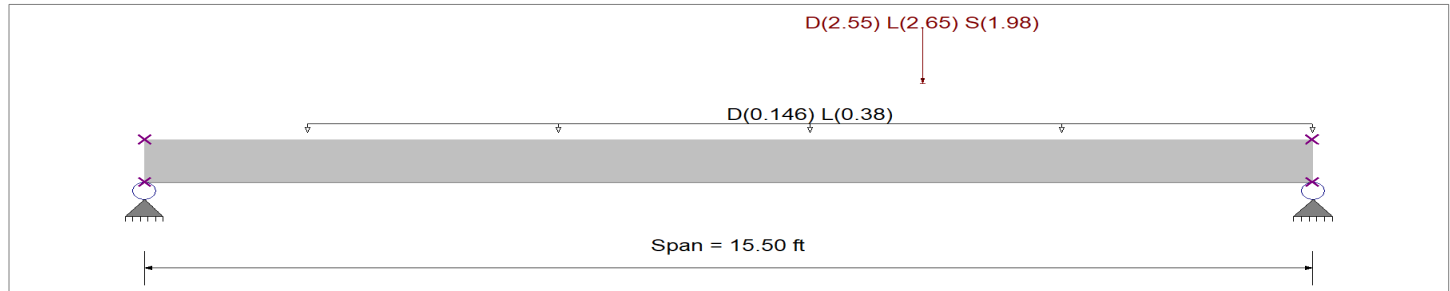
CG ENGINEERING

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DESCRIPTION: BM_R (RXN for Beam Right of Moment Frame)

General Beam Properties

Elastic Modulus 29,000.0 ksi
 Span #1 Span Length = 15.50 ft Area = 10.0 in² Moment of Inertia = 100.0 in⁴



Applied Loads

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

Load for Span Number 1

Uniform Load : D = 0.1460, L = 0.380 k/ft, Extent = 2.160 --> 15.50 ft, Tributary Width = 1.0 ft

Point Load : D = 2.550, L = 2.650, S = 1.980 k @ 10.333 ft

DESIGN SUMMARY

Maximum Bending =	31.891 k-ft	Maximum Shear =	7.464 k
Load Combination	+D+0.750L+0.750S	Load Combination	+D+L
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1
Location of maximum on span	10.308 ft	Location of maximum on span	15.500 ft
Maximum Deflection			
Max Downward Transient Deflection	0.269 in		690
Max Upward Transient Deflection	0.001 in		171451
Max Downward Total Deflection	0.434 in		428
Max Upward Total Deflection	0.001 in		130638

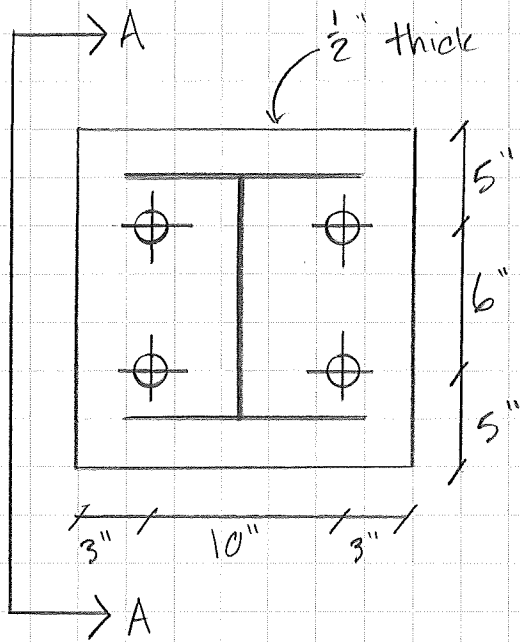
Vertical Reactions

Support notation : Far left is #'

