civil & structural
engineering & planning

STRUCTURAL CALCULATIONS

Rudolf Residence

8253 W. Mercer Way
Mercer Island, WA

05/11/2018

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

CG Project No.: 15227.10

Project Location
8253 W. Mercer Way
Mercer Island, WA

Project Description
This project involves the construction of a new 3-story single family residence. The house will be framed with premanufactured roof and floor trusses. The walls will be light wood framed shear walls. The foundation will be concrete auger cast piles and grade beams.

Scope of Work
Provide structural calculations and construction documents in accordance with current building code.

Basis of Design

Roof Loads	Dead	15 psf
	Snow	25 psf
Floor Loads	Dead	15 psf
	Garage	55 psf
	Live	40 psf
	Deck	60 psf

Wind Parameters
110 MPH Wind Speed, 3-Sec Gust
Exposure Category C
Kzt = 1.3

Seismic Parameters
Sds = 0.961
I_E = 1.0
Light Wood Framed Shearwalls
R= 6.5

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

Description: Project Summary		By: DTR	Date: 5/10/2018
		Checked	Date
		Scale: NTS	Sheet No.
Project: Rudolf Res.		Job No.: 15227.10	0-1

Gravity Design Loads

Roof DL

Roofing Material	2.5	psf
1/2 Sheathing	1.5	psf
Insulation	1.0	psf
1/2 Gypsum	2.0	psf
Trusses @ 24" OC	3.0	psf
M/E	1.2	psf
Misc	1.5	psf
	12.7	psf
USE	15.0	psf

Floor DL

Flooring Material	2.0	psf
3/4 Sheathing	2.3	psf
Insulation	1.0	psf
1/2 Gypsum	2.0	psf
Truss @ 16" OC	4.2	psf
M/E	1.0	psf
Misc	1.5	psf
	14.0	psf
USE	15.0	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	psf

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Description: Gravity Design Loads		By: MBB	Date: 03/05/18
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Residence		Job No.	1-2
		15227.10	

Beam Span Table - Floor 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	815	522	362	266	204	160	117	-	-	-	-	-	-	-	-	-	-
3 1/2 x 5 1/2 LSL	1340	858	546	344	230	162	118	-	-	-	-	-	-	-	-	-	-
4x8 HF #2	1270	902	627	460	353	279	226	186	155	122	-	-	-	-	-	-	-
3 1/2 x 7 1/4 LSL	2275	1456	1011	743	522	367	267	201	155	122	-	-	-	-	-	-	-
6x8 DF #2	1880	1203	836	614	470	371	301	249	209	178	153	134	114	-	-	-	-
2 11/16 x 9 1/4 PSL	2405	1924	1603	1374	1193	889	648	487	375	295	236	192	158	132	111	-	-
4x10 HF #2	1620	1296	942	692	530	419	339	280	236	201	173	151	133	113	-	-	-
3 1/2 x 9 1/4 PSL	3130	2504	2087	1789	1553	1169	852	640	493	388	310	252	208	173	146	124	106
5 1/4 x 9 1/4 PSL	4695	3756	3130	2683	2328	1753	1278	960	739	582	466	379	312	260	219	186	160
2 11/16 x 9 1/2 PSL	2470	1976	1647	1411	1235	965	704	529	407	320	256	209	172	143	121	103	-
3 1/2 x 9 1/2 LSL	3160	2528	2107	1646	1260	953	694	522	402	316	253	206	170	141	119	101	-
3 1/2 x 9 1/2 PSL	3215	2572	2143	1837	1608	1270	926	696	536	421	337	274	226	188	159	135	116
6x10 DF #2	2960	1930	1340	984	754	596	482	399	335	285	246	214	188	167	149	134	118
5 1/4 x 9 1/2 PSL	4825	3860	3217	2757	2413	1905	1389	1043	804	632	506	412	339	283	238	202	174
7 x 9 1/2 PSL	6430	5144	4287	3674	3215	2540	1852	1391	1072	843	675	549	452	377	318	270	231
2 11/16 x 11 1/4 PSL	2925	2340	1950	1671	1463	1300	1104	890	686	539	432	351	289	241	203	173	148
3 1/2 x 11 1/4 LSL	3740	2992	2493	2137	1740	1375	1114	866	667	525	420	342	281	235	198	168	144
3 1/2 x 11 1/4 PSL	3810	3048	2540	2177	1905	1693	1438	1155	889	700	560	455	375	313	264	224	192
6x12 DF #2	3585	2829	1964	1443	1105	873	707	584	491	418	361	314	276	245	218	196	177
5 1/4 x 11 1/4 PSL	5710	4568	3807	3263	2855	2538	2157	1739	1340	1054	844	686	565	471	397	337	289
2 11/16 x 11 7/8 PSL	3085	2468	2057	1763	1543	1371	1222	1010	804	632	506	412	339	283	238	202	174
3 1/2 x 11 7/8 LSL	3950	3160	2633	2257	1930	1525	1235	1018	784	617	494	402	331	276	232	198	169
3 1/2 x 11 7/8 PSL	4020	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)

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Suite 200
Edmonds, WA 98020

Description: Beam Span Table		By: DTR	Date: 03/05/18
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Res.		Job No. 15227.10	1-3

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	-
3 1/2 x 9 1/4 PSL	3600	2880	2400	2057	1785	1411	1143	944	789	620	497	404	333	277	234	199	170
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)

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Suite 200
Edmonds, WA 98020

Description: Beam Span Table		By: DTR	Date: 03/05/18
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Res.		Job No.: 15227.10	1-4

HF Column & HF Sill Plate Capacity TABLE

IBC 2015, NDS 2015

	6	7	8	9	10	11	12	13	14	15	16
(2) 2x4 HF Stud	5,149	**4,121**	**3,311**	**2,693**	**2,224**	**1,862**	**1,579**	**1,355**	**1,175**	**1,028**	**906**
P_{SILL}	**4,784**	-	-	-	-	-	-	-	-	-	-
(3) 2x4 HF Stud	9,220	7,723	**6,382**	**5,281**	**4,406**	**3,715**	**3,166**	**2,726**	**2,369**	**2,076**	**1,834**
P_{SILL}	**6,910**	**6,910**	-	-	-	-	-	-	-	-	-
(4) 2x4 HF Stud	12,294	10,298	8,510	**7,041**	**5,875**	**4,953**	**4,221**	**3,635**	**3,159**	**2,769**	**2,445**
P_{SILL}	**8,505**	**8,505**	**8,505**	-	-	-	-	-	-	-	-
(2) 3x4 HF Stud	10,245	8,581	**7,091**	**5,868**	**4,896**	**4,128**	**3,518**	**3,029**	**2,632**	**2,307**	**2,038**
P_{SILL}	**7,619**	**7,619**	-	-	-	-	-	-	-	-	-
(3) 3x4 HF Stud	15,367	12,872	10,637	**8,802**	**7,343**	**6,191**	**5,277**	**4,543**	**3,948**	**3,461**	**3,057**
P_{SILL}	**10,631**	**10,631**	**10,631**	-	-	-	-	-	-	-	-
(2) 2x6 HF Stud	7,951	**6,405**	**5,164**	**4,210**	**3,481**	**2,917**	**2,476**	**2,125**	**1,843**	**1,613**	**1,423**
P_{SILL}	**7,518**	-	-	-	-	-	-	-	-	-	-
(3) 2x6 HF Stud	16,730	15,297	13,636	11,927	**10,333**	**8,934**	**7,746**	**6,750**	**5,918**	**5,221**	**4,634**
P_{SILL}	**10,859**	**10,859**	**10,859**	**10,859**	-	-	-	-	-	-	-
(4) 2x6 HF Stud	23,902	22,755	21,314	19,614	17,764	15,903	14,146	**12,558**	**11,158**	**9,942**	**8,891**
P_{SILL}	**13,365**	**13,365**	**13,365**	**13,365**	**13,365**	**13,365**	**13,365**	-	-	-	-
4x6 HF #2	14,409	11,327	9,009	**7,286**	**5,993**	**5,006**	**4,239**	**3,633**	**3,147**	**2,751**	**2,425**
P_{SILL}	**8,328**	**8,328**	**8,328**	-	-	-	-	-	-	-	-
4x8 HF #2	18,744	14,808	11,809	**9,566**	**7,876**	**6,583**	**5,577**	**4,782**	**4,142**	**3,622**	**3,193**
P_{SILL}	**10,277**	**10,277**	**10,277**	-	-	-	-	-	-	-	-
4x10 HF #2	23,562	18,717	14,972	**12,150**	**10,015**	**8,377**	**7,101**	**6,090**	**5,277**	**4,615**	**4,069**
P_{SILL}	**13,112**	**13,112**	**13,112**	-	-	-	-	-	-	-	-
6x6 DF #2	19,595	18,889	17,995	16,908	15,659	14,315	**12,960**	**11,665**	**10,475**	**9,407**	**8,463**
P_{SILL}	**13,087**	**13,087**	**13,087**	**13,087**	**13,087**	**13,087**	-	-	-	-	-
6x8 DF #2	25,830	24,899	23,721	22,288	20,642	18,870	17,083	**15,377**	**13,808**	**12,400**	**11,156**
P_{SILL}	**16,149**	**16,149**	**16,149**	**16,149**	**16,149**	**16,149**	**16,149**	-	-	-	-
6x10 DF #2	28,621	27,790	26,739	25,450	23,929	22,224	**20,420**	**18,614**	**16,885**	**15,285**	**13,835**
P_{SILL}	**20,604**	**20,604**	**20,604**	**20,604**	**20,604**	**20,604**	-	-	-	-	-

* columns designed per Column Table

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250 4th Ave. South
Suite 200

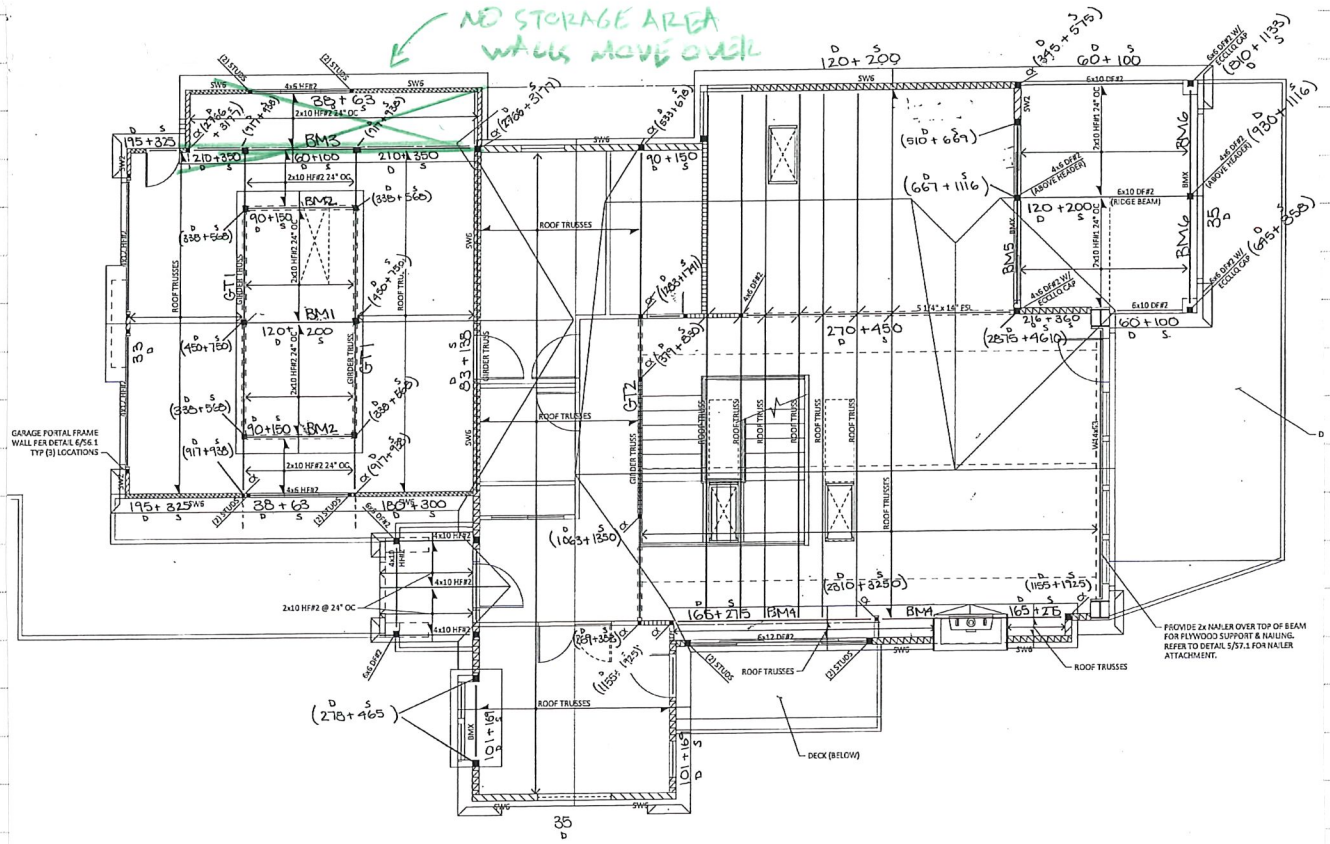
Description		By DTR	Date 03/05/18
Wood Column Capacity Table		Checked	Date
		Scale	Sheet No.
Project Rudolf Res.		Job No. 15227.10	1-5

Roof Framing Key Plan

* loads on walls in (plf), loads on columns in (lb)

DL = 15psf

SL = 25psf



C G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: Roof Framing Key Plan	By: MBB	Date: 3/5/18
		Checked	Date
		Scale: NTS	Sheet No.: 1-6
	Project: Rudolf Residence	Job No.: 15227.10	

Third Floor Framing Key Plan

* loads on walls in (plf), loads on columns in (lb)

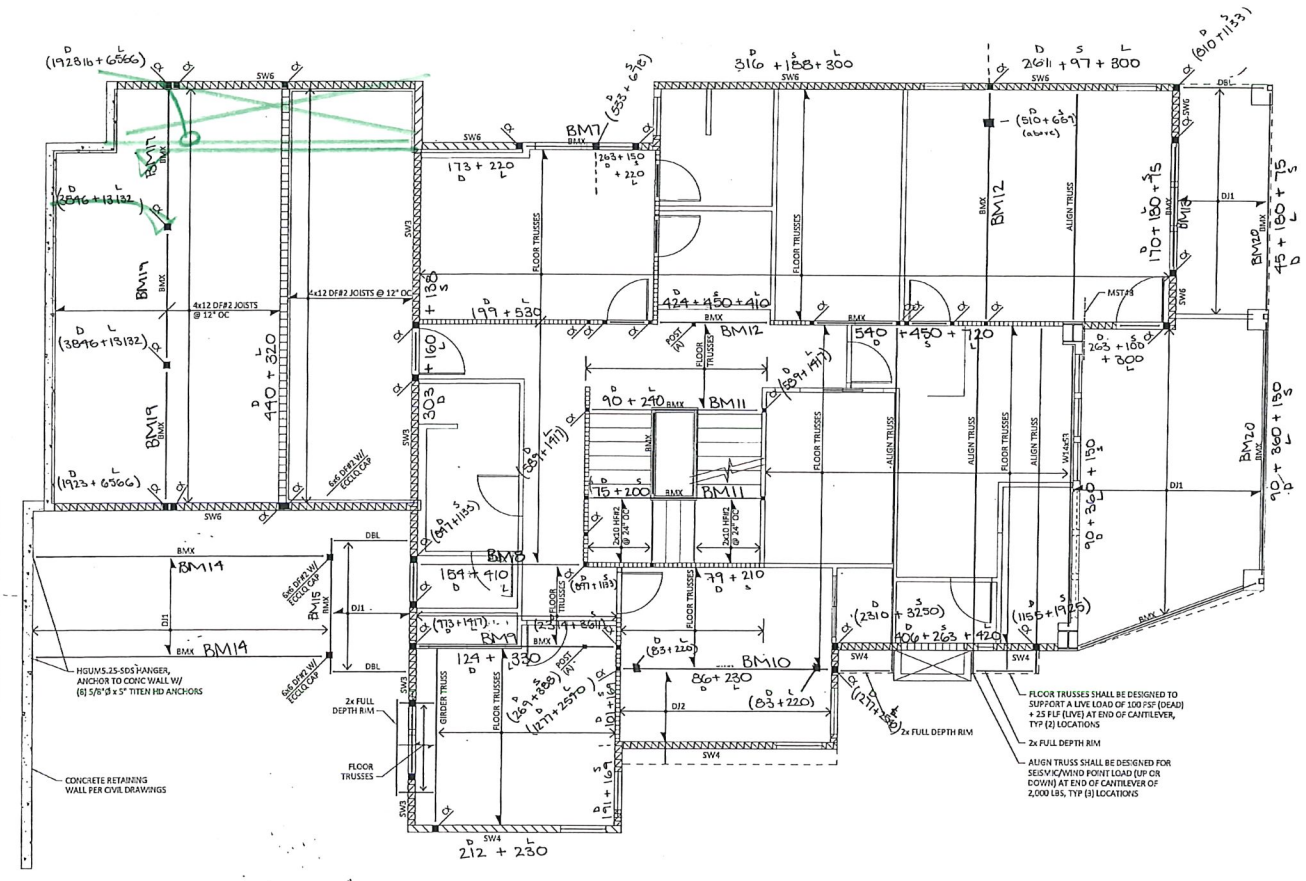
DL = 15psf

SL = 25psf

LL = 40 psf

LL deck = 60 psf

Garage DL = 40psf



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		Checked	Date
		Scale: NTS	Sheet No.: 1-7
	Project: Rudolf Residence	Job No.: 15227.10	

