C.G ENGINEERING

civil & structural
engineering & planning

Town of
La Conner
Planning Department
Received
10-17-23

STRUCTURAL CALCULATIONS

The Talmon

306 Centre Street
La Conner, WA 98257

JOSEPH G. GALUSHA
STATE OF WASHINGTON
39553
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Joe Galusha
7/21/23

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

GENERAL INDEX

The Talmon
Centre Street
La Conner, WA 98257

SECTION NO. 1

The Talmon
Centre Street
La Conner, WA 98257

SECTION TITLE: Main Building Gravity Design

SECTION INDEX

250 4th Ave S Edmonds, WA 98020
p. 425.778.8500 f. 425.778.5536

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

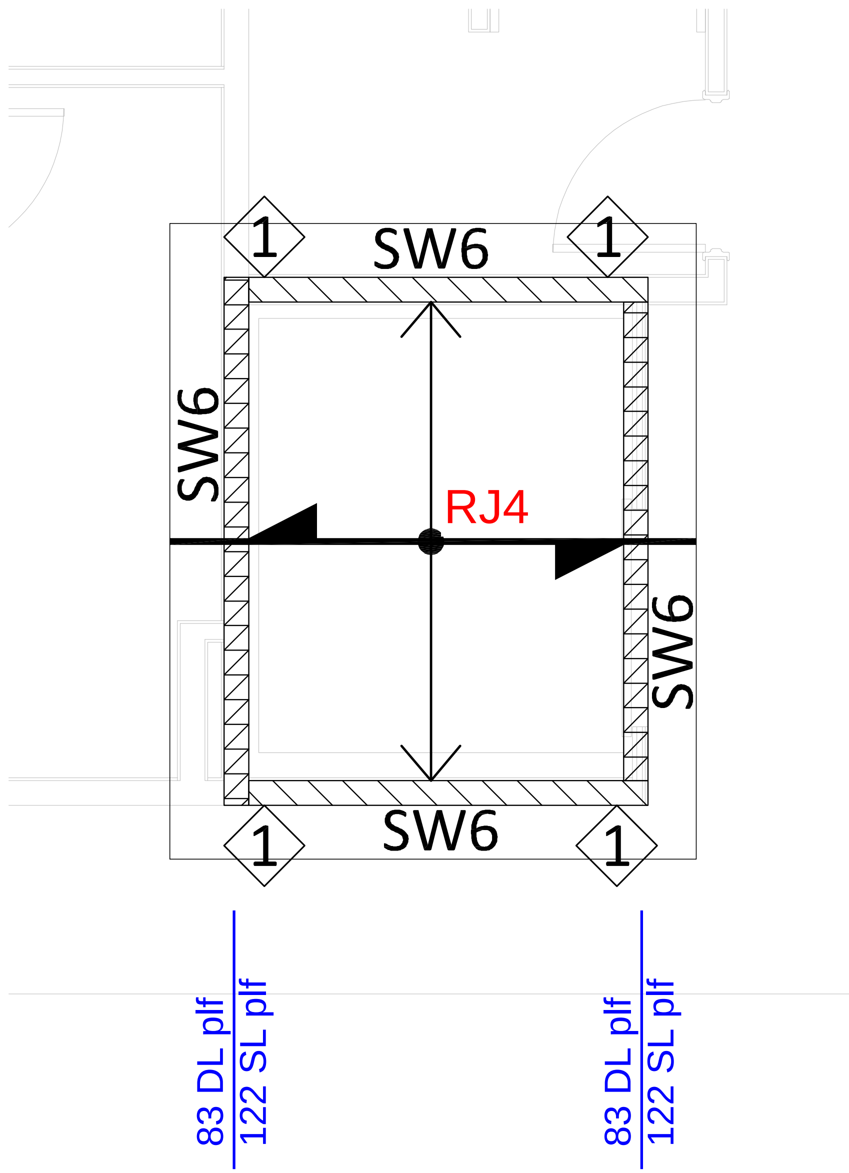
Roof LL (Snow)	25.0	psf

Floor LL	40.0	psf
Deck LL	60.0	psf

ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description: Gravity Design Loads	By: CCC	Date: 07/21/23		
		Checked:	Date:		
		Scale:	Sheet No.		
	Project: The Talmon	Job No.: 23154.10			

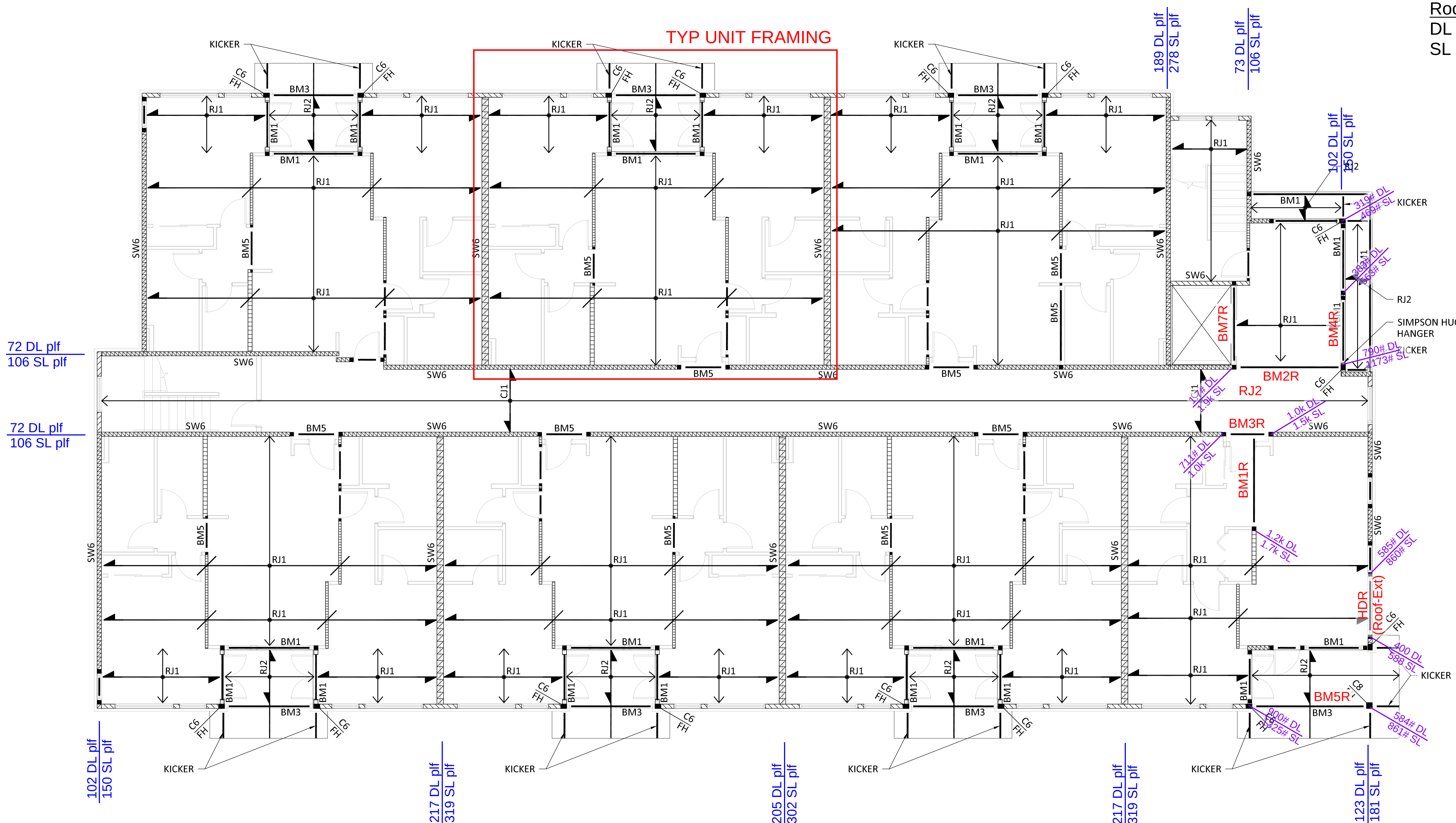
UPPER ROOF FRAMING KEY PLAN

Roof Loading:
DL = 17 PSF
SL = 25 PSF



ROOF FRAMING KEY PLAN

Roof Loading:
DL = 17 PSF
SL = 25 PSF

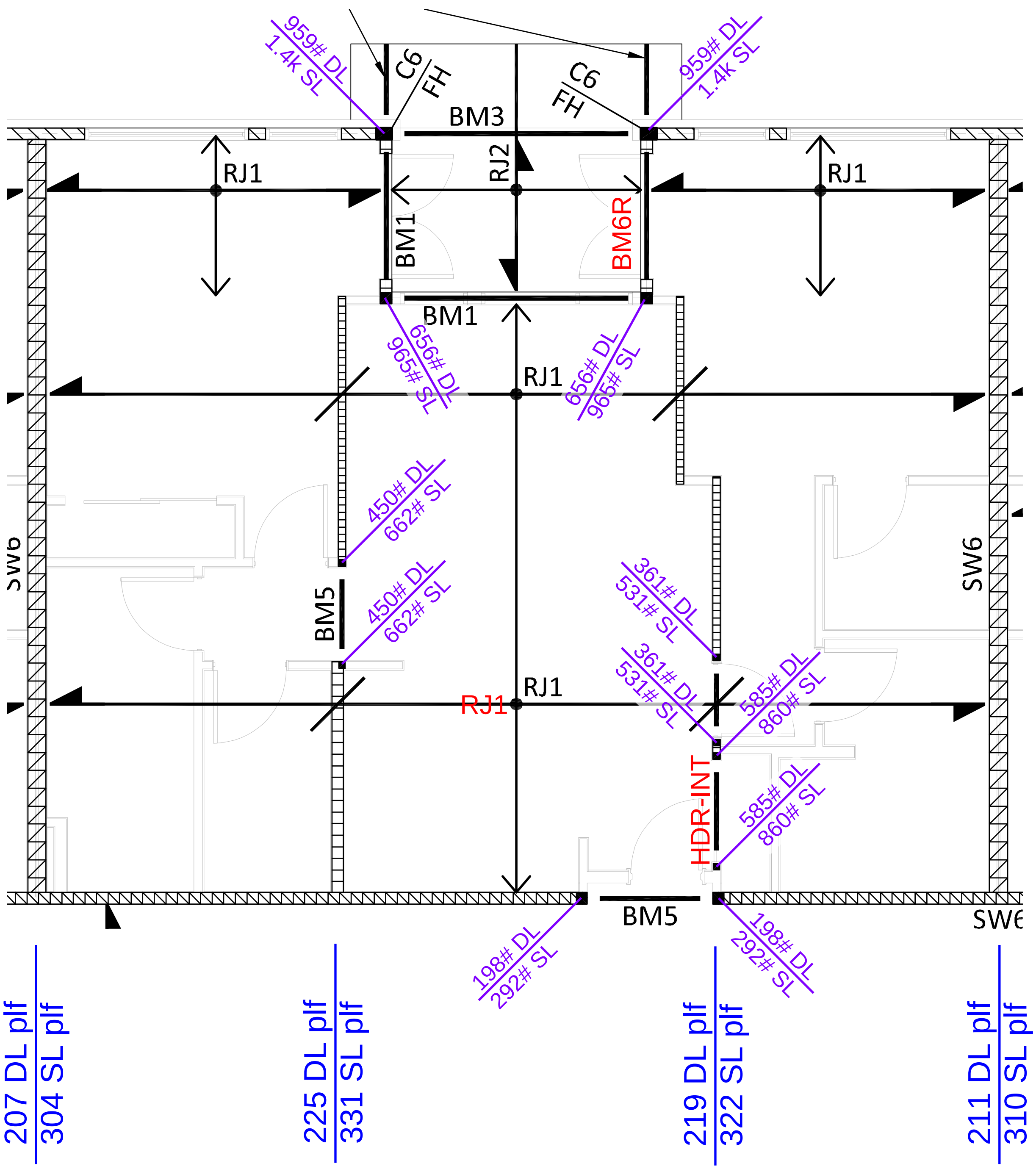


*Non-typical framing indicated on this overall key plan

1 ROOF FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"

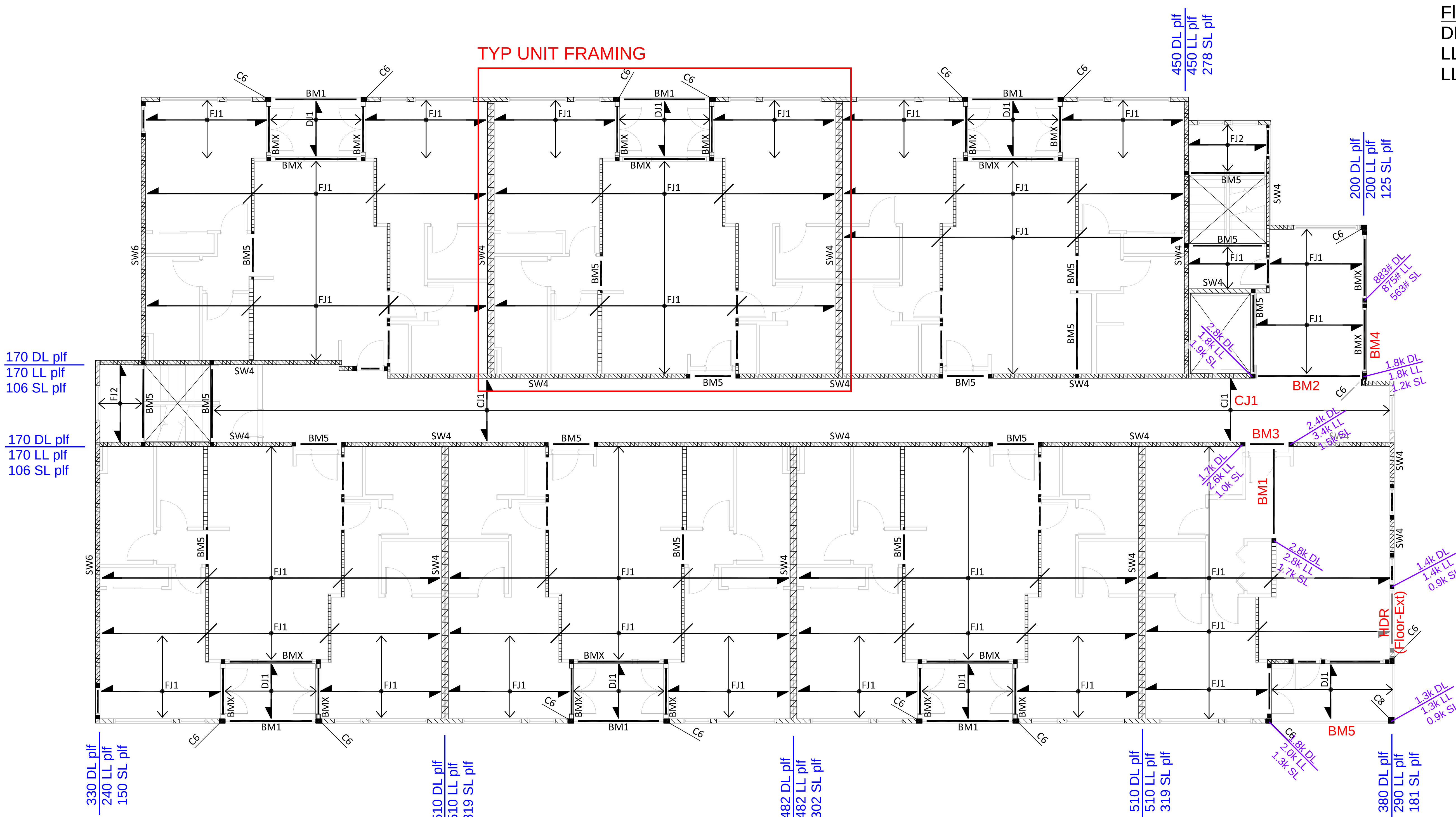
5'-0" 0 5'-0" 10'-0"

TYP UNIT FRAMING KEY PLAN
(ROOF)



THIRD FLOOR FRAMING KEY PLAN

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



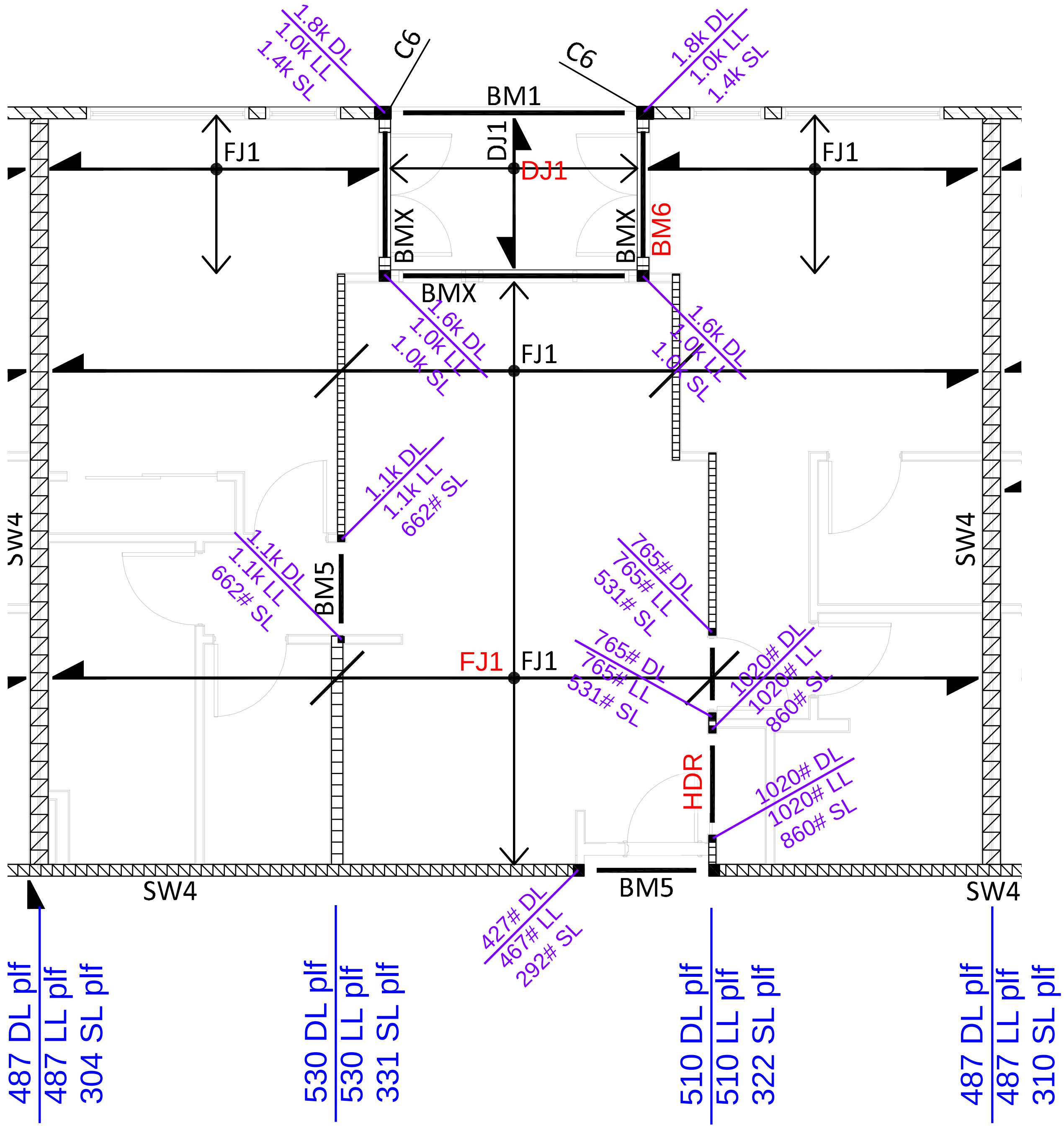
*Non-typical framing indicated on this overall key plan

1 THIRD FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"

5'-0" 0 5'-0" 10'-0"

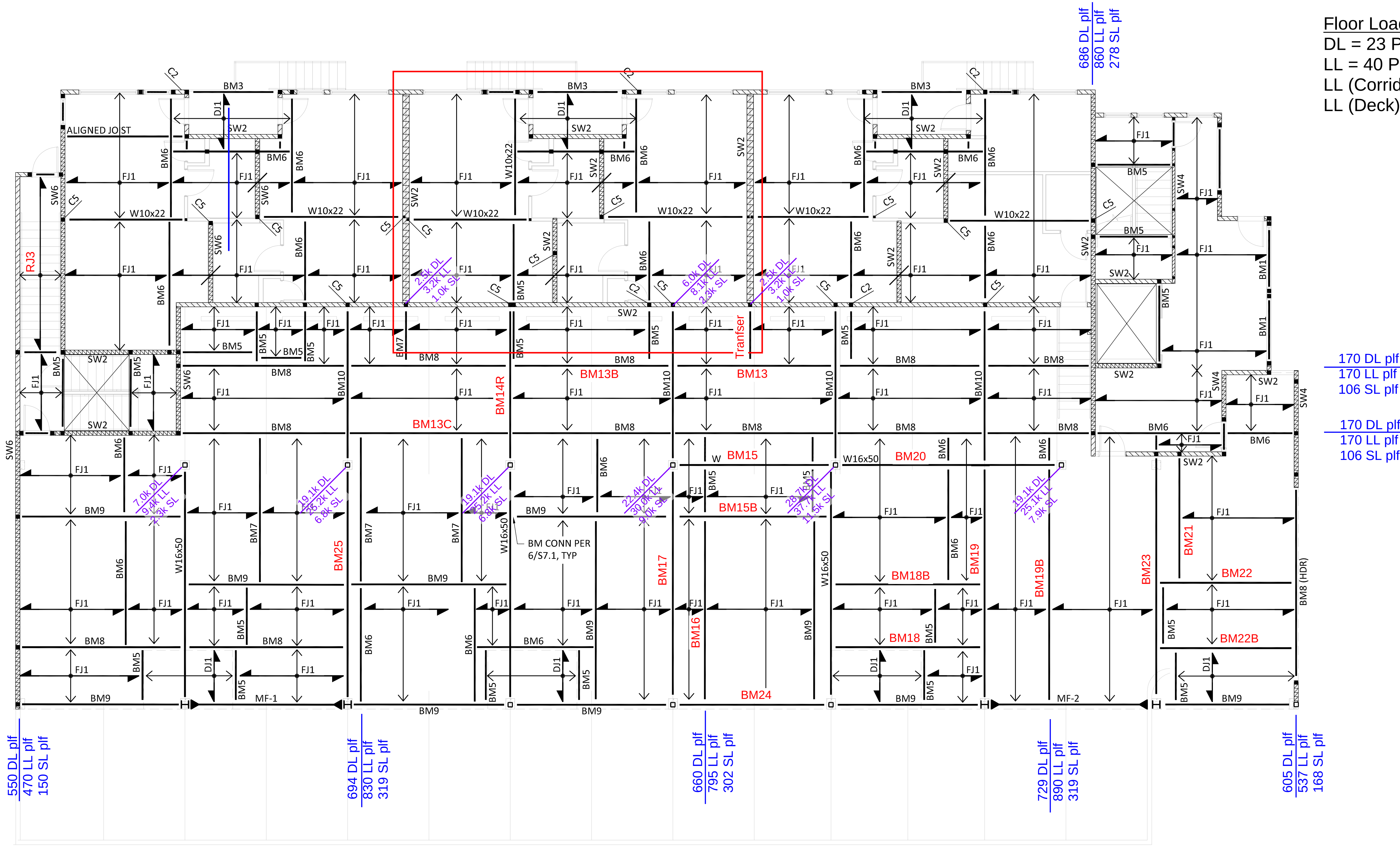
1.005

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

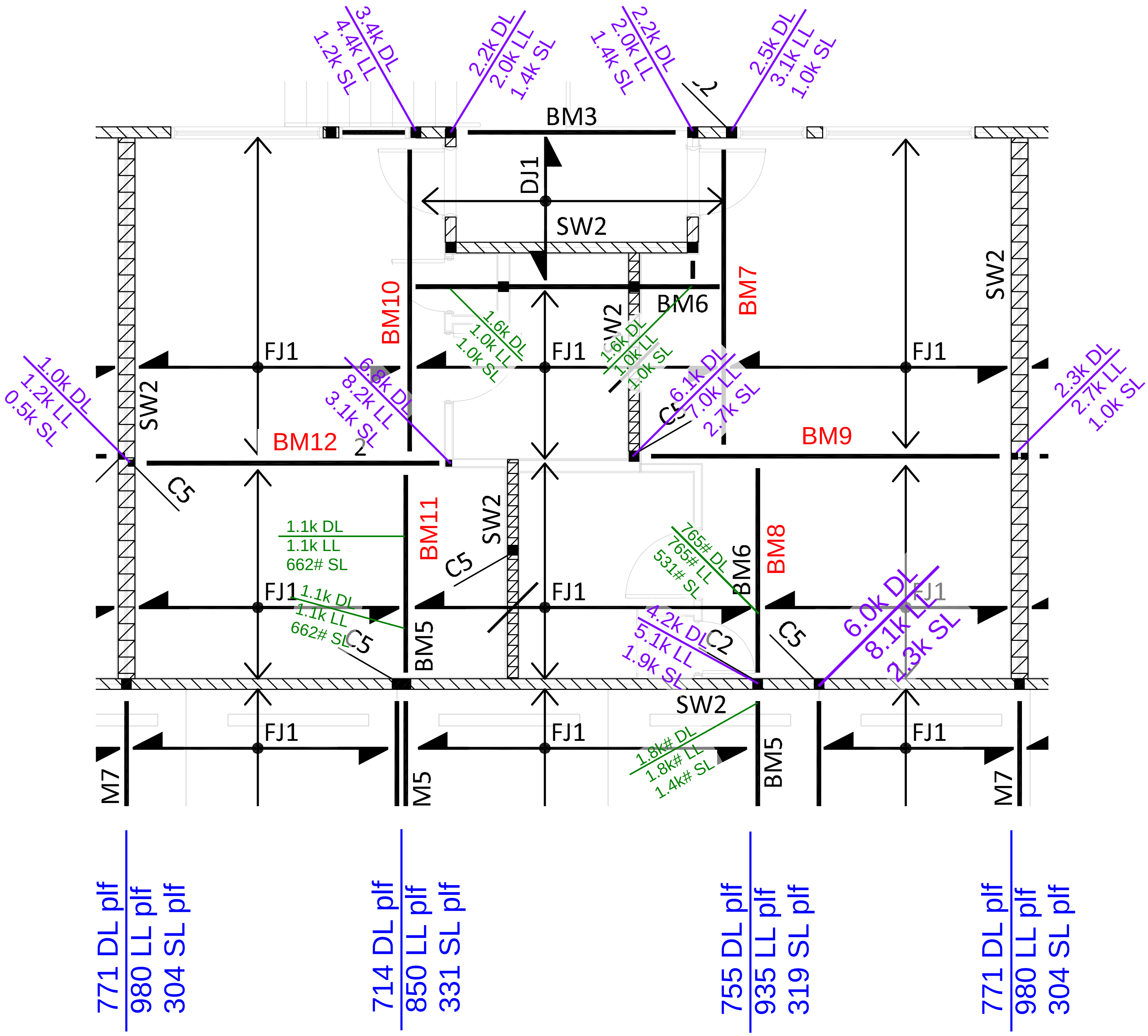


1 SECOND FLOOR FRAMING PLAN

SCALE: 1/8" = 1'-0"