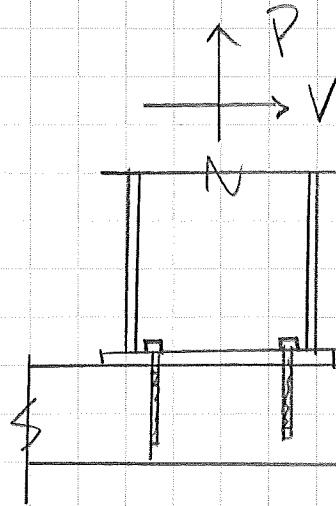
Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.753	7.464
Overall MINimum		
D Only	1.688	2.809
+D+L	4.753	7.464
+D+S	2.348	4.129
+D+0.750L	3.987	6.300
+D+0.750L+0.750S	4.482	7.290
+0.60D	1.013	1.686
L Only	3.065	4.654
S Only	0.660	1.320

Base Plate Design + Anchorage



(RODS CONSERVATIVELY
CHECKED WITH 10K OF UPLIFT)



$$P_{max} = 0 \text{ lbs} \quad (\text{max uplift w/ } \Omega)$$

$$V_{max} = 10584 \text{ lbs} \quad (\text{max shear})$$

Per HILTI, use (4) $\frac{3}{4}$ " ϕ
THREADED RODS W/ 5" MIN EMBED

Provide 6' x 6' sq. ft.



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

Description

Lateral Design

Project

THE TALMON

By ERH

Checked

Scale

Job No.

23154.10

Date 07.17.23

Date

Sheet No.



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 E-Mail:

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 Date: 7/21/2023

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Specifier's comments:

1 Input data

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 3/4

Effective embedment depth: $h_{ef} = 5.000$ in.

Material: ASTM F 1554

Proof: Design method ACI 318-14 / CIP

Stand-off installation: $e_b = 0.000$ in. (no stand-off); $t = 0.500$ in.

Anchor plate: $l_x \times l_y \times t = 16.000$ in. x 16.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)

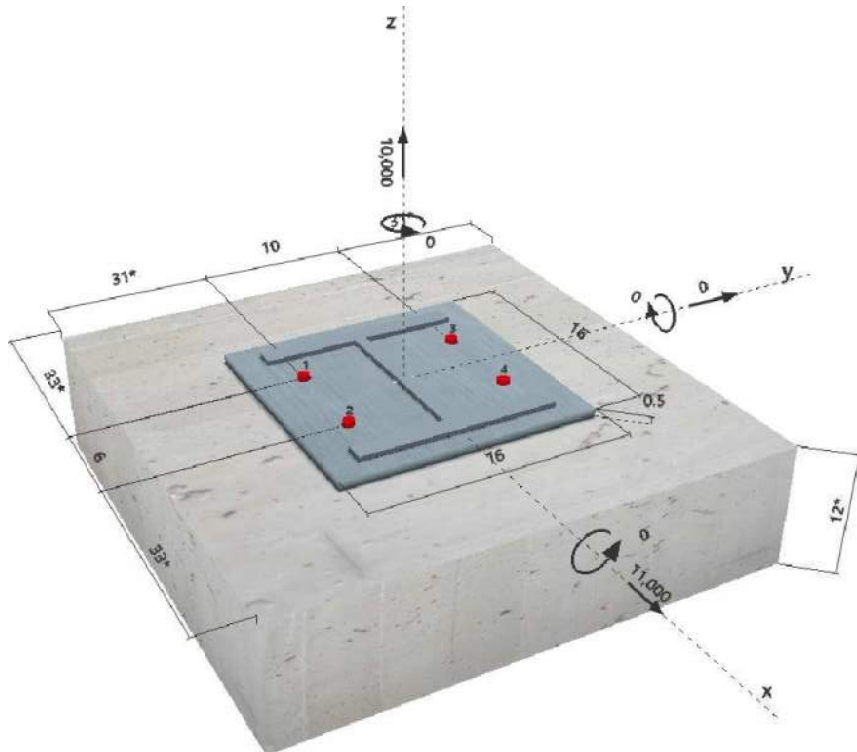
Profile: W shape (AISC); (L x W x T x FT) = 13.400 in. x 12.400 in. x 0.790 in. x 1.250 in.

Base material: cracked concrete, 2500, $f'_c = 2500$ psi; $h = 12.000$ in.