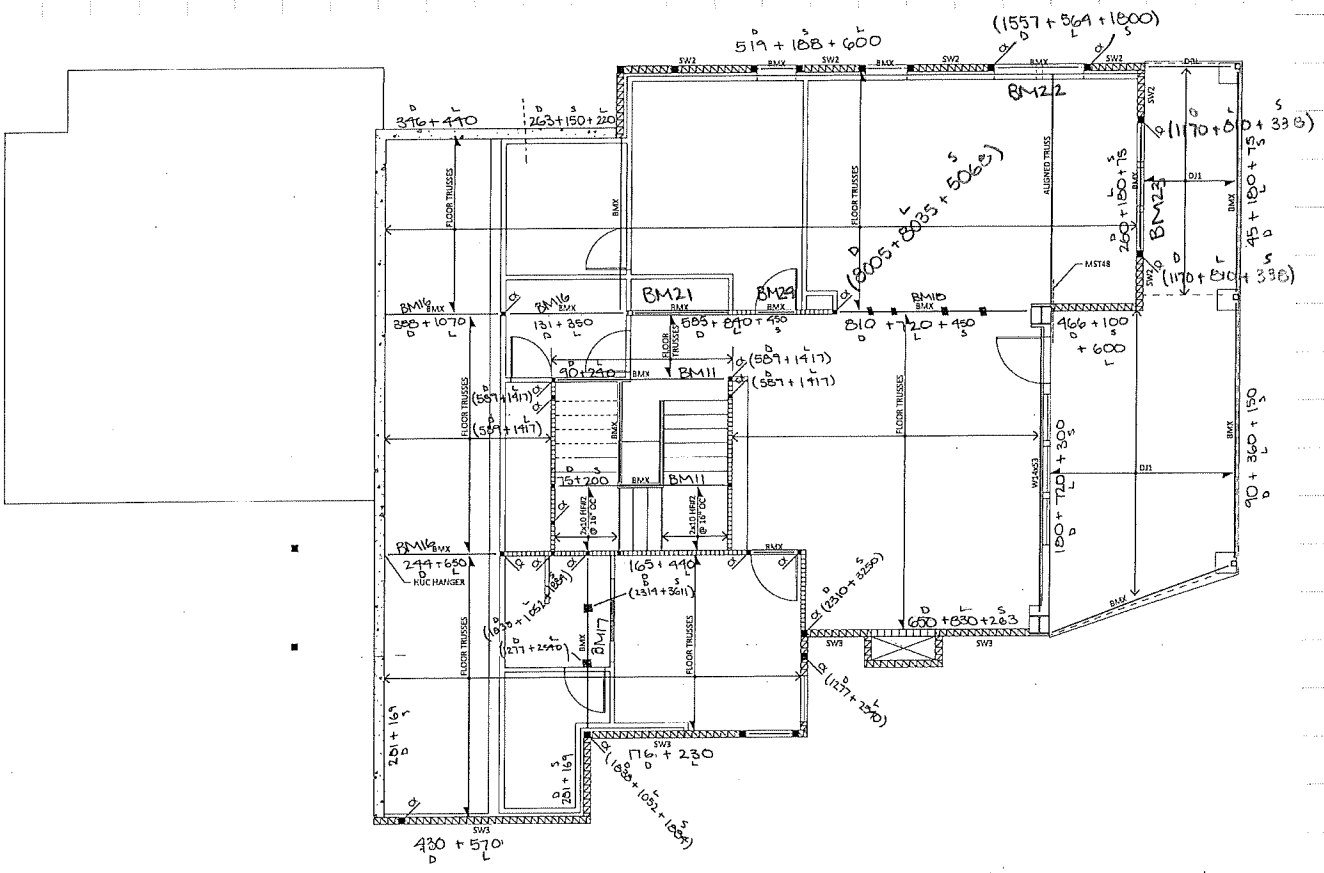
Second Floor Framing Key Plan

* loads on walls in (plf), loads on columns in (lb)

DL = 15 psf

SL = 25 psf

LL = 40 psf



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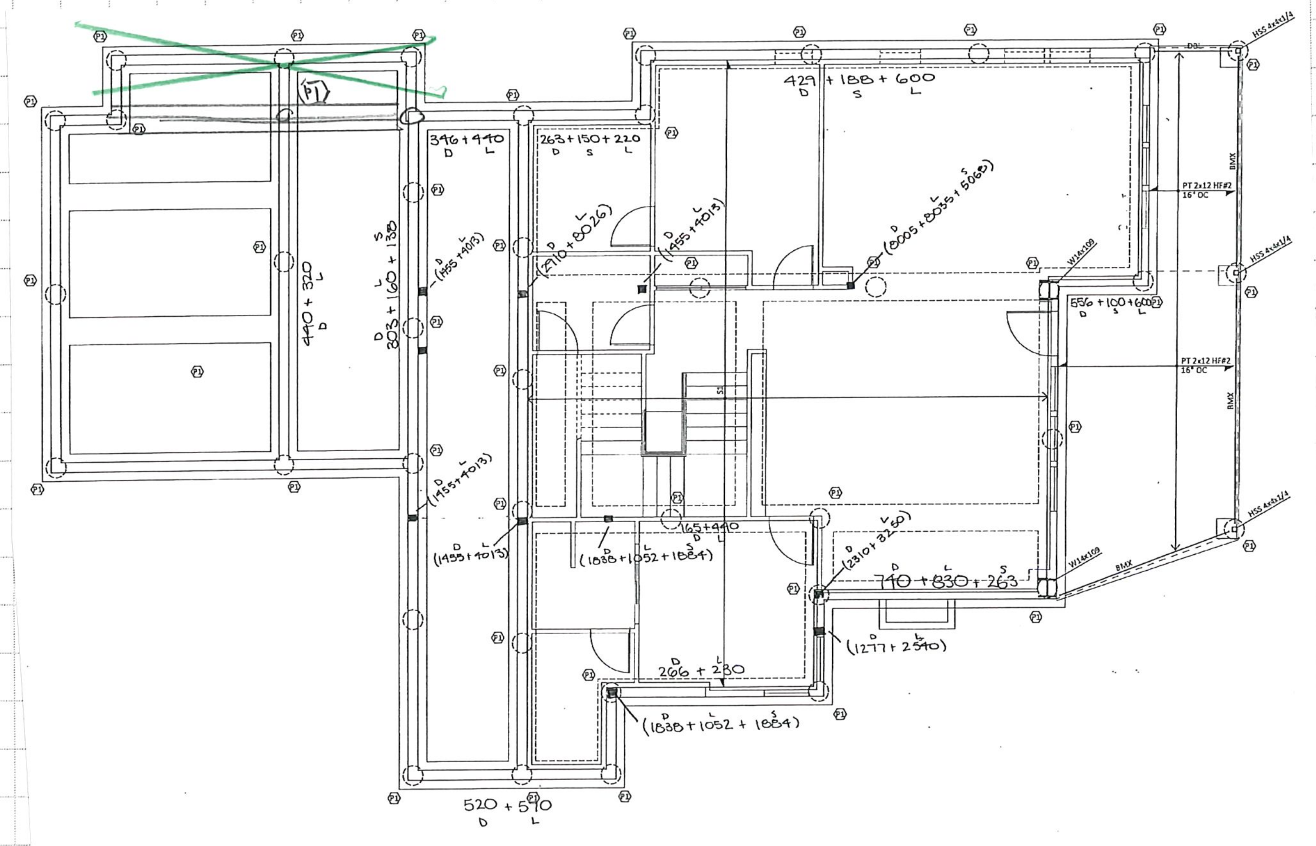
		Checked	Date
		Scale: NTS	Sheet No.: 1-8
	Project: Rudolf Residence	Job No.: 15227.10	

Foundation Key Plan

* loads on Walls in (plf), loads on columns in (lb)

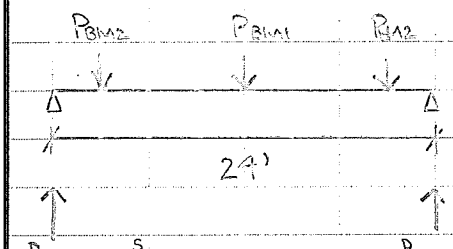


C.G ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: Foundation Key Plan	By: MBB	Date: 3/5/18
		Checked:	Date:
		Scale: NTS	Sheet No.
	Project: Rudolf Residence	Job No. 15227.10	1-9

GT 2

$P_{BM1} = \left(\frac{7.5'}{2}\right)(120\,plf^{D} + 200\,plf^{S})$ @ 12'

$= 450\,lb^{D} + 750\,lb^{S}$

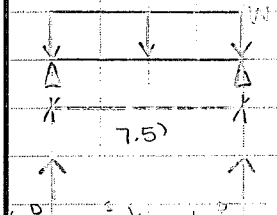


$P_{BM2} = \left(\frac{7.5'}{2}\right)(90\,plf^{D} + 150\,plf^{S})$ @ 4', 20'

$= 338\,lb^{D} + 563\,lb^{S}$

917 lb + 983 lb 917 lb + 983 lb * only gravity reactions

BM1



$W = \left(\frac{16'}{2}\right)(15\,psf^{D} + 25\,psf^{S})$

$= 120\,plf^{D} + 200\,plf^{S}$

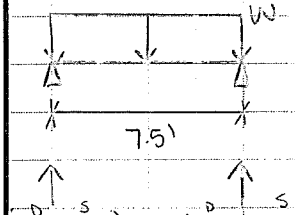
$= 320\,plf$

(450 + 750) lb (450 + 750) lb

$W_{allow} = 405\,plf$ (79%)

Per Roof Beam Span Table, 4x8 HF #2 is adequate (minimum)

BM2



$W = \left(\frac{(8' + 4')}{2}\right)(15\,psf^{D} + 25\,psf^{S})$

$= 90\,plf^{D} + 150\,plf^{S}$

$= 240\,plf$

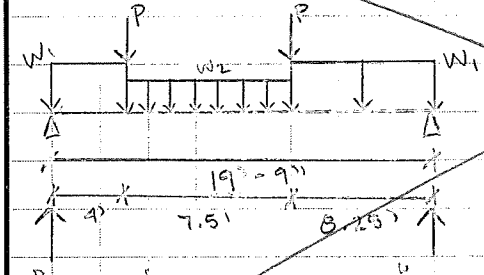
$V_{max} = 915\,lb$ (36%)

$M_{max} = 1715\,lb\text{-}ft$ (92%)

(333 + 567) lb (355 + 567) lb

Per Woodworks, 4x6 HF #2 is adequate

BM3



$W_1 = \left(\frac{(4' + 14')}{2}\right)(15\,psf^{D} + 25\,psf^{S})$ 0' → 4', 11.5' → 19.75'

$= 210\,plf^{D} + 350\,plf^{S}$

$W_2 = \left(\frac{(4' + 4')}{2}\right)(15\,psf^{D} + 25\,psf^{S})$ 4' → 11.5'

$= 60\,plf^{D} + 100\,plf^{S}$

$P = 917\,lb^{D} + 983\,lb^{S}$ @ 9', 11.5'

2766 lb + 3177 lb 2766 lb + 3177 lb

$V_{max} = 6425\,lb$ (30%), $M_{max} = 30122\,lb\text{-}ft$ (53%)

Per Woodworks, Use 5 1/4" x 16" 20F PSL

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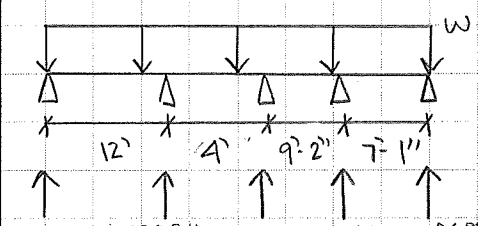
		Checked	Date
		Scale: NTS	Sheet No.: 1-10
	Project: Rudolf Residence	Job No.: 15227.15	

GT2

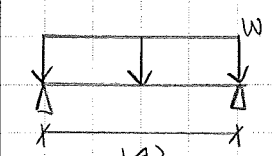
$$w = \left(\frac{11'}{2}\right)\left(15^{D}_{psf} + 25^{S}_{psf}\right)$$
$$= 83^{D}_{plf} + 138^{S}_{plf}$$

* only need reactions



533 lb	1283 lb	3791 lb	1063 lb	269 lb (D)
+ 678 lb	+ 1741 lb	+ 8301 lb	+ 1350 lb	+ 388 lb (S)
= 1211 lb	= 3024 lb	= 1209 lb	= 2412 lb	= 657 lb

BM4



$$w = \left(\frac{22'}{2}\right)\left(15^{D}_{psf} + 25^{S}_{psf}\right)$$
$$= 165^{D}_{plf} + 275^{S}_{plf}$$
$$= 440_{plf}$$

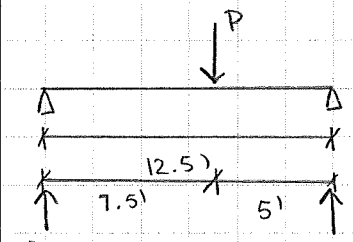
$w_{allow} = 540$ plf (81%)

$(1155^{D} + 1925^{S})$ $(1155^{D} + 1925^{S})$

Per Roof Beam Span Table, 3½ x 9½ 2.0E PSL is adequate

BM5

$$P = 667^{D}\text{ lb} + 1116^{S}\text{ lb}$$



$V_{max} = 1156$ lb (9%)

$M_{max} = 5699$ lb·ft (17%)

377^{D} lb + 447^{S} 510^{D} lb + 669^{S} lb

Per Woodworks, use 3½" x 16" 2.0E PSL

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

Description: Gravity Design		By: MBB	Date: 3/1/18
		Checked	Date
		Scale: NTS	Sheet No.
Project: Rudolf Residence		Job No.: 15227.15	1-12

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:09	BM5

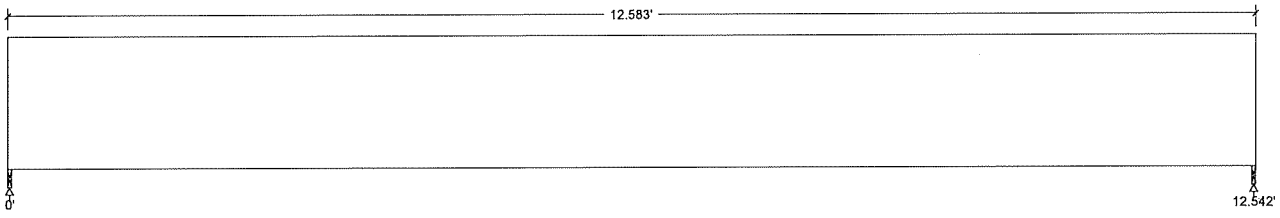
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Pd	Dead	Point		7.54		667		lbs
Ps	Snow	Point		7.54		1116		lbs
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	377		510
Snow	447		669
Factored:			
Total	824		1179
Bearing:			
Capacity			
Beam	1313		1313
Support	1211		1211
Des ratio			
Beam	0.63		0.90
Support	0.68		0.97
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	1.11		1.11
Fcp sup	625		625

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

BM5

PSL, PSL, 2.0E, 3-1/2"x16"

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

Total length: 12.58'; Clear span: 12.5'; volume = 4.9 cu.ft.