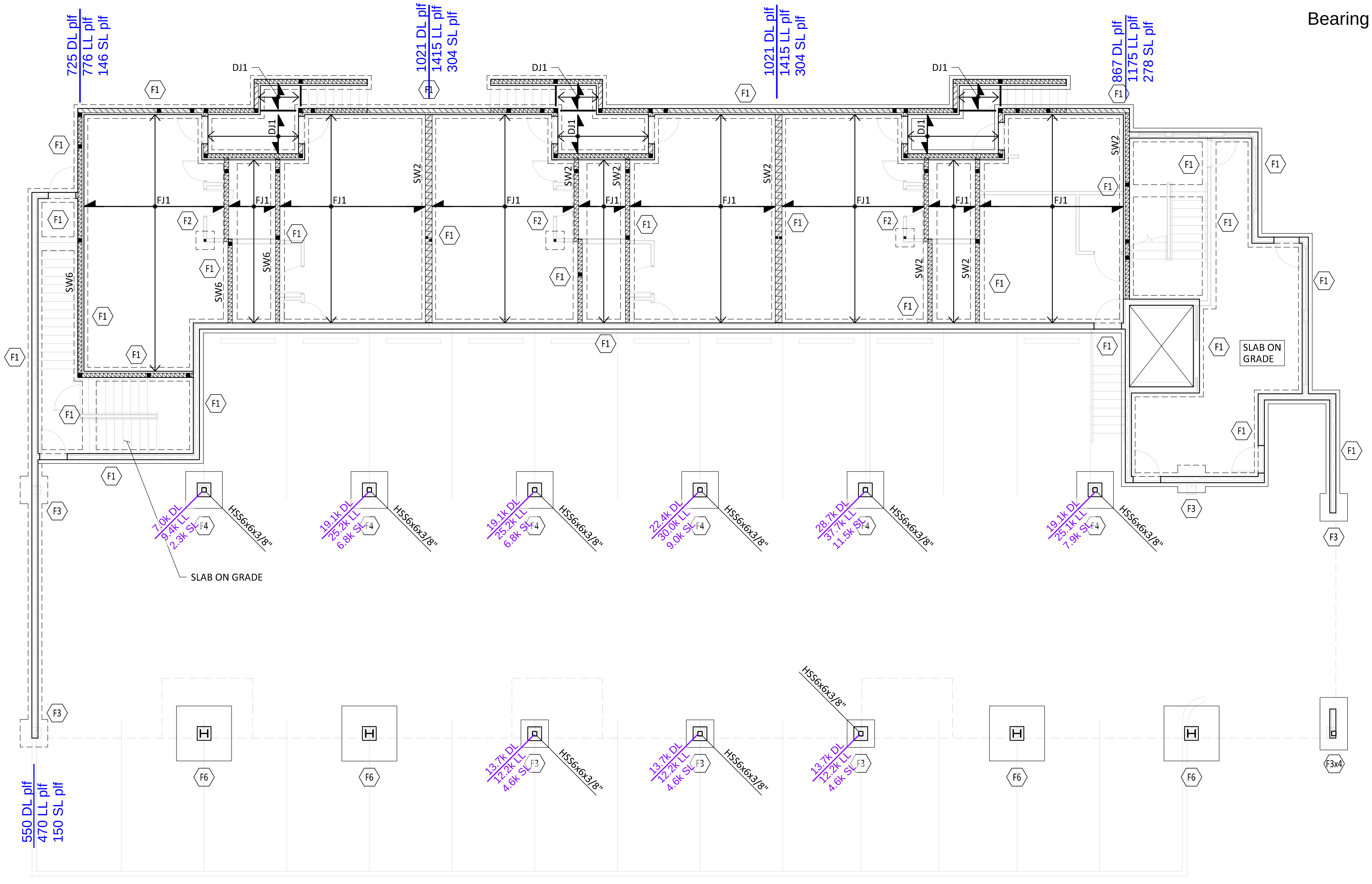
5'-0" 0 5'-0" 10'-0"

TYP UNIT FRAMING KEY PLAN
(2nd FLOOR)



FOUNDATION KEY PLAN

Bearing Capacity: 4000 psf



1 FOUNDATION PLAN

SCALE: 1/8" = 1'-0"

5'-0" 0 5'-0" 10'-0"

1.009

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} <= 3.0**
5. **Table values include the Size Factor (C_F) and the Load Duration Factor (C_D)**

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

Description	Beam Span Table	By	CCC	Date	06/19/23
		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)																	
	Span Length in Feet																
Beam	**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} <= 4.0**
5. **Table values include the Size Factor (C_F)**

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

Description	Beam Span Table	By	CCC	Date	06/19/23
		Checked		Date	
		Scale		Sheet No.	
Project	The Talmon	Job No.	23154.10		

BEARING WALL TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

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

Notes:

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

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

Description: Bearing Wall Capacity Table	By: CCC	Date: 07/21/23
	Checked	Date
	Scale	Sheet No.
Project: The Talmon	Job No. 23154.10	

DF Column & DF Sill Plate Capacity TABLE

IBC 2018, NDS 2018

Date modified 10-2-14

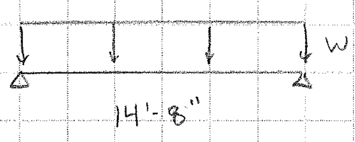
	6	7	8	9	10	11	12	13	14	15	16
(2) 2x4 DF stud	5,781	4,680	3,786	3,092	2,559	2,146	1,823	1,565	1,358	1,188	1,048
P_{SILL}	-	-	-	-	-	-	-	-	-	-	-
(3) 2x4 DF Stud	10,202	8,671	7,239	6,029	5,050	4,270	3,645	3,143	2,734	2,398	2,119
P_{SILL}	-	-	-	-	-	-	-	-	-	-	-
(4) 2x4 DF Sud	13,603	11,561	9,652	8,039	6,734	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	-	-	-	-	-	-	-

C.G ENGINEERING
250 4th Ave. South
Suite 200

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

Joist Design:

RJ1:

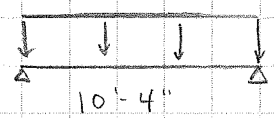


w = 17 psf DL + 25 psf SL

M =	65%	2.4 k-ft
V =	35%	623 lbs
Δ_T =	L/449	0.401"

Use 11 7/8" TJI 110 @ 24" OC

RJ2 (corridor):

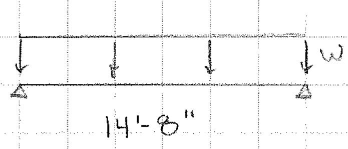


w = 17 psf DL + 25 psf SL

M =	78%	1158 lbs-ft
V =	31%	383 lbs
Δ_T =	L/340	0.221"

Use 2x8 HF #2 @ 24" OC

FJ1:

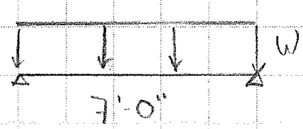


w = 23 psf DL + 40 psf LL

M =	73%	2.3 k-ft
Δ_T =	L/567	0.314"
TJI Rating =		47

Use 11 7/8" TJI 110 @ 16" OC

CJ1:

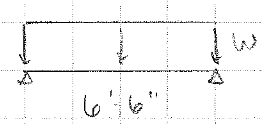


w = 23 psf DL + 40 psf LL

M =	64%	514 ft-lbs
V =	31%	250 lbs
Δ_T =	L/423	0.20"

Use 2x6 HF #2 @ 16" OC

DJ1:



w = 23 psf DL + 60 psf LL

M =	77%	615 lbs-ft
V =	37%	309 lbs
Δ_L =	L/608	0.132"

Use 2x6 HF #2 @ 16" OC

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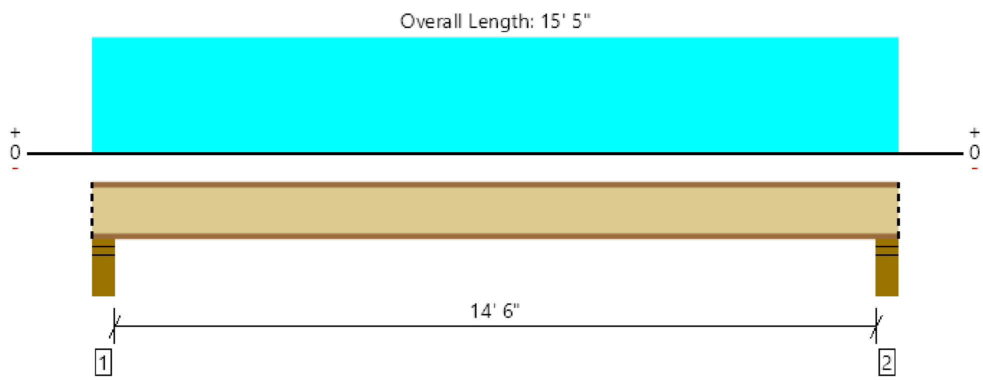
250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
www.cgengineering.com

Description: Joist Design	By: CCC	Date: 6/20/23
	Checked	Date
	Scale	Sheet No.
Project: The Talmon	Job No.: 23154.10	

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.

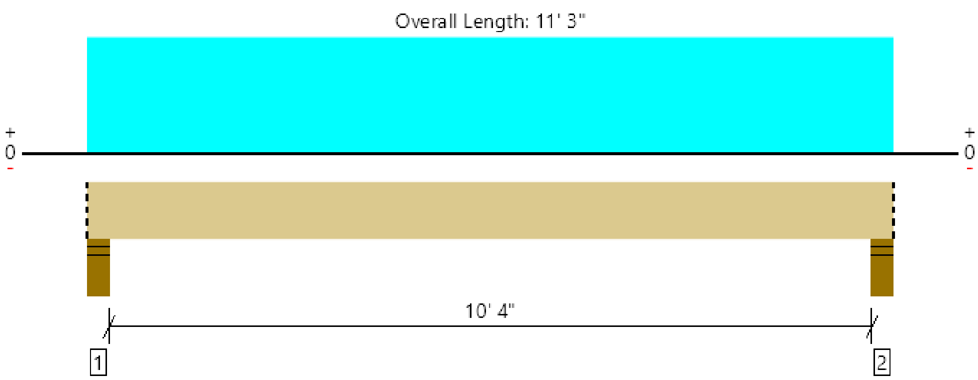
Vertical Load	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 15' 5"	24"	17.0	25.0	Default Load

Weyerhaeuser Notes

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

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)

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

- 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.

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

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	

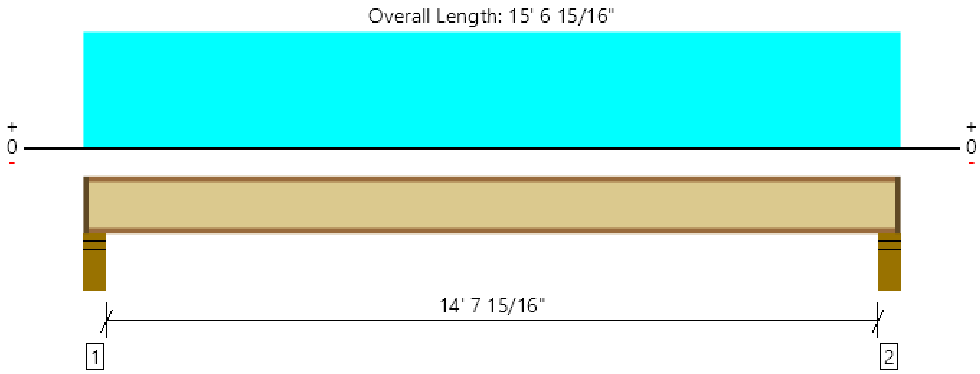
FORTEWEB

MEMBER REPORT

PASSED

Level, FJ1

1 piece(s) 11 7/8" TJI® 110 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	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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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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

Weyerhaeuser

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 18, 2023 10:22	CJ1

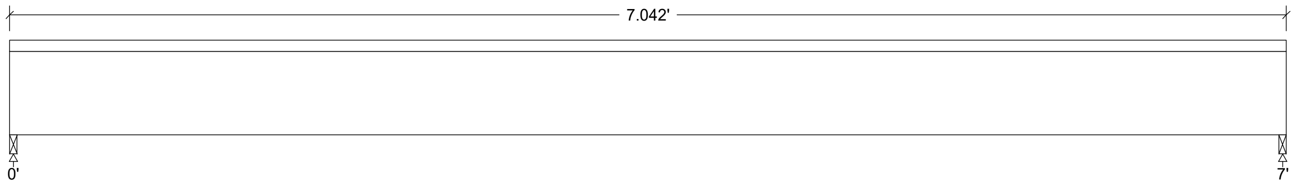
Design Check Calculation Sheet

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:

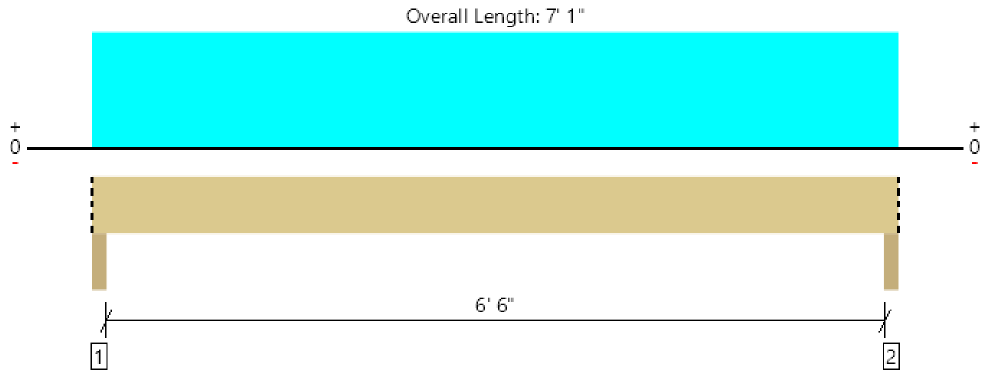
```
V max = 294, V design = 254 (NDS 3.4.3.1(a)) lbs; M(+) = 514 lbs-ft
EI = 27.04e06 lb-in^2
"Live" deflection is due to all non-dead loads (live, wind, snow…)
Total deflection = 1.50 permanent + "live"
```

Design Notes:

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. 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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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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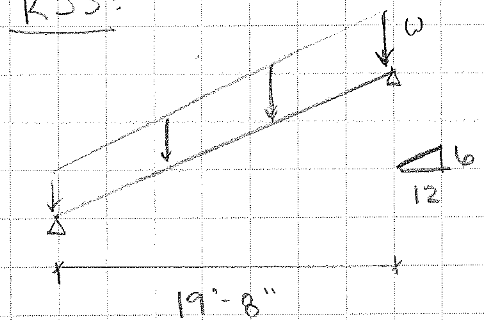
RJ3:

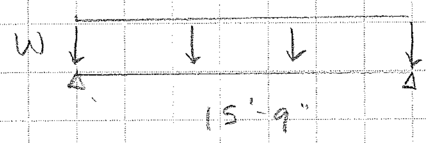


$w = 17$ psf DL + 25 psf SL

$M = 94\%$ 4.1 k-ft
$V = 44\%$ 828 lbs
$\Delta_T = L/217$ 1.23"

| Use 11 7/8" TJI 210 @ 24" OC |

FJ1 (@ Crawlspace):



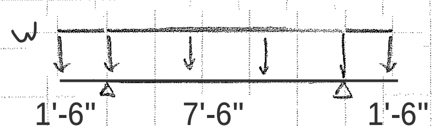
$w = 17$ psf DL + 40 psf LL

$M = 84\%$ 2.66 k-ft
$V = 42\%$ 662 lbs
$\Delta_T = L/467$ 0.409

| Use 11 7/8" TJI 110 @ 16" OC |

TJI Rating = 44

RJ4:



w = 17psf DL + 25psf SL

M = 24%
V = 16%
$\Delta_T = L/632$

USE 2x10 HF #2 @ 24" OC

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www.cgengineering.com

Description Joist Design	By CCC	Date 7/18/23
	Checked	Date
	Scale	Sheet No.
Project The Talmon	Job No. 23154.10	

COMPANY	PROJECT
July 21, 2023 15:33	RJ4

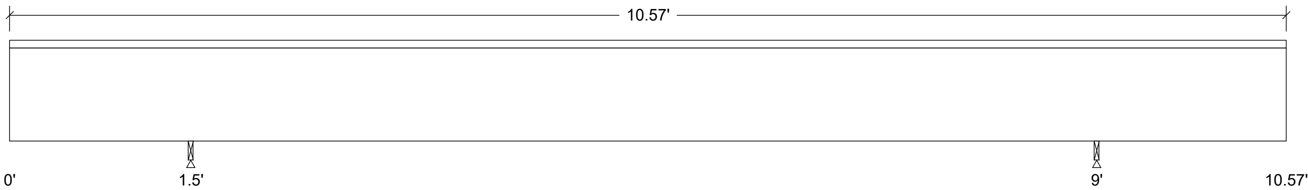
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat- tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full Area	No			17.00(24.0")		psf
SL	Snow	Full Area	Yes			25.00(24.0")		psf

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



Unfactored:		
Dead	178	181
Snow	266	270
Factored:		
Total	444	452
Bearing:		
Capacity		
Joist	532	532
Support	586	586
Des ratio		
Joist	0.84	0.85
Support	0.76	0.77
Load comb	#5	#8
Length	0.50*	0.50*
Min req'd	0.42	0.43
Cb	1.75	1.75
Cb min	1.75	1.75
Cb support	1.25	1.25
Fcp sup	625	625

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

Lumber-soft, Hem-Fir, No.2, 2x10 (1-1/2"x9-1/4")

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

Roof joist spaced at 24.0" c/c; Total length: 10.56'; Clear span: 1.5', 7.438', 1.563'; Volume = 1.0 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 = 27	Fv' = 172	psi	fv/Fv' = 0.16
Bending(+)	fb = 292	Fb' = 1237	psi	fb/Fb' = 0.24
Bending(-)	fb = 58	Fb' = 799	psi	fb/Fb' = 0.07
Deflection:				
Interior Live	0.02 = < L/999	0.38 = L/240	in	0.07
Total	0.05 = < L/999	0.50 = L/180	in	0.09
Cantil. Live	-0.02 = < L/999	0.15 = L/120	in	0.10
Total	-0.03 = L/632	0.20 = L/90	in	0.14

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	LC#
Fv'	150	1.15	1.00	1.00	-	-	-	-	1.00	1.00	8
Fb'+	850	1.15	1.00	1.00	1.000	1.100	-	1.15	1.00	1.00	4
Fb'-	850	1.15	1.00	1.00	0.646	1.100	-	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	4
Emin'	0.47 million		1.00	1.00	-	-	-	-	1.00	1.00	4

CRITICAL LOAD COMBINATIONS:

```
Shear      : LC #8  = D + S (pattern: sSS)
Bending(+): LC #4  = D + S (pattern: sSs)
Bending(-): LC #2  = D + S
Deflection: LC #4  =   (live)
            LC #4  =   (total)
Bearing    : Support 1 - LC #5 = D + S (pattern: SSs)
             Support 2 - LC #8 = D + S (pattern: sSS)
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:

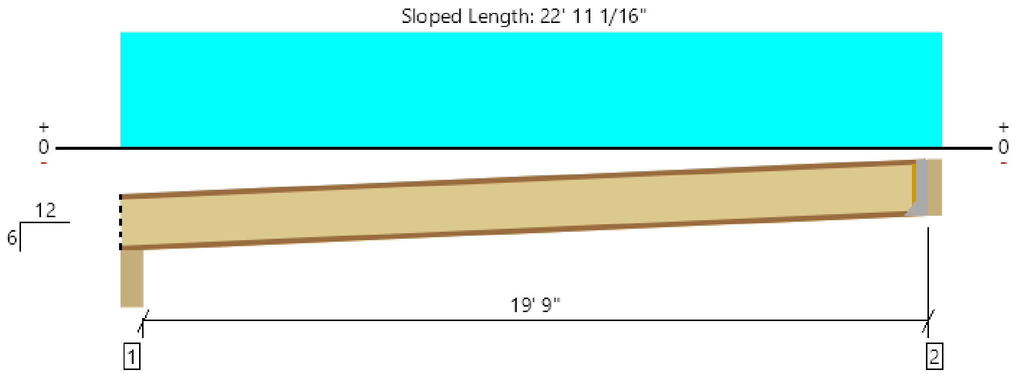
```
V max = 320, V design = 253 (NDS 3.4.3.1(a)) lbs; M(+) = 521 lbs-ft; M(-) = 104 lbs-ft
EI = 128.61e06 lb-in^2
"Live" deflection is due to all non-dead loads (live, wind, snow…)
Total deflection = 1.50 permanent + "live"
Lateral stability(-):  Lu = 7.50'  Le = 13.13'  RB = 25.4; Lu based on full span
```

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. Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
4. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

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"

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 6/12

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.

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

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

Weyerhaeuser

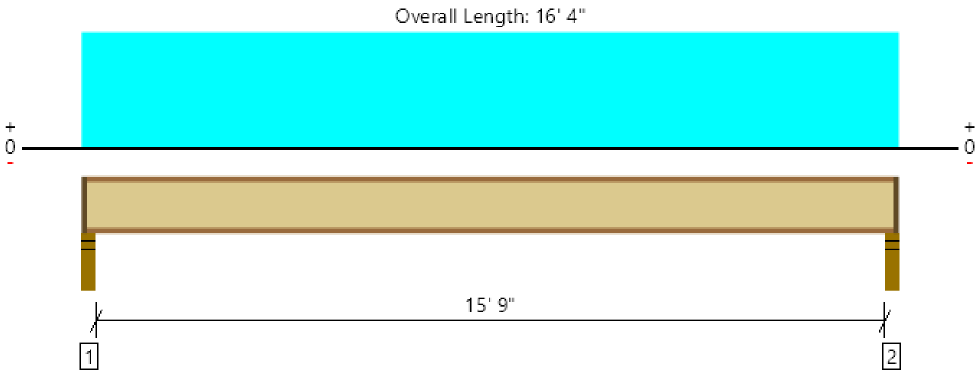
FORTEWEB

MEMBER REPORT

PASSED

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

Weyerhaeuser Notes

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

Weyerhaeuser

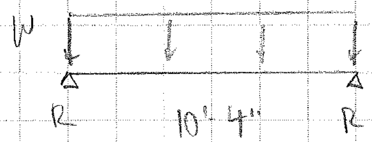
7/21/2023 4:46:54 PM UTC

ForteWEB v3.6, Engine: V8.3.0.43, Data: V8.1.4.1

File Name: Telmon

1.023

BM1R:



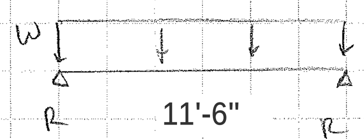
w = (227 DL + 334 SL) plf

R = 1.2 k DL + 1.7 k SL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

* From Roof Beam Table

BM2R:



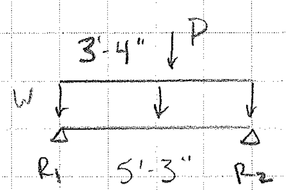
w = (72 DL + 106 SL) plf

R = 415 # DL + 610 # SL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

* From Roof Beam Table

BM3R:



P = 1.2 k DL + 1.7 k SL

w = (89 DL + 131 SL) plf

R_1 = 711 # DL + 1022 # SL

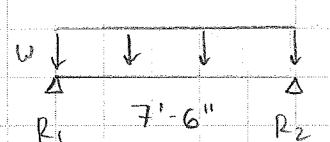
R_2 = 1032 # DL + 1477 # SL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 19% 4.2 k-ft

V = 24% 2.4 kips

BM4R:



w = (102 DL + 150 SL) plf

R_1 = 383 # DL + 563 # SL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

* From Roof Beam Table

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

Description: Roof Framing Design		By: CCC	Date: 6/19/23
		Checked	Date
		Scale	Sheet No.
Project: The Talmon		Job No. 23154.10	

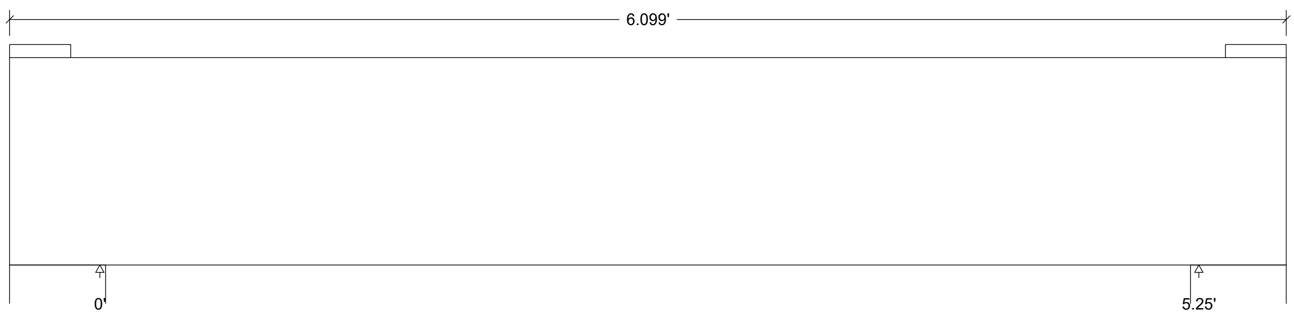
	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 10:10	BM3R

Design Check Calculation Sheet
WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude 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	fv = 79	Fv' = 334	psi	fv/Fv' = 0.24
Bending(+)	fb = 618	Fb' = 3268	psi	fb/Fb' = 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

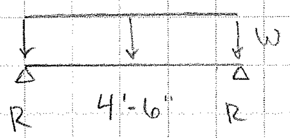
Additional Data:

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-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"
 Lateral stability(+):  Lu = 5.25'  Le = 10.81'  RB = 11.2
```

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.

HDR (Roof - Interior):

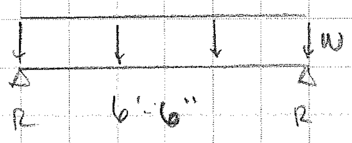


w = (219 DL + 332 SL) plf
R = 493 # DL + 747 # SL

| Use 4x6 HF #2 | * From Beam Table

Max Span = 5'-0"

HDR (Roof - Exterior):

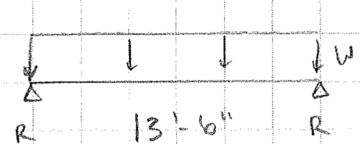


w = (123 DL + 181 SL) plf
R = 400 # DL + 588 # SL

| Use 4x6 HF #2 | * From Beam Table

Max Span = 7'-0"

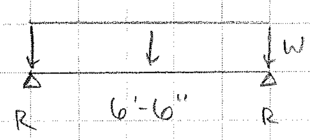
BM5R:



w = (114 DL + 168 SL) plf
R = 770 # DL + 1134 # SL

| Use 3½" x 11⅞" PSL | * From Beam Table

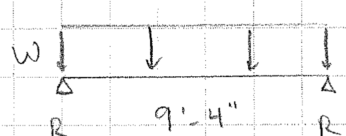
BM6R:



w = (115 DL + 169 SL) plf
R = 375 # DL + 550 # SL

| Use 3½" x 11⅞" PSL | * From Beam Table

BM7R:



w = (275 DL + 272 SL) plf

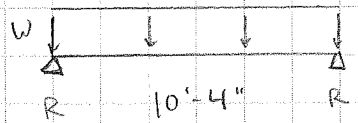
R = 1.3k DL + 1.3k SL

| Use 3½" x 11⅞" PSL | * From Roof Table

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

Description Roof Framing	By CCC	Date 6/20/23
	Checked	Date
	Scale	Sheet No.
Project The Talmon	Job No. 23154.10	

BM1:



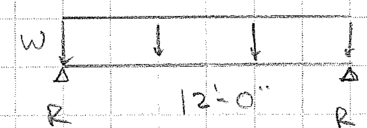
w = (307 DL + 534 LL) plf

R = 1.6 k DL + 2.8 k LL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

* From Beam Table

BM2:



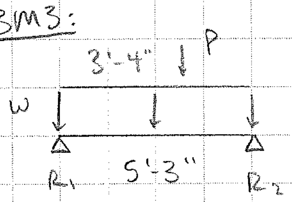
w = (86 DL + 150 LL) plf

R = 518 # DL + 900 # LL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

*From Beam Table

BM3:



P = 1.6 k DL + 2.8 k LL

w = (121 DL + 525 LL) plf

R_1 = 951# DL + 2.6 k LL

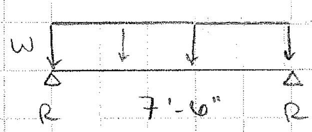
R_2 = 1.4 k DL + 3.4 k LL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 38% 7.4 k-ft

V = 47% 4.5 kips

BM4:

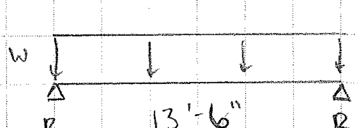


w = (133 DL + 230 LL) plf

R_1 = 500 # DL + 875# LL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 9\tfrac{1}{2}" \text{ PSL}}$ *From Beam Table

BM5:



w = (75 DL + 195 LL) plf

R = 506 # DL + 1.3 k LL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

* From Beam Table

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

Description Floor Framing Design	By CCC	Date 6/20/23
	Checked	Date
	Scale	Sheet No.
Project The Talmon	Job No. 23154.10	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY July 21, 2023 10:31	PROJECT BM3

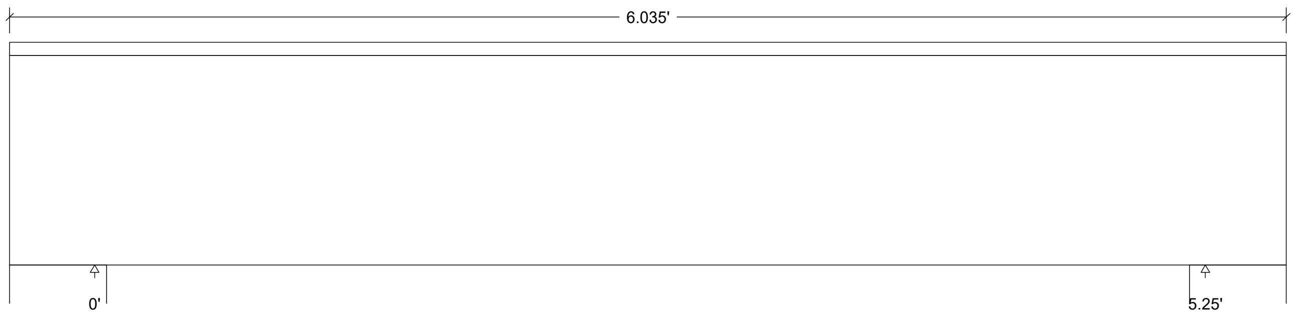
Design Check Calculation Sheet

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	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	-	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
```

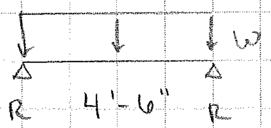
CALCULATIONS:

```
 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:

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.