Reinforcement: tension: condition B, shear: condition B;
 edge reinforcement: none or < 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]



Input data and results must be checked for agreement with the existing conditions and for plausibility!
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2 Load case/Resulting anchor forces

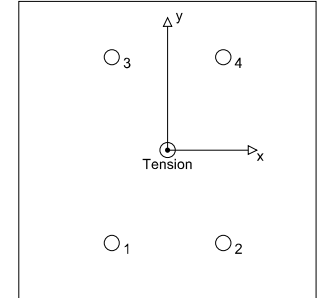
Load case: Design loads

Anchor reactions [lb]

Tension force: (+ Tension, - Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2500	2750	2750	0
2	2500	2750	2750	0
3	2500	2750	2750	0
4	2500	2750	2750	0

max. concrete compressive strain: - [%]
 max. concrete compressive stress: - [psi]
 resulting tension force in (x/y)=(0.000/0.000): 10000 [lb]
 resulting compression force in (x/y)=(0.000/0.000): 0 [lb]



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	2500	14529	18	OK
Pullout Strength*	2500	6867	37	OK
Concrete Breakout Strength**	10000	16435	61	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading ** anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
 $\phi N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in. ²]	f_{uta} [psi]
0.33	58000

Calculations

N_{sa} [lb]
19372

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
19372	0.750	14529	2500

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
 $N_p = 8 A_{org} f'_c$ ACI 318-14 Eq. (17.4.3.4)
 $\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{org} [in. ²]	λ_a	f'_c [psi]
1.000	0.65	1.000	2500

Calculations

N_p [lb]
13080

Results

N_{pN} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{ductile}$	ϕN_{pN} [lb]	N_{ua} [lb]
13080	0.700	0.750	1.000	6867	2500

Input data and results must be checked for agreement with the existing conditions and for plausibility!
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E-Mail:			

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3.3 Concrete Breakout Strength

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-14 Eq. (17.4.2.1b)}$$

$$\phi N_{cbg} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

$$A_{Nc} \text{ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-14 Eq. (17.4.2.1c)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)}$$

$$N_b = k_c \lambda_a \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-14 Eq. (17.4.2.2a)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
5.000	0.000	0.000	31.000	1.000
c_{ac} [in.]	k_c	λ_a	f_c [psi]	
-	24	1.000	2500	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
525.00	225.00	1.000	1.000	1.000	1.000	13416

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕN_{cbg} [lb]	N_{ua} [lb]
31305	0.700	0.750	1.000	16435	10000



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4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	2750	7555	37	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	11000	43827	26	OK
Concrete edge failure in direction x+**	11000	21049	53	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-14 Eq. (17.5.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.33	58000

Calculations

V_{sa} [lb]
11623

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
11623	0.650	7555	2750

4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right] \quad \text{ACI 318-14 Eq. (17.5.3.1b)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

$$A_{Nc} \text{ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-14 Eq. (17.4.2.1c)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)}$$

$$N_b = k_c \lambda_a \sqrt{f_c} h_{ef}^{1.5} \quad \text{ACI 318-14 Eq. (17.4.2.2a)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	5.000	0.000	0.000	31.000
$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c [psi]
1.000	-	24	1.000	2500

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
525.00	225.00	1.000	1.000	1.000	1.000	13416

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cp} [lb]	V_{ua} [lb]
62610	0.700	1.000	1.000	43827	11000

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4.3 Concrete edge failure in direction x+

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}} \right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b \quad \text{ACI 318-14 Eq. (17.5.2.1b)}$$

$\phi V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

$$A_{vc0} = 4.5 c_{a1}^2 \quad \text{ACI 318-14 Eq. (17.5.2.1c)}$$

$$\psi_{ec,v} = \left(\frac{1}{1 + \frac{2e_v}{3c_{a1}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.5.2.5)}$$

$$\psi_{ed,V} = 0.7 + 0.3 \left(\frac{c_{a2}}{1.5c_{a1}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.5.2.6b)}$$

$$\psi_{h,v} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0 \quad \text{ACI 318-14 Eq. (17.5.2.8)}$$

$$V_b = \left(7 \left(\frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda_a \sqrt{f_c} c_{a1}^{1.5} \quad \text{ACI 318-14 Eq. (17.5.2.2a)}$$