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

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 31	Fv' = 334	psi	fv/Fv' = 0.09
Bending(+)	fb = 458	Fb' = 2689	psi	fb/Fb' = 0.17
Live Defl'n	0.03 = <L/999	0.42 = L/360	in	0.08
Total Defl'n	0.07 = <L/999	0.63 = L/240	in	0.10

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.15	-	1.00	0.806	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D+S, V max = 1156, V design = 1156 lbs

Bending(+): LC #2 = D+S, M = 5699 lbs-ft

Deflection: LC #2 = D+S (live)

LC #2 = D+S (total)

D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake

All LC's are listed in the Analysis output

Load combinations: ASCE 7-10 / IBC 2015

CALCULATIONS:

Deflection: EI = 2389e06 lb-in2

"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)

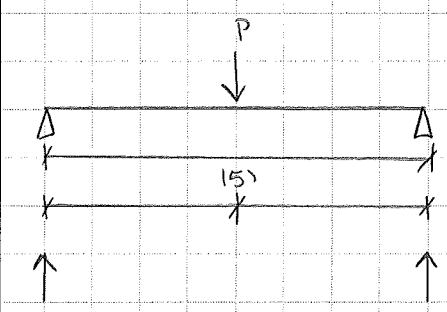
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.

Lateral stability(+): Lu = 12.56' Le = 24.44' RB = 19.6

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

BM6



465 lb (D) + 558 lb (S)

465 lb (D) + 558 lb (S)

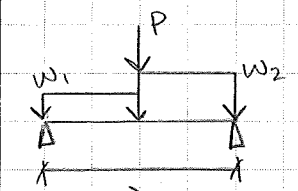
$P = 667^{D}\text{ lb} + 1116^{S}\text{ lb}$

$V_{max} = 999$ lb (8%)

$M_{max} = 7178$ lb·ft (29%)

Per woodworks, 3 1/2 x 16" PSL adequate

BM7



606 (D) + 483 (S)

606 (D) + 483 (S)

$w_1 = 173^{D}\text{ plf} + 220^{L}\text{ plf}$

$w_2 = 263^{D}\text{ plf} + 150^{S}\text{ plf} + 220^{L}\text{ plf}$

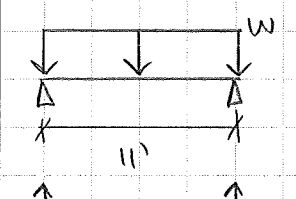
$P = 533^{D}\text{ lb} + 678^{S}\text{ lb}$

$V_{max} = 2271$ lb (50%)

$M_{max} = 4501$ lb·ft (99%)

Per wood works, 4 x 10 HF #2 is adequate

BM8



897 (D) + 1133 (S)

897 (D) + 1133 (S)

$w = \left(\frac{15.5' + 5'}{2}\right)\left(15^{D}\text{ psf} + 40^{L}\text{ psf}\right)$

$w = 154^{D}\text{ plf} + 410^{L}\text{ plf}$

$w = 564$ plf

Use 3 1/2 x 16 2.0E PSL

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: Gravity Design	By: MBB	Date: 3/1/18
		Checked	Date
		Scale: NTS	Sheet No.: 1 - 14
	Project: Rudolf Residence	Job No.: 15227.15	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:10	BM6

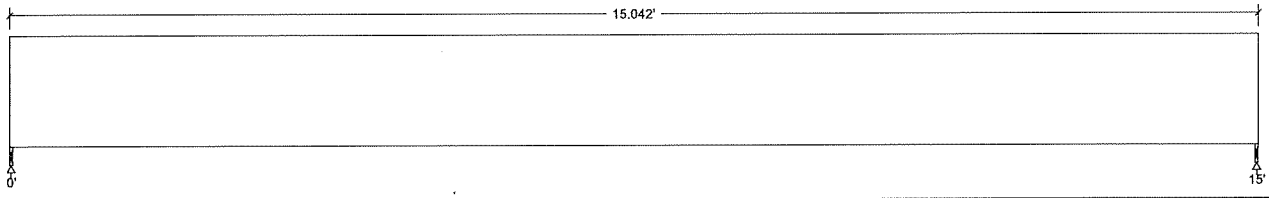
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Pd	Dead	Point		7.52		667		lbs
Ps	Snow	Point		7.52		1116		lbs
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	465		465
Snow	558		558
Factored:			
Total	1023		1023
Bearing:			
Capacity			
Beam	1313		1313
Support	1211		1211
Des ratio			
Beam	0.78		0.78
Support	0.84		0.84
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	1.11		1.11
Fcp sup	625		625

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

BM6

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 15.04'; Clear span: 14.958'; volume = 5.8 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 27	Fv' = 334	psi	fv/Fv' = 0.08
Bending(+)	fb = 577	Fb' = 2451	psi	fb/Fb' = 0.24
Live Defl'n	0.06 = <L/999	0.50 = L/360	in	0.11
Total Defl'n	0.12 = <L/999	0.75 = L/240	in	0.16

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.15	-	1.00	0.735	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+S, V max = 999, V design = 999 lbs
Bending(+): LC #2 = D+S, M = 7178 lbs-ft
Deflection: LC #2 = D+S (live)
LC #2 = D+S (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 15.00' Le = 28.44' RB = 21.1

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 2, 2018 12:15	BM7

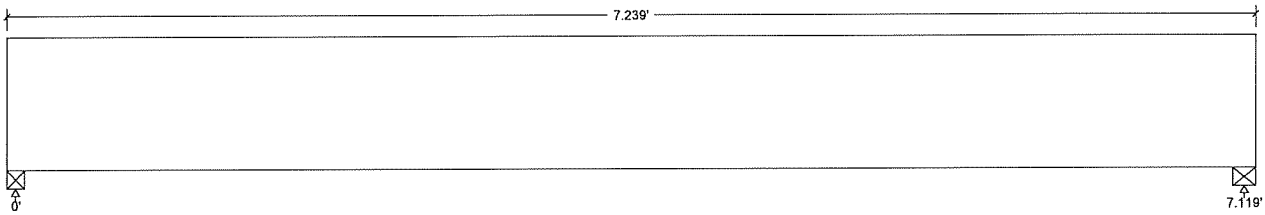
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
d1	Dead	Partial UDL		0.11	4.11	173.0	173.0	plf
l1	Live	Partial UDL		0.11	4.11	220.0	220.0	plf
d2	Dead	Partial UDL		4.11	7.11	263.0	263.0	plf
l2	Live	Partial UDL		4.11	7.11	220.0	220.0	plf
s2	Snow	Partial UDL		4.11	7.11	150.0	150.0	plf
d3	Dead	Point		4.11		533		lbs
s3	Snow	Point		4.11		678		lbs
Self-weight	Dead	Full UDL				6.7		plf

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



Unfactored:			
Dead	919		1142
Live	771		769
Snow	391		737
Factored:			
Total	1791		2271
Bearing:			
Capacity			
Beam	1791		2271
Support	3061		3881
Des ratio			
Beam	1.00		1.00
Support	0.59		0.59
Load comb	#3		#3
Length	1.26		1.60
Min req'd	1.26		1.60
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

BM8

Lumber-soft, Hem-Fir, No.2, 4x10 (3-1/2"x9-1/4")

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 7.24'; Clear span: 7.0'; volume = 1.6 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 86	Fv' = 172	psi	fv/Fv' = 0.50
Bending(+)	fb = 1082	Fb' = 1153	psi	fb/Fb' = 0.94
Live Defl'n	0.06 = <L/999	0.24 = L/360	in	0.26
Total Defl'n	0.16 = L/542	0.36 = L/240	in	0.44

Additional Data:

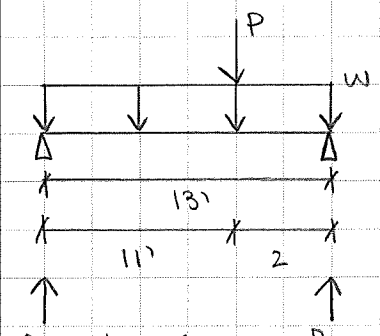
FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	150	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	3
Fb'+	850	1.15	1.00	1.00	0.983	1.200	1.00	1.00	1.00	1.00	-	3
Fcp'	405	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.3 million		1.00	1.00	-	-	-	-	1.00	1.00	-	3
Emin'	0.47 million		1.00	1.00	-	-	-	-	1.00	1.00	-	3

CRITICAL LOAD COMBINATIONS:
Shear : LC #3 = D+.75(L+S), V max = 2271, V design = 1859 lbs
Bending(+): LC #3 = D+.75(L+S), M = 4501 lbs-ft
Deflection: LC #3 = D+.75(L+S) (live)
LC #3 = D+.75(L+S) (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 300e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 7.13' Le = 13.94' RB = 11.2

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design 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.

BM9



$P = 269^{D}\ lb + 388^{S}\ lb$

$w = \left(\frac{5' + 11.5'}{2}\right)(15^{D}\ psf + 40^{L}\ psf)$

$= 124^{D}\ plf + 330^{L}\ plf$

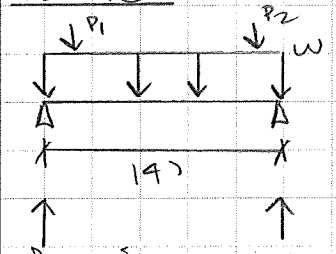
$(977^{D} + 2181^{L} + 611^{S})\ lb \qquad (1162^{D} + 2182^{L} + 327^{S})\ lb$

$V_{max} = 3318\ lb\ (25\%)$

$M_{max} = 10411\ lb \cdot ft\ (39\%)$

Per Woodworks, 3½ x 16 PSL is adequate

BM10



$w = 86^{D}\ plf + 230^{L}\ plf$

$P_1 = P_2 = (6')(15^{D}\ psf + 25^{S}\ psf)\left(\frac{12'}{2}\right) = 540^{D} + 900^{S}$ @ 1', 13'

$(1277^{D} + 2590^{S})\ lb \qquad (1277^{D} + 2590^{S})\ lb$

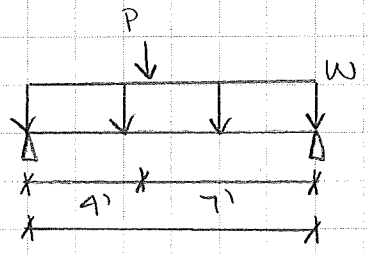
$V_{max} = 3796\ lb\ (24\%)$

$M_{max} = 9859\ lb \cdot ft\ (31\%)$

Per Woodworks, 3½ x 16 2.0E PSL is adequate

BM11

* Use worst case stair loading



$w = 90^{D}\ plf + 240^{L}\ plf$

$P = \left(\frac{4'}{2}\right)\left(\frac{5.5'}{2}\right)(15^{D}\ psf + 40^{L}\ psf)$

$= 83^{D}\ lb + 220^{L}\ lb$

$(612^{D} + 1977^{L})\ lb \qquad (589^{D} + 1417^{L})\ lb$

$V_{max} = 2077\ lb\ (28\%)$

$M_{max} = 5894\ lb \cdot ft\ (98\%)$

Per Woodworks, 3½ x 9½ 2.0E PSL is adequate

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

Description: Gravity Design		By: MBB	Date: 3/1/18
		Checked	Date
		Scale: NTS	Sheet No.: 1-17
Project: Rudolf Residence		Job No.: 15227.15	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:43	BM9

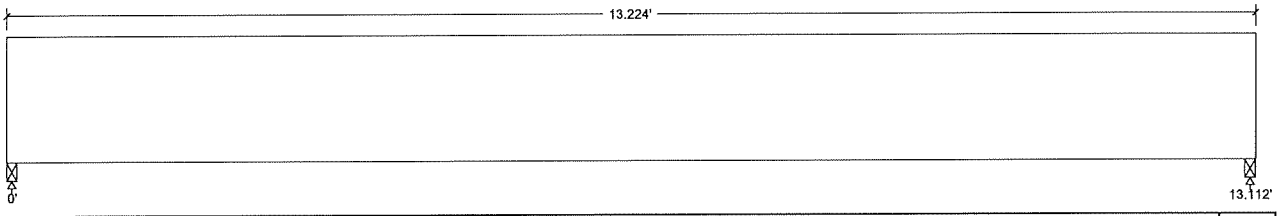
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Pd	Dead	Point		11.11		269		lbs
Ps	Snow	Point		11.11		388		lbs
d	Dead	Full UDL				124.0		plf
l	Live	Full UDL				330.0		plf
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	977		1162
Live	2181		2182
Snow	61		327
Factored:			
Total	3158		3344
Bearing:			
Capacity			
Beam	3423		3625
Support	3158		3344
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#2		#2
Length	1.30		1.38
Min req'd	1.30**		1.38**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM9

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 13.22'; Clear span: 13'; volume = 5.1 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 71	Fv' = 290	psi	fv/Fv' = 0.25
Bending(+)	fb = 837	Fb' = 2454	psi	fb/Fb' = 0.34
Live Defl'n	0.09 = <L/999	0.44 = L/360	in	0.21
Total Defl'n	0.16 = <L/999	0.66 = L/240	in	0.24

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.846	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 3318, V design = 2662 lbs
Bending(+): LC #2 = D+L, M = 10411 lbs-ft
Deflection: LC #2 = D+L (live)
LC #2 = D+L (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
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: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow...)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 13.13' Le = 25.38' RB = 19.9

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:44	BM10

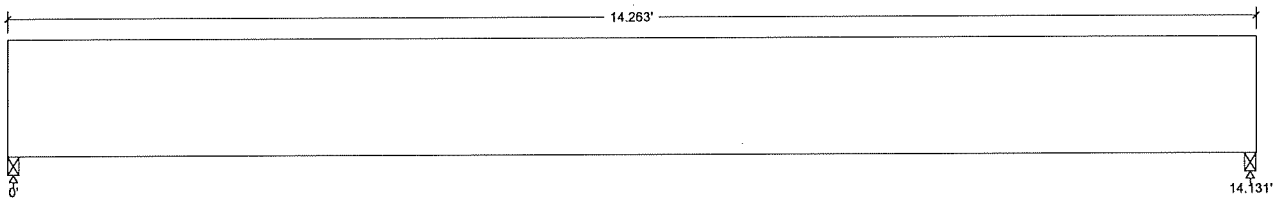
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pattern	Location [ft] Start	End	Magnitude Start	End	Unit
d	Dead	Full UDL				86.0		plf
s	Snow	Full UDL				230.0		plf
d1	Dead	Point		1.13		540		lbs
d2	Dead	Point		13.13		540		lbs
s1	Snow	Point		1.13		900		lbs
s2	Snow	Point		13.13		900		lbs
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	1277		1277
Snow	2540		2540
Factored:			
Total	3817		3817
Bearing:			
Capacity			
Beam	4137		4137
Support	3817		3817
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#2		#2
Length	1.58		1.58
Min req'd	1.58**		1.58**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM10

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 14.26'; Clear span: 14'; volume = 5.5 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 80	Fv' = 334	psi	fv/Fv' = 0.24
Bending(+)	fb = 792	Fb' = 2535	psi	fb/Fb' = 0.31
Live Defl'n	0.10 = <L/999	0.47 = L/360	in	0.22
Total Defl'n	0.18 = L/956	0.71 = L/240	in	0.25

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.15	-	1.00	0.760	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+S, V max = 3796, V design = 2970 lbs
Bending(+): LC #2 = D+S, M = 9859 lbs-ft
Deflection: LC #2 = D+S (live)
LC #2 = D+S (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow...)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 14.13' Le = 27.06' RB = 20.6

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:44	BM11

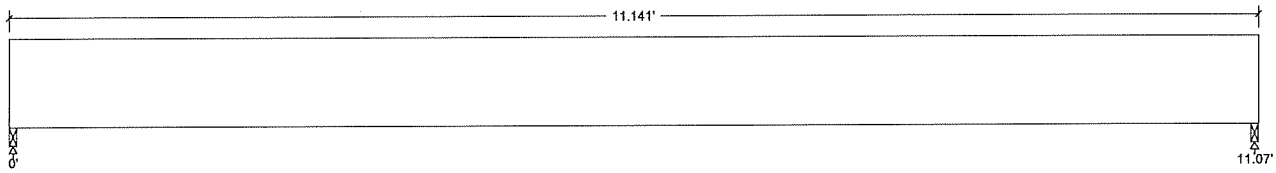
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
dl	Dead	Full UDL				90.0		plf
ll	Live	Full UDL				240.0		plf
Pdl	Dead	Point		4.07		83		lbs
Pll	Live	Point		4.07		220		lbs
Self-weight	Dead	Full UDL				10.4		plf

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



Unfactored:			
Dead	612		589
Live	1477		1417
Factored:			
Total	2089		2006
Bearing:			
Capacity			
Beam	2264		2174
Support	2089		2006
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#2		#2
Length	0.86		0.83
Min req'd	0.86**		0.83**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM11

PSL, PSL, 2.0E, 3-1/2"x9-1/2"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 11.14'; Clear span: 11'; volume = 2.6 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 81	Fv' = 290	psi	fv/Fv' = 0.28
Bending(+)	fb = 1332	Fb' = 2796	psi	fb/Fb' = 0.48
Live Defl'n	0.18 = L/731	0.37 = L/360	in	0.49
Total Defl'n	0.29 = L/451	0.55 = L/240	in	0.53

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.964	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

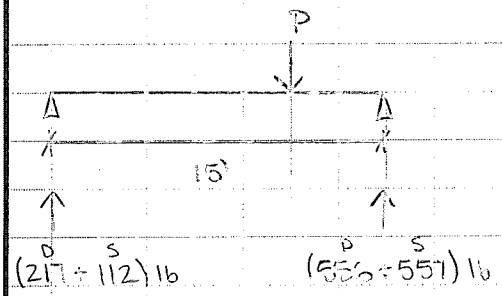
CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 2077, V design = 1795 lbs
Bending(+): LC #2 = D+L, M = 5844 lbs-ft
Deflection: LC #2 = D+L (live)
LC #2 = D+L (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 500e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow...)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 11.06' Le = 20.44' RB = 13.8

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

BM12

$P = 510^{D}\text{ lb} + 669^{S}\text{ lb}$ @ 12.5'



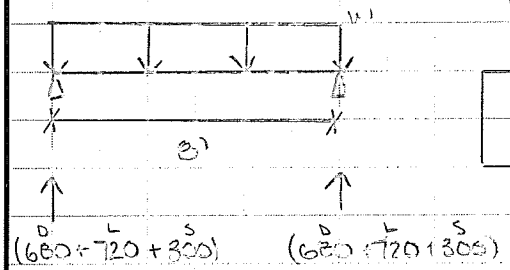
V_{max} = 1095 lb (9%)

M_{max} = 2750 lb-ft (9%)

Per Wood Works 3 1/2 x 16" 2.0E PSL is adequate

BM13

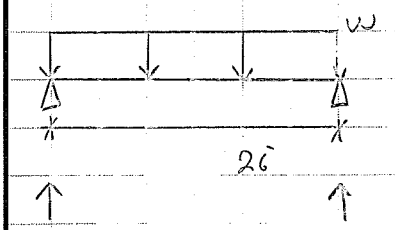
$w = 170^{D} + 180^{L} + 75^{S} = 425$ plf



Per Roof Beam Span Table, 3 1/2 x 7 1/4 LSL is adequate.