HDR (Floor - Interior):

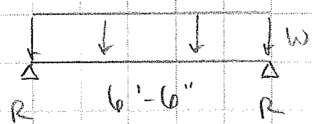


w = (293 DL + 510 LL) plf
R = 660# DL + 1150# LL

$\boxed{\text{Use 4x8 HF \#2}}$ * From Beam Table

Max Span = 5'-0"

HDR (Floor - Exterior):

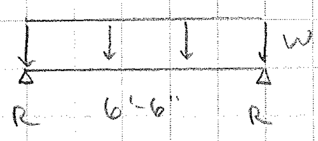


w = (167 DL + 290 LL) plf
R = 543# DL + 943# LL

$\boxed{\text{Use 4x8 HF \#2}}$ * From Beam Table

Max Span = 7'-0"

BM6:



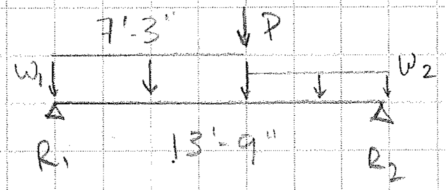
w = (155 DL + 270 LL) plf
R = 504# DL + 878# LL

$\boxed{\text{Use 3 1/2" x 11 7/8" PSL}}$ * From Beam Table

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

Description Floor Framing Design		By CCC	Date 6/20/23
		Checked	Date
		Scale	Sheet No.
Project The Talmon		Job No.	

BM7:

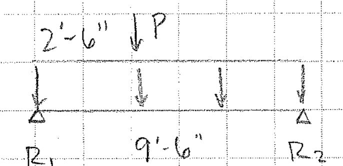


P = 1066 DL + 667 LL + 667 SL
W_1 = (694 DL + 830 LL + 319 SL) plf
W_2 = (132 DL + 230 LL) plf
R_1 = 4.3k DL + 5.0k LL + 2.0k SL
R_2 = 2.5k DL + 3.1k LL + 1.0k SL

| Use 7" x 11 7/8" PSL |

M = 75% 29.7 k-ft
V = 49% 7.9 kips
Δ_T = L/293 0.56"

BM8:



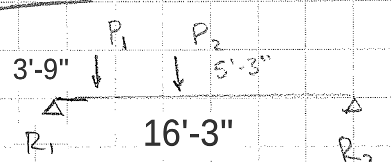
P = 765 DL + 765 LL + 531 SL
W = (755 DL + 935 LL + 319 SL) plf

R_1 = 4.2k DL + 5.1k LL + 1.9k SL
R_2 = 3.9k DL + 4.7k LL + 1.7k SL

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

M = 71% 21.0 k-ft
V = 61% 9.2 kips
Δ_T = L/430 0.26"

BM9:



P_1 = 4.3k DL + 5.0k LL + 2.0k SL
P_2 = 3.9k DL + 4.7k LL + 1.7k SL

R_1 = 6.1k DL + 7.0k LL + 2.7k SL
R_2 = 2.3k DL + 2.7k LL + 1.0k SL

| Use W10x22 |

M = 86% 55.7 k-ft
V = 27% 13.6 kips
Δ_T = L/313 0.62"

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY July 21, 2023 10:37	PROJECT BM7

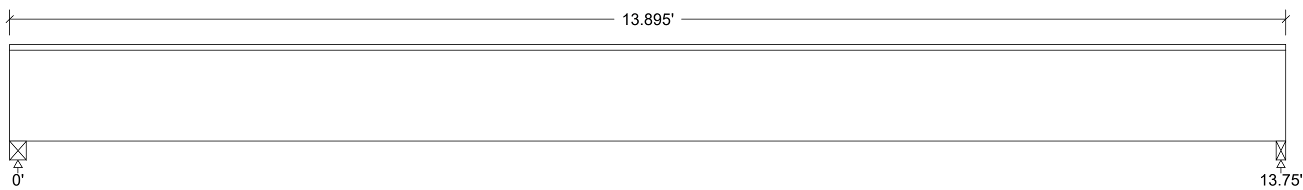
Design Check Calculation Sheet

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) :



	0'	13.75'
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	fv = 143	Fv' = 290	psi	fv/Fv' = 0.49
Bending(+)	fb = 2165	Fb' = 2903	psi	fb/Fb' = 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	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 = 9345, V design = 7915 (NDS 3.4.3.1(a)) lbs; M(+) = 29676 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:

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.

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 | CG ENGINEERING | (c) ENERCALC INC 1983-2023

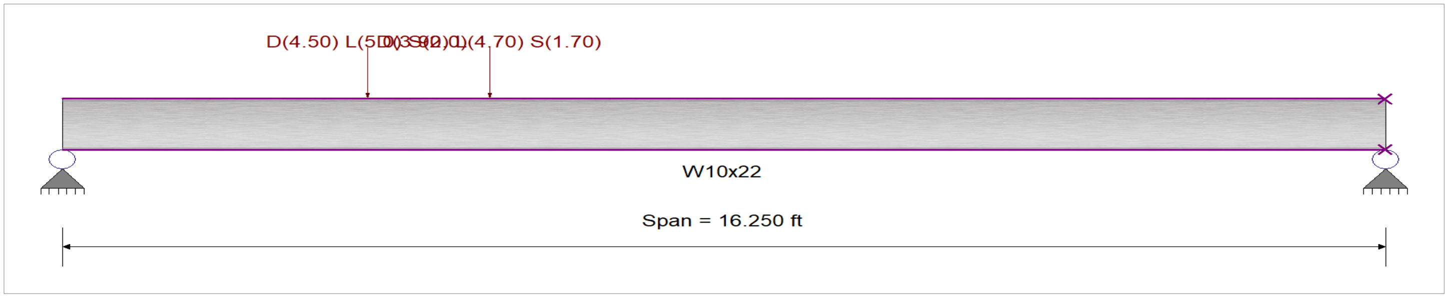
DESCRIPTION: BM9

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
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

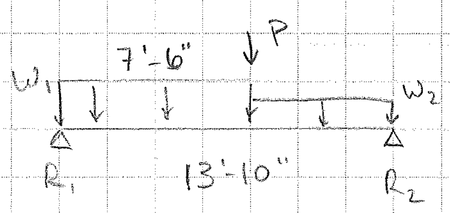
Max Downward Transient Deflection	0.328 in	Ratio =	594	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.622 in	Ratio =	313	>=240.	Span: 1 : +D+0.750L+0.750S
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

Vertical Reactions

Support notation : Far left is #1 | 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:



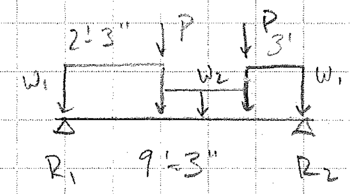
P = 800 DL + 500 LL + 500 SL
w_1 = (714 DL + 850 LL + 331 SL) plf
w_2 = (138 DL + 240 LL) plf

R_1 = 4.5k DL + 5.2k LL + 2.0 k SL
R_2 = 2.6 k DL + 3.2k LL + 1.0 k SL

$\boxed{\text{Use } 7" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 50%
V = 76%
Δ_T = L/288

BM11:



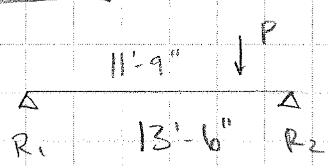
P = 1.1k DL + 1.1k LL + 662 SL
w_1 = (714 DL + 850 LL + 331 SL) plf
w_2 = (181 DL + 315 LL) plf

R_1 = 4.7k DL + 2.8k LL + 1.6k SL
R_2 = 4.4 k DL + 3.0k LL + 1.6 k SL

$\boxed{\text{Use } 3\tfrac{1}{2}" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 80% 15.8 k-ft
V = 71% 5.7 kips

BM12:



P = 8.9 k DL + 8.2 k LL + 3.6 k SL

R_1 = 1.1 k DL + 1.0 k LL + 0.5 k SL
R_2 = 7.8 k DL + 7.2 k LL + 3.1 k SL

$\boxed{\text{Use } 7" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 64% 25.5 k-ft
V = 93% 15 kips
Δ_T = L/470 0.34"

C.G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description Beam Design	By CCC	Date 7/11/23
		Checked	Date
		Scale	Sheet No.
	Project The Talmon	Job No. 23157.10	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	July 21, 2023 15:09	BM10

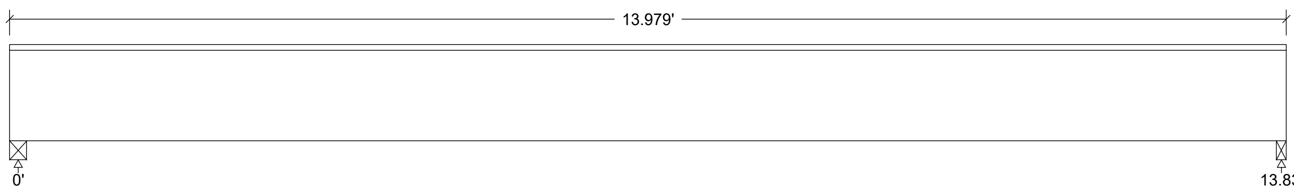
Design Check Calculation Sheet

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) :



	0'	13.83'
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	fv = 145	Fv' = 290	psi	fv/Fv' = 0.50
Bending(+)	fb = 2190	Fb' = 2903	psi	fb/Fb' = 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-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:

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.

COMPANY	PROJECT
July 21, 2023 11:27	BM11

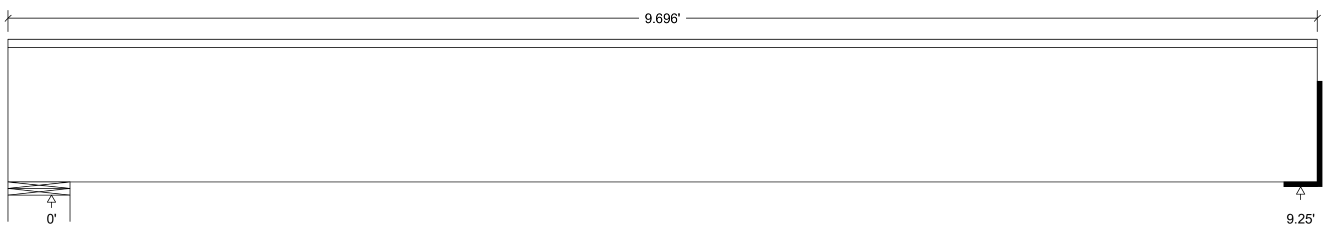
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
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	fv = 205	Fv' = 290	psi	fv/Fv' = 0.71
Bending(+)	fb = 2310	Fb' = 2903	psi	fb/Fb' = 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-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:

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.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 11:33	BM12

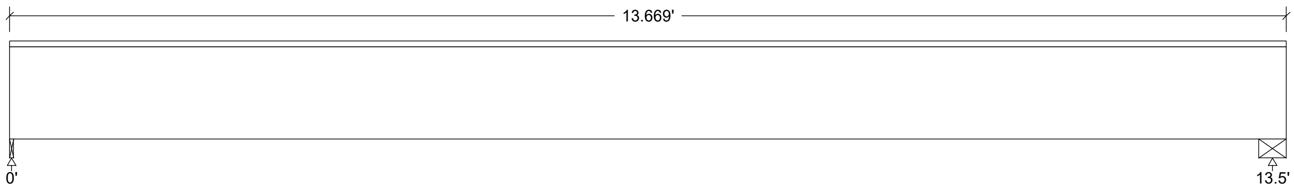
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude 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) :



	0'	13.5'
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	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
```

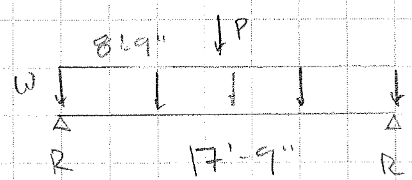
CALCULATIONS:

```
V max = 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:

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.

BM13:



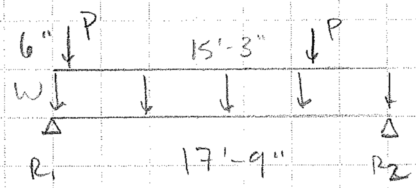
P = 2.4k DL + 3.0k LL + 1.0k SL
w = (170 DL + 170 LL + 106 SL) plf

R = 2.8k DL + 3.1k LL + 1.5k SL

M = 73% 36.9 k-ft
V = 29% 5.4 kips
Δ_T = L/242 0.87"

$\boxed{\text{Use } 5\frac{1}{4}" \times 14" \text{ PSL}}$

BM13B:



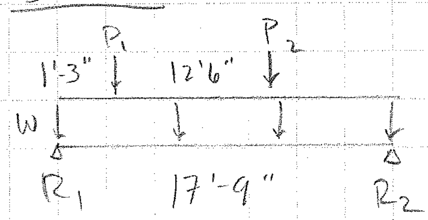
P = 2.4k DL + 2.9k LL + 1.0k SL
w = (170 DL + 170 LL + 106 SL) plf

R_1 = 4.2k DL + 4.8k LL + 2.0k SL
R_2 = 3.7k DL + 4.1k LL + 1.8k SL

$\boxed{\text{Use } 5\frac{1}{4}" \times 14" \text{ PSL}}$

M = 55% 21.9 k-ft
V = 51% 7.2 kips
Δ_T = L/334 0.64"

BM13C:



P_1 = 5.7k DL + 6.8k LL + 2.6k SL
P_2 = 5.8k DL + 6.9k LL + 2.6k SL
w = (170 DL + 170 LL + 106 SL) plf

R_1 = 8.6k DL + 9.9k LL + 4.1k SL
R_2 = 6.0k DL + 6.9k LL + 3.0k SL

M = 96% 62.7 k-ft
V = 78% 14.3 kips
Δ_T = L/285 0.75"

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

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Description	By CCC	Date 7/10/23
	Checked	Date
	Scale	Sheet No.
Project The Talmon	Job No. 23154.10	

	COMPANY	PROJECT
	July 21, 2023 13:15	BM13

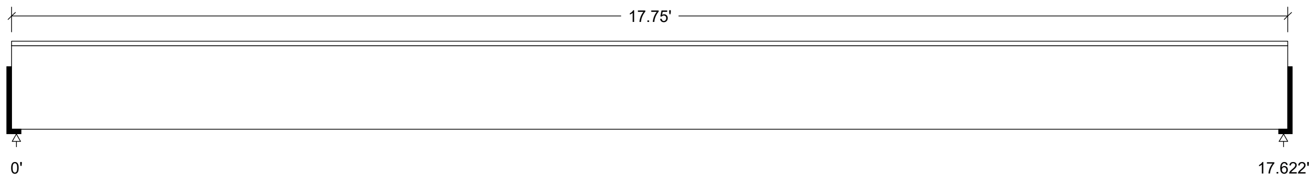
Design Check Calculation Sheet

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	fv = 110	Fv' = 290	psi	fv/Fv' = 0.38
Bending(+)	fb = 2583	Fb' = 2851	psi	fb/Fb' = 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(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 = 5811, V design = 5392 (NDS 3.4.3.1(a)) lbs; M(+) = 36920 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:

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.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 13:41	BM13B

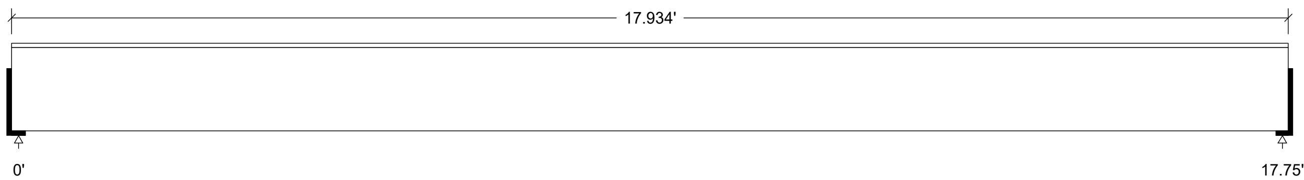
Design Check Calculation Sheet

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) :



	0'	17.75'
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	fv = 147	Fv' = 290	psi	fv/Fv' = 0.51
Bending(+)	fb = 1530	Fb' = 2851	psi	fb/Fb' = 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(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 = 7217, V design = 7217 (NDS 3.4.3.1(a)) lbs; M(+) = 21865 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:

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.

	COMPANY	PROJECT
	July 21, 2023 13:58	BM13C

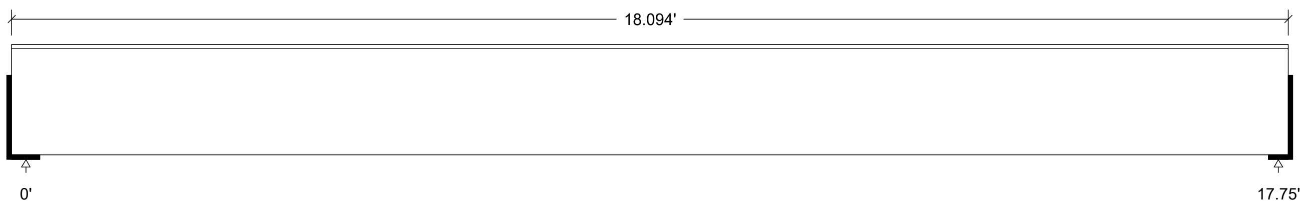
Design Check Calculation Sheet

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	fv* = 227	Fv' = 290	psi	fv*/Fv' = 0.78
Bending(+)	fb = 2655	Fb' = 2772	psi	fb/Fb' = 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	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
```

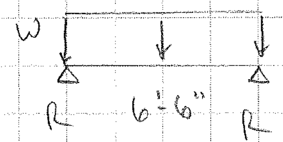
CALCULATIONS:

```
V max = 18394, V design* = 14315 (NDS 3.4.3.1(a)) lbs; M(+) = 62732 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:

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.

Transfer BM:

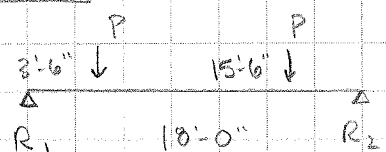


w = (738 DL + 923 LL + 304 SL) plf
ASD Total = 1661 plf

R = 2.4k DL + 3.0k LL + 1.0k SL

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

BM15:



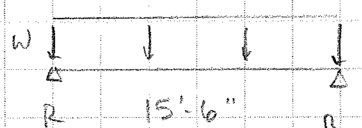
P = 3.0k DL + 3.7k LL + 1.3k SL

R_1 = 2.8k DL + 3.5k LL + 1.2k SL
R_2 = 3.2k DL + 3.9k LL + 1.4k SL

$\boxed{\text{Use } 7" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 56% 22.1 k-ft
V = 44% 7.1 kips
Δ_T = L/279 0.77"

BM16:

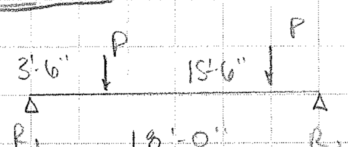


w = (658 DL + 790 LL + 300 SL) plf
ASD = 1476 plf
R = 5.1k DL + 6.2k LL + 2.4k SL

$\boxed{\text{Use } 5\tfrac{1}{4}" \times 18" \text{ PSL}}$

M = 66%
V = 48%
Δ_T = L/445

BM15B:



P = 5.1k DL + 6.1k LL + 2.3k SL

R_1 = 4.8k DL + 5.8k LL + 2.2k SL
R_2 = 5.4k DL + 6.4k LL + 2.4k SL

$\boxed{\text{Use } 5\tfrac{1}{4}" \times 18" \text{ PSL}}$

M = 58%
V = 65%
Δ_T = L/431

C.G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description Beam Design	By CCC	Date 7/11/23
		Checked	Date
		Scale	Sheet No.
	Project The Talmon	Job No. 23154.10	

COMPANY	PROJECT
July 21, 2023 14:17	BM15

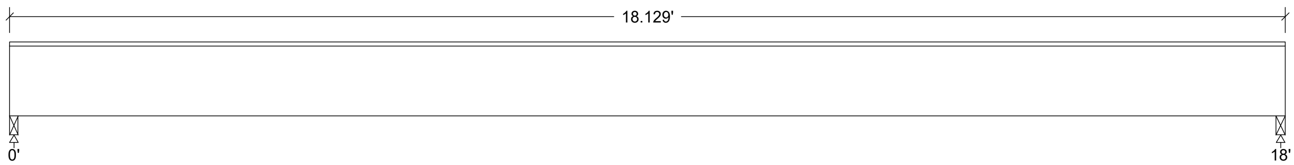
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pattern	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-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:

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.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 14:18	BM16

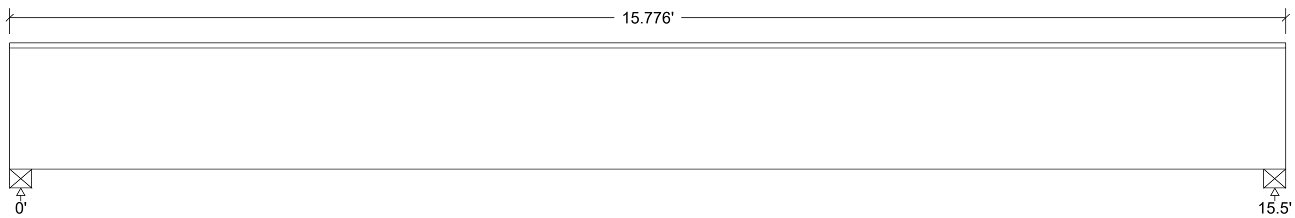
Design Check Calculation Sheet

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	fv = 140	Fv' = 290	psi	fv/Fv' = 0.48
Bending(+)	fb = 1841	Fb' = 2772	psi	fb/Fb' = 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-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:

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.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	July 21, 2023 14:20	BM15B

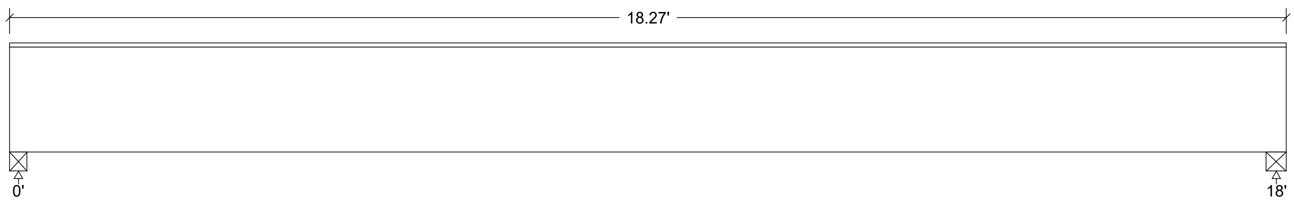
Design Check Calculation Sheet

WoodWorks Sizer 2023

Loads:

Load	Type	Distribution	Pattern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude 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) :



	0'	18'
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	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
```

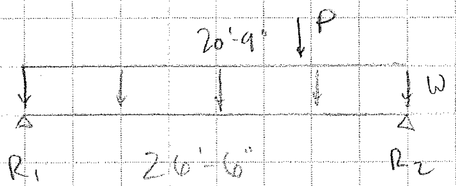
CALCULATIONS:

```
V max = 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:

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.