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
20.667	31.000	0.000	1.000	12.000

l_0 [in.]	λ_a	d_a [in.]	f_c [psi]	$\psi_{parallel,V}$
5.000	1.000	0.750	2500	1.000

Calculations

A_{Vc} [in. ²]	A_{Vc0} [in. ²]	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{b,V}$	V_b [lb]
864.00	1922.00	1.000	1.000	1.607	41618

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cbg} [lb]	V_{ua} [lb]
30070	0.700	1.000	1.000	21049	11000

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.608	0.523	5/3	78	OK

$$\beta_{NV} = \beta_N^c + \beta_V^c \leq 1$$

6 Warnings

- Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading! Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength, Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by Ω_0 .

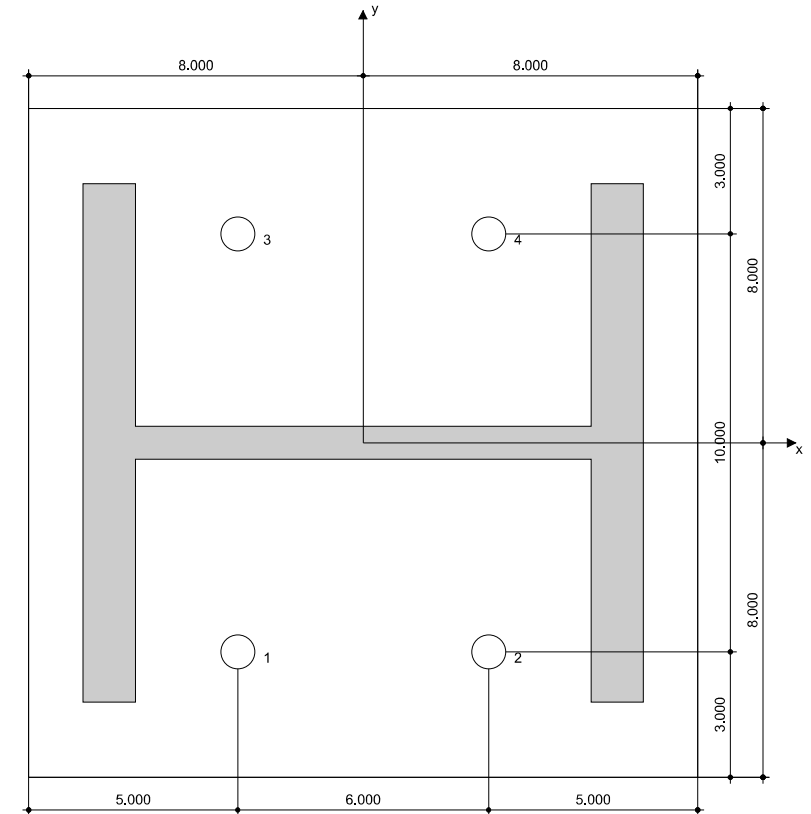
Fastening meets the design criteria!

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7 Installation data

Anchor plate, steel: -
 Profile: W shape (AISC); 13,400 x 12,400 x 0.790 x 1,250 in.
 Hole diameter in the fixture: $d_f = 0.813$ in.
 Plate thickness (input): 0.500 in.
 Recommended plate thickness: not calculated
 Drilling method: Hammer drilled
 Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 3/4
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 5.000 in.
Minimum thickness of the base material: 6.000 in.



Coordinates Anchor in.

Anchor	x	y	C_{-x}	C_{+x}	C_{-y}	C_{+y}
1	-3,000	-5,000	33,000	39,000	31,000	41,000
2	3,000	-5,000	39,000	33,000	31,000	41,000
3	-3,000	5,000	33,000	39,000	41,000	31,000
4	3,000	5,000	39,000	33,000	41,000	31,000



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8 Remarks; Your Cooperation Duties

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