BM14

$w = \left(\frac{6.5'}{2}\right)(15^{D}\text{ psf} + 60^{L}\text{ psf} + 25^{S}\text{ psf})$



V_{max} = 1795 lb (26%)

M_{max} = 9876 lb-ft (75%)

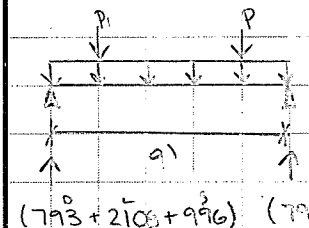
$(877^{D} + 2586^{L} + 1078^{S})$ lbs

Per woodworks, 5 1/8" x 18" 20F-1.5E Glulam Western Species is adequate

BM15

$P = (555^{D} + 1230^{L} + 783^{S})$ lb @ 1' & 8'

$w = (15/2 \times 15^{D} + 60^{L} + 25^{S})$ psf



V_{max} = 3393 lb (47%)

M_{max} = 9985 lb-ft (27%)

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

Description: Gravity Design		By: MBB	Date: 3/1/18
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Residence		Job No. 15227.10	1 - 21

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:45	BM12

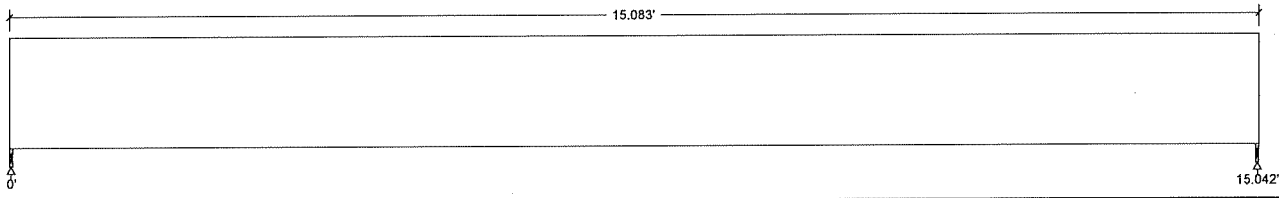
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Pd	Dead	Point		12.54		510		lbs
Ps	Snow	Point		12.54		669		lbs
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	217		556
Snow	112		557
Factored:			
Total	329		1113
Bearing:			
Capacity			
Beam	1313		1313
Support	1211		1211
Des ratio			
Beam	0.25		0.85
Support	0.27		0.92
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	1.11		1.11
Fcp sup	625		625

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

BM12

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 15.08'; Clear span: 15'; volume = 5.9 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 29	Fv' = 334	psi	fv/Fv' = 0.09
Bending(+)	fb = 221	Fb' = 2447	psi	fb/Fb' = 0.09
Live Defl'n	0.02 = <L/999	0.50 = L/360	in	0.03
Total Defl'n	0.05 = <L/999	0.75 = L/240	in	0.06

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.15	-	1.00	0.734	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:

```
Shear     : LC #2  = D+S, V max =   1090, V design =   1090 lbs
Bending(+): LC #2  = D+S, M =    2750 lbs-ft
Deflection: LC #2  = D+S  (live)
            LC #2  = D+S  (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
```