BM17:



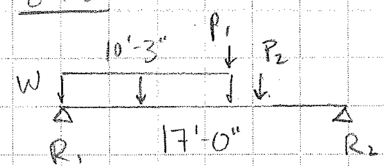
P = 10.2 k DL + 12.2 k LL + 4.6 k SL
W = (138 DL + 240 LL) plf

R_1 = 4.0k DL + 5.0 k LL + 1.0 k SL
R_2 = 9.8 k DL + 12.7 k LL + 3.6 k SL

M = 68%
V = 23%
Δ_T = L/353

$\boxed{\text{Use W16x40}}$

BM18:



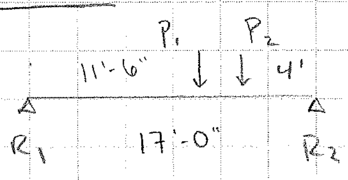
P_1 = 1.9 k DL + 1.4 k LL + 1.0 k SL
P_2 = 2.5 k DL + 3.0 k LL + 1.1 k SL
W = (75 DL + 195 LL) plf

R_1 = 2.1 k DL + 2.9 k LL + 0.7 k SL
R_2 = 3.1 k DL + 3.5 k LL + 1.4 k SL

M = 92%	36.8 k-ft
V = 46%	6.6 kips
Δ_T = L/257	0.79"

$\boxed{\text{Use 5 1/4" x 14" PSL}}$

BM18B:



P_1 = 2.5 k DL + 3.0 k LL + 1.1 k SL
P_2 = 4.6 k DL + 5.6 k LL + 2.1 k SL

R_1 = 1.9 k DL + 2.3 k LL + 0.8 k SL
R_2 = 5.2 k DL + 6.3 k LL + 2.4 k SL

M = 74%	48.1 k-ft
V = 63%	11.6 kips
Δ_T = L/460	0.44"

$\boxed{\text{Use 5 1/4" x 18" PSL}}$

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

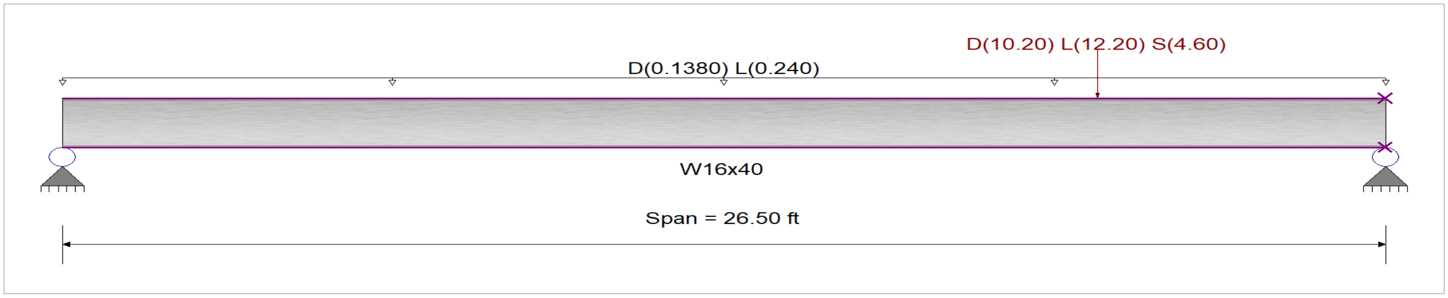
DESCRIPTION: BM17

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
		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
July 21, 2023 14:24	BM18

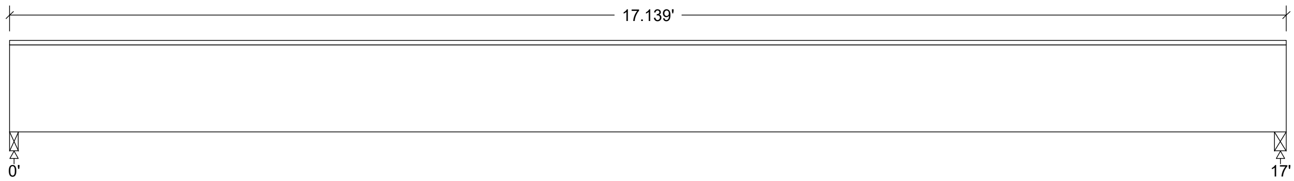
Design Check Calculation Sheet

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	fv = 135	Fv' = 290	psi	fv/Fv' = 0.46
Bending(+)	fb = 2573	Fb' = 2851	psi	fb/Fb' = 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-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:

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.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 14:25	BM18B

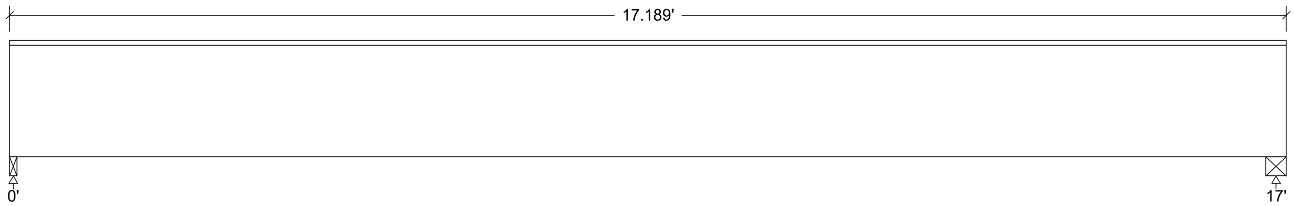
Design Check Calculation Sheet

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) :



	0'	17'
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	fv = 184	Fv' = 290	psi	fv/Fv' = 0.63
Bending(+)	fb = 2037	Fb' = 2772	psi	fb/Fb' = 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	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
```

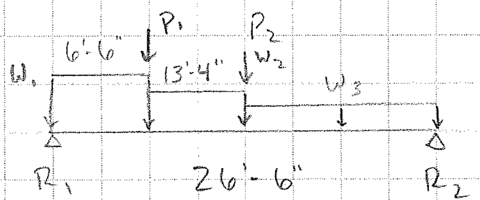
CALCULATIONS:

```
V max = 11572, V design = 11572 (NDS 3.4.3.1(a)) lbs; M(+) = 48128 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:

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.

BM19:



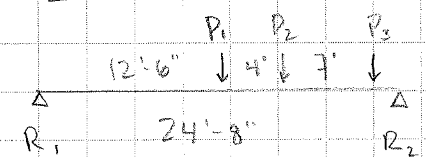
$w_1 = (158 \text{ DL} + 275 \text{ LL})$ plf
$P_1 = 3.1\text{k DL} + 3.5\text{k LL} + 1.4\text{ k SL}$
$w_2 = (144 \text{ DL} + 250 \text{ LL})$ plf
$P_2 = 5.2\text{k DL} + 6.3\text{k LL} + 2.4\text{k SL}$
$w_3 = (129 \text{ DL} + 225 \text{ LL})$ plf

$R_1 = 6.9\text{k DL} + 9.2\text{ k LL} + 2.3\text{ k SL}$
$R_2 = 5.2\text{k DL} + 7.1\text{k LL} + 1.6\text{ k SL}$

Use W16x40

$M = 72\%$	114.2 k-ft
$V = 16\%$	16 kips
$\Delta_T = L/319$	1.0"

BM20:



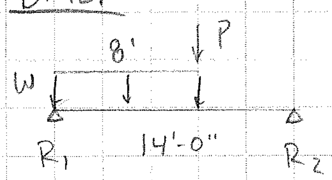
$P_1 = 5.8\text{k DL} + 7.1\text{k LL} + 2.6\text{k SL}$
$P_2 = 10.4\text{ k DL} + 15.4\text{k LL} + 3.8\text{k SL}$
$P_3 = 9.7\text{ k DL} + 11.8\text{ k LL} + 4.2\text{k SL}$

$R_1 = 6.8\text{k DL} + 9.2\text{ k LL} + 2.7\text{ k SL}$
$R_2 = 19.1\text{ k DL} + 25.1\text{ k LL} + 7.9\text{ k SL}$

Use W16x50

$M = 92\%$	210.3 k-ft
$V = 36\%$	44.2 kips
$\Delta_T = L/275$	1.05"

BM21:



$P = 2.8\text{k DL} + 2.8\text{k LL} + 1.7\text{ k SL}$
$w = (706 \text{ DL} + 843 \text{ LL} + 333 \text{ SL})$ 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 k-ft
$V = 65\%$	9.2 kips
$\Delta_T = L/260$	0.65"

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

Description: Beam Design	By: CCC	Date: 7/12/23
	Checked:	Date:
	Scale:	Sheet No.:
Project: The Talmon	Job No.: 23154.10	

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING

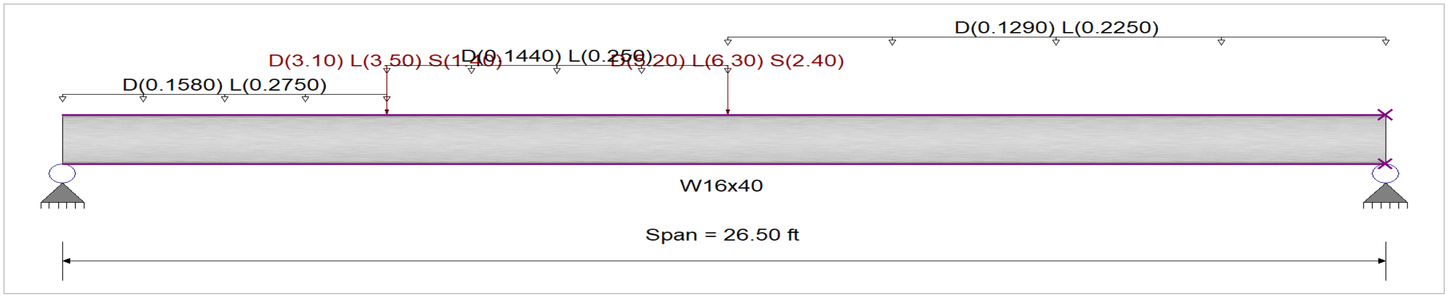
DESCRIPTION: BM19

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
		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	0.567 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 #1 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

Steel Beam

Project File: Talmon.ec6

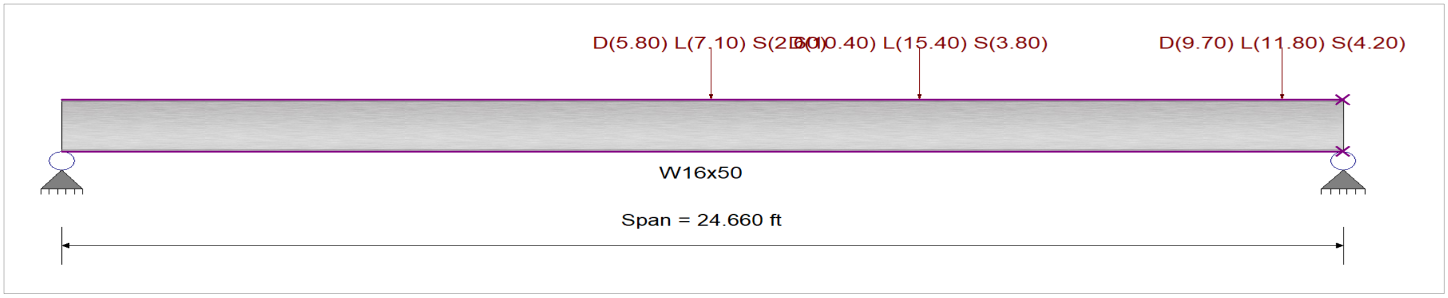
DESCRIPTION: BM20

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
		Location of maximum on span	23.533 ft
Span # where maximum occurs	Span # 1	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 #1 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

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	July 21, 2023 14:42	BM21

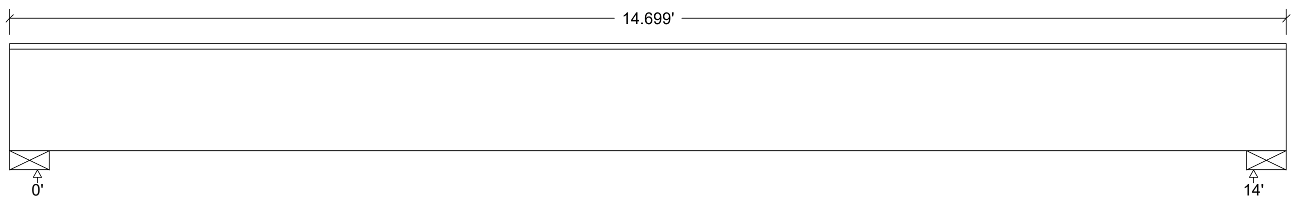
Design Check Calculation Sheet

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) :



	0'	14'
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
```

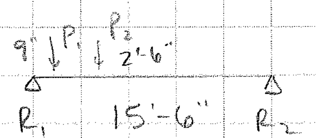
CALCULATIONS:

```
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:

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.

BM22:



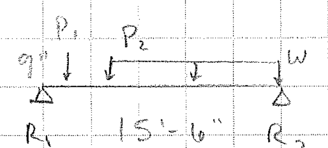
P_1 = 2.6 k DL + 3.0 k LL + 1.2 k SL
P_2 = 5.2 k DL + 6.0 k LL + 2.6 k SL

R_1 = 6.8k DL + 7.9 k LL + 3.3 k SL
R_2 = 1.0 k DL + 1.1 k LL + 0.5 k SL

$\boxed{\text{Use } 5\tfrac{1}{4}" \times 14" \text{ PSL}}$

V = 84% 12 kips
M = 67% 27 k-ft
Δ_T = L/466 0.40"

BM22B:



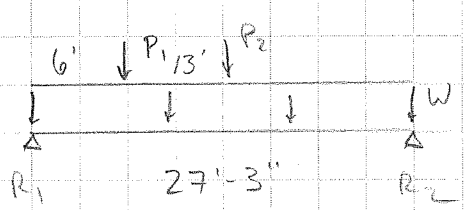
P_1 = 2.6 k DL + 3.0 k LL + 1.2 k SL
P_2 = 879 DL + 879 LL + 550 SL
w = (196 DL + 375 LL + 78 SL) plf

R_1 = 4.3k DL + 5.6 k LL + 2.0 k SL
R_2 = 1.7 k DL + 3.1 k LL + .7 k SL

$\boxed{\text{Use } 5\tfrac{1}{4}" \times 11\tfrac{7}{8}" \text{ PSL}}$

M = 71% 20.7 k-ft
V = 66% 8.0 kips
Δ_T = L/275 0.67"

BM23:



w = (138 DL + 240 LL) plf
P_1 = 4.3k DL + 5.6 k LL + 2.0 k SL
P_2 = 6.8 k DL + 7.9 k LL + 3.3 k SL

R_1 = 8.7 k DL + 11.7 k LL + 3.3 k SL
R_2 = 6.2 k DL + 8.4 k LL + 2.1 k SL

$\boxed{\text{Use W16} \times 40}$

M = 91%
V = 21%
Δ_T = L/246

C.G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description Beam Design	By CCC	Date 7/12/23
		Checked	Date
		Scale	Sheet No.
	Project The Talman	Job No. 23154.10	

![WoodWorks SOFTWARE FOR WOOD DESIGN]	**COMPANY** July 21, 2023 14:42	**PROJECT** BM22

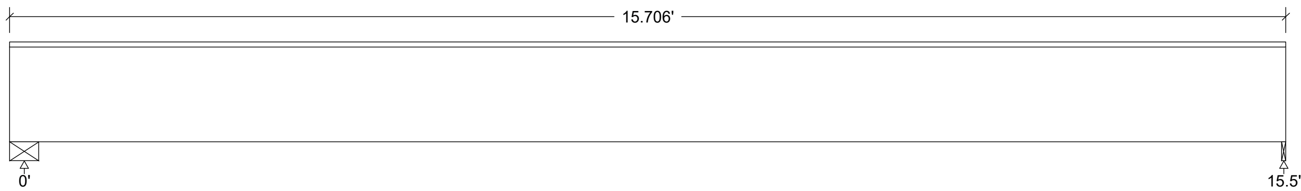
Design Check Calculation Sheet

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	fv* = 245	Fv' = 290	psi	fv*/Fv' = 0.84
Bending(+)	fb = 1890	Fb' = 2851	psi	fb/Fb' = 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#
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 = 14723, V design* = 11994 (NDS 3.4.3.1(a)) lbs; M(+) = 27006 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:

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.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	July 21, 2023 14:43	BM22B

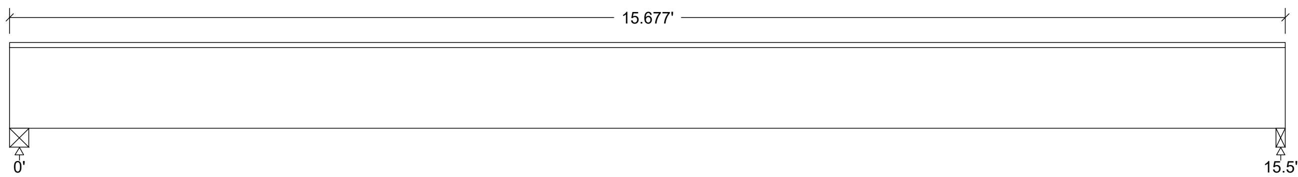
Design Check Calculation Sheet

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	fv* = 192	Fv' = 290	psi	fv*/Fv' = 0.66
Bending(+)	fb = 2015	Fb' = 2903	psi	fb/Fb' = 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#
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 #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-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:

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.

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING

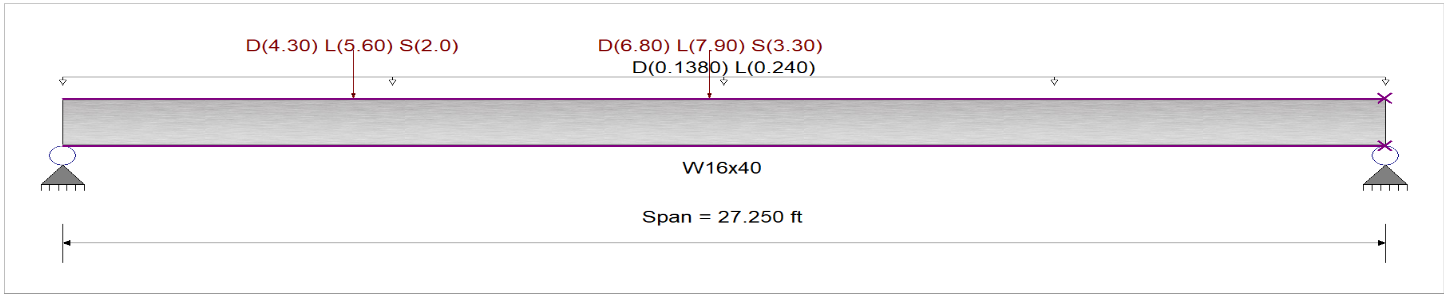
DESCRIPTION: BM23

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
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

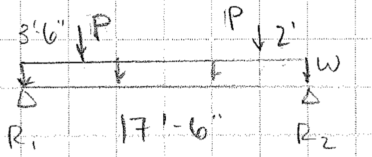
Max Downward Transient Deflection	0.752 in	Ratio =	434	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.329 in	Ratio =	246	>=240.	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<240.0	n/a

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:



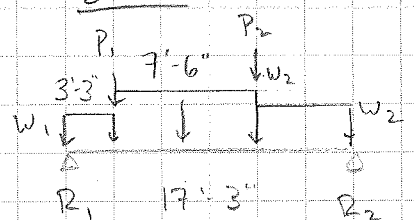
P = 5.1 k DL + 6.1 k LL + 2.3 k SL
w = 200 DL plf

R_1 = 6.4 k DL + 5.6 k LL + 2.1 k SL
R_2 = 7.3 k DL + 6.6 k LL + 2.5 k SL

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

M = 62%
V = 74%
Δ_T = L/384

BM14R:



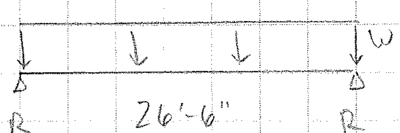
P_1 = 8.6 k DL + 10.4 k LL + 3.9 k SL
P_2 = 7 k DL + 7.7 k LL + 3.5 k SL
w_1 = 173 DL + 300 LL
w_2 = 414 DL + 720 LL
w_3 = 135 DL + 235 LL

$\boxed{\text{Use W12x35}}$

R_1 = 12.1 k DL + 15.8 k LL + 4.5 k SL
R_2 = 8 k DL + 10.4 k LL + 2.9 k SL

M = 88%. Δ_T = L/279

BM19B:



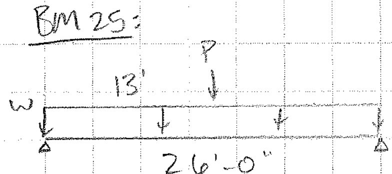
w = (729 DL + 890 LL + 319 SL) plf

R = 9.7 k DL + 11.8 k LL + 4.2 k SL

M = 79%. Δ_T = L/262

$\boxed{\text{Use W16x40}}$ V = 22%.

BM25:



P = 10.2 k DL + 12.2 k LL + 4.6 k SL
w = (144 DL + 250 LL)
R = 7.0 k DL + 9.4 k LL + 2.3 k SL

$\boxed{\text{Use W16x40}}$ M = 98%. Δ_T = L/256

C.G ENGINEERING
250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
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Description: Beam Design	By: CCC	Date: 7/20/23
	Checked	Date
	Scale	Sheet No.
Project: The Talmon	Job No.: 23154.10	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY July 21, 2023 14:47	PROJECT BM24

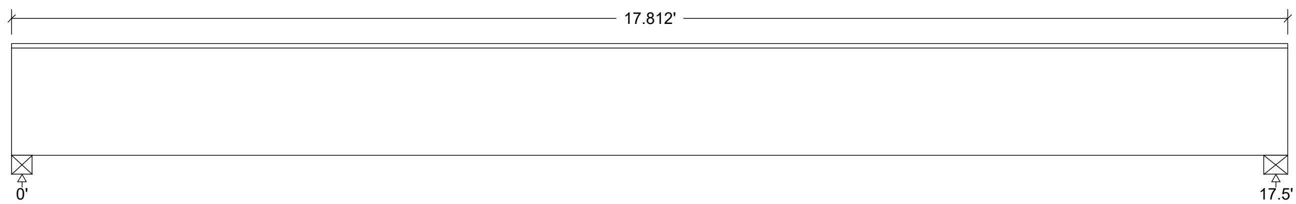
Design Check Calculation Sheet

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) :



	0'	17.5'
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:

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 (c) ENERCALC INC 1983-2023

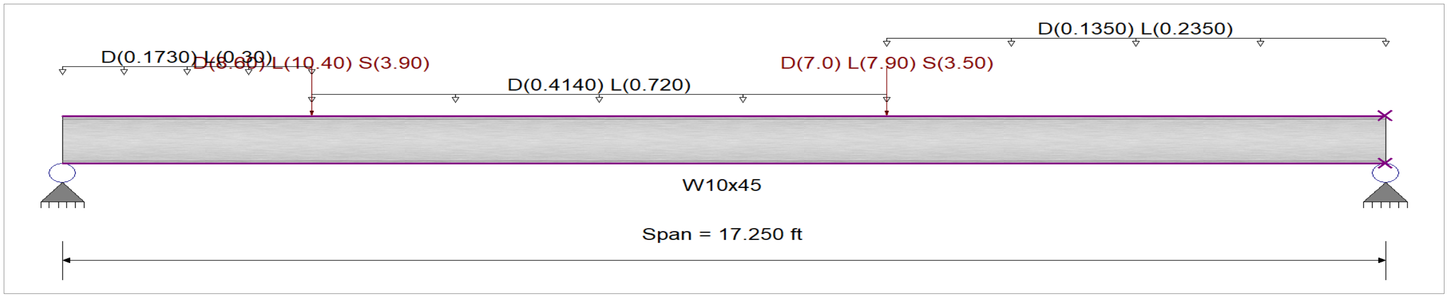
DESCRIPTION: BM14R

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
		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	0.480 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 #1 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

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

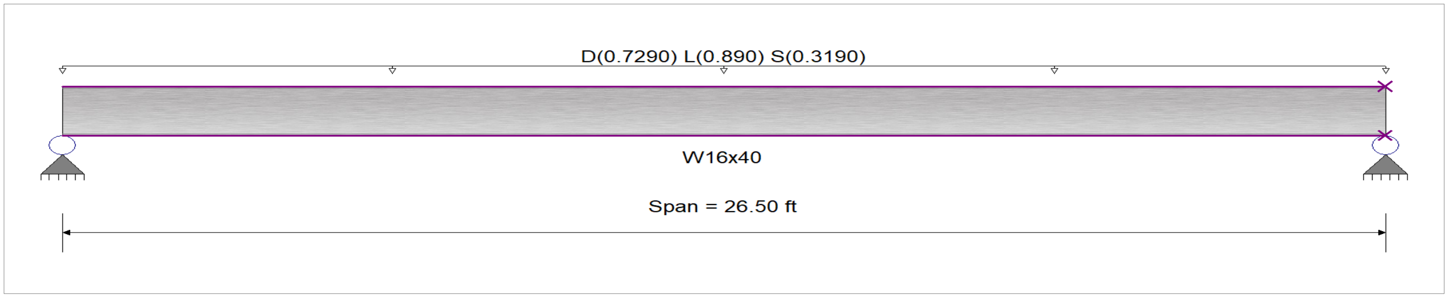
DESCRIPTION: BM19B

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 #1 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

Steel Beam

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

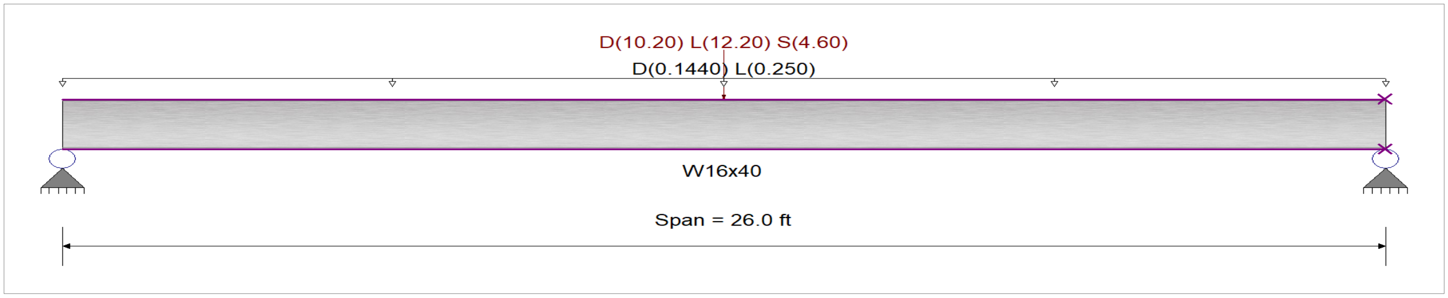
DESCRIPTION: BM25

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
		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

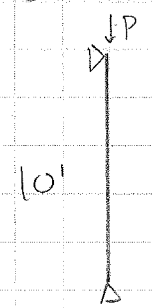
Vertical Reactions

Support notation : Far left is #1   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 kips ← Max Column Loading

Axial = 99%

Foundation Design:

Bearing Pressure = 4000 psf

Max Wall line Load = (1021 DL + 1415 LL + 304 SL) plf
ASD = 2436 plf

Typ Cont FTG = 1'-6" width → 6000 plf > 2436 plf

$\boxed{\text{Typ 1'-6" wide Cont FTG Adequate}}$

Pad FTG Design:

Max Point load on Cont FTG = 12 kips
If Point Load greater than 12k, use pad footing...

Size	Capacity
2'x2'	16 k
3'x3'	36 k
4'x4'	64 k

C G ENGINEERING
250 4th Ave. South
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Edmonds, WA 98020
425.778.8500
www.cgengineering.com

Description	By CCC	Date 7/21/23
	Checked	Date
	Scale	Sheet No.
Project The Talman	Job No. 23154.10	

Steel Column

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

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**	Overall Column Height	10.0 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade		Brace condition :	
Fy : Steel Yield	36.0 ksi		
E : Elastic Bending Modulus	29,000.0 ksi		

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 =	**0.9932**	: 1
Load Combination	D Only	
Location of max.above base	0.0	ft
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
PASS Maximum Shear Stress Ratio	**0.0**	: 1
Load Combination	0.0	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Va : Applied	0.0	k
Vn / Omega : Allowable	0.0	k

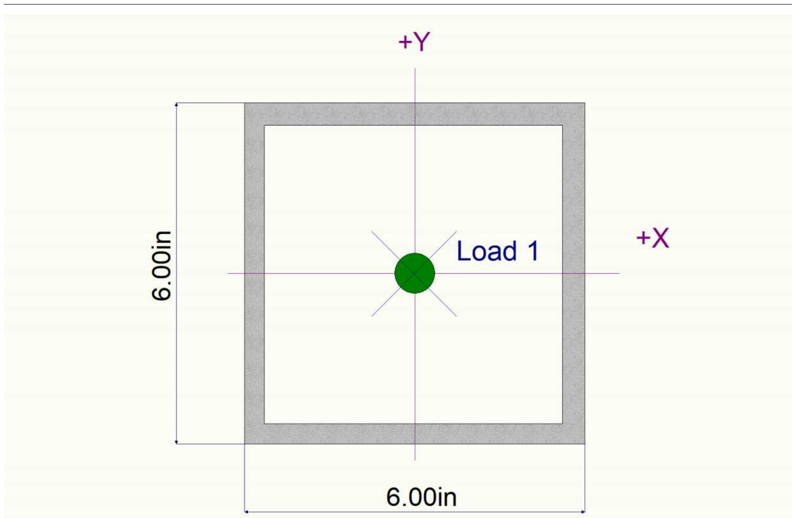
Maximum Load Reactions . .

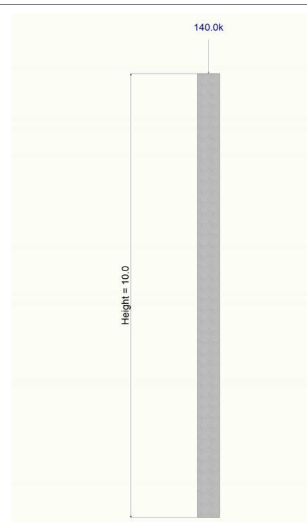
Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

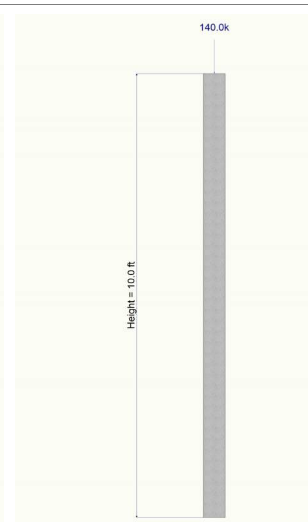
Maximum Load Deflections . . .

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 :				

Sketches







SECTION NO. 2

The Talmon
Centre Street
La Conner, WA 98257

SECTION TITLE: Main Building Lateral Design

SECTION INDEX

250 4th Ave S Edmonds, WA 98020
p. 425.778.8500 f. 425.778.5536

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

From Computer Program:
Short & 1-Sec Period Mapped
Acceleration Parameters (MCE)

S_S =	1.200	Lat. =	48.393	
S_1 =	0.427	Long. =	-122.493	
$S_{MS} = F_aS_S$ =	1.440	F_a =	1.200	ASCE 7-16 (Eq. 11.4-1)
$S_{M1} = F_vS_1$ =	0.811	F_v =	1.900	ASCE 7-16 (Eq. 11.4-2)
$S_{DS} = (2/3)S_{MS}$ =	0.960			ASCE 7-16 (Eq. 11.4-3)
$S_{D1} = (2/3)S_{M1}$ =	0.541			ASCE 7-16 (Eq. 11.4-4)
$T_a = C_th_n^x$ =	0.27	C_t =	0.02	ASCE 7-16 (Table 12.8-2)
		h_n =	32	
		x =	0.75	ASCE 7-16 (Table 12.8-2)
R Factor =	6.5	(Wood shear walls)		ASCE 7-16 (Table 12.2-1)
I_E Factor =	1.0			ASCE 7-16 (Table 1.5-2)

Seismic Base Shear

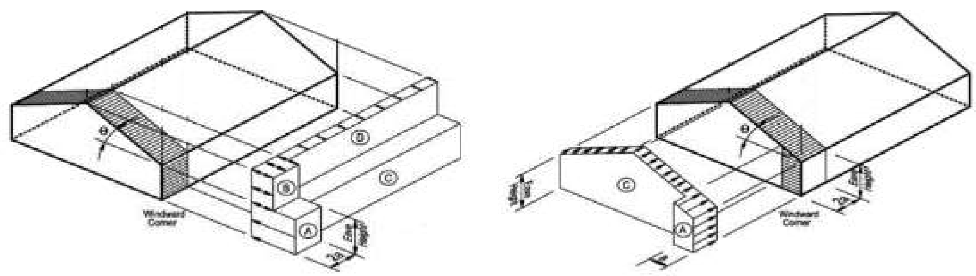
$V = 0.044S_{DS}I$ =	0.042 W	(Minimum Force)	ASCE 7-16 (Eq. 12.8-5)
$V = (S_{DS}IW)/R$ =	**0.148 W**	**(Governing Force)**	ASCE 7-16 (Eq. 12.8-2)
$V = (S_{D1}IW)/RT_a$ =	0.309 W	(Maximum Force)	ASCE 7-16 (Eq. 12.8-3)

C.G ENGINEERING 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	Section 1609.4	Section 26.7.3
Basic Wind Speed	V= 98		Per Jurisdiction
Risk Category	I_W= II		Table 1.5-1
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		Figure 28.6-1
Roof Angle	Angle= 0.0		
Design Wind Pressure, p_{s3}	p_{s30A}= 15.3		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30B}= -7.9		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30C}= 10.1		Figure 28.6-1
Design Wind Pressure, p_{s3}	p_{s30D}= -4.7		Figure 28.6-1
Height/Exposure Adjustm	λ_{max}= 1.40		
Topo. Effect Coeff., K_{zt}	K_{zt}= 1.00		

$V_{asd} = V_{ult} * \sqrt{0.6}$ Section 1609.3.1



Design Wind Load	ULT	ASD
	$p_s=\lambda*Kzt*p_{s30}$	$p_s=\lambda*Kzt*ps30*0.6$
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
	$p_s=\lambda*Kzt*p_{s30}$	$p_s=\lambda*Kzt*ps30*0.6$
$p_{s30A,C}$=	16.0	9.6
$p_{s30B,D}$=	8.0	4.8

C G ENGINEERING 250 4th Ave South Suite 200 Edmonds, WA 98020	Description: Wind Summary	By: ERH Checked: Scale: NTS	Date: 7/6/2023 Date: Sheet No.
	Project: The Talmon	Job No. 23154.10	

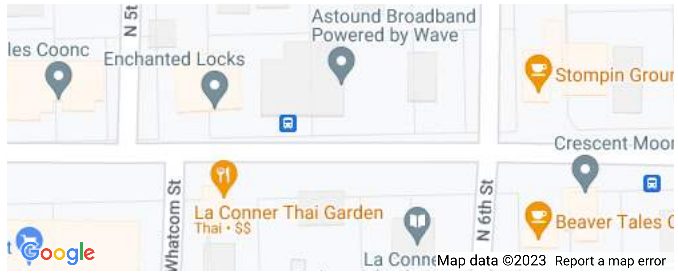
⚠ This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

ℹ The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address:	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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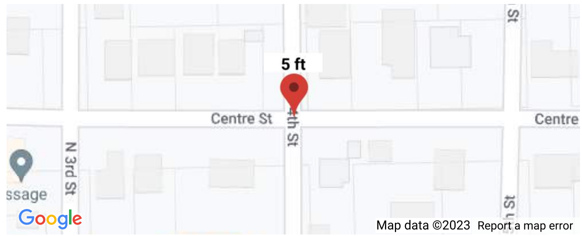
ATC Hazards by Location

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)
SsRT	1.199	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.32	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.427	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.477	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	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_1, 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.

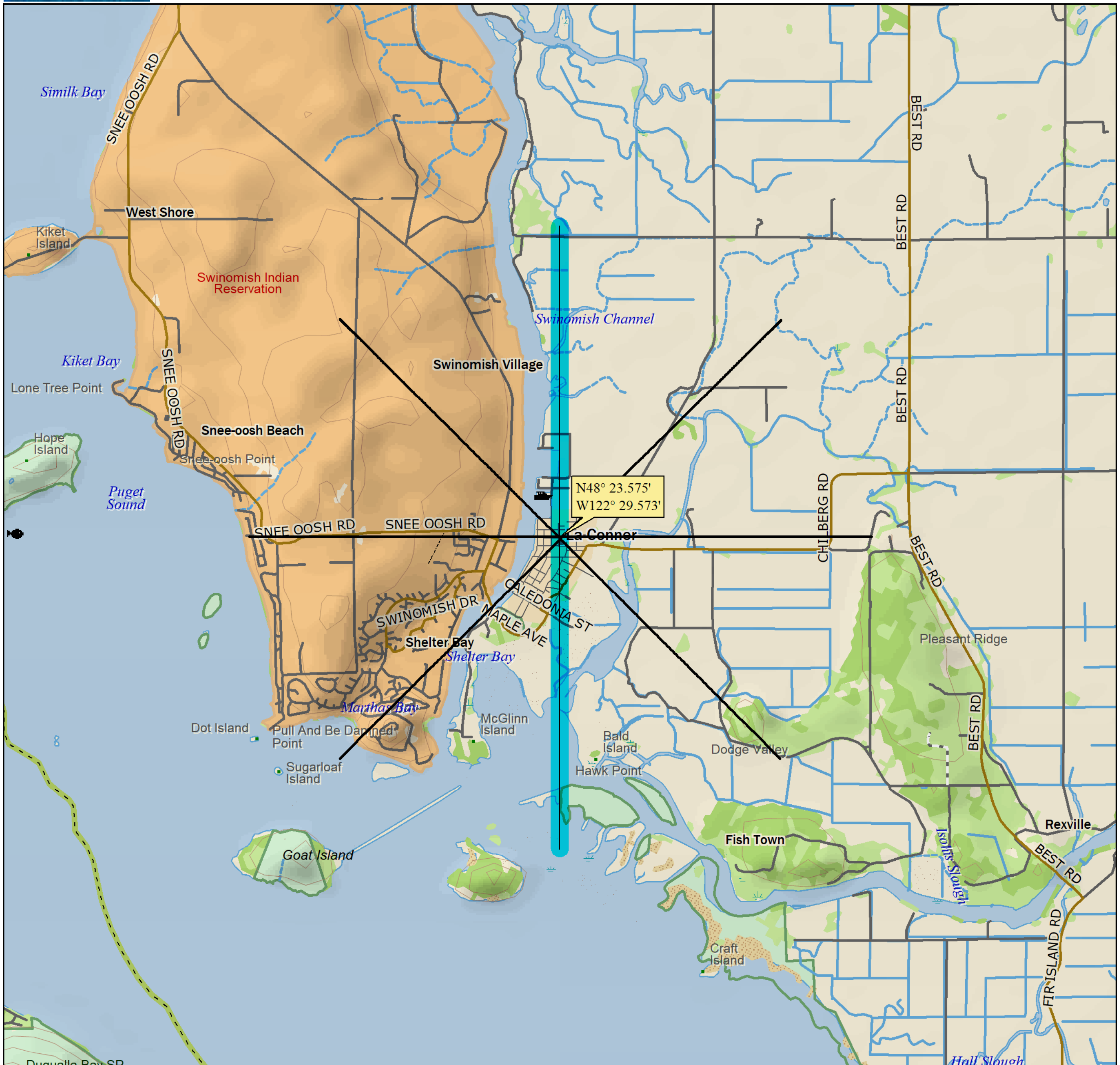
To evaluate the liquefaction potential of the site, we analyzed the following factors:

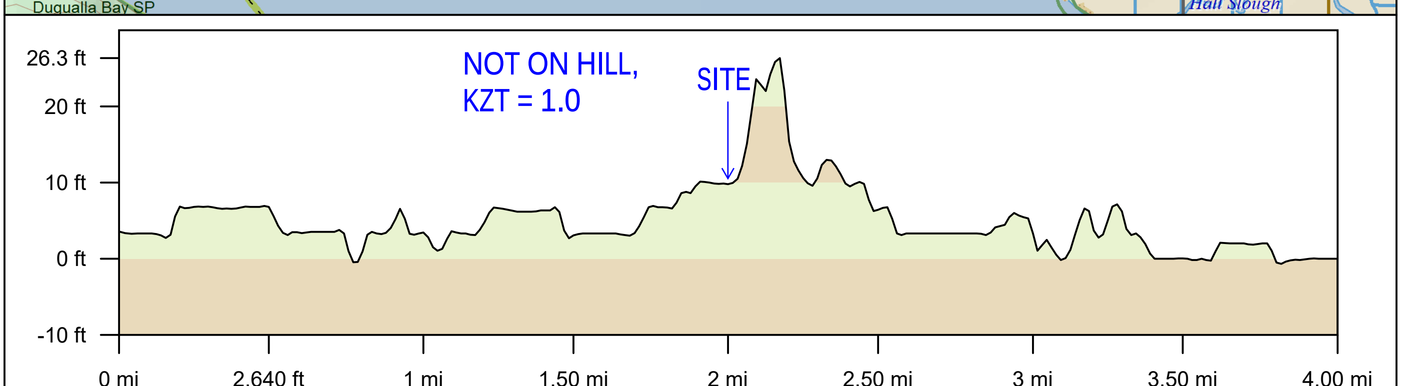
1) Soil type and plasticity
2) Groundwater depth
3) Relative soil density
4) Initial confining pressure
5) Maximum anticipated intensity and duration of ground shaking

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.

DELORME Topo North America™ 10



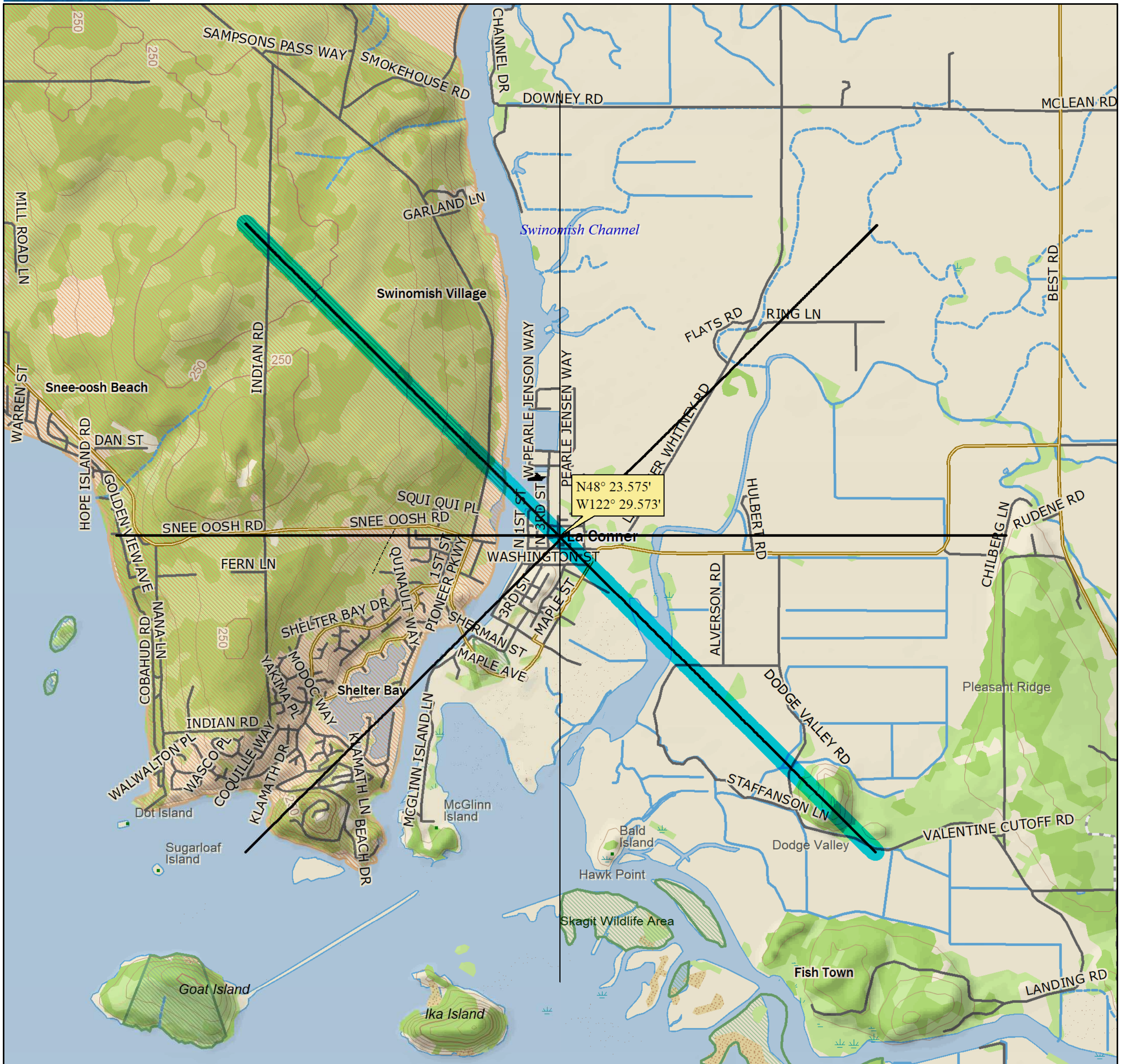


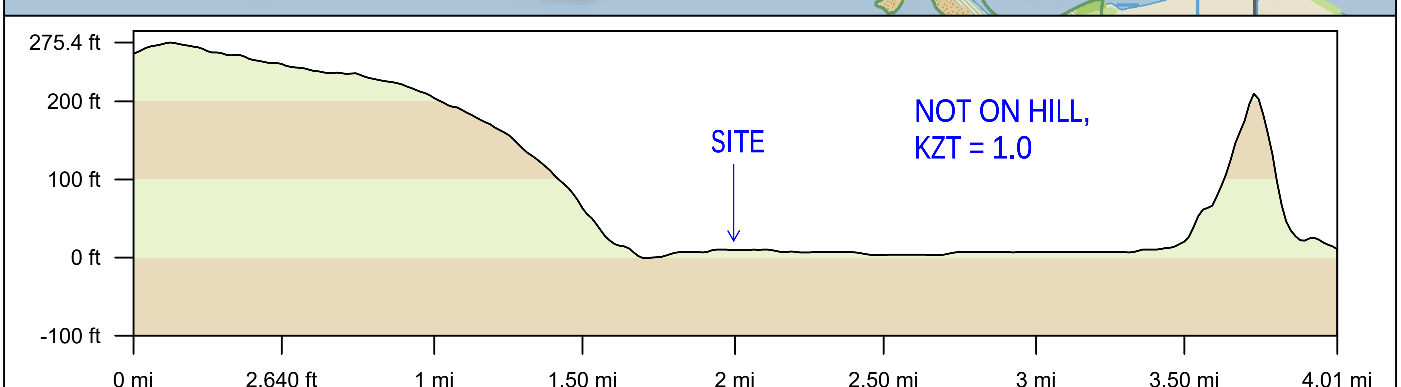
Lin Dist: 4.0 mi	Terr Dist: 4.0 mi	Elev Gain: -3.5 ft	Avg Grade: 0
Climb Elev: 70.2 ft	Desc Elev: 73.8 ft	Max. Elev: 26.3 ft	Min. Elev: -0.7 ft
Climb Dist: 1.7 mi	Desc Dist: 1.8 mi		

TN MN (14.7°E)

Scale 1 : 62,500

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

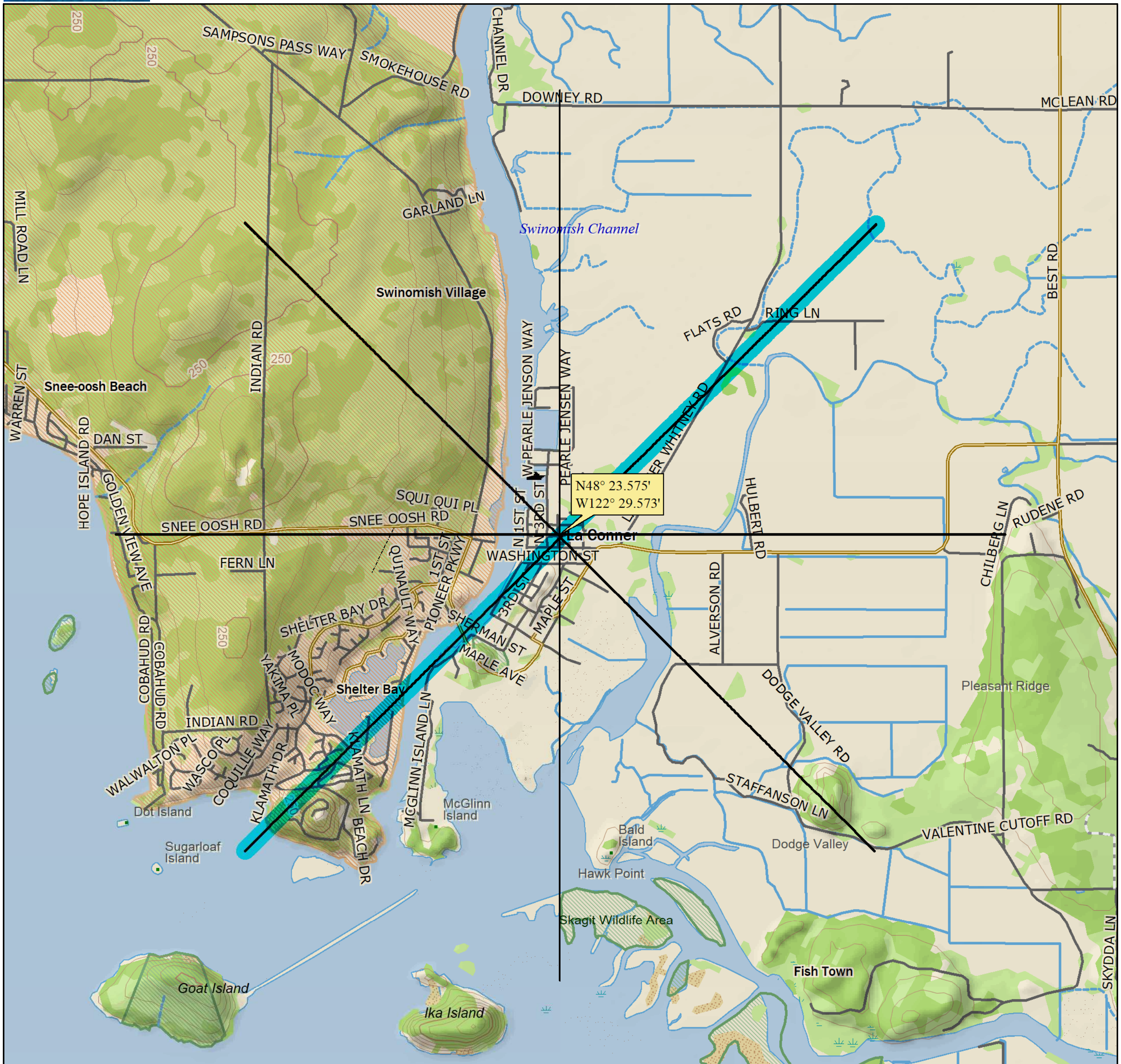


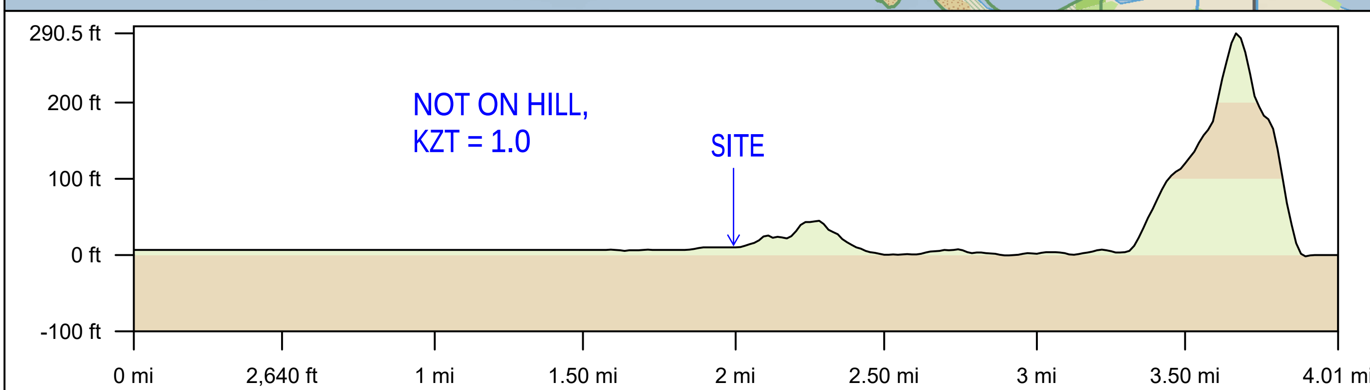


Lin Dist: 4.0 mi	Terr Dist: 4.0 mi	Elev Gain: -250.7 ft	Avg Grade: 3
Climb Elev: 242.0 ft	Desc Elev: 492.7 ft	Max. Elev: 275.4 ft	Min. Elev: -0.9 ft
Climb Dist: 1.3 mi	Desc Dist: 2.2 mi		

TN
MN (14.7°E)

Scale 1 : 43,750
1" = 3,645.8 ft
Data Zoom 12-2

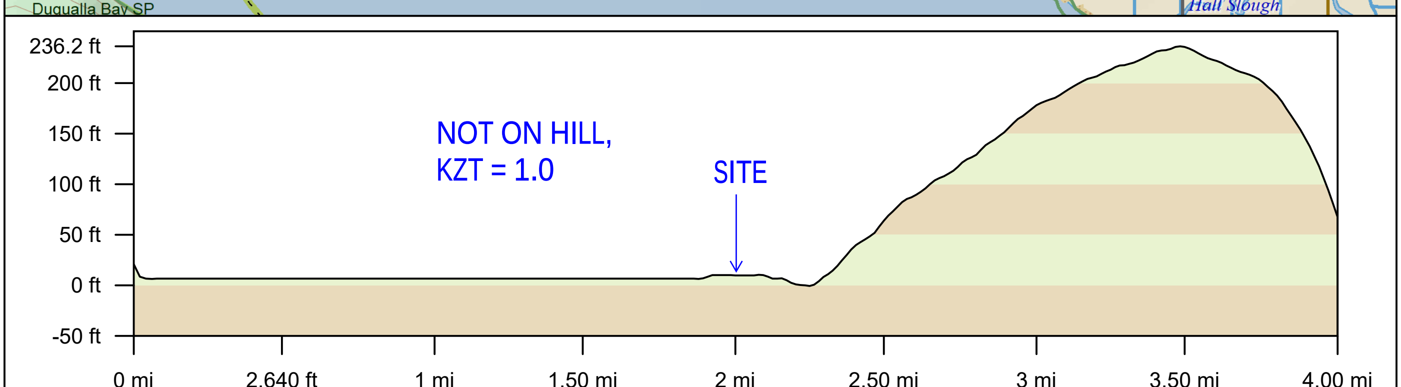




Lin Dist: 4.0 mi	Terr Dist: 4.0 mi	Elev Gain: -6.6 ft	Avg Grade: 3
Climb Elev: 355.9 ft	Desc Elev: 362.4 ft	Max. Elev: 290.5 ft	Min. Elev: -1.7 ft
Climb Dist: 1.3 mi	Desc Dist: 1.1 mi		

TN
MN (14.7°E)

Scale 1 : 43,750
1" = 3,645.8 ft
Data Zoom 12-2



Lin Dist: 4.0 mi	Terr Dist: 4.0 mi	Elev Gain: 46.4 ft	Avg Grade: 2
Climb Elev: 241.6 ft	Desc Elev: 195.2 ft	Max. Elev: 236.2 ft	Min. Elev: -0.6 ft
Climb Dist: 1.4 mi	Desc Dist: 4,473.6 ft		

TN
MN (14.7°E)

Scale 1 : 62,500

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

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)	$\frac{w_x * h_x^k}{\Sigma w_i * h_i^k}$	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)	Σw_i (kips)	F_i (kips)	ΣF_i (kips)	$\frac{\Sigma F_i * w_{px}}{\Sigma w_i}$	F_{px} (Min) $0.2S_{DS}Iw_{px}$	F_{px} (Max) $0.4S_{DS}Iw_{px}$	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**

ENGINEERING 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

Diaphragm Level	Story Height (ft)	Elevation (ft)	B (ft)	L (ft)	X-Dir p,asd (K)	Y-Dir p,asd (K)
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

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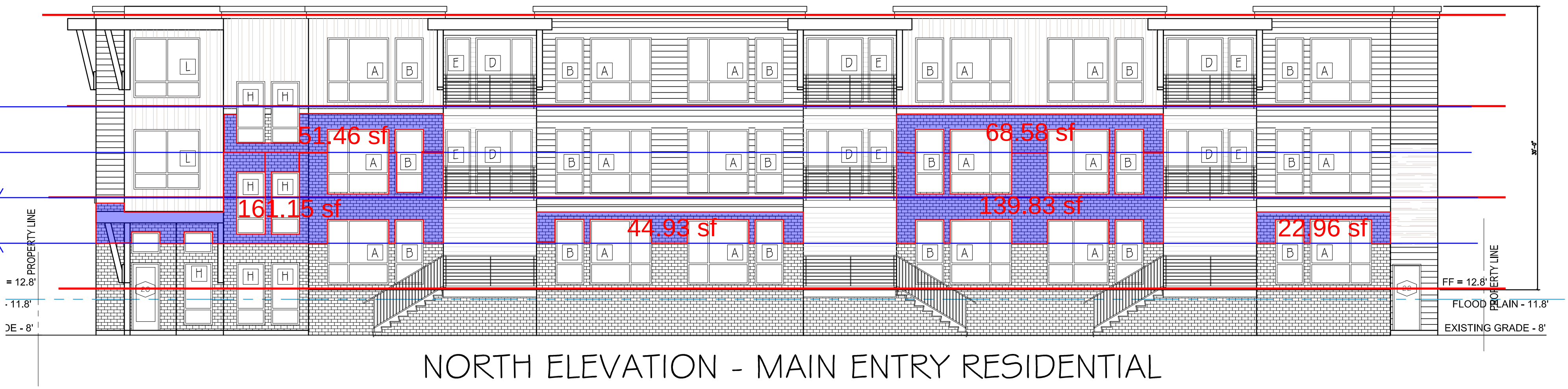
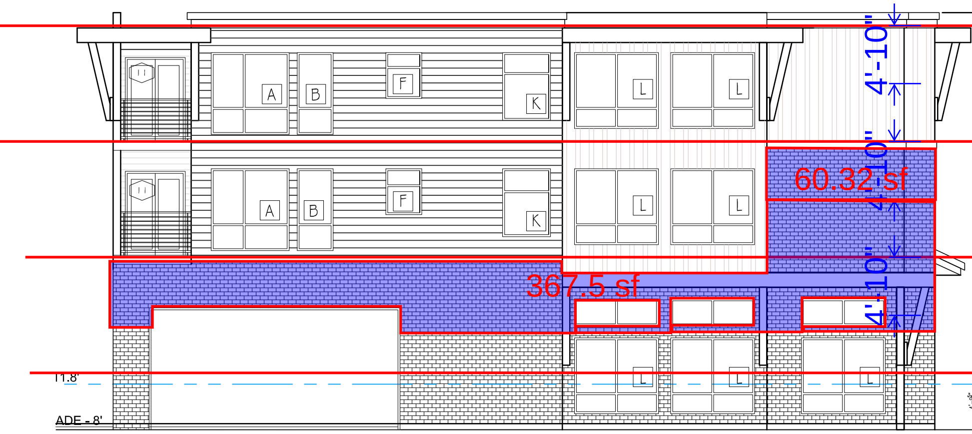
250 4th Ave. South
Suite 200
Edmonds, WA 98020