CALCULATIONS:

```
Deflection:  EI =   2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+):  Lu = 15.06'  Le = 28.50'  RB = 21.1
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

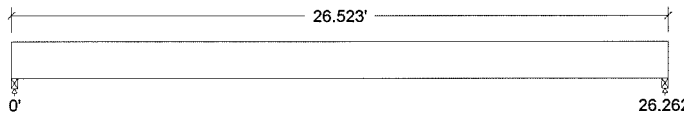
WoodWorks®	COMPANY	PROJECT
	May 4, 2018 10:31	BM14-B

Design Check Calculation Sheet
WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
d	Dead	Full Area				15.00(3.25')		psf
s	Snow	Full Area				25.00(3.25')		psf
Load4	Live	Full Area				60.00(3.25')		psf
Self-weight	Dead	Full UDL				17.6		plf

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



Unfactored:			
Dead	877		877
Live	2586		2586
Snow	1078		1078
Factored:			
Total	3625		3625
Bearing:			
Capacity			
Beam	3625		3625
Support	7232		7232
Des ratio			
Beam	1.00		1.00
Support	0.50		0.50
Load comb	#3		#3
Length	3.14		3.14
Min req'd	3.14		3.14
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.07		1.07
Fcp sup	625		625

BM14

Glulam-Bal., West Species, 20F-1.5E WS, 5-1/8"x18"

12 laminations, 5-1/8" maximum width,
Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 26.52'; Clear span: 26'; volume = 17.0 cu.ft.
Service: wet; Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 49	Fv' = 171	psi	fv/Fv' = 0.29
Bending(+)	fb = 977	Fb' = 1301	psi	fb/Fb' = 0.75
Live Defl'n	0.71 = L/442	0.88 = L/360	in	0.81
Total Defl'n	1.17 = L/269	1.31 = L/240	in	0.89

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	195	1.00	0.88	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2000	1.00	0.80	1.00	0.813	0.939	1.00	1.00	1.00	1.00	-	2
Fcp'	425	-	0.53	1.00	-	-	-	-	1.00	-	-	-
E'	1.5 million		0.83	1.00	-	-	-	-	1.00	-	-	3
Eminy'	0.63 million		0.83	1.00	-	-	-	-	1.00	-	-	3

Only the lesser of CL and CV is applied, as per NDS 5.3.6

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D+L, V max = 3431, V design = 3005 lbs
Bending(+): LC #2 = D+L, M = 22529 lbs-ft
Deflection: LC #3 = D+.75(L+S) (live)
LC #3 = D+.75(L+S) (total)

D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015

CALCULATIONS:

Deflection: EI = 3736e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 2.00(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 26.25' Le = 48.31' RB = 19.9

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:46	BM15

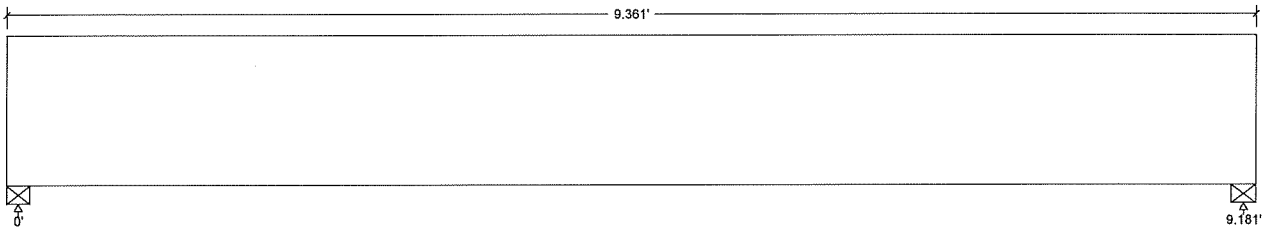
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
dl	Dead	Full Area				15.00(2.50')		psf
ll	Live	Full Area				60.00(2.50')		psf
sl	Snow	Full Area				25.00(2.50')		psf
Pd	Dead	Point		1.00		555		lbs
Pd2	Dead	Point		8.00		555		lbs
Pl1	Live	Point		1.00		1250		lbs
Pl2	Live	Point		8.00		1250		lbs
Ps1	Snow	Point		1.00		781		lbs
Ps2	Live	Point		8.00		781		lbs
Self-weight	Dead	Full UDL				9.0		plf

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



Unfactored:			
Dead	793		750
Live	2108		2577
Snow	996		370
Factored:			
Total	3121		3328
Bearing:			
Capacity			
Beam	3121		3328
Support	5082		5418
Des ratio			
Beam	1.00		1.00
Support	0.61		0.61
Load comb	#3		#2
Length	2.10		2.24
Min req'd	2.10		2.24
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

BM15

Glulam-Bal., West Species, 20F-1.5E WS, 3-1/2"x13-1/2"

9 laminations, 3-1/2" maximum width,
Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 9.36'; Clear span: 9'; volume = 3.1 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 97	Fv' = 195	psi	fv/Fv' = 0.50
Bending(+)	fb = 521	Fb' = 1862	psi	fb/Fb' = 0.28
Live Defl'n	0.05 = <L/999	0.31 = L/360	in	0.17
Total Defl'n	0.08 = <L/999	0.46 = L/240	in	0.17

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	195	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2000	1.00	1.00	1.00	0.931	1.000	1.00	1.00	1.00	1.00	-	2
Fcp'	425	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.5 million	1.00	1.00	-	-	-	-	-	1.00	-	-	2
Eminy'	0.63 million	1.00	1.00	-	-	-	-	-	1.00	-	-	2

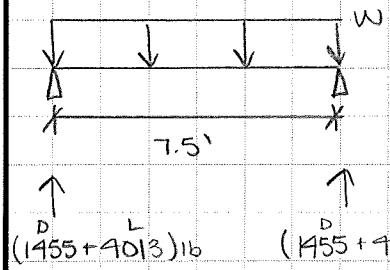
CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 3062, V design = 3062 lbs
Bending(+): LC #2 = D+L, M = 4614 lbs-ft
Deflection: LC #2 = D+L (live)
LC #2 = D+L (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 1076e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow...)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 9.19' Le = 18.31' RB = 15.6

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

BM16 *use worst case

$W = 388^{D}\ plf + 1070^{L}\ plf$



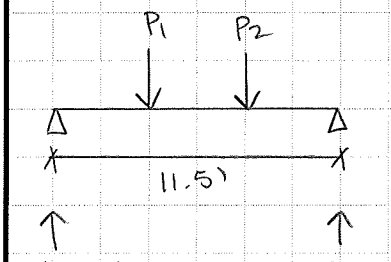
$(1455^{D} + 4013^{L})\ lb$ $(1455^{D} + 4013^{L})\ lb$

$V_{max} = 5681\ lb\ (33\%)$

$M_{max} = 10937\ lb\cdot ft\ (32\%)$

Per woodworks, 3½ x 16" 2.0E PSL is adequate

BM17



$(1957^{D} + 1488^{L} + 1727^{S})\ lb$ $(1838^{D} + 1052^{L} + 1894^{S})\ lb$

$P_1 = 1277^{D}\ lb + 2510^{L}\ lb$ @ 4'-9" (4.75')

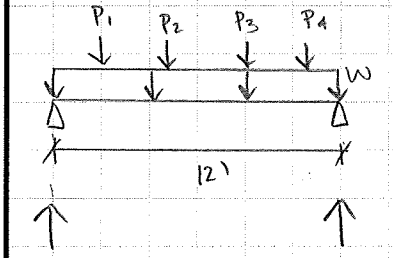
$P_2 = 2314^{D}\ lb + 3611^{S}\ lb$ @ 6'

$V_{max} = 4367\ lb\ (35\%)$

$M_{max} = 22233\ lb\cdot ft\ (64\%)$

Per woodworks, 3½ x 16 2.0E PSL is adequate

BM18



8128 D
8354 L
5071 S

6105 D
5496 L
3472 S

$W = 810^{D}\ plf + 720^{L}\ plf + 450^{S}\ plf$

$P_1 = 1755^{D}\ lb + 1350^{S}\ lb + 2520^{L}\ lb$ @ 1.33' (BM21)

$P_2 = 810^{D}\ lb + 675^{S}\ lb + 1080^{L}\ lb$ @ 1.83'

$P_3 = 810^{D}\ lb + 675^{S}\ lb + 1080^{L}\ lb$ @ 4.83'

$P_4 = 217^{D}\ lb + 112^{S}\ lb$ @ 7' (BM12)

Per Woodworks, 5¼ x 16" 2.0E PSL is adequate

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: Gravity Design	By: MBB	Date: 3/5/18
		Checked	Date
		Scale: NTS	Sheet No.: 1-25
	Project: Rudolf Residence	Job No.: 15227.10	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:48	BM16

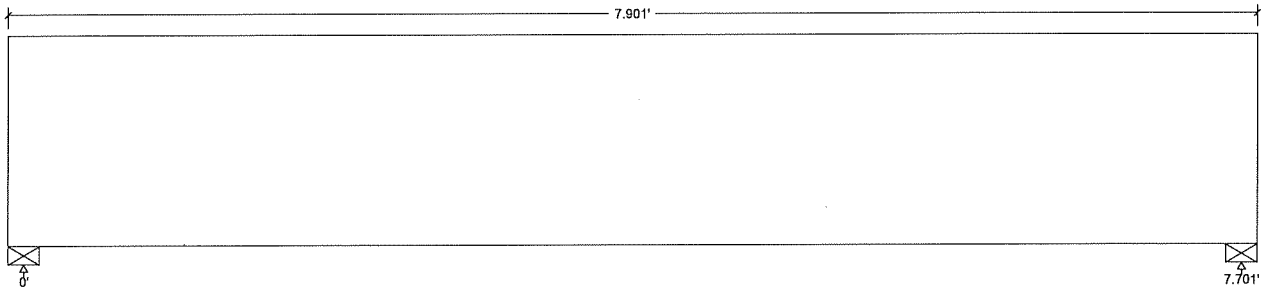
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
d	Dead	Full UDL				388.0		plf
l	Live	Full UDL				1070.0		plf
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	1600		1600
Live	4227		4227
Factored:			
Total	5827		5827
Bearing:			
Capacity			
Beam	6316		6316
Support	5827		5827
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#2		#2
Length	2.41		2.41
Min req'd	2.41**		2.41**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM16

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 7.9'; Clear span: 7.5'; volume = 3.1 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 96	Fv' = 290	psi	fv/Fv' = 0.33
Bending(+)	fb = 879	Fb' = 2735	psi	fb/Fb' = 0.32
Live Defl'n	0.04 = <L/999	0.26 = L/360	in	0.14
Total Defl'n	0.06 = <L/999	0.39 = L/240	in	0.14

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.943	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	2
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 5681, V design = 3566 lbs
Bending(+): LC #2 = D+L, M = 10937 lbs-ft
Deflection: LC #2 = D+L (live)
LC #2 = D+L (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 7.69' Le = 15.88' RB = 15.8

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 1, 2018 16:48	BM17

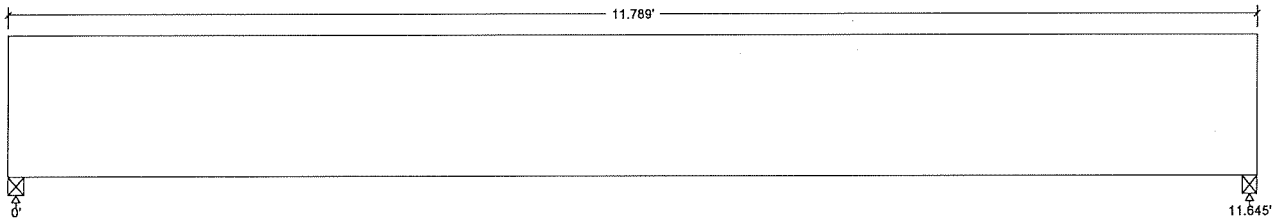
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Pd1	Dead	Point		4.90		1277		lbs
Pl1	Live	Point		4.90		2540		lbs
Pd2	Dead	Point		6.15		2314		lbs
Ps2	Snow	Point		6.15		3611		lbs
Self-weight	Dead	Full UDL				17.5		plf

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



Unfactored:			
Dead	1957		1838
Live	1488		1052
Snow	1727		1884
Factored:			
Total	4367		4041
Bearing:			
Capacity			
Beam	4734		4379
Support	4367		4041
Des ratio			
Beam	0.92		0.92
Support	1.00		1.00
Load comb	#3		#3
Length	1.80		1.67
Min req'd	1.80**		1.67**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM17

PSL, PSL, 2.0E, 3-1/2"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 11.79'; Clear span: 11.5'; volume = 4.6 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 116	Fv' = 334	psi	fv/Fv' = 0.35
Bending(+)	fb = 1787	Fb' = 2772	psi	fb/Fb' = 0.64
Live Defl'n	0.11 = <L/999	0.39 = L/360	in	0.28
Total Defl'n	0.24 = L/586	0.58 = L/240	in	0.41

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.15	-	1.00	-	-	-	-	1.00	-	1.00	3
Fb'+	2900	1.15	-	1.00	0.831	1.00	-	1.00	1.00	-	-	3
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million		-	1.00	-	-	-	-	1.00	-	-	3
Eminy'	1.04 million		-	1.00	-	-	-	-	1.00	-	-	3

CRITICAL LOAD COMBINATIONS:
Shear : LC #3 = D+.75(L+S), V max = 4367, V design = 4343 lbs
Bending(+): LC #3 = D+.75(L+S), M = 22233 lbs-ft
Deflection: LC #3 = D+.75(L+S) (live)
LC #3 = D+.75(L+S) (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 2389e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 11.63' Le = 23.00' RB = 19.0

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN

COMPANY	PROJECT
Mar. 6, 2018 13:34	BM18

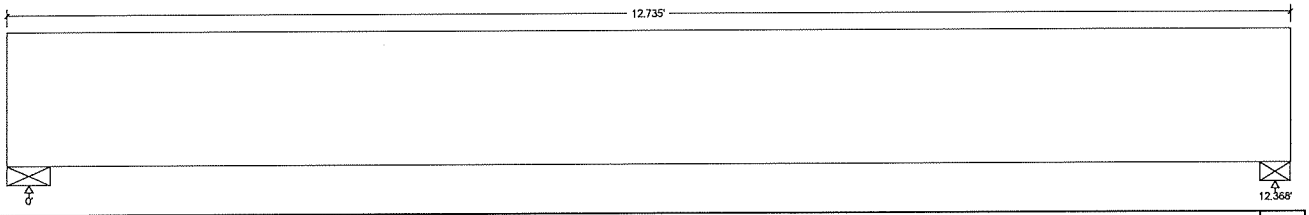
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
d1	Dead	Full UDL				810.0		plf
d2	Dead	Point		1.76		1755		lbs
d3	Dead	Point		2.26		810		lbs
d4	Dead	Point		5.26		810		lbs
d5	Dead	Point		7.43		217		lbs
l1	Live	Full UDL				720.0		plf
l2	Live	Point		1.76		2520		lbs
l3	Live	Point		2.26		1080		lbs
l4	Live	Point		5.26		1080		lbs
s1	Snow	Full UDL				450.0		plf
s2	Snow	Point		1.76		1350		lbs
s3	Snow	Point		2.26		675		lbs
s4	Snow	Point		5.26		675		lbs
s5	Snow	Point		7.43		112		lbs
Self-weight	Dead	Full UDL				26.2		plf

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



Unfactored:			
Dead	8128		6105
Live	8354		5496
Snow	5071		3472
Factored:			
Total	18196		12831
Bearing:			
Capacity			
Bearn	20380		14371
Support	18196		12831
Dem ratio			
Bearn	0.89		0.89
Support	1.00		1.00
Load comb	#3		#3
Length	5.18		3.65
Min req'd	5.18**		3.65**
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.

BM18

PSL, PSL, 2.0E, 5-1/4"x16"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 12.74'; Clear span: 12'; volume = 7.4 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv* = 245	Fv' = 290	psi	fv'/Fv' = 0.85
Bending(+)	fb = 2200	Fb' = 2814	psi	fb/Fb' = 0.78
Live Defl'n	0.19 = L/795	0.41 = L/360	in	0.45
Total Defl'n	0.43 = L/346	0.62 = L/240	in	0.69

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

Additional Data:

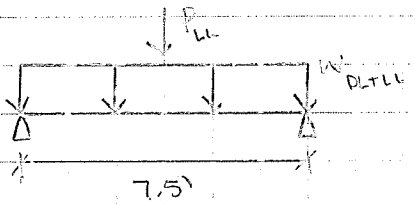
FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.970	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million	-	-	1.00	-	-	-	-	1.00	-	-	3
Eminy'	1.04 million	-	-	1.00	-	-	-	-	1.00	-	-	3

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 16151, V design* = 13743 lbs
Bending(+): LC #2 = D+L, M = 41070 lbs-ft
Deflection: LC #3 = D+.75(L+S) (live)
LC #3 = D+.75(L+S) (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 3584e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 12.38' Le = 24.19' RB = 13.0

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

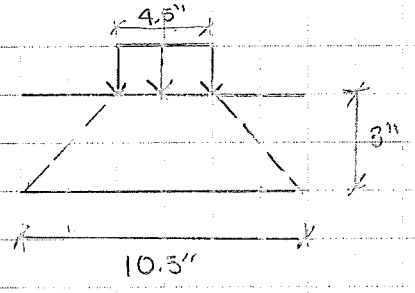
Garage



$W = (55 + 90)$ plf (DL + LL)

$P = 2000$ lb

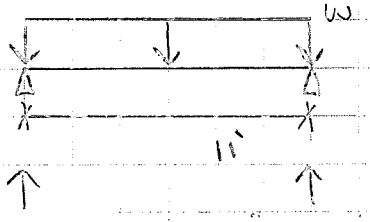
By inspection, Pt load controls



Concrete Topping

Per woodworks, 4x12 DF #2 joists are adequate

BM19



$W = \left(\frac{15'}{2}\right)(55\,psf + 40\,psf) + 2000\,lb$ (D, L; point load non-concurrent)

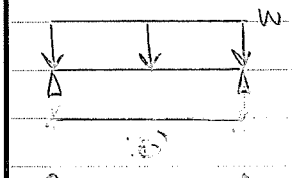
11,000 lbs

Per wood works, 5 1/8 x 11 7/8" 2.0E PSL is adequate.

Deck

BM20 at max tributary

$W = (90 + 360) = 450$ plf (D, L)



$V_{max} = 4203$ lb (41 %)

$M_{max} = 19819$ lb-ft (81 %)

4334 lb 4234 lb

Per Woodworks, 5 1/8 x 15" 20F 1.5E Glulam (use Alaska Cedar Glulam)

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description: Gravity Design	By: MBB	Date: 3/5/13
		Checked	Date
		Scale: NTS	Sheet No.: 1-29
	Project: Rudolf Residence	Job No.: 15227.10	

WoodWorks®	COMPANY	PROJECT
	May 10, 2018 10:34	BM19 - continuous

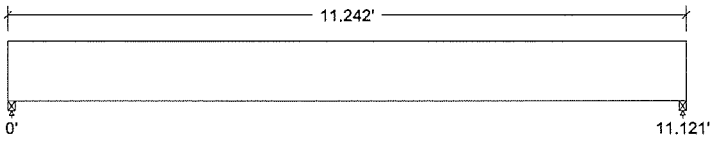
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
Load1	Dead	Full Area				55.00(7.50')		psf
Load2	Live	Full Area				40.00(7.50')		psf
Load3	Live	Point		5.50		2000		lbs
Self-weight	Dead	Full UDL				19.5		plf

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



Unfactored:			
Dead	2427		2427
Live	2708		2664
Factored:			
Total	5136		5091
Bearing:			
Capacity			
Beam	5752		5703
Support	5136		5091
Des ratio			
Beam	0.89		0.89
Support	1.00		1.00
Load comb	#2		#2
Length	1.46		1.45
Min req'd	1.46**		1.45**
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.

BM19

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

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

Total length: 11.24'; Clear span: 11'; volume = 4.9 cu.ft.

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

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 104	Fv' = 290	psi	fv/Fv' = 0.36
Bending(+)	fb = 1641	Fb' = 2861	psi	fb/Fb' = 0.57