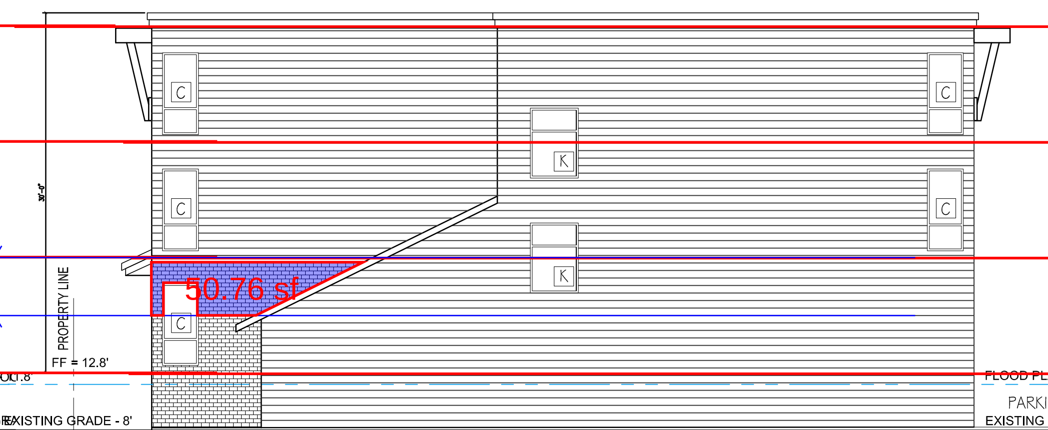
Description	Wind Design	By	ERH	Date	07/17/23
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	The Talmon	Job No.	23154.10		



NORTH ELEVATION - MAIN ENTRY RESIDENTIAL



EAST ELEVATION - GARAGE ENTRY



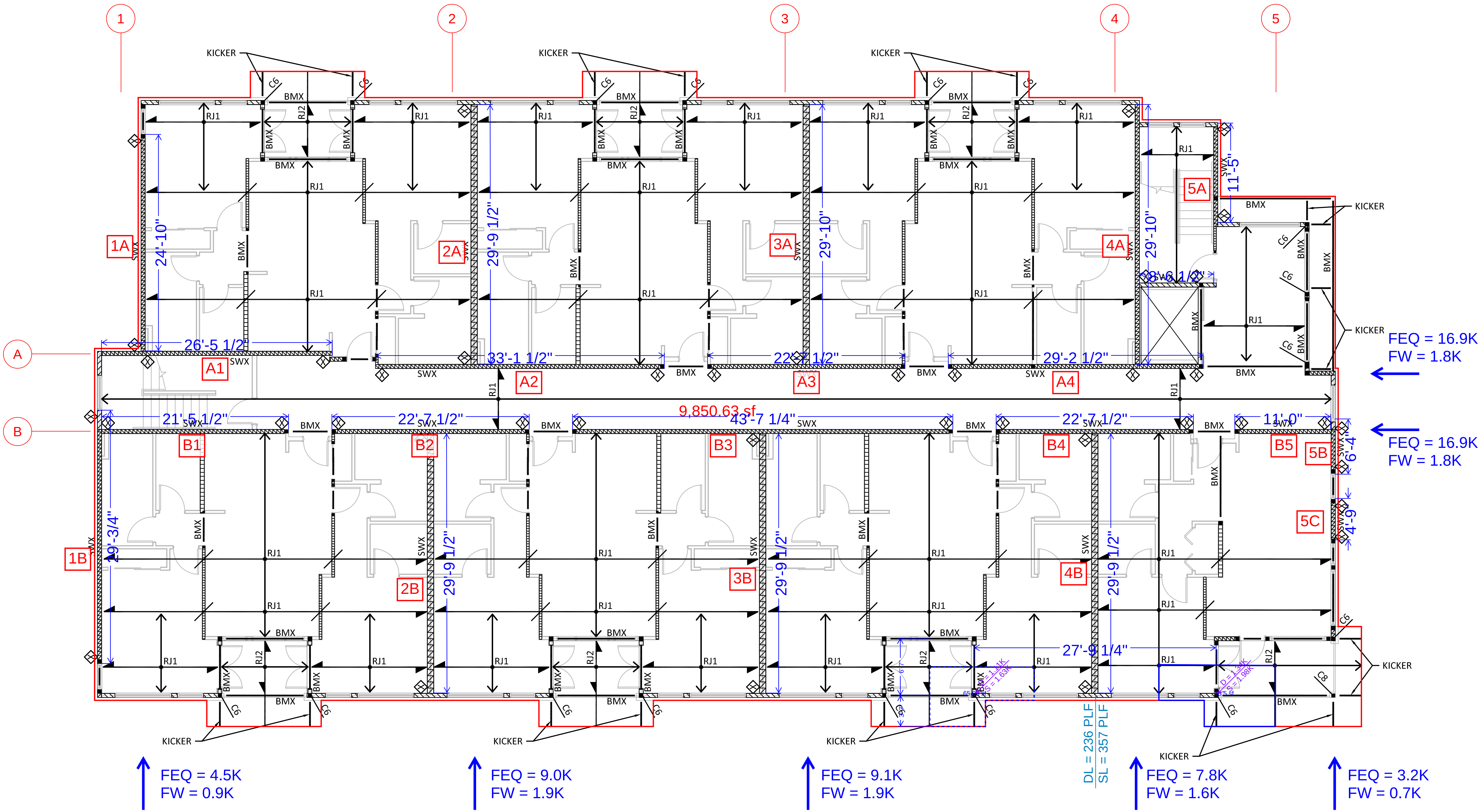
WEST ELEVATION

BRICK AREA KEY PLAN

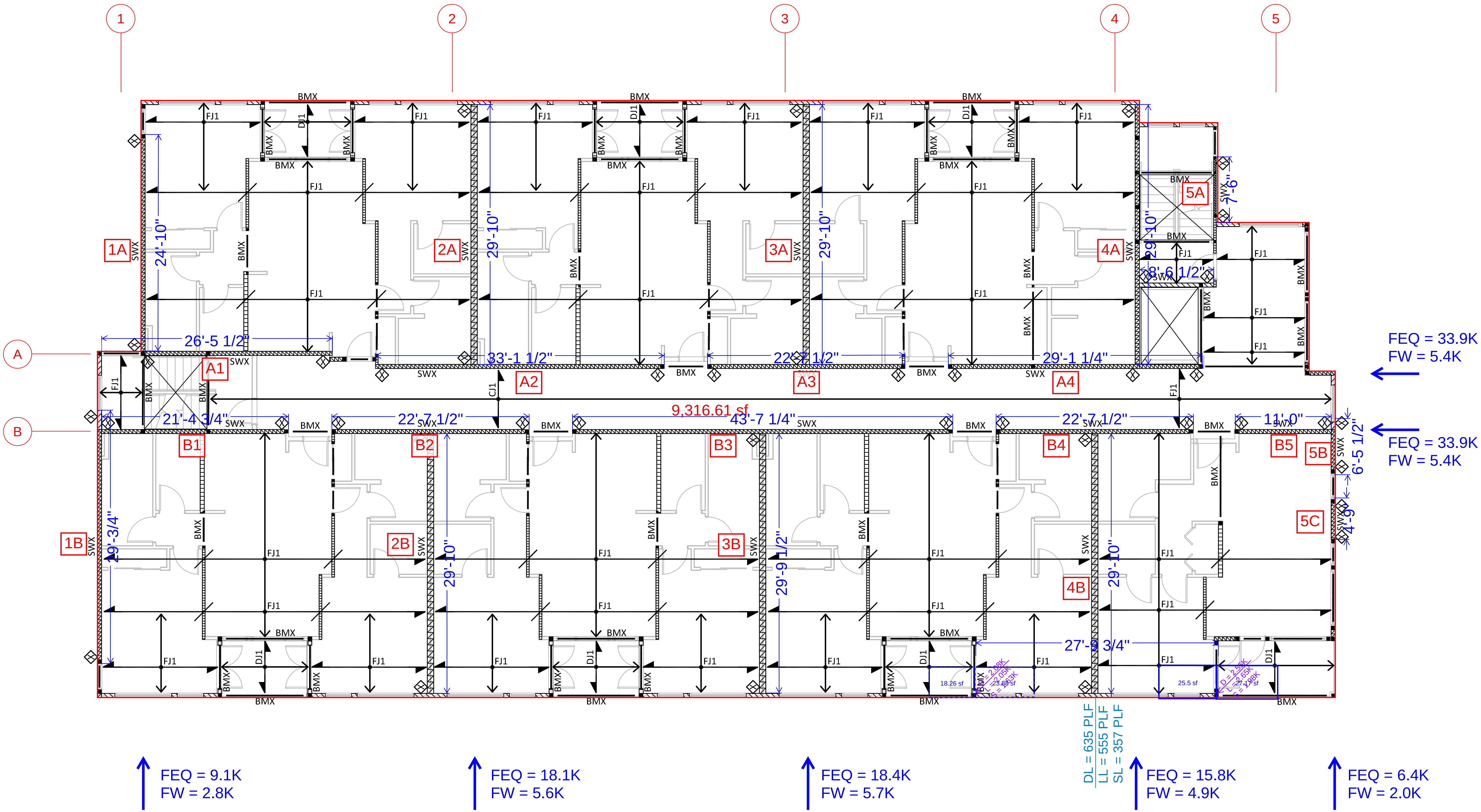
BRICK AREA = 967 SF
BRICK DENSITY = 4.5LB/65.25CI = 0.069PCI = 119 PCF
2" BRICK VENEER = 119 PCF x (2"/12IN/FT) = 19.83 PSF
BRICK WEIGHT = 967 SF x 19.83 PSF = 19.18 K

SEISMIC FORCE = 0.148 x 19.18K = 2.84K
--> APPLY AT LOWER LEVEL

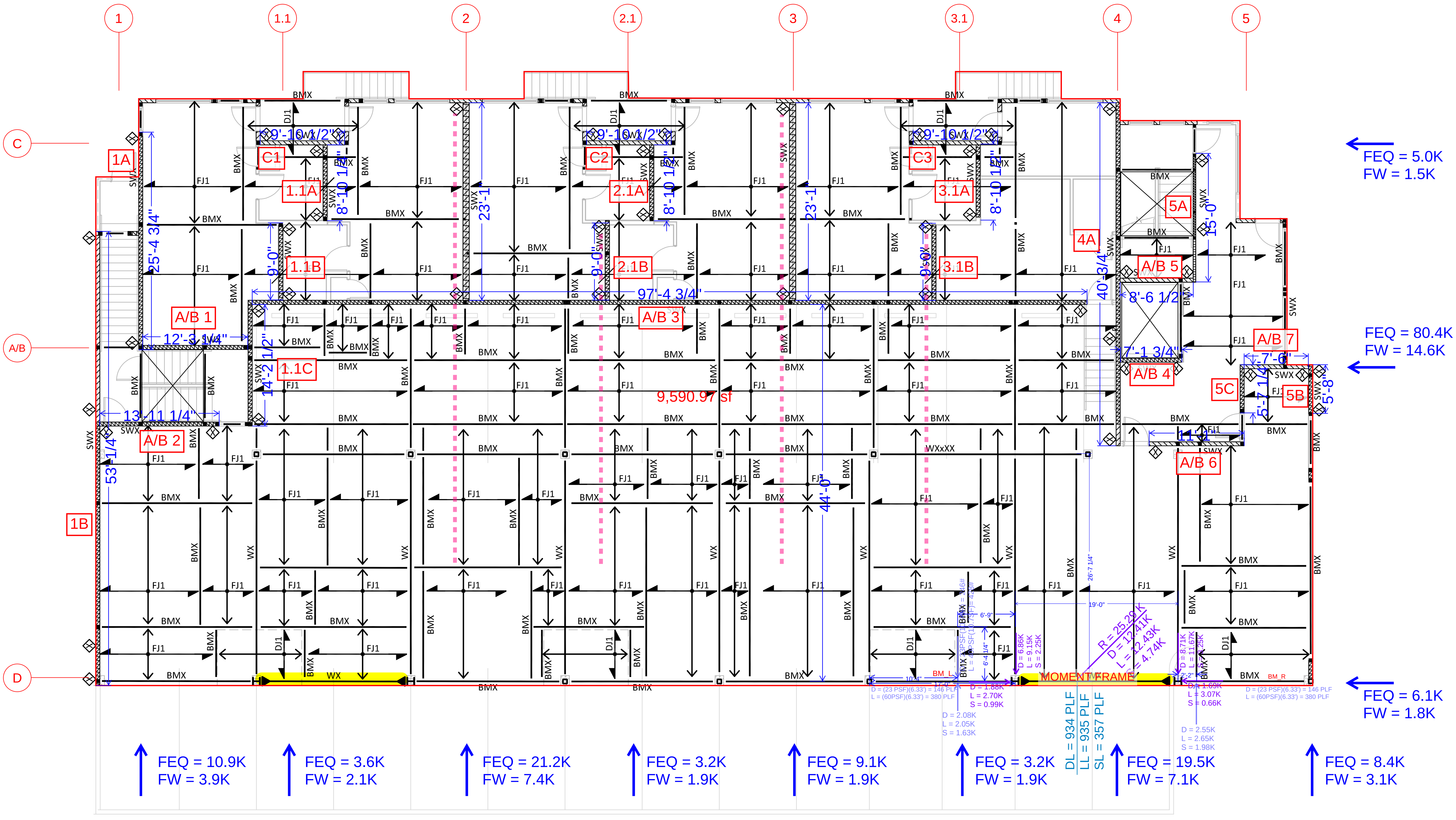
UPPER FLOOR LATERAL KEY PLAN



MAIN FLOOR LATERAL KEY PLAN



LOWER FLOOR LATERAL KEY PLAN



2.015

Upper Floor Shear Walls - Walls Below the Roof Framing

X - Direction Walls

Fx (EQ) =	33.7	kips (Story Shear)	
Fx (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

Wall Line	Wall Mark	SW Length	Trib Width	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6-0.14S_{DS})$DL End i	EQ End j	Wind 0.6 * DL End i	Wind End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
A	1	26.46	1	1.0	141	11	SW6	26.0	1.4	0.1	0.9	0.9	1.2	1.2	0.5	0.5	None	None	16.9	1.8	3.5
	2	33.125	-	1.0	141	11	SW6	32.6	1.4	0.1	1.1	1.1	1.5	1.5	0.3	0.3	None	None	-	-	3.5
	3	22.625		1.0	141	11	SW6	22.1	1.4	0.1	0.8	0.8	1.0	1.0	0.6	0.6	MST37	MST37	-	-	3.5
	4	29.21		1.0	141	11	SW6	28.7	1.4	0.1	1.0	1.0	1.3	1.3	0.4	0.4	None	None	-	-	3.5
	5	8.54	-	1.0	141	11	SW6	8.0	1.4	0.2	0.2	0.2	0.2	0.2	1.3	1.3	MST37	MST37	-	-	0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
B	1	21.42	1	1.0	139	11	SW6	20.9	1.4	0.1	0.7	0.7	0.9	0.9	0.6	0.6	MST37	MST37	16.9	1.8	3.5
	2	22.625	-	1.0	139	11	SW6	22.1	1.4	0.1	0.8	0.8	1.0	1.0	0.6	0.6	MST37	MST37	-	-	3.5
	3	43.58	-	1.0	139	11	SW6	43.1	1.4	0.1	1.5	1.5	1.9	1.9	0.0	0.0	None	None	-	-	3.5
	4	22.625	-	1.0	139	11	SW6	22.1	1.4	0.1	0.8	0.8	1.0	1.0	0.6	0.6	MST37	MST37	-	-	3.5
	5	11	-	1.0	139	11	SW6	10.5	1.4	0.2	0.4	0.4	0.5	0.5	1.0	1.0	MST37	MST37	-	-	3.5
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
		Σ	2.0																33.7	3.6	

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

C G ENGINEERING

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	

Upper Floor Shear Walls - Walls Below the Roof Framing

Y - Direction Walls

Fy (EQ) =	33.7	kips (Story Shear)	
Fy (wind) =	7.0	kips (Story Shear)	
Wy =	237	PLF	seismic
Wy =	49	PLF	wind

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

Wall Line	Wall Mark	SW Length	Trib Width	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6\text{-}0.14S_{DS})$DL End i	EQ $(0.6\text{-}0.14S_{DS})$DL End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
1	A	24.83	19	1.0	84	12	SW6	24.3	0.8	0.2	1.0	1.0	1.3	1.3	0.0	0.0	None	None	4.5	0.9	5.0
	B	29	-	1.0	84	12	SW6	28.5	0.8	0.2	1.2	1.2	1.5	1.5	0.0	0.0	None	None	-	-	5.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
2	A	29.8	38	1.0	151	22	SW6	29.3	1.5	0.3	2.0	2.0	2.5	2.5	0.0	0.0	None	None	9.0	1.9	11.5
	B	29.8	-	1.0	151	22	SW6	29.3	1.5	0.3	2.0	2.0	2.5	2.5	0.0	0.0	None	None	-	-	11.5
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
3	A	29.8	38.5	1.0	153	23	SW6	29.3	1.5	0.3	2.0	2.0	2.5	2.5	0.0	0.0	None	None	9.1	1.9	11.5
	B	29.8	-	1.0	153	23	SW6	29.3	1.5	0.3	2.0	2.0	2.5	2.5	0.0	0.0	None	None	-	-	11.5
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
4	A	29.8	33	1.0	131	19	SW6	29.3	1.3	0.3	1.8	1.8	2.3	2.3	0.0	0.0	None	None	7.8	1.6	10.0
	B	29.8	-	1.0	131	19	SW6	29.3	1.3	0.3	1.8	1.8	2.3	2.3	0.0	0.0	None	None	-	-	10.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
5	A	11.42	13.5	1.0	142	21	SW6	10.9	1.4	0.3	0.2	0.2	0.3	0.3	1.2	1.2	MST37	MST37	3.2	0.7	0.0
	B	6.42	-	1.0	142	21	SW6	5.9	1.5	0.3	0.3	0.3	0.4	0.4	1.2	1.2	MST37	MST37	-	-	6.5
	C	4.75	-	1.0	142	21	SW6	4.3	1.5	0.3	0.2	0.2	0.3	0.3	1.3	1.3	MST37	MST37	-	-	6.5
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
								-0.5			0.0	0.0	0.0	0.0							0.0
		Σ	**142.0**																**33.7**	**7.0**	

Input Cell

Input Cell w/ Formula

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

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

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 From Above EQ	Line Load From Above Wind	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6\text{-}0.14S_{DS})$DL End i	EQ $(0.6\text{-}0.14S_{DS})$DL End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Net Uplift From Above End i	Net Uplift From Above End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
			kips
Re-Calc	-	6.5	kips

Input Cell

Input Cell w/ Formula

C G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020	Description: Main Floor Shear Walls X-Direction	By: ERH	Date: 07/17/23
		Checked	Date
		Scale: NTS	Sheet No.
	Project: The Talmon	Job No. 23154.10	

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 From Above EQ	Line Load From Above Wind	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6\text{-}0.14S_{DS})$DL End i	EQ $(0.6\text{-}0.14S_{DS})$DL End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Net Uplift From Above End i	Net Uplift From Above End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
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
	B	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	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
2	A	29.8	38	9.0	1.9	1.0	304	67	SW4	29.3	3.0	0.9	2.4	2.4	3.1	3.1	0.0	0.0	0.6	0.6	MST37	MST37	18.1	5.6	11.5
	B	29.8	-	-	-	1.0	304	67	SW4	29.3	3.0	0.9	2.4	2.4	3.1	3.1	0.0	0.0	0.6	0.6	MST37	MST37	-	-	11.5
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
3	A	29.8	38.5	9.1	1.9	1.0	308	68	SW4	29.3	3.0	0.9	2.4	2.4	3.1	3.1	0.0	0.0	0.6	0.6	MST37	MST37	18.4	5.7	11.5
	B	29.8	-	-	-	1.0	308	68	SW4	29.3	3.0	0.9	2.4	2.4	3.1	3.1	0.0	0.0	0.6	0.6	MST37	MST37	-	-	11.5
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
4	A	29.8	33	7.8	1.6	1.0	264	58	SW4	29.3	2.6	0.8	2.2	2.2	2.8	2.8	0.0	0.0	0.4	0.4	None	None	15.8	4.9	10.0
	B	29.8	-	-	-	1.0	264	58	SW4	29.3	2.6	0.8	2.2	2.2	2.8	2.8	0.0	0.0	0.4	0.4	None	None	-	-	10.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
5	A	7.5	13.5	3.2	0.7	1.0	345	76	SW4	7.0	3.6	1.1	0.2	0.2	0.2	0.2	1.2	0.0	4.6	3.4	MST60	MST48	6.4	2.0	0.0
	B	6.42	-	-	-	1.0	345	76	SW4	5.9	3.6	1.1	0.4	0.4	0.5	0.5	1.2	1.2	4.5	4.5	MST60	MST60	-	-	6.5
	C	4.75	-	-	-	1.0	345	76	SW4	4.3	3.7	1.2	0.3	0.3	0.3	0.3	1.3	1.3	4.8	4.8	MST60	MST60	-	-	6.5
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
		Σ	142.0	33.7	7.0																		67.8	21.0	

Input Cell
Input Cell w/ Formula

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

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

Lower Floor Shear Walls - Walls Below the Main Floor Framing

X - Direction Walls

Fx (EQ) =	23.7	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 =	346	PLF	seismic
Wx =	105	PLF	wind

Wall Line	Wall Mark	SW Length	Trib Width	Line Load From Above EQ	Line Load From Above Wind	EQ, WL 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ $(0.6\text{-}0.14S_{DS})$DL End i	EQ $(0.6\text{-}0.14S_{DS})$DL End j	Wind 0.6 * DL End i	Wind 0.6 * DL End j	Net Uplift From Above End i	Net Uplift From Above End j	Governing Net Uplift End i	Governing Net Uplift End j	Hold-down End i	Hold-down End j	EQ Line Load	Wind Line Load	DL Trib
A/B	1	12.25	36.5	67.8	10.8	1.0	509	66	SW2	11.8	5.1	0.9	0.2	0.2	0.3	0.3	2.2	2.2	7.1	7.1	HDU11	HDU11	80.4	14.6	0.0
	2	13.92	-	-	-	1.0	509	66	SW2	13.4	5.1	0.9	0.3	0.3	0.4	0.4	2.6	0.0	7.4	4.8	HDU11	HDU8	-	-	0.0
	3	97.4	-	-	-	1.0	509	66	SW2	96.9	5.0	0.9	2.0	2.0	2.5	2.5	0.0	0.0	3.0	3.0	HDU4	HDU4	-	-	0.0
	4	7.2	-	-	-	1.0	509	66	SW2	6.7	5.3	1.0	0.1	0.1	0.2	0.2	2.0	2.0	7.2	7.2	HDU11	HDU11	-	-	0.0
	5	8.54	-	-	-	1.0	509	66	SW2	8.0	5.2	1.0	0.2	0.2	0.2	0.2	4.0	4.0	9.1	9.1	HDU11	HDU11	-	-	0.0
	6	11.08	-	-	-	1.0	509	66	SW2	10.6	5.2	0.9	0.2	0.2	0.3	0.3	0.0	0.0	4.9	4.9	HDU8	HDU8	-	-	0.0
	7	7.5	-	-	-	1.0	509	66	SW2	7.0	5.3	1.0	0.2	0.2	0.2	0.2	0.0	0.0	5.1	5.1	HDU8	HDU8	-	-	0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
				~~33.9~~	~~5.4~~	0.0	NA	######	######	-0.5	######	######	0.0	0.0	0.0	0.0	2.6	2.6	####	####	######	######	33.9	5.4	0.0
			-	-	-	0.0	NA	######	######	-0.5	######	######	0.0	0.0	0.0	0.0	2.5	2.5	####	####	######	######	-	-	0.0
			-	-	-	0.0	NA	######	######	-0.5	######	######	0.0	0.0	0.0	0.0	1.0	1.0	####	####	######	######	-	-	0.0
			-	-	-	0.0	NA	######	######	-0.5	######	######	0.0	0.0	0.0	0.0	2.5	2.5	####	####	######	######	-	-	0.0
			-	-	-	0.0	NA	######	######	-0.5	######	######	0.0	0.0	0.0	0.0	3.6	3.6	####	####	######	######	-	-	0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
C	1	9.875	14.5	0.0	0.0	1.0	170	37	SW6	9.4	1.7	0.5	0.2	0.2	0.3	0.3	0.0	0.0	1.5	1.5	HDU2	HDU2	5.0	1.5	0.0
	2	9.875	-	-	-	1.0	170	37	SW6	9.4	1.7	0.5	0.2	0.2	0.3	0.3	0.0	0.0	1.5	1.5	HDU2	HDU2	-	-	0.0
	3	9.875	-	-	-	1.0	170	37	SW6	9.4	1.7	0.5	0.2	0.2	0.3	0.3	0.0	0.0	1.5	1.5	HDU2	HDU2	-	-	0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
D	1	1	17.5	0.0	0.0	0.2	NA	######	######	0.5	######	######	0.0	0.0	0.0	0.0	####	####	####	####	######	######	6.1	1.8	0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
																									0.0
									MOMENT FRAMES																0.0
																									0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			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
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
		Σ	68.5	101.7	16.2																		125.4	23.4	

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			
Nil	-	0	kips
None	-	0.5	kips
HDU2	(2)-2x HF	2.215	kips
HDU4	(2)-2x HF	3.3	kips
HDU5	(2)-2x HF	4.1	kips
HDU8	4x DF#2	7.0	kips
HDU11	6x6 DF#1	9.5	kips
HDU14	6x6 DF#1	14.4	kips
			kips
			kips
Re-Calc	-	14.5	kips

Input Cell
Input Cell w/ Formula

ENGINEERING	Description: Lower Floor Shear Walls	By: ERH	Date: 07/17/23
	X-Direction	Checked	Date
		Scale: NTS	Sheet No.
250 4th Ave. South Suite 200 Edmonds, WA 98020	Project: The Talmon	Job No.: 23154.10	

Lower Floor Shear Walls - Walls Below the Main Floor Framing

Y - Direction Walls

Fy (EQ) =	23.7	kips (Story Shear)
Fy (wind) =	14.0	kips (Story Shear)