Live Defl'n	0.13 = <L/999	0.37 = L/360	in	0.34
Total Defl'n	0.22 = L/612	0.56 = L/240	in	0.39

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.985	1.00	-	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, V max = 5092, V design = 4323 lbs

Bending(+): LC #2 = D+L, M = 16869 lbs-ft

Deflection: LC #2 = D+L (live)

LC #2 = D+L (total)

D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake

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: ASCE 7-10 / IBC 2015

CALCULATIONS:

Deflection: EI = 1612e06 lb-in2

"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)

Total Deflection = 1.00(Dead Load Deflection) + Live Load Deflection.

Lateral stability(+): Lu = 11.13' Le = 21.13' RB = 10.4

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 5, 2018 16:00	BM20

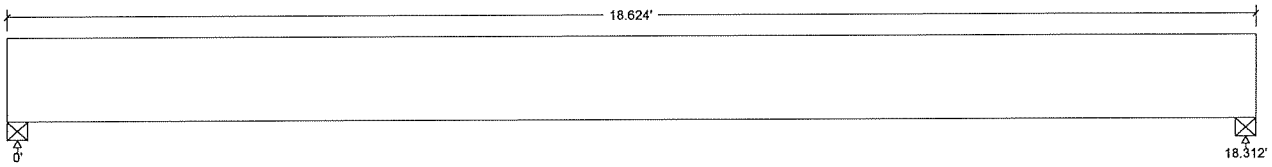
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
d	Dead	Full UDL				90.0		plf
l	Live	Full UDL				360.0		plf
Self-weight	Dead	Full UDL				14.6		plf

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



Unfactored:			
Dead	972		972
Live	3352		3352
Factored:			
Total	4325		4325
Bearing:			
Capacity			
Beam	4325		4325
Support	8628		8628
Des ratio			
Beam	1.00		1.00
Support	0.50		0.50
Load comb	#2		#2
Length	3.75		3.75
Min req'd	3.75		3.75
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.07		1.07
Fcp sup	625		625

BM20

Glulam-Bal., West Species, 20F-1.5E WS, 5-1/8"x15"

10 laminations, 5-1/8" maximum width,
Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 18.62'; Clear span: 18'; volume = 9.9 cu.ft.
Service: wet; Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 70	Fv' = 171	psi	fv/Fv' = 0.41
Bending(+)	fb = 1216	Fb' = 1507	psi	fb/Fb' = 0.81
Live Defl'n	0.51 = L/434	0.61 = L/360	in	0.83
Total Defl'n	0.80 = L/274	0.92 = L/240	in	0.87

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	195	1.00	0.88	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2000	1.00	0.80	1.00	0.942	0.991	1.00	1.00	1.00	1.00	-	2
Fcp'	425	-	0.53	1.00	-	-	-	-	1.00	-	-	-
E'	1.5 million		0.83	1.00	-	-	-	-	1.00	-	-	2
Eminy'	0.63 million		0.83	1.00	-	-	-	-	1.00	-	-	2

Only the lesser of CL and CV is applied, as per NDS 5.3.6

CRITICAL LOAD COMBINATIONS:

Shear : LC #2 = D+L, V max = 4254, V design = 3601 lbs
Bending(+): LC #2 = D+L, M = 19477 lbs-ft
Deflection: LC #2 = D+L (live)
LC #2 = D+L (total)

D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015

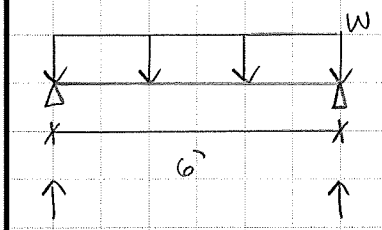
CALCULATIONS:

Deflection: EI = 2162e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow...)
Total Deflection = 2.00(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 18.31' Le = 33.69' RB = 15.2

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

BM21

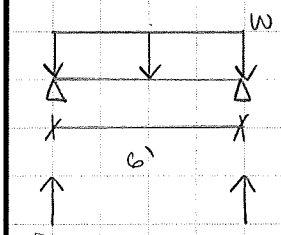


$W = 585_{D} + 840_{L} + 450_{S}$ plf
$= 1875$ plf

Wallow = 2467 plf (76%)

Per Beam Span Table,
3½ x 9½ 2.0E PSL is adequate

BM22

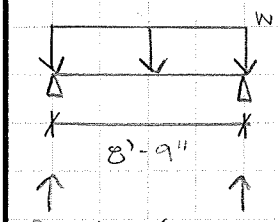


$W = 519_{D} + 188_{L} + 600_{S}$ plf
$= 1307$ plf

Wallow = 2467 plf (53%)

Per Beam Span Table,
3½ x 9½ 2.0E PSL is adequate

BM23

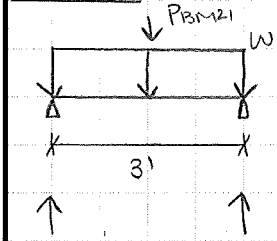


$W = 260_{D} + 180_{L} + 75_{S}$ plf
$= 515$ plf

Wallow = 1982 plf (34%)

Per Beam Span Table, 3½ x 9½ 2.0E PSL is adequate

BM24



$W = 585_{D} + 840_{L} + 450_{S}$ plf
$P_{BM21} = 1755_{D}$ lb + 2520_{L} lb + 1350_{S} lb
Vmax = 2160 lb (47%)
Mmax = 1620 lb·ft (58%)

Per Woodworks, 3½ x 7¼ 2.0E PSL is adequate

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

Description: Gravity Design		By: MBB	Date: 3/5/18
		Checked	Date
		Scale: NTS	Sheet No.: 1-32
Project: Rudolf Residence		Job No.: 15227.10	

WoodWorks® SOFTWARE FOR WOOD DESIGN	COMPANY	PROJECT
	Mar. 6, 2018 13:31	BM24

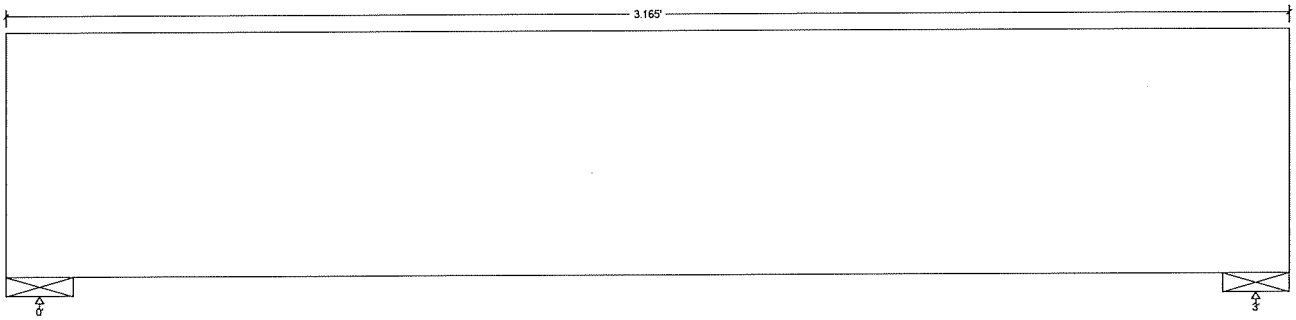
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pattern	Location [ft] Start	End	Magnitude Start	End	Unit
d	Dead	Full UDL				585.0		plf
l	Live	Full UDL				840.0		plf
s	Snow	Full UDL				450.0		plf
Pd	Dead	Point		1.58		1755		lbs
Pl	Live	Point		1.58		2520		lbs
Ps	Snow	Point		1.58		1350		lbs
Self-weight	Dead	Full UDL				7.9		plf

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



Unfactored:			
Dead	1815		1815
Live	2589		2589
Snow	1387		1387
Factored:			
Total	4798		4798
Bearing:			
Capacity			
Bean	5200		5200
Support	4798		4798
Des ratio			
Bean	0.92		0.92
Support	1.00		1.00
Load comb	#3		#3
Length	1.98		1.98
Min req'd	1.98**		1.98**
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

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

BM23

PSL, PSL, 2.0E, 3-1/2"x7-1/4"

Supports: All - Timber-soft Beam, D.Fir-L No.2
Total length: 3.17'; Clear span: 2.835'; volume = 0.6 cu.ft.
Lateral support: top= at supports, bottom= at supports;

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 195	Fv' = 290	psi	fv/Fv' = 0.67
Bending(+)	fb = 1886	Fb' = 2884	psi	fb/Fb' = 0.65
Live Defl'n	0.02 = <L/999	0.10 = L/360	in	0.21
Total Defl'n	0.04 = L/913	0.15 = L/240	in	0.26

Additional Data:

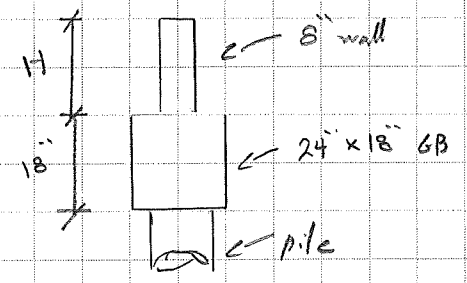
FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	290	1.00	-	1.00	-	-	-	-	1.00	-	1.00	2
Fb'+	2900	1.00	-	1.00	0.994	1.00	-	1.00	1.00	-	-	2
Fcp'	750	-	-	1.00	-	-	-	-	1.00	-	-	-
E'	2.0 million	-	-	1.00	-	-	-	-	1.00	-	-	3
Eminy'	1.04 million	-	-	1.00	-	-	-	-	1.00	-	-	3

CRITICAL LOAD COMBINATIONS:
Shear : LC #2 = D+L, V max = 4287, V design = 3303 lbs
Bending(+): LC #2 = D+L, M = 4818 lbs-ft
Deflection: LC #3 = D+.75(L+S) (live)
LC #3 = D+.75(L+S) (total)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lo=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASCE 7-10 / IBC 2015
CALCULATIONS:
Deflection: EI = 222e06 lb-in2
"Live" deflection = Deflection from all non-dead loads (live, wind, snow…)
Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
Lateral stability(+): Lu = 3.00' Le = 6.19' RB = 6.6

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2015), the National Design Specification (NDS 2015), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. SCL-BEAMS (Structural Composite Lumber): the attached SCL selection is for preliminary design only. For final member design contact your local SCL manufacturer.
4. Size factors vary from one manufacturer to another for SCL materials. They can be changed in the database editor.
5. FIRE RATING: LVL, PSL and LSL are not rated for fire endurance.

Typical Grade Beam



$H_{min} = 4'-0"$ (conservative assumption)

Foundation weight =

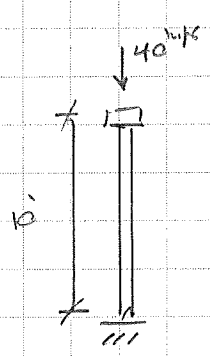
$(150_{pcf})\left[\frac{8}{12} \times 4' + 2 \times 1.5\right] = 850_{plf}$

Note: @ some locations wall height is over 4'-0". Extra weight is included w/ specific beam calc.

Per following calcs → 18" DEEP x 22" WIDE BEAM w/ (4) #6 bars T&B stirrups @ 7" o.c. $f_c' = 4000$ psi

Typical Pile

Per Geotech: • 16" ϕ pile capacity = 40 kips
• Assume point of fixity 10' below grade



Notes: 1.) Refer to additional calcs for lateral capacity of pile. Added axial load increases lateral capacity thus it was excluded from lateral calc.
2.) 40 kip load is assume Live Load for worst case load factor

Per Enercalc → 16" ϕ pile w/ (6) #6 bars $f_c' = 4000$ psi

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description Foundations	By PTH	Date 3/2/18
		Checked	Date
		Scale	Sheet No. 1-34
	Project Rudolf Res.	Job No. 15227.15	

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Concrete Column

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Typical Concrete Column

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

f'c : Concrete 28 day strength	=	4 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		ASTM A615 Bars Used
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

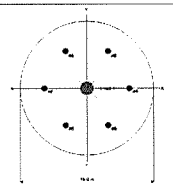
Overall Column Height = 10.0 ft
End Fixity: Top Free, Bottom Fixed

Brace condition for deflection (buckling) along columns
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 16.0in Diameter, Column Edge to Rebar
Edge Cover = 2.50in

Column Reinforcing : 6 - #6 bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 2,094.40 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft above base, L = 40.0 k

DESIGN SUMMARY

Load Combination +1.20D+1.60L
Location of max.above base 9.933 ft

Maximum Stress Ratio **0.125 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	66.513 k	φ * Pn =	531.06 k
Mu-x =	0.0 k-ft	φ * Mn-x =	0.0 k-ft
Mu-y =	0.0 k-ft	φ * Mn-y =	0.0 k-ft
Mu Angle =	0.0 deg		
Mu at Angle =	0.0 k-ft	φMn at Angle =	0.0 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	833.03 k
Pnmin : Nominal Min. Tension Axial Capacity	k
φ Pn, max : Usable Compressive Axial Capacity	531.06 k
φ Pn, min : Usable Tension Axial Capacity	k

Maximum SERVICE Load Reactions . .

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

Maximum SERVICE Load Deflections . . .

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

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

General Section Information φ = 0.750 β = 0.850 θ = 0.850
ρ : % Reinforcing 1.313 % Rebar % Ok
Reinforcing Area 2.640 in^2
Concrete Area 201.062 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D			9.93	2.93	531.06					0.000			0.006
+1.20D+1.60L			9.93	66.51	531.06					0.000			0.125
+1.20D+0.50L			9.93	22.51	531.06					0.000			0.042

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Concrete Column

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Typical Concrete Column

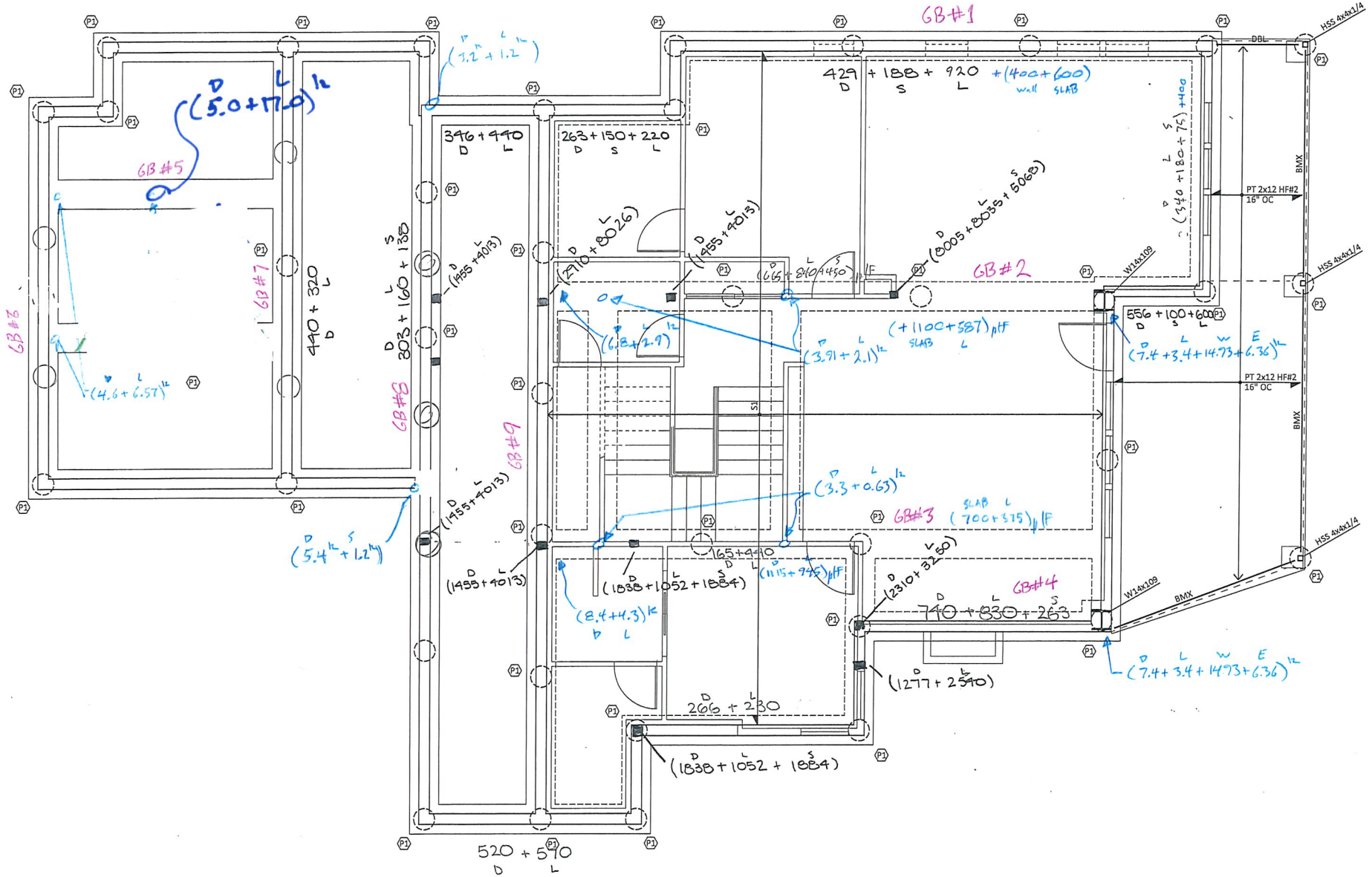
Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D			9.93	2.51	531.06					0.000			0.005
+0.90D			9.93	1.88	531.06					0.000			0.004

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Foundation key Plan



Concrete Beam

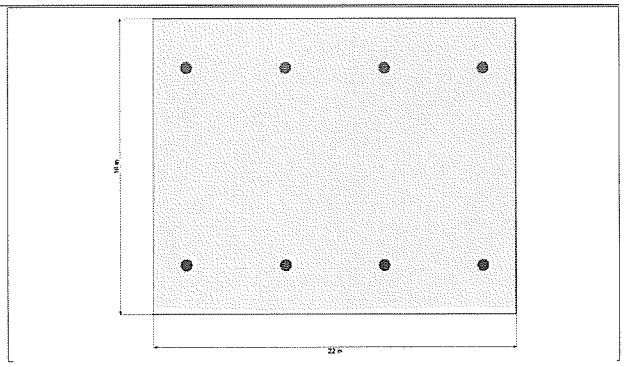
Description : Grade Beam #1

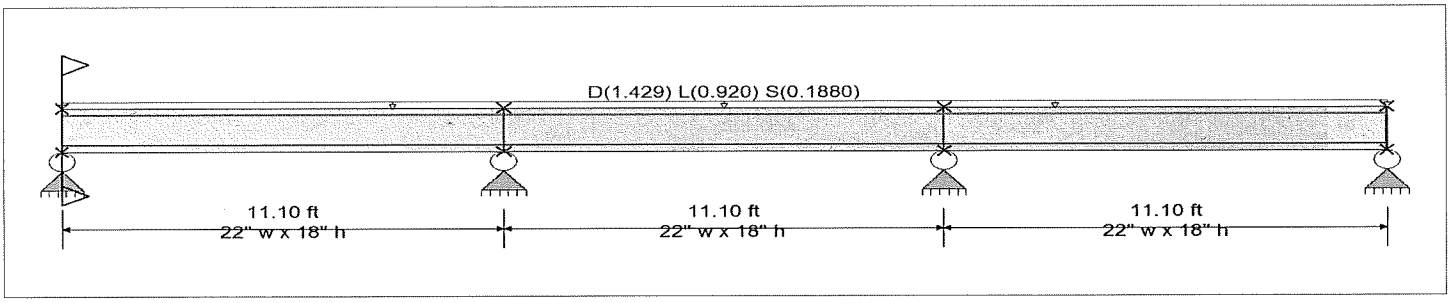
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

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





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.10 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.10 ft in this span

Span #2 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.10 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.10 ft in this span

Span #3 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.10 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.10 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 1.429, L = 0.920, S = 0.1880

Uniform Load on ALL spans : D = 1.429, L = 0.920, S = 0.1880 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.394** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.005 in	Ratio =	26852 >=36
Mu : Applied	-46.318 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable	117.418 k-ft	Max Downward Total Deflection	0.015 in	Ratio =	8990 >=18
Location of maximum on span	0.000 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 >=18
Span # where maximum occurs	Span # 3				

Shear Stirrup Requirements

Between 0.00 to 0.00 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 0.07 to 8.81 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 8.88 to 12.28 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 12.36 to 20.94 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 21.02 to 24.42 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 24.49 to 33.23 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

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Concrete Beam

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Lic. # : KW-06006155 — Licensee : CG ENGINEERING

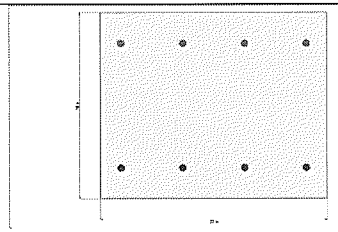
Description : Grade Beam #2

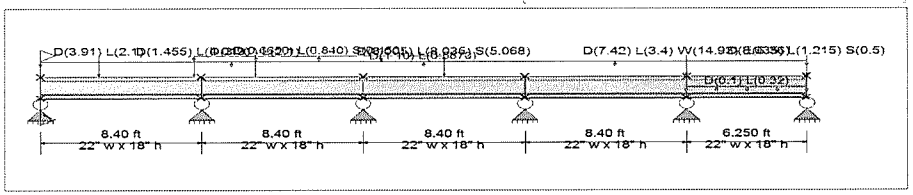
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	= 4.0 ksi	ϕ Phi Values	Flexure :	0.90	
fr = f'c^1/2 * 7.50	= 474.342 psi		Shear :	0.750	
ψ Density	= 145.0 pcf	β 1	=	0.850	
λ LtWt Factor	= 1.0				
Elastic Modulus	= 3,122.0 ksi	Fy - Stirrups		60.0 ksi	
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi	
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3	
		Number of Resisting Legs Per Stirru	=	2	





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 11.250 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 11.250 ft in this span
Span #2 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 11.750 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 11.750 ft in this span
Span #3 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span
Span #4 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span
Span #5 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 6.250 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 6.250 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 1.10, L = 0.5870

Uniform Load on ALL spans : D = 1.10, L = 0.5870 k/ft

Partial Length Uniform Load : D = 0.6650, L = 0.840, S = 0.450 k/ft, Extent = 8.0 -->> 21.0 ft

Load for Span Number 1
Point Load : D = 1.455, L = 4.013 k @ 8.0 ft

Point Load : D = 3.910, L = 2.10 k @ 3.080 ft

Load for Span Number 2
Point Load : D = 3.910, L = 2.10 k @ 2.80 ft

Load for Span Number 3

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Engineer:
Project Descr:

Project ID:

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06006155 — Licensee : CG ENGINEERING

Description : Grade Beam #2

Applied Loads

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

Point Load : D = 8.005, L = 8.035, S = 5.068 k @ 4.20 ft

Load for Span Number 5
Point Load : D = 7.420, L = 3.40, W = 14.930, E = 6.360 k @ 0.0 ft

Point Load : D = 8.035, L = 1.215, S = 0.50 k @ 6.250 ft

Uniform Load : D = 0.10, L = 0.320 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.407 : 1	Maximum Deflection			
Section used for this span	Typical Section	Max Downward Transient Deflection	0.003 in	Ratio =	31574 >=36
Mu : Applied	-47.769 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable	117.418 k-ft	Max Downward Total Deflection	0.007 in	Ratio =	13667 >=18
Location of maximum on span	0.000 ft	Max Upward Total Deflection	-0.001 in	Ratio =	82600 >=18
Span # where maximum occurs	Span # 3				

Shear Stirrup Requirements

Between 0.00 to 7.64 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 7.83 to 10.12 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 10.31 to 14.89 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 15.08 to 20.05 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 20.24 to 21.76 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 21.95 to 25.01 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 25.20 to 33.41 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 33.60 to 33.60 ft, PhiVc < Vu, Req'd Vs = 1.384, use stirrups spaced at 7.000 in
Between 33.74 to 39.71 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

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Title Block Line 6

Project Title:
Engineer:
Project Descr:
Project ID:

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

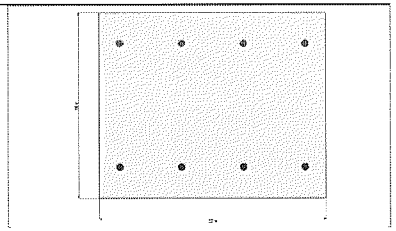
Description : Grade Beam #3

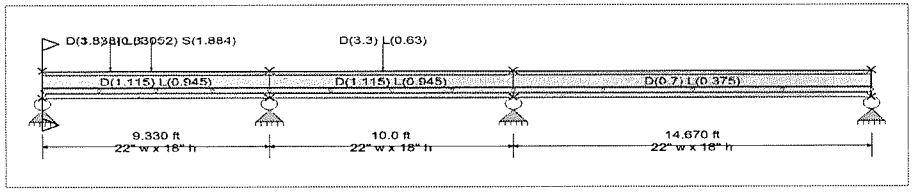
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in
Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.250 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.250 ft in this span
Span #2 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.750 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.750 ft in this span
Span #3 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 14.670 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 14.670 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads

Load for Span Number 1
Uniform Load : D = 1.115, L = 0.9450 k/ft, Tributary Width = 1.0 ft

Point Load : D = 3.30, L = 0.630 k @ 2.80 ft

Point Load : D = 1.838, L = 1.052, S = 1.884 k @ 4.50 ft

Load for Span Number 2
Uniform Load : D = 1.115, L = 0.9450 k/ft, Tributary Width = 1.0 ft

Point Load : D = 3.30, L = 0.630 k @ 4.670 ft

Load for Span Number 3
Uniform Load : D = 0.70, L = 0.3750 k/ft, Tributary Width = 1.0 ft

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Grade Beam #3

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.369 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.005 in	Ratio = 32076 >=360
Mu : Applied		-43.328 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Mn * Phi : Allowable		117.418 k-ft	Max Downward Total Deflection	0.023 in	Ratio = 7545 >=180
Location of maximum on span		0.000 ft	Max Upward Total Deflection	-0.001 in	Ratio = 104257 >=180
Span # where maximum occurs		Span # 3			

Shear Stirrup Requirements

Between 0.00 to 0.19 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 0.25 to 7.15 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 7.22 to 10.13 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 10.20 to 18.33 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 18.40 to 19.92 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 20.01 to 33.90 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Title Block Line 1
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Project Descr:

Project ID:

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

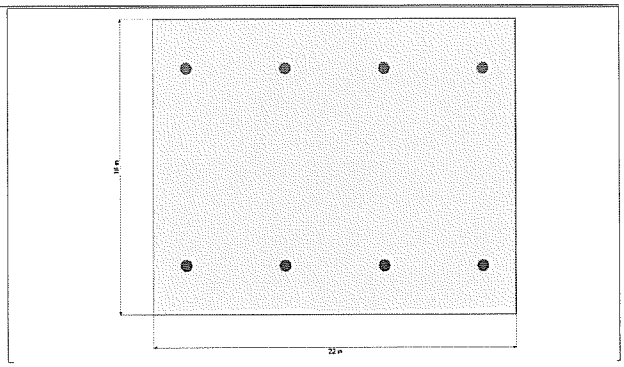
Description : Grade Beam #4

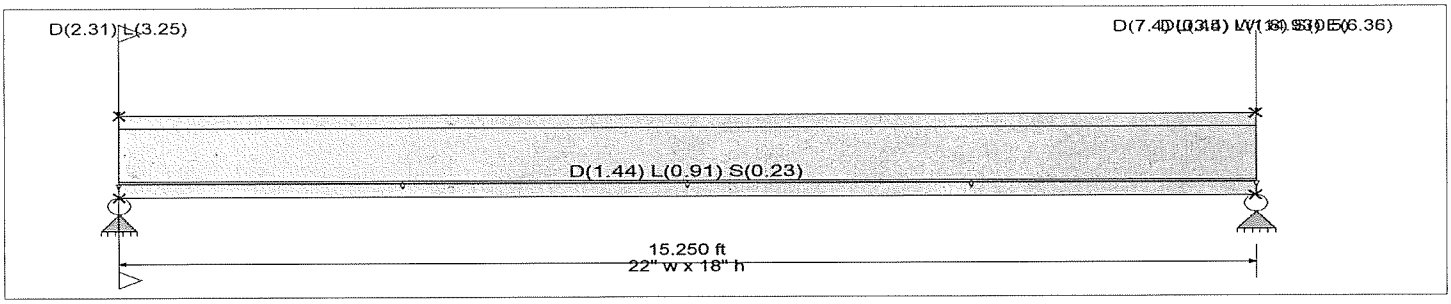
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	= 4.0 ksi	ϕ Phi Values	Flexure :	0.90	
fr = f'c$^{1/2}$ * 7.50	= 474.342 psi		Shear :	0.750	
ψ Density	= 145.0 pcf	β_1	=	0.850	
λ LtWt Factor	= 1.0				
Elastic Modulus	= 3,122.0 ksi	Fy - Stirrups		40.0 ksi	
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi	
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3	
		Number of Resisting Legs Per Stirrup =		2	





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in
Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 15.250 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 15.250 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 1.440, L = 0.910, S = 0.230 k/ft, Tributary Width = 1.0 ft

Point Load : D = 7.40, L = 3.40, W = 14.930, E = 6.360 k @ 15.250 ft

Point Load : D = 2.310, L = 3.250 k @ 0.0 ft

Point Load : D = 0.450, L = 1.80, S = 0.50 k @ 15.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.935** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.033 in	Ratio =	5517 >=36
Mu : Applied	109.824 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable	117.418 k-ft	Max Downward Total Deflection	0.259 in	Ratio =	705 >=18
Location of maximum on span	7.639 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 >=18
Span # where maximum occurs	Span # 1				

Shear Stirrup Requirements

Between 0.00 to 3.56 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 3.58 to 11.67 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 11.69 to 15.22 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in

Concrete Beam

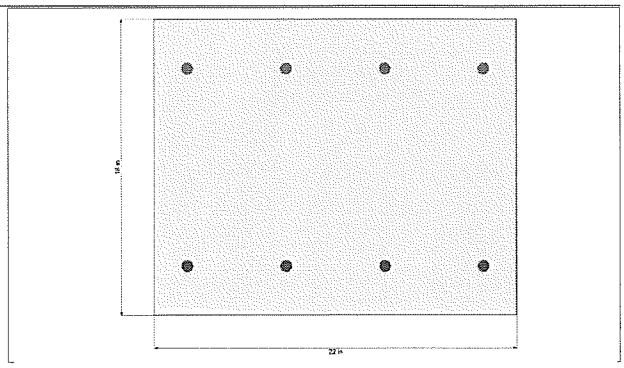
Description : Grade Beam #5

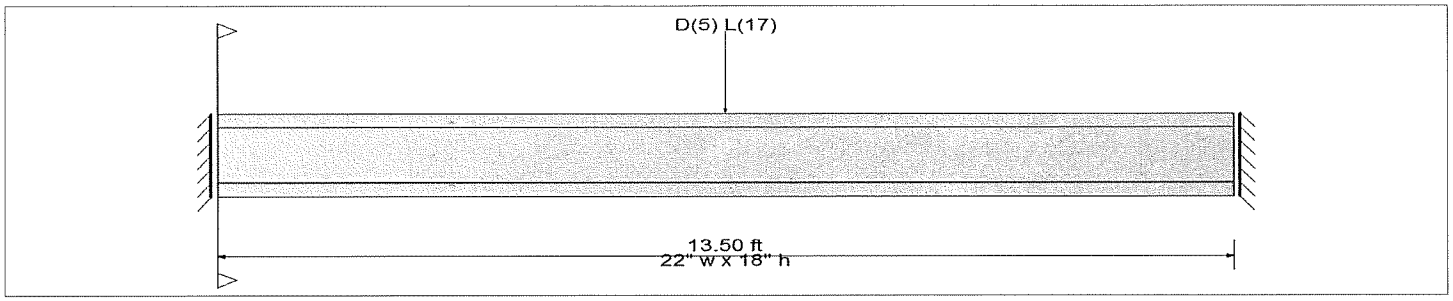
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	= 4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c$^{1/2}$ * 7.50	= 474.342 psi		Shear :	0.750
ψ Density	= 145.0 pcf	β_1	=	0.850
λ LtWt Factor	= 1.0			
Elastic Modulus	= 3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3
		Number of Resisting Legs Per Stirrup =		2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in
Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Point Load : D = 5.0, L = 17.0 k @ 6.750 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.539** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.011 in	Ratio =	14376 >=36
Mu : Applied	-63.292 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable	117.418 k-ft	Max Downward Total Deflection	0.016 in	Ratio =	9897 >=18
Location of maximum on span	13.500 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 >=18
Span # where maximum occurs	Span # 1				

Shear Stirrup Requirements

Entire Beam Span Length : PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in

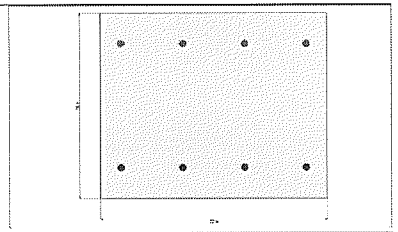
Description : Grade Beam #6

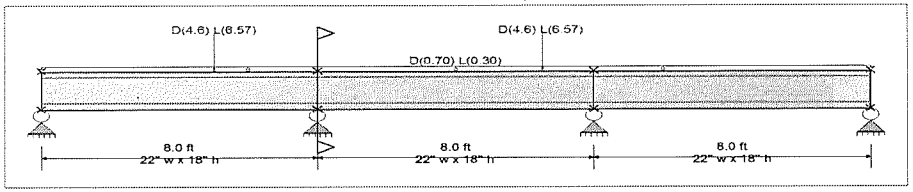
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	= 4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c $^{1/2}$ * 7.50	= 474.342 psi		Shear :	0.750
ψ Density	= 145.0 pcf	β_1	=	0.850
λ LtWt Factor	= 1.0			
Elastic Modulus	= 3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	= 60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	= 29,000.0 ksi	Stirrup Bar Size #		3
		Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Span #2 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Span #3 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.70, L = 0.30

Uniform Load on ALL spans : D = 0.70, L = 0.30 k/ft

Load for Span Number 1
Point Load : D = 4.60, L = 6.570 k @ 5.0 ft

Load for Span Number 2
Point Load : D = 4.60, L = 6.570 k @ 6.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.242** : 1	Maximum Deflection			
Section used for this span	**Typical Section**	Max Downward Transient Deflection	0.002 in	Ratio =	41531 >=36
Mu : Applied	-28.363 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable	117.418 k-ft	Max Downward Total Deflection	0.005 in	Ratio =	18450 >=18
Location of maximum on span	0.000 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 >=18
Span # where maximum occurs	Span # 2				

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Engineer:
Project Descr:

Project ID:

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155

Licensee : CG ENGINEERING

Description : Grade Beam #6

Shear Stirrup Requirements

Between 0.00 to 5.60 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 5.65 to 7.95 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 8.00 to 14.77 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 14.83 to 15.95 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 16.00 to 23.95 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Title Block Line 1
You can change this area
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and then using the "Printing &
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Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID:

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

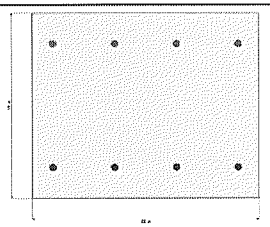
Description : Grade Beam #7

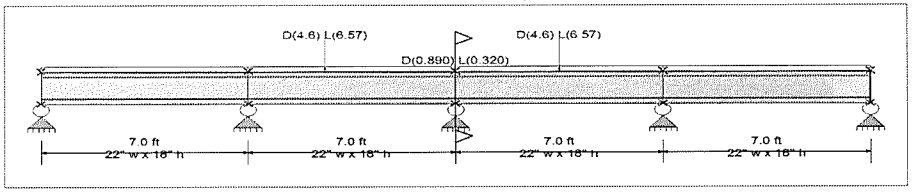
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Span #2 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Span #3 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Span #4 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.890, L = 0.320

Uniform Load on ALL spans : D = 0.890, L = 0.320 k/ft

Load for Span Number 2
Point Load : D = 4.60, L = 6.570 k @ 2.60 ft

Load for Span Number 3
Point Load : D = 4.60, L = 6.570 k @ 3.50 ft

Title Block Line 1
You can change this area
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Project Title:
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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Grade Beam #7

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.200** : 1	Maximum Deflection			
Section used for this span		**Typical Section**	Max Downward Transient Deflection	0.001 in	Ratio =	82085 >=36
Mu : Applied		-23.441 k-ft	Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Mn * Phi : Allowable		117.418 k-ft	Max Downward Total Deflection	0.002 in	Ratio =	42331 >=18
Location of maximum on span		0.000 ft	Max Upward Total Deflection	0.000 in	Ratio =	999 >=18
Span # where maximum occurs		Span # 3				

Shear Stirrup Requirements

Entire Beam Span Length : Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

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Concrete Beam

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

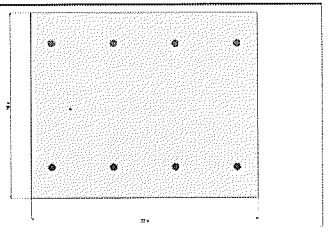
Description : Grade Beam #8

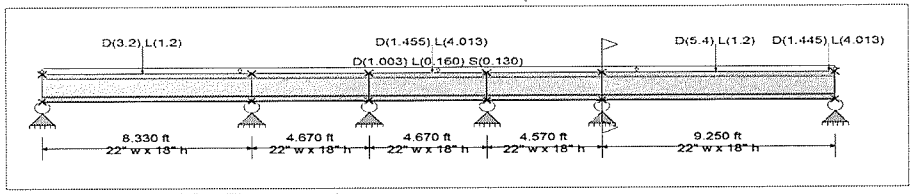
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
			E - Stirrups	=	29,000.0 ksi
fy - Main Rebar	=	60.0 ksi	Stirrup Bar Size #		3
E - Main Rebar	=	29,000.0 ksi			
Number of Resisting Legs Per Stirrup	=	2			





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 8.330 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 8.330 ft in this span
Span #2 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span
Span #3 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span
Span #4 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 13.50 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 13.50 ft in this span
Span #5 Reinforcing....	
4-#6 at 3.0 in from Bottom, from 0.0 to 9.250 ft in this span	4-#6 at 3.0 in from Top, from 0.0 to 9.250 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 1.003, L = 0.160, S = 0.130

Uniform Load on ALL spans : D = 1.003, L = 0.160, S = 0.130 k/ft

Load for Span Number 1
Point Load : D = 3.20, L = 1.20 k @ 4.0 ft

Load for Span Number 3
Point Load : D = 1.455, L = 4.013 k @ 2.50 ft

Load for Span Number 5
Point Load : D = 5.40, L = 1.20 k @ 4.50 ft

Point Load : D = 1.445, L = 4.013 k @ 9.0 ft

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Concrete Beam

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Description : Grade Beam #8

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.240 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.001 in	Ratio = 85909 >=36
Mu : Applied		28.175 k-ft	Max Upward Transient Deflection	0.000 in	Ratio = 0 <360
Mn * Phi : Allowable		117.418 k-ft	Max Downward Total Deflection	0.008 in	Ratio = 14021 >=18
Location of maximum on span		4.625 ft	Max Upward Total Deflection	-0.001 in	Ratio = 53480 >=18
Span # where maximum occurs		Span # 5			

Shear Stirrup Requirements

Between 0.00 to 22.14 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 22.24 to 22.24 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 22.45 to 31.07 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 31.28 to 31.28 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in

Concrete Beam

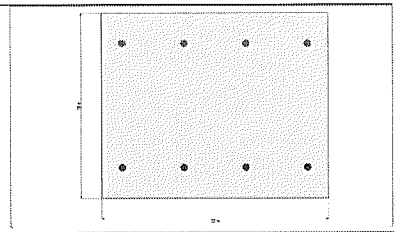
Description : Grade Beam #9

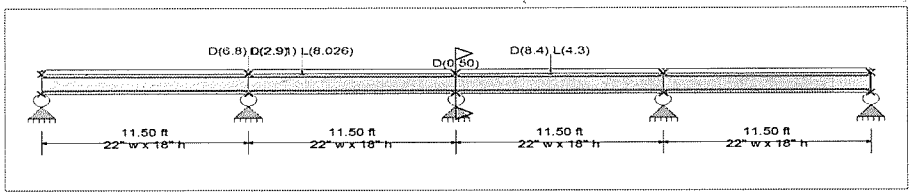
CODE REFERENCES

Calculations per ACI 318-14, IBC 2015, ASCE 7-10
Load Combination Set : ASCE 7-10

Material Properties

f'c	=	4.0 ksi	ϕ Phi Values	Flexure :	0.90
fr = f'c^1/2 * 7.50	=	474.342 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	3,122.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirru	=	2





Cross Section & Reinforcing Details

Rectangular Section, Width = 22.0 in, Height = 18.0 in

Span #1 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span

Span #2 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span

Span #3 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span

Span #4 Reinforcing....
4-#6 at 3.0 in from Bottom, from 0.0 to 11.50 ft in this span
4-#6 at 3.0 in from Top, from 0.0 to 11.50 ft in this span

Applied Loads

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

Beam self weight calculated and added to loads
Loads on all spans...
D = 0.50

Uniform Load on ALL spans : D = 0.50 k/ft

Load for Span Number 2
Point Load : D = 2.910, L = 8.026 k @ 3.0 ft

Point Load : D = 6.80, L = 2.90 k @ 0.0 ft

Load for Span Number 3
Point Load : D = 8.40, L = 4.30 k @ 5.250 ft

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1
Lic. # : KW-06005155
Licensee : CG ENGINEERING

Concrete Beam

Description : Grade Beam #9

DESIGN SUMMARY

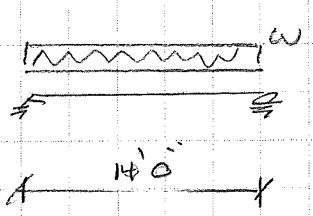
Design OK

Maximum Bending Stress Ratio	=	0.298 : 1	Maximum Deflection		
Section used for this span		Typical Section	Max Downward Transient Deflection	0.004 in	Ratio = 36861 >=36
Mu : Applied		-35.044 k-ft	Max Upward Transient Deflection	-0.002 in	Ratio = 55881 >=36
Mn * Phi : Allowable		117.418 k-ft	Max Downward Total Deflection	0.011 in	Ratio = 12950 >=18
Location of maximum on span		0.000 ft	Max Upward Total Deflection	0.000 in	Ratio = 999 >=18
Span # where maximum occurs		Span # 3			

Shear Stirrup Requirements

Between 0.00 to 11.38 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 11.50 to 12.23 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 12.35 to 22.88 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in
Between 23.00 to 23.12 ft, PhiVc/2 < Vu <= PhiVc, Req'd Vs = Min 9.6.3.3, use stirrups spaced at 7.000 in
Between 23.24 to 45.88 ft, Vu < PhiVc/2, Req'd Vs = Not Reqd 9.6.3.1, use stirrups spaced at 0.000 in

Structural Slab

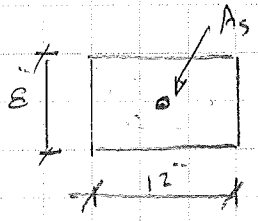


$w = (10^{D} + 40^{L})$ psf + self weight

Assume 6" slab → w_u = 166 psf (LRFD)

$$M_u = wl^2/8 = \frac{(166)(14.0)^2}{8} = 48.8^{k\text{-}in}/ft$$

$$V_u = wl/2 = \frac{(166)(14)}{2} = 1.16^{kips}/ft$$



$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.372(60)}{0.85(3.0)(12)} = 0.73"$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.372)(60)(3 - 0.73/2)$$

$$= 52.7^{k\text{-}in}/ft \quad \text{OK}$$

$$\phi V_n = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{4000}\,12 \times 3$$

$$= 3.4^{kips}/ft \quad \text{OK}$$

6" slab w/ #5 bars @ 10" o.c.

Temp. steel = $A_s \times 0.0014 = 6" \times 12" \times 0.0018 = 0.13^{in^2}/ft$

#4 bars @ 18" o.c.

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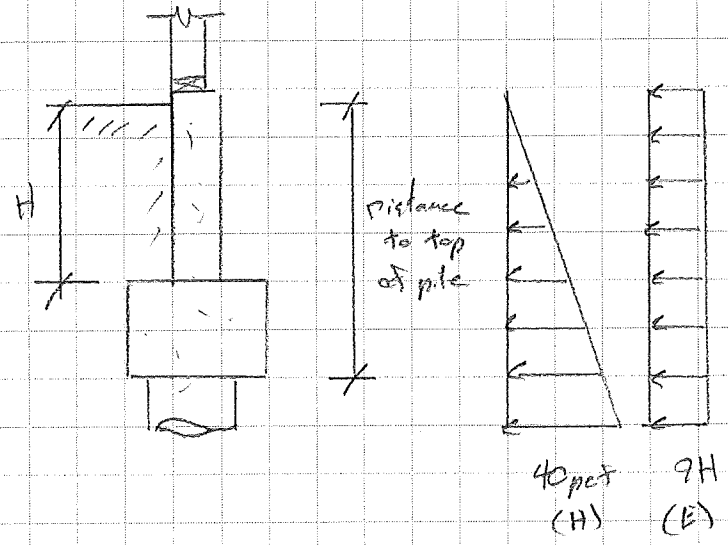
250 4th Ave. South
Suite 200
Edmonds, WA 98020
425.778.8500
www.cgengineering.com

Description Gravity - Concrete		By DTR	Date 3/16/11
		Checked	Date
		Scale	Sheet No.
Project Rudolf Res.		Job No. 15227.10	1-46

Typical Interior Retaining Wall



Controlling Load Combo

1.6 H + 1.0 E

Note: Only @ locations where slab is not present to resist sliding

Forces @ Bottom of Wall

H	Moment (M_u)	Shear (V_u)
4'	$13^{k'}$/ft	0.72^{k}/ft
6'	$32.9^{k'}$/ft	1.17^{k}/ft

8" conc. wall w/ #5 @ 15" o.c.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.248(60)}{0.85(4)(12)} = 0.365''$$

$$\phi M_n = A_s f_y \left(d - a/2\right) = 0.9(0.248)(60)\left(4 - 0.365/2\right) = 51.1^{k'} \quad \text{(o.k.)}$$

$$\phi V_n = 2\sqrt{f'_c}\, b d = 0.75(2)\sqrt{4000}\,(12)(4) = 4.6^{k} \quad \text{(o.k.)}$$

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Edmonds, WA 98020
425.778.8500
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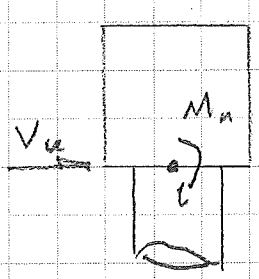
Description Retaining walls		By DTR	Date 2/28/18
		Checked	Date
		Scale	Sheet No.
Project Rudolf Res		Job No. 15227.15	1-47

Interior Retaining Wall (cont...)

Forces @ Bottom of Grade Beam

H	M_u	V_u	
5.5'	24.6 k'/ft	0.98 k/ft	} Design piles to resist these forces
7.5'	51.4 k'/ft	1.47 k/ft	



Space piles under 4' walls @ 6'-0" o.c.

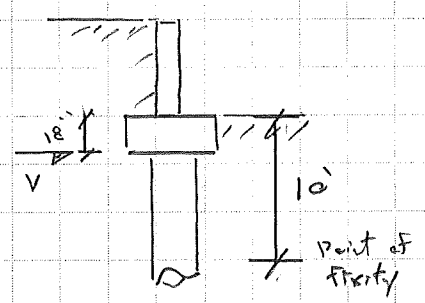
$V_{total} = 5.88^{k}$
$M_{total} = 12.3^{k'}$ Per Enercalc → 16" ϕ pile w/ (6) #6

Space piles under 6' wall @ 4'-0" o.c.

$V_{total} = 5.88^{k}$
$M_{total} = 17.3^{k'}$ Per Enercalc → 16" ϕ pile w/ (6) #6

Pile Capacity

Per Geotech: 1.) Passive resistance = 8.0 k/pile
2.) Assume piles are fixed 10' below grade



16" ϕ pile w/ (6) #6 (no moment @ top) = 5.0 kips (LRFD) = 7.15 kips (ASD)

point load located @ grade, used for lateral analysis

CG ENGINEERING 250 4th Ave. South Suite 200 Edmonds, WA 98020 425.778.8500 www.cgengineering.com	Description Retaining Walls	By DTR	Date 2/28/18
		Checked	Date
		Scale	Sheet No.
	Project Rudolf Res.	Job No. 1527.15	1-48

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Concrete Column

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

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Description : Pile Supporting 4' Retaining Wall

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

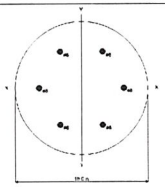
f'c : Concrete 28 day strength	=	4.0 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	0.50 %
Max. Reinf.	=	8.0 %

Overall Column Height = 10.0 ft
End Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 0.80
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 0.80

Column Cross Section

Column Dimensions : 16.0in Diameter, Column Edge to Rebar Edge Cover = 2.50in

Column Reinforcing : 6.0 - #6 bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 2,094.40 lbs * Dead Load Factor
BENDING LOADS . . .
Lat. Point Load at 8.50 ft creating Mx-x, E = 5.880 k
Moment acting about X-X axis, E = 12.30 k-ft

DESIGN SUMMARY

Load Combination +1.20D+E
Location of max.above base 9.933 ft

Maximum Stress Ratio **0.945 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	2.513 k	φ * Pn =	2.444 k
Mu-x =	-62.280 k-ft	φ * Mn-x =	-66.621 k-ft
Mu-y =	0.0 k-ft	φ * Mn-y =	0.0 k-ft
Mu Angle =	0.0 deg		
Mu at Angle =	62.280 k-ft	φMn at Angle =	65.901 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	833.03 k
Pnmin : Nominal Min. Tension Axial Capacity	k
φ Pn, max : Usable Compressive Axial Capacity	531.06 k
φ Pn, min : Usable Tension Axial Capacity	k

Maximum SERVICE Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 5.880 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.3635 in at 10.0 ft above base
for load combination E Only
Along X-X 0.0in at 0.0 ft above base
for load combination

General Section Information . φ = 0.750 β = 0.850 θ = 0.850
ρ : % Reinforcing 1.313 % Rebar % Ok
Reinforcing Area 2.640 in^2
Concrete Area 201.062 in^2

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D			9.93	2.93	531.06					0.000			0.006
+1.20D			9.93	2.51	531.06					0.000			0.005

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Concrete Column

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Description : Pile Supporting 4' Retaining Wall

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.20D+E	Actual		9.93	2.51	2.44	1.000	-62.28			0.000	62.28	65.90	**0.945**
+0.90D			9.93	1.88	531.06					0.000			**0.004**
+0.90D+E	Actual		9.93	1.88	2.44	1.000	-62.28			0.000	62.28	65.90	**0.945**

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.70E	0.0000	in	0.000	ft	0.254	in	10.000	ft
+D+0.5250E	0.0000	in	0.000	ft	0.191	in	10.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.70E	0.0000	in	0.000	ft	0.254	in	10.000	ft
E Only	0.0000	in	0.000	ft	0.360	in	9.933	ft

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Concrete Column

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ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Pile Supporting 6' Retaining Wall

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

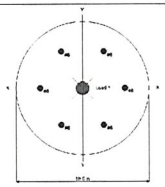
f'c : Concrete 28 day strength	=	4.0 ksi
E =	=	3,122.0 ksi
Density	=	150.0 pcf
β	=	0.850
fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits		*ASTM A615 Bars Used*
Min. Reinf.	=	0.50 %
Max. Reinf.	=	8.0 %

Overall Column Height = 10.0 ft
End Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0

Column Cross Section

Column Dimensions : 16.0in Diameter, Column Edge to Rebar Edge Cover = 2.50in

Column Reinforcing : 6.0 - #6 bars



Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 2,094.40 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft above base, D = 10.0 k
BENDING LOADS . . .
Lat. Point Load at 8.50 ft creating Mx-x, E = 5.880 k
Moment acting about X-X axis, E = 17.132 k-ft

DESIGN SUMMARY

Load Combination +0.90D+E
Location of max.above base 9.933 ft

Maximum Stress Ratio **0.976 : 1**
Ratio = (Pu^2+Mu^2)^.5 / (PhiPn^2+PhiMn^2)^.5

Pu =	10.885 k	φ * Pn =	9.879 k
Mu-x =	-67.112 k-ft	φ * Mn-x =	-69.430 k-ft
Mu-y =	0.0 k-ft	φ * Mn-y =	0.0 k-ft
Mu Angle =	0.0 deg		
Mu at Angle =	67.112 k-ft	φMn at Angle =	68.733 k-ft

Pn & Mn values located at Pu-Mu vector intersection with capacity curve

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	833.03 k
Pnmin : Nominal Min. Tension Axial Capacity	k
φ Pn, max : Usable Compressive Axial Capacity	531.06 k
φ Pn, min : Usable Tension Axial Capacity	k

Maximum SERVICE Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	5.880 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.4039 in at 10.0 ft above base
for load combination E Only
Along X-X 0.0in at 0.0 ft above base
for load combination

General Section Information φ = 0.750 β = 0.850 θ = 0.850
ρ : % Reinforcing 1.313 % Rebar % Ok
Reinforcing Area 2.640 in^2
Concrete Area 201.062 in^2

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project Descr:
Project ID:

Printed: 1 MAR 2018, 1:41PM

Concrete Column

File = R:\2015 Projects\15227.20 Rudolf Residence Civil Design_Structural\Engineering\Gravity\grade beams.e
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.1

Lic. # : KW-06005155 — Licensee : CG ENGINEERING

Description : Pile Supporting 6' Retaining Wall

Governing Load Combination Results

Governing Factored Load Combination	Moment X-X	Moment Y-Y	Dist. from base ft	Axial Load k Pu	φ * Pn	Bending Analysis k-ft δx	δx * Mux	δy	δy * Muy	Alpha (deg)	δ Mu	φ Mn	Utilization Ratio
+1.40D			9.93	16.93	531.06					0.000			**0.032**
+1.20D			9.93	14.51	531.06					0.000			**0.027**
+1.20D+E	Actual		9.93	14.51	13.56	1.000	-67.11			0.000	67.11	70.12	**0.957**
+0.90D			9.93	10.88	531.06					0.000			**0.020**
+0.90D+E	Actual		9.93	10.88	9.88	1.000	-67.11			0.000	67.11	68.73	**0.976**

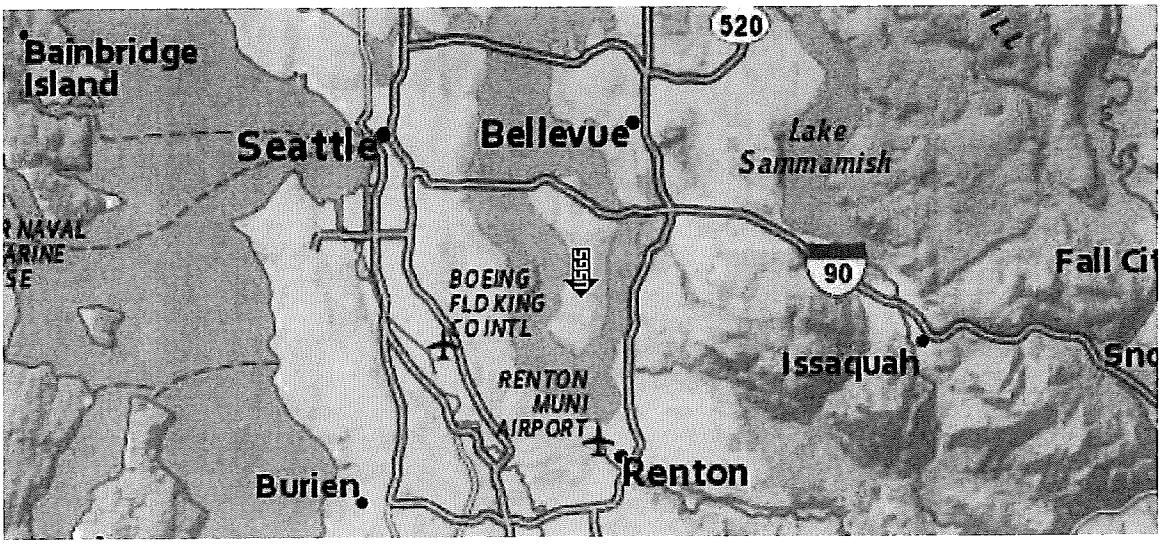
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.70E	0.0000	in	0.000	ft	0.283	in	10.000	ft
+D+0.5250E	0.0000	in	0.000	ft	0.212	in	10.000	ft
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.70E	0.0000	in	0.000	ft	0.283	in	10.000	ft
E Only	0.0000	in	0.000	ft	0.400	in	9.933	ft

USGS Design Maps Summary Report

User-Specified Input