Story HT =	10
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.96

Wy =	167 PLF	seismic
Wy =	99 PLF	wind

				Line Load							EQ	Wind	EQ		Wind		Net Uplift		Governing				EQ	Wind	
Wall	Wall	SW	Trib	From Above		EQ, WL	EQ	Wind	SW	Reduced	Gross	Gross	$(0.6\text{-}0.14S_{DS})$DL		0.6 * DL		From Above		Net Uplift		Hold-down		Line	Line	DL
Line	Mark	Length	Width	EQ	Wind	2w/h	Shear	Shear	Callout	HD Length	Uplift	Uplift	End i	End j	End i	End j	End i	End j	End i	End j	End i	End j	Load	Load	Trib
1	A	25.4	11	9.1	2.8	1.0	99	25	SW6	24.9	1.0	0.3	1.3	1.3	1.7	1.7	0.5	0.5	0.1	0.1	None	None	10.9	3.9	6.0
	B	53	-	-	-	1.0	99	25	SW6	52.5	1.0	0.3	1.9	1.9	2.5	2.5	0.3	0.3	0.0	0.0	None	None	-	-	3.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
1.1	A	8.83	21.5	0.0	0.0	1.0	99	25	SW6	8.3	1.0	0.4	0.5	0.5	0.6	0.6	0.0	0.0	0.5	0.5	HDU2	HDU2	3.6	2.1	6.5
	B	9	-	-	-	1.0	99	25	SW6	8.5	1.0	0.4	0.5	0.5	0.7	0.7	0.0	0.0	0.5	0.5	None	None	-	-	7.0
	C	14.2	-	-	-	1.0	99	25	SW6	13.7	1.0	0.4	0.6	0.6	0.8	0.8	0.0	0.0	0.4	0.4	None	None	-	-	4.5
2	A	23.08	18.5	18.1	5.6	1.0	519	130	SW2	22.6	5.1	1.8	1.9	1.9	2.4	2.4	0.6	0.6	3.8	3.8	HDU5	HDU5	21.2	7.4	11.5
										-0.5			0.0	0.0	0.0	0.0	0.6	0.6							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
2.1	A	8.83	19	0.0	0.0	1.0	519	130	SW2	8.3	5.3	1.9	0.5	0.5	0.6	0.6	0.0	0.0	4.8	4.8	HDU8	HDU8	3.2	1.9	6.5
	B	9	-	-	-	1.0	519	130	SW2	8.5	5.3	1.9	0.5	0.5	0.7	0.7	0.0	0.0	4.8	4.8	HDU8	HDU8	-	-	7.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
3	A	23.08	19	18.4	5.7	1.0	527	132	SW2	22.6	5.2	1.8	1.8	1.8	2.4	2.4	0.6	0.6	4.0	4.0	HDU5	HDU5	21.6	7.6	11.0
										-0.5			0.0	0.0	0.0	0.0	0.6	0.6							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
3.1	A	8.83	19	0.0	0.0	1.0	527	132	SW2	8.3	5.4	1.9	0.5	0.5	0.6	0.6	0.0	0.0	4.9	4.9	HDU8	HDU8	3.2	1.9	6.5
	B	9	-	-	-	1.0	527	132	SW2	8.5	5.4	1.9	0.5	0.5	0.7	0.7	0.0	0.0	4.9	4.9	HDU8	HDU8	-	-	7.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
4	A	40	22.5	15.8	4.9	1.0	488	127	SW2	39.5	4.8	1.7	2.1	2.1	2.7	2.7	0.4	0.4	3.1	3.1	HDU4	HDU4	19.5	7.1	6.0
										-0.5			0.0	0.0	0.0	0.0	0.4	0.4							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
5	A	15	11.5	6.4	2.0	1.0	319	85	SW4	14.5	3.2	1.2	0.5	0.5	0.6	0.6	0.0	3.4	2.7	6.1	HDU4	HDU8	8.4	3.1	2.5
	B	5.67	-	-	-	1.0	319	85	SW4	5.2	3.4	1.3	0.2	0.2	0.3	0.3	4.5	0.0	7.6	3.1	HDU11	HDU4	-	-	4.0
	C	5.58	-	-	-	1.0	319	85	SW4	5.1	3.4	1.3	0.2	0.2	0.3	0.3	0.0	0.0	3.2	3.2	HDU4	HDU4	-	-	4.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
										-0.5			0.0	0.0	0.0	0.0	####	####							0.0
		Σ	142.0	67.8	21.0																		91.5	35.0	

Input Cell
Input Cell w/ Formula

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

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

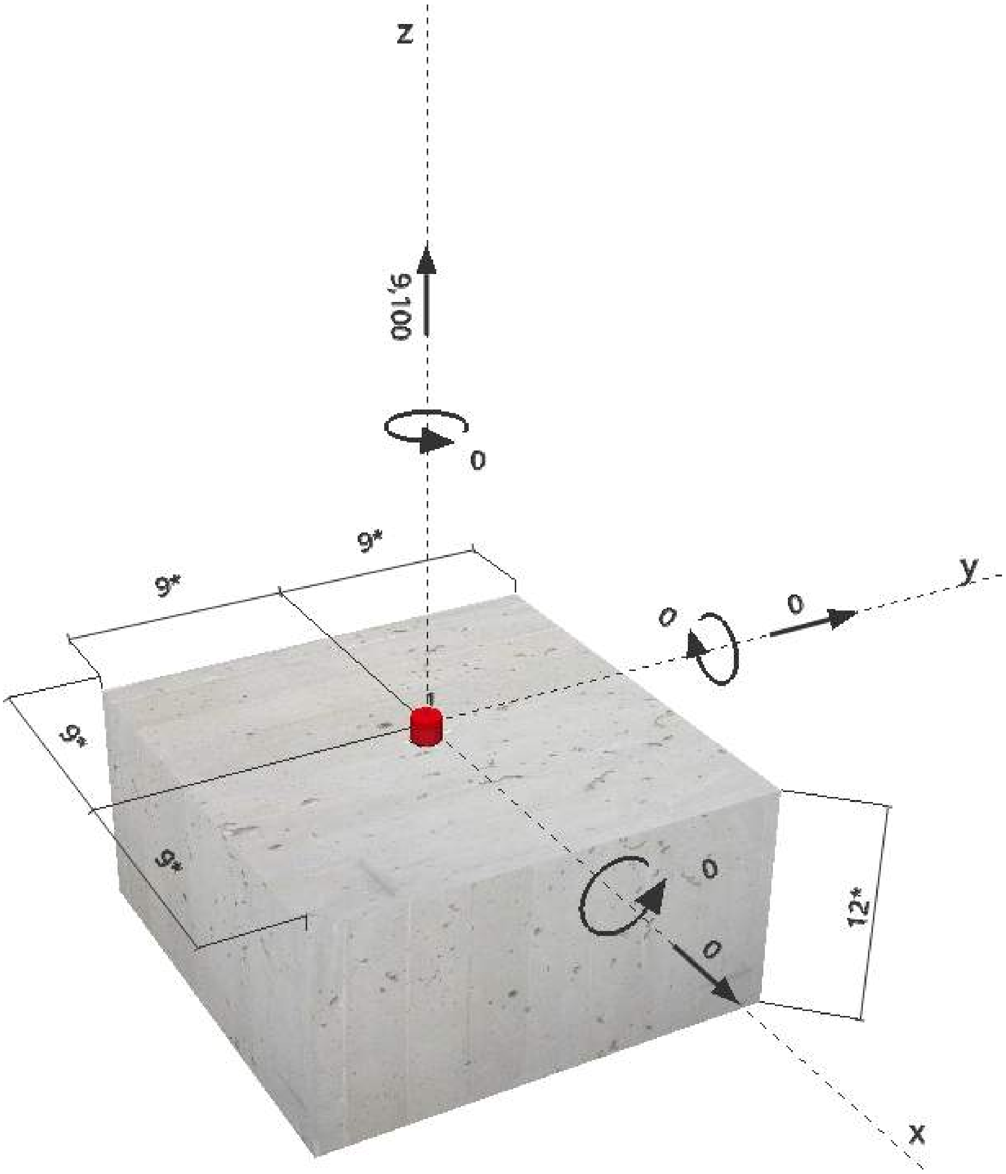
Company:	CG Engineeering	Page:	1
Specifier:		Project:	The Talmon
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	7/18/2023
E-Mail:			

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]



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

$N_{sa} = A_{se,N} f_{uta}$ 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]
0.61	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

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8 A_{brg} f'_c$ 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_{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

3.3 Concrete Breakout Strength

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

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

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

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

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

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

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

$N_b = k_c\ \lambda_a \sqrt{f_c'}\ h_{ef}^{1.5}$ 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]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
17636	0.700	0.750	1.000	9259	9100

Company: CG Engineeering
Specifier:
Address:
Phone I Fax: |
E-Mail:

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 Engineeering	Page:	5
Specifier:		Project:	The Talmon
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	7/18/2023
E-Mail:			

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

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
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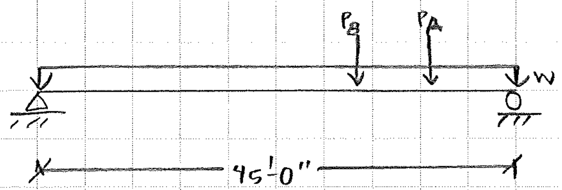
LATERAL DESIGN

CHECK DIAPHRAGM



$P_A = P_B = 33.9^k$ (ASD EQ)

w: $23.5k / 68.5' = 343$ PLF (ASD EQ)

$V_{max} = 59.321$ (SEE ENERCALC)

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

– Blocked Diaphragm Required –

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

Blocked Wood Structural Panel Diaphragms[1,2,3,4,5]

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.)	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): 6			6			4			3		
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	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
	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
Sheathing and Single-Floor	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
			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
	10d	1-1/2	15/32	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
			19/32	2	640	21	14	850	13	9.5	$\boxed{1280}$	18	12	1460	28	17
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15

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250 4th Ave. South
Suite 200
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Description: Lateral Design	By: SRH	Date: 7/13/13
	Checked:	Date:
	Scale: NTS	Sheet No.
Project: The Talmon	Job No. 23154.10	

General Beam Analysis

Project File: Talmon.ec6

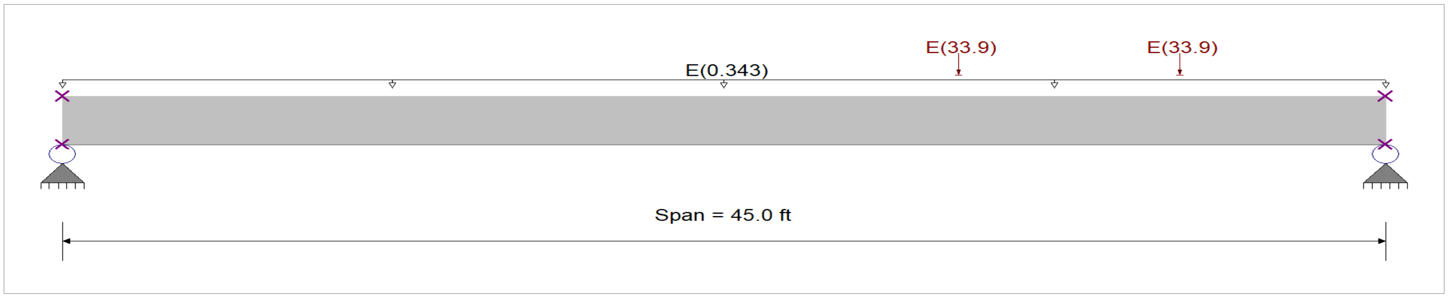
LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING

DESCRIPTION: Lower Level Diaphragm Analysis

General Beam Properties

Elastic Modulus 29,000.0 ksi

Span #1 Span Length = 45.0 ft Area = 10.0 in^2 Moment of Inertia = 100.0 in^4



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

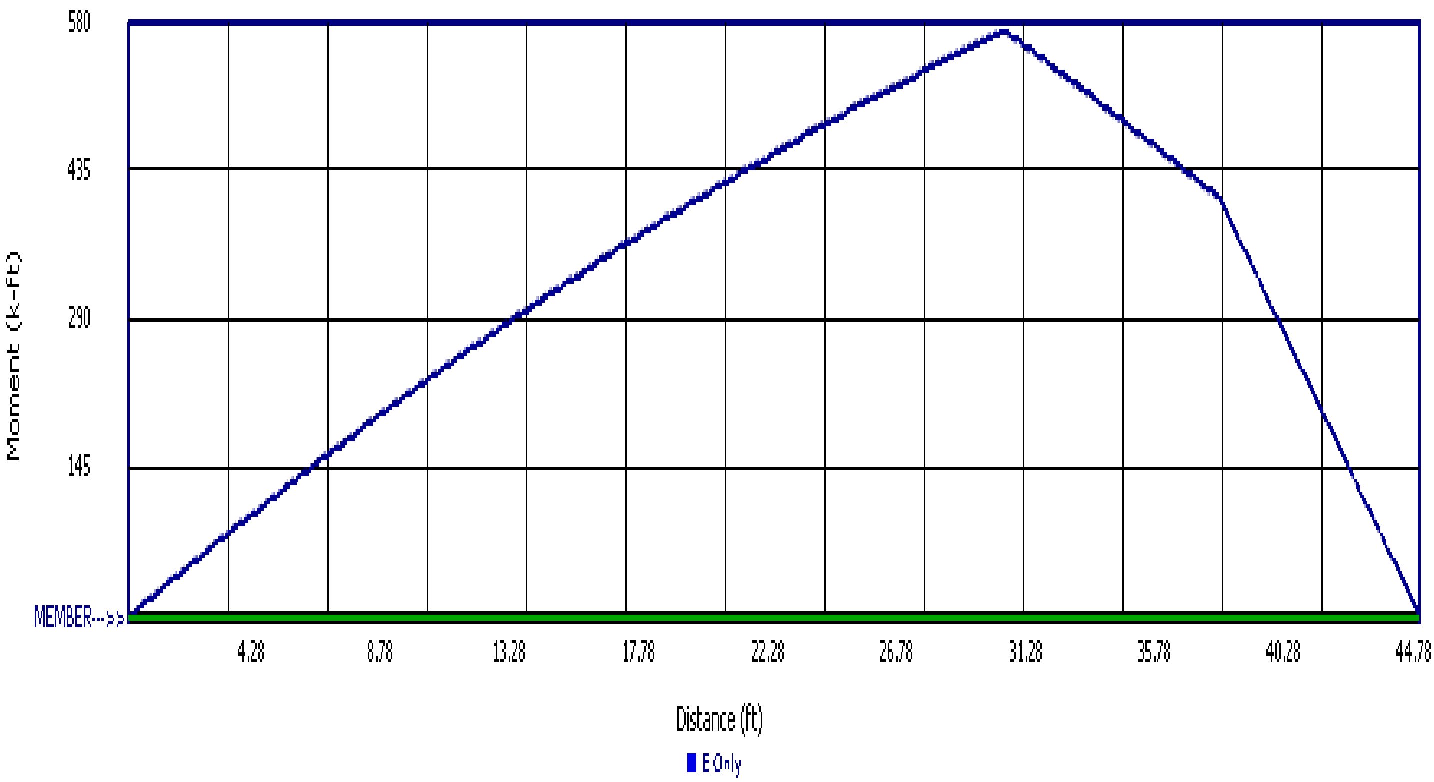
Load Combination	Support 1	Support 2
Overall MAXimum	23.914	59.321
Overall MINimum		
E Only * 0.70	16.740	41.525
E Only * 0.5250	12.555	31.143
E Only	23.914	59.321

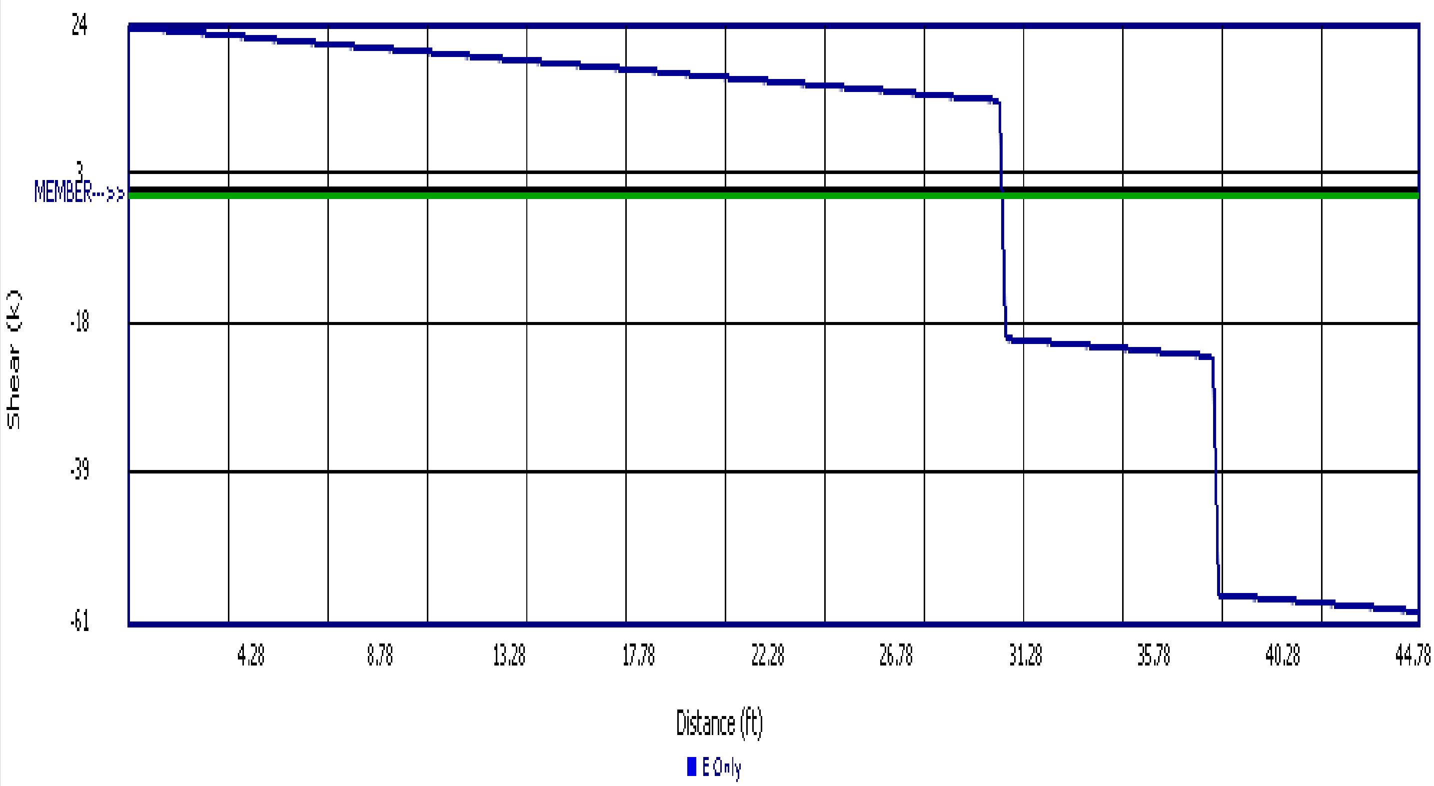
General Beam Analysis

Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: Lower Level Diaphragm Analysis



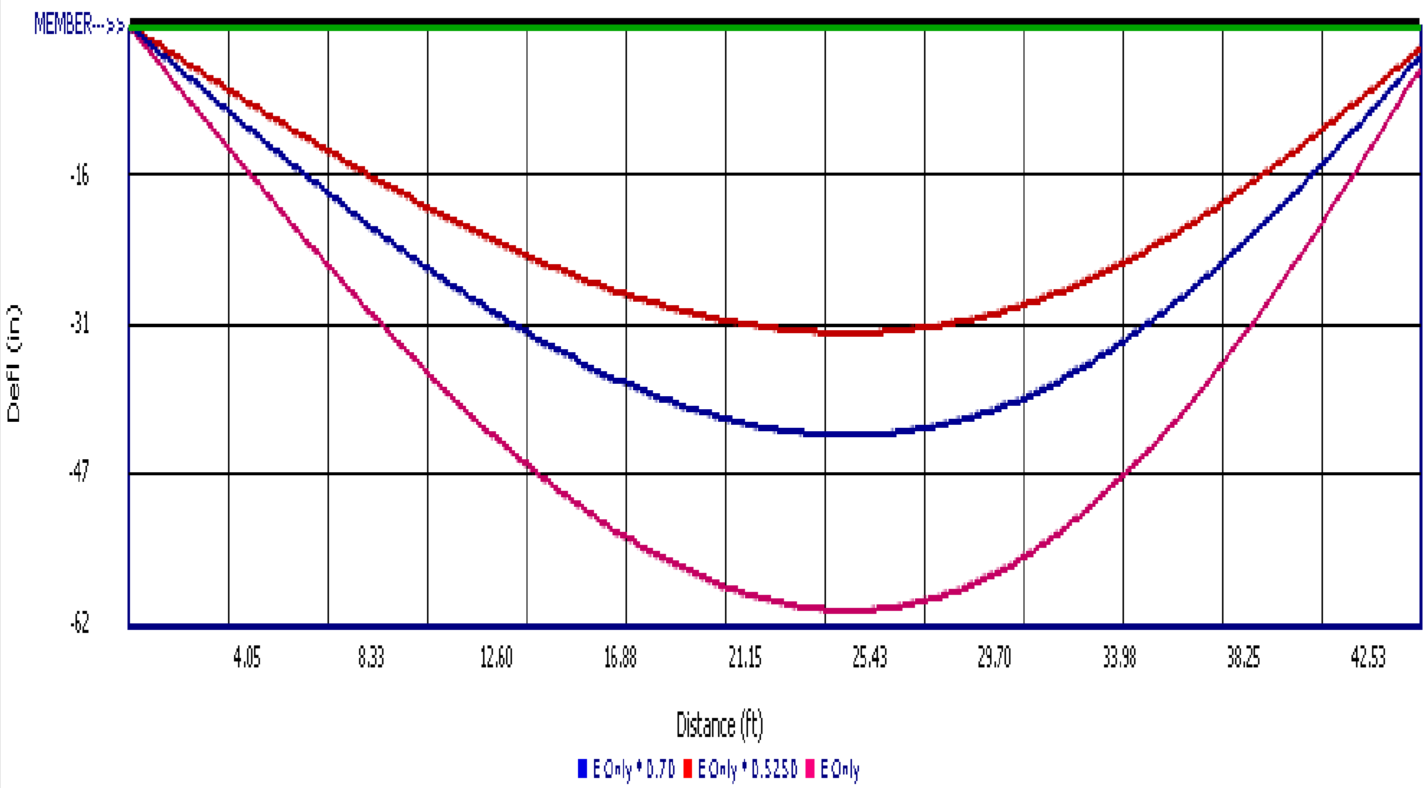


General Beam Analysis

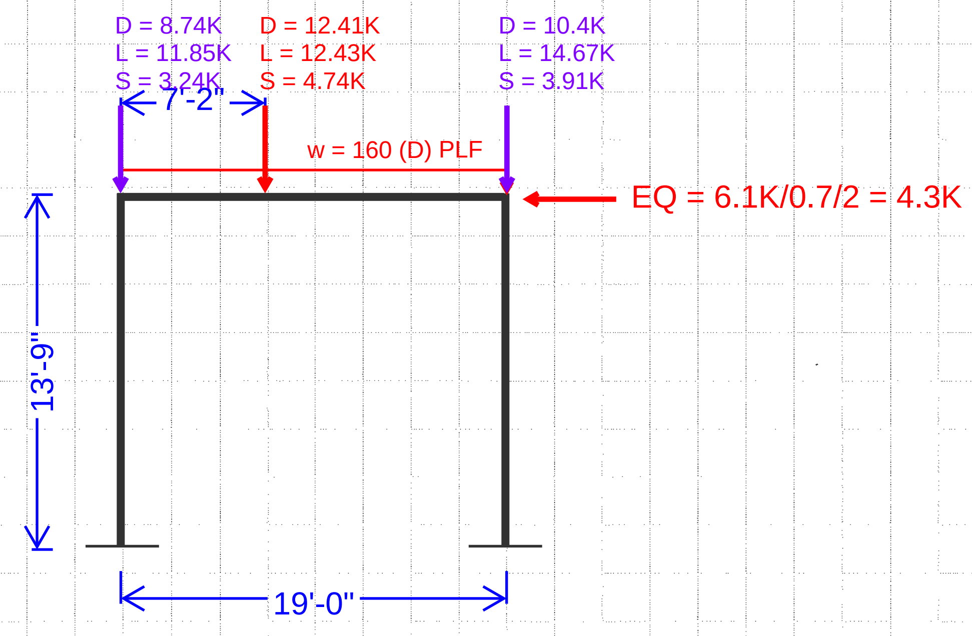
Project File: Talmon.ec6

LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: Lower Level Diaphragm Analysis



MOMENT FRAME DESIGN



REFERT TO RAM ELEMENTS MOMENT FRAME MODEL OUTPUT

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

Description		By ERH	Date 07.07.23
	MOMENT FRAME	Checked	Date
		Scale NTS	Sheet No.
Project	THE TALMON	Job No. 23154.10	

Design of Special Moment Frames (Reduced Beam Section)

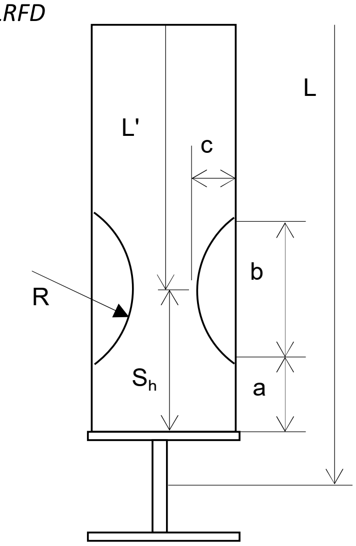
Per AISC Seismic Design Manual 2016 - LRFD

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	*in^3*
r_y=	2.59	*in*
R_y=	1.1	*in^2*
F_y=	50	*ksi*
F_u=	65	*ksi*

Column Properties:

Size=	W12X136	
H=	165	*in*
A_g=	39.9	*in^2*
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	*in^3*
r_y=	3.16	*in*



Beam Limitations (per AISC 358 5.3-1)

Depth ≤ W36

Weight ≤ 300 lbs/ft

tf ≤ 1.75

Clear Span-to-Depth Ratio ≥ 7.0 (for SMF only)

a = .6*bf =	6.1	*in (Eq. 5.8-1)*
b = .75*d =	7.8	*in (Eq. 5.8-2)*
c = .2*bf =	2.0	*in (Eq. 5.8-3)*
Radius =	4.8	*in*

(refer to RBS detail sheets by CG Eng)

Protected Zone= 15.2 *in*

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 *in^3*	(AISC 358 Eq. 5.8-4)
$C_{pr}=(F_y+F_u)/2F_y \leq 1.2$ =	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)

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		Scale	Sheet No.
	Project The Talmon	Job No. 22332.10	

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2016 - LRFD

Compute Shear Force @ Center of RBS

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

Compute Maximum Moment @ Face of Column

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

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

$M_{pe} = R_yF_yZ_x =$	4691.5	*kip-in*	(AISC 358 Eq. 5.8-7)
$\mathbf{M_f \leq \phi_d M_{pe}}$	**OKAY**	*(for ϕ_d = 1.0)*	(AISC 358 Eq. 5.8-8)

Check Column-Beam Relationship Limitations

$M_{uv} = V_{RBS}*(a+b/2+d_c/2) =$	956	*kip-in*	
$\Sigma M^*_{pb} = \Sigma(M_{pr}+M_{uv})$	4456	*kip-in*	(Seismic Provisions Eq. E3-3a)
	(assuming single beam-column)		
$P_{uc} =$	46	*kips (See RAM model LC: 1.2D + 1.0E + 1.0L)*	
$\Sigma M^*_{pc} = \Sigma Z_{xc}(F_{yc}-P_{uc}/A_g)$	10453	*kip-in*	(Seismic Provisions Eq. E3-2a)
	(assuming single beam-column)		
$\mathbf{\Sigma M^*_{pc}/\Sigma M^*_{pb} \geq 1.0 =}$	**2.3**	**OKAY**	

Check Shear Strength of the Beam

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

*note: beam web is full-penetration welded to column face
*note: assumes reduced area for (2) 13/16" erection bolt holes

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	Project The Talmon	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

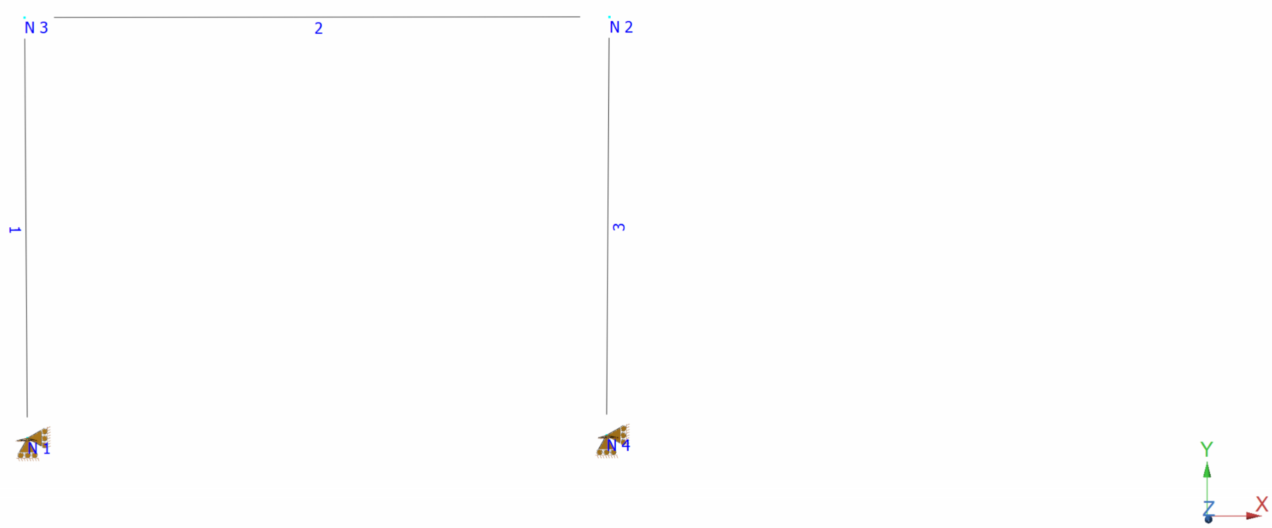
$R_u = \Sigma M_f/(d-t_f)$ =	423	*kips*	
$.75*F_y*A_g$ =	1496	*kips* ------->	USE J10-11
$R_u \le \phi Rn$ =	448	*kips*	**OKAY! DOUBLER PLATE NOT REQ'D**

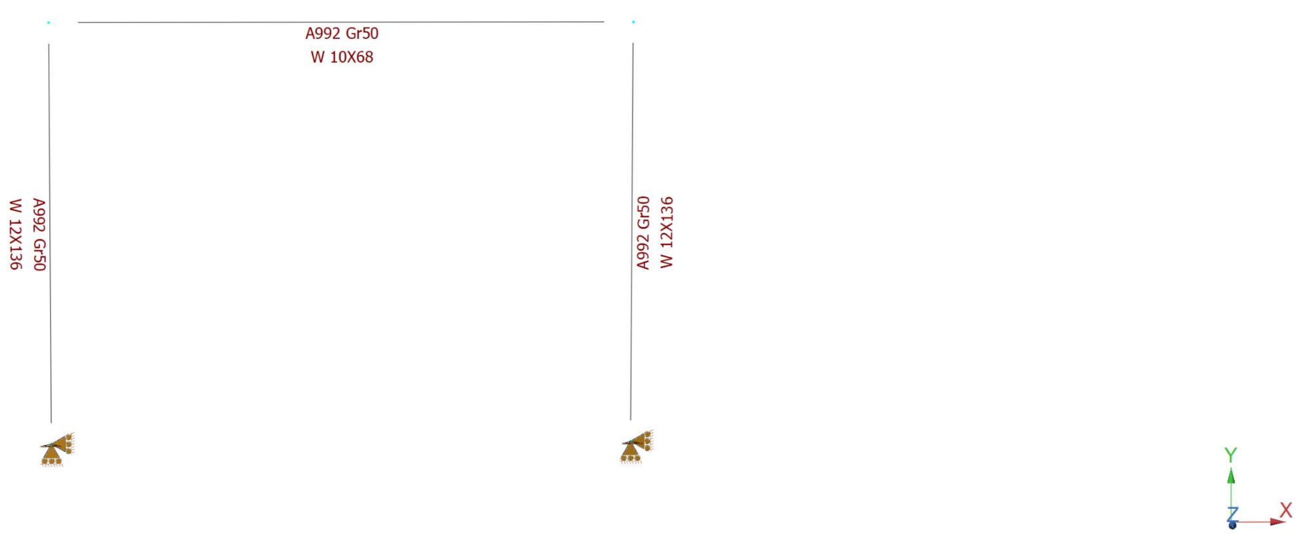
Check Continuity Plate Requirements

$t_{cf} \ge .4\sqrt{(1.8 b_{bf} t_{bf} F_{yb} R_{yb}/(F_{yc}*R_{yc}))}$ =	1.5	**REQUIRED**
$t_{cf} \ge b_{bf}/6$ =	1.7	**REQUIRED**

If required, continuity plates shall match thickness of beam flanges

ENGINEERING 250 4th Ave South Suite 200 Edmonds, WA 98020	Description Moment Frame Calculation	By LS Checked Scale	Date 7/17/2023 Date Sheet No.
	Project The Talmon	Job No. 22332.10	


N 3
2
N 2
1
3
N 1
N 4
Y
Z
X


A992 Gr50
W 10X68
A992 Gr50
W 12X136
A992 Gr50
W 12X136
Y
Z
X

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Current Date: 7/14/2023 4:54 PM
Units system: English
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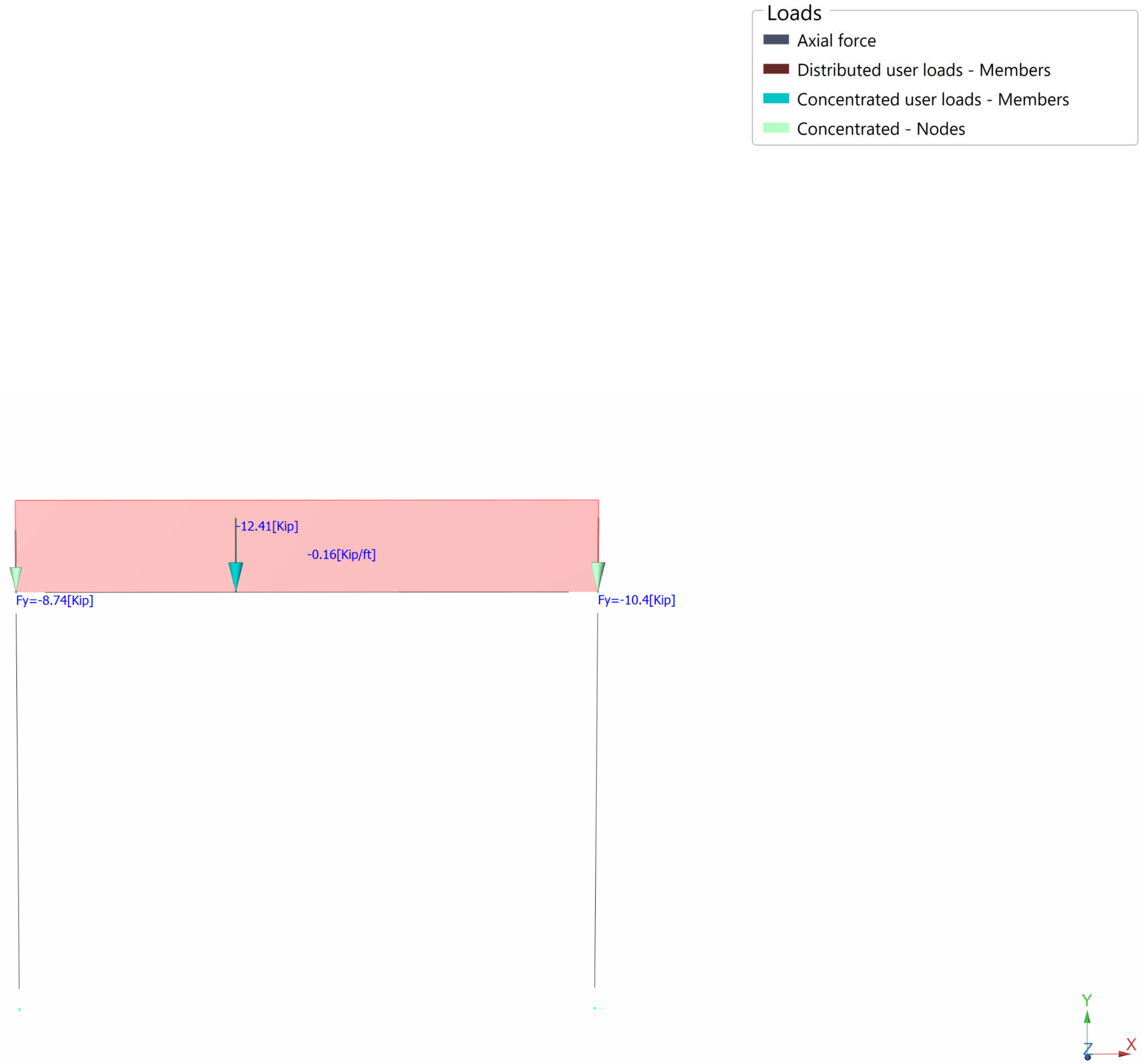
Load condition: DL=Dead Load



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Units system: English
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Load condition: L=Live Load

RAM® Elements CONNECT Edition

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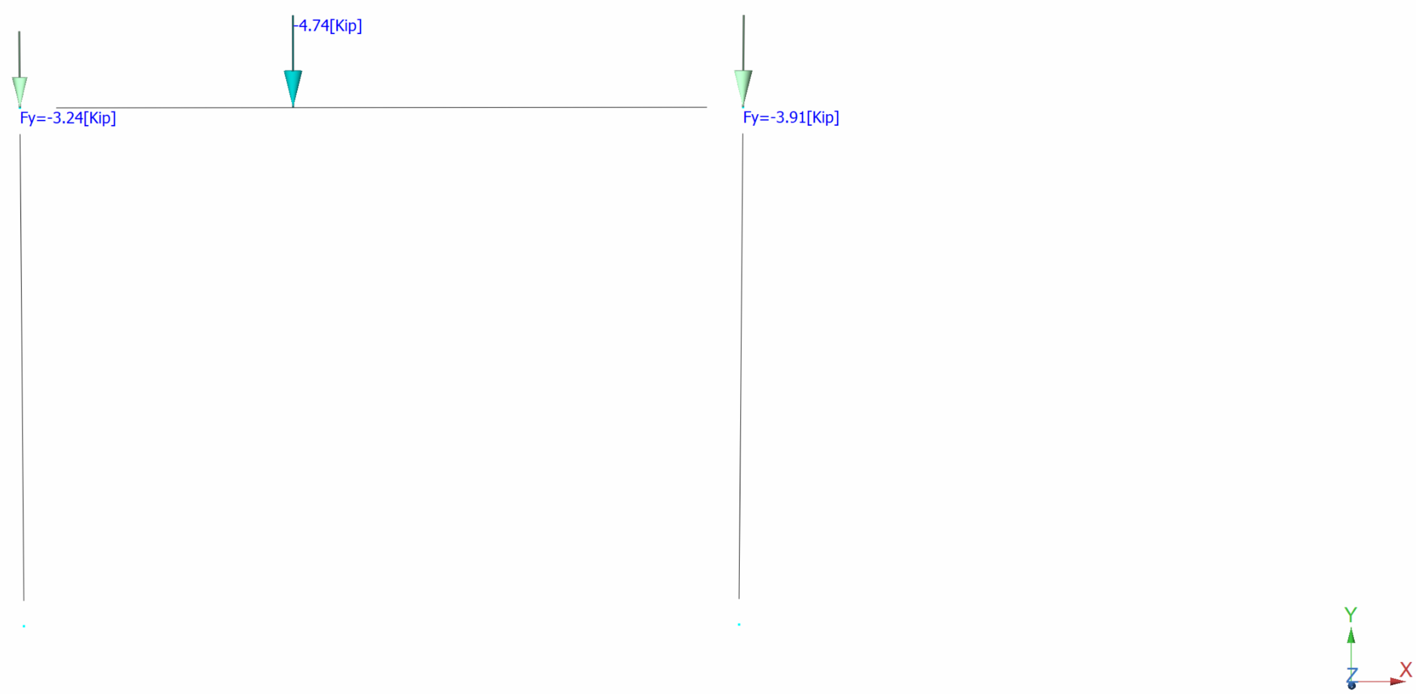
Current Date: 7/14/2023 4:55 PM
Units system: English
File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\Moment Frame.retx
Load condition: S=Snow Load

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



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Current Date: 7/21/2023 11:17 AM
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Load condition: Eq=Seismic Load

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Current Date: 7/17/2023 9:46 AM
Units system: English
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**	

RAM® Elements CONNECT Edition

HP Inc.

Current Date: 7/21/2023 11:19 AM
Units system: English
File name: R:_2023 Projects\23154.10 The Talmon_Structural\Engineering\Lateral Design\RAM\Moment Frame.retx

Analysis result

Nodal displacements envelope

Note.- **lc** 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

		Translation						Rotation					
Node		**X** [in]	**lc**	**Y** [in]	**lc**	**Z** [in]	**lc**	**Rx** [Rad]	**lc**	**Ry** [Rad]	**lc**	**Rz** [Rad]	**lc**
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

RAM® Elements
CONNECT Edition

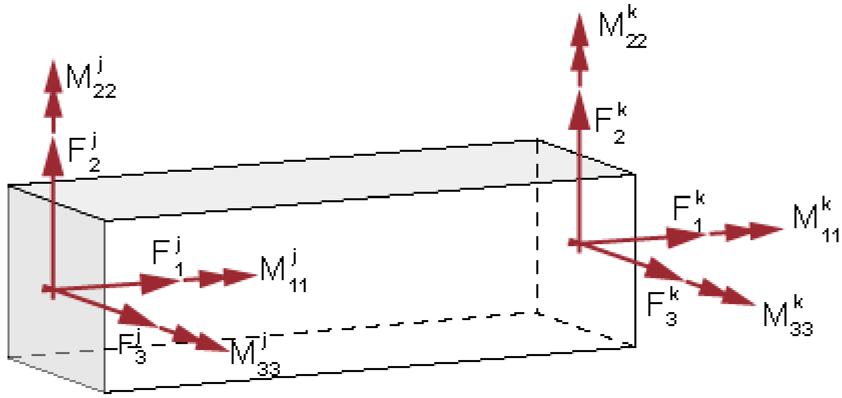
HP Inc.

Current Date: 7/17/2023 10:01 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



Sign convention with possitive values

CONDITION: **Vu=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	13.83142	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

2.043

RAM Elements CONNECT Edition

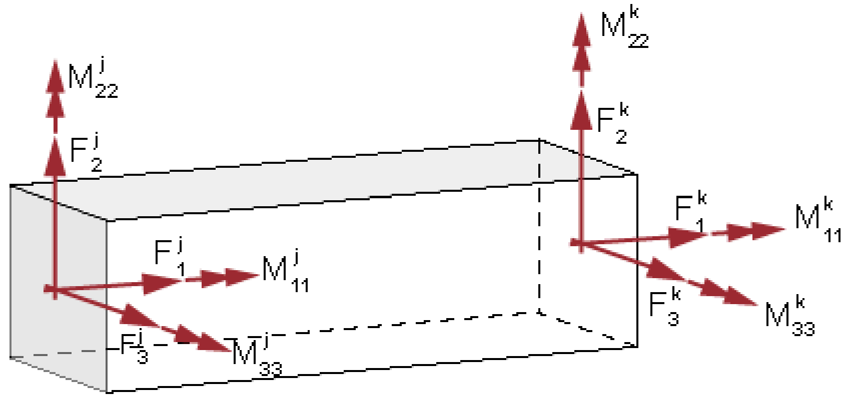
HP Inc.

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



Sign convention with possitive values

CONDITION: **Vu=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

RAM Elements CONNECT Edition

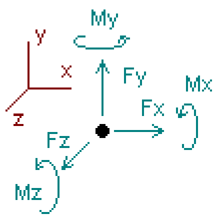
HP Inc.

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.- **lc** 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

		Forces						Moments					
Node		**Fx** [Kip]	**lc**	**Fy** [Kip]	**lc**	**Fz** [Kip]	**lc**	**Mx** [Kip*ft]	**lc**	**My** [Kip*ft]	**lc**	**Mz** [Kip*ft]	**lc**
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

General Beam Analysis

Project File: Talmon.ec6

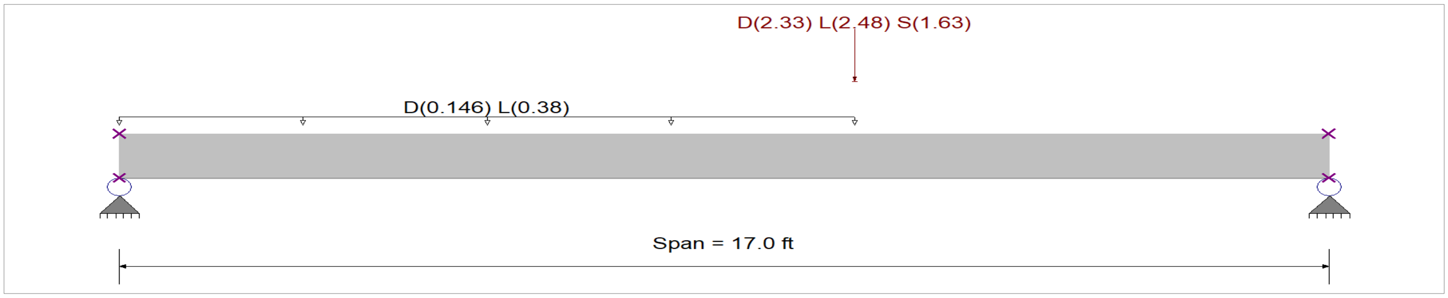
LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING (c) ENERCALC INC 1983-2023

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 in^2 Moment of Inertia = 100.0 in^4



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

2.046

General Beam Analysis

Project File: Talmon.ec6

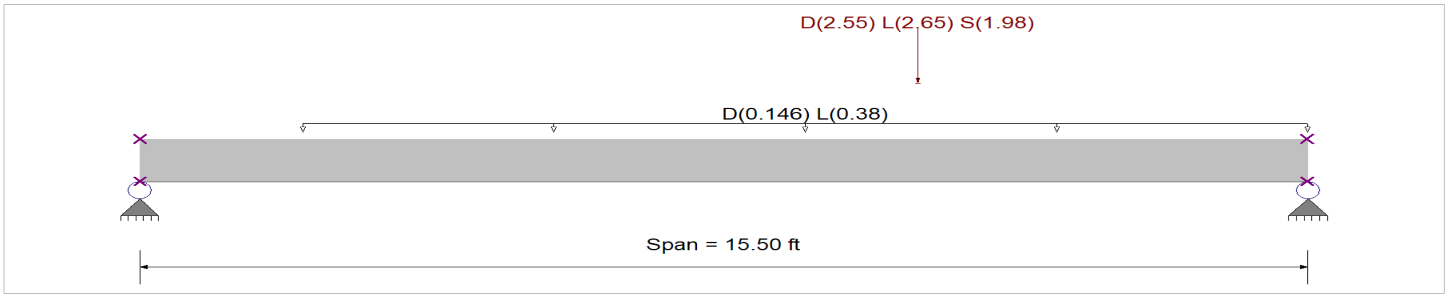
LIC# : KW-06015244, Build:20.23.05.25 CG ENGINEERING

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 in^2 Moment of Inertia = 100.0 in^4



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

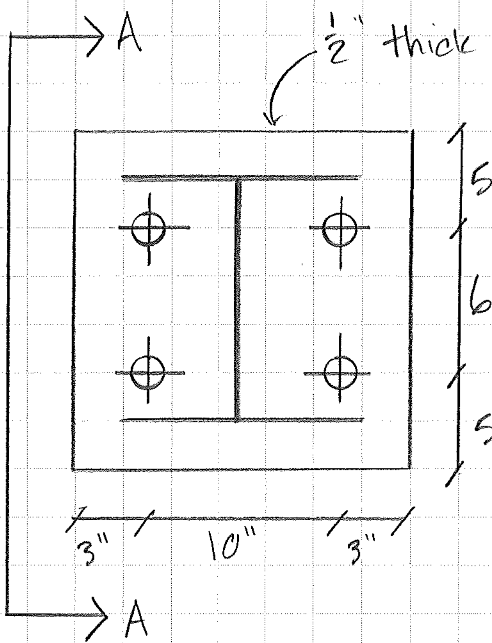
Vertical Reactions

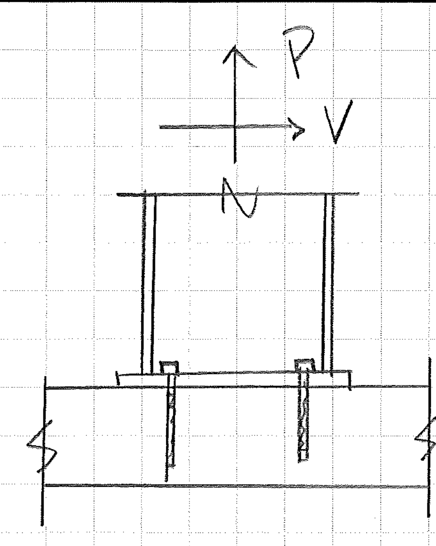
Support notation : Far left is #´ Values in KIPS

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

2.047

Base Plate Design + Anchorage





P_{max} = 0 lbs (max uplift w/ Ω)
V_{max} = 10584 lbs (max shear)

(RODS CONSERVATIVELY CHECKED WITH 10K OF UPLIFT)

Per HILTI, use (4) 3/4" ϕ THREADED RODS W/ 5" MIN EMBED

Provide 6'×6' sq. ftg.

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

Description: Lateral Design		By: ERH	Date: 07.17.23
		Checked:	Date:
		Scale:	Sheet No.:
Project: THE TALMON		Job No.: 23154.10	

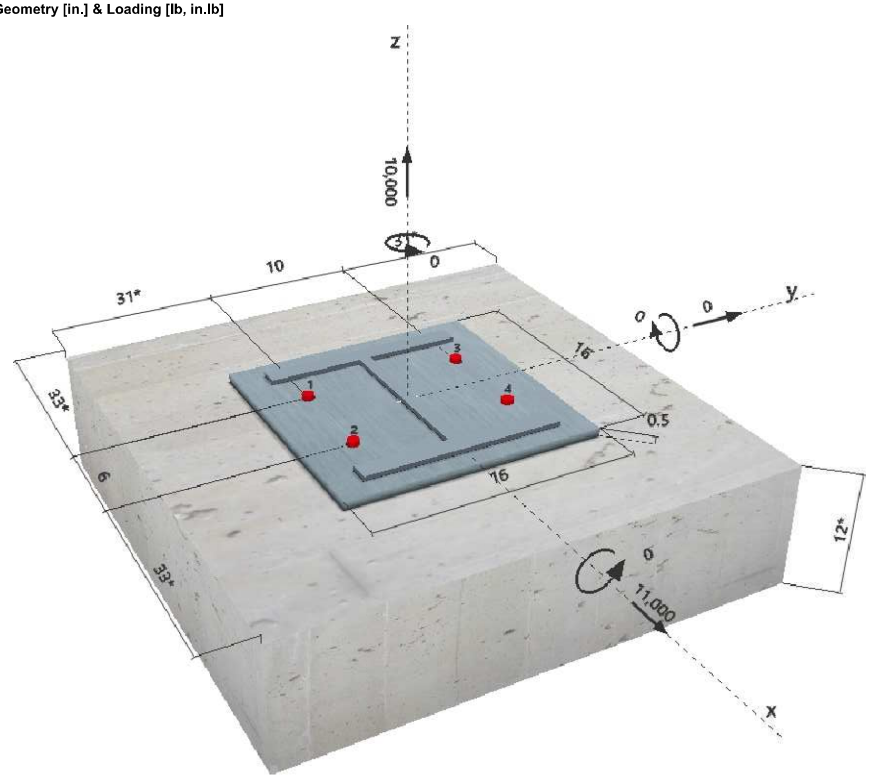
Company:	CG Engineering	Page:	1
Specifier:	ERH	Project:	The Talmon
Address:	250 4th Ave S Ste 200 Edmonds, WA 98020	Sub-Project I Pos. No.:	
Phone I Fax:	(425) 778-8500 | (425) 778-5536	Date:	7/21/2023
E-Mail:			

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 x l_y x 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]



Company:	CG Engineering	Page:	2
Specifier:	ERH	Project:	The Talmon
Address:	250 4th Ave S Ste 200 Edmonds, WA 98020	Sub-Project I Pos. No.:	
Phone I Fax:	(425) 778-8500 | (425) 778-5536	Date:	7/21/2023
E-Mail:			

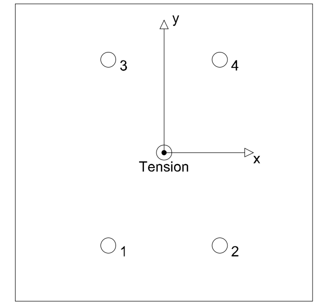
2 Load case/Resulting anchor forces

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}	$\phi\, N_{sa}$ [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)
$N_p = 8\, A_{brg}\, f'_c$ 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_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.65	1.000	2500

Calculations

N_p [lb]
13080

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, N_{pn}$ [lb]	N_{ua} [lb]
13080	0.700	0.750	1.000	6867	2500

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-14 Eq. (17.4.2.1b)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	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]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
31305	0.700	0.750	1.000	16435	10000

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

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$	ACI 318-14 Eq. (17.5.1.2b)
$\phi\ V_{steel} \geq V_{ua}$	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}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
11623	0.650	7555	2750

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$	ACI 318-14 Eq. (17.5.3.1b)
$\phi\ V_{cpg} \geq V_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	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_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
62610	0.700	1.000	1.000	43827	11000

4.3 Concrete edge failure in direction x+

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

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

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\, c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

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

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

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

$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\, c_{a1}^{1.5}$ 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_e [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}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
864.00	1922.00	1.000	1.000	1.607	41618

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [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

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

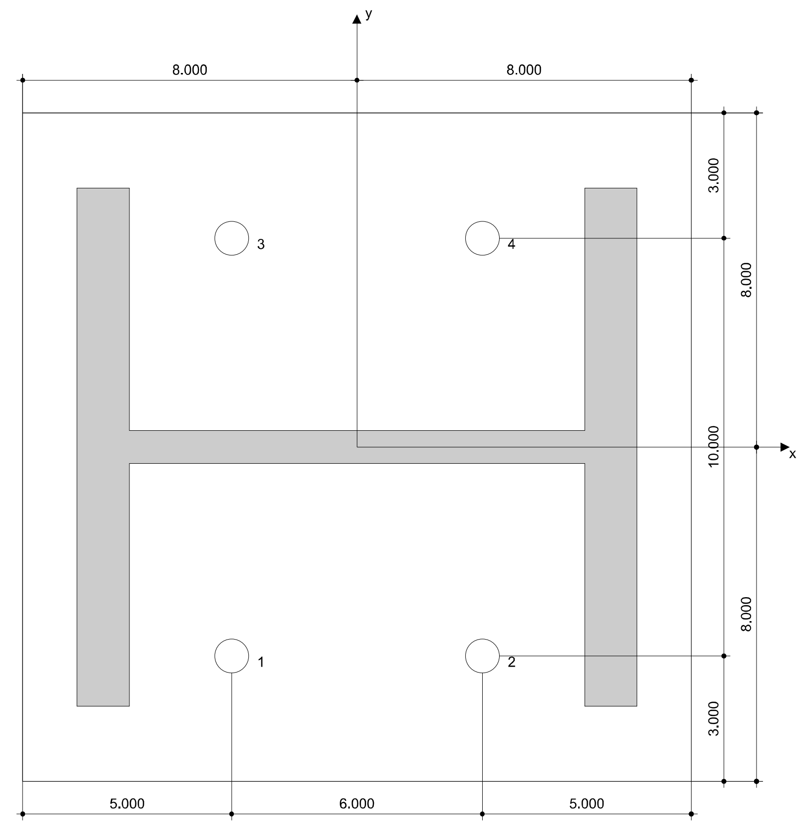
- Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading! Input data and results must be checked for agreement with the existing conditions 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!

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

Company:	CG Engineering	Page:	7
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8 Remarks; Your Cooperation Duties

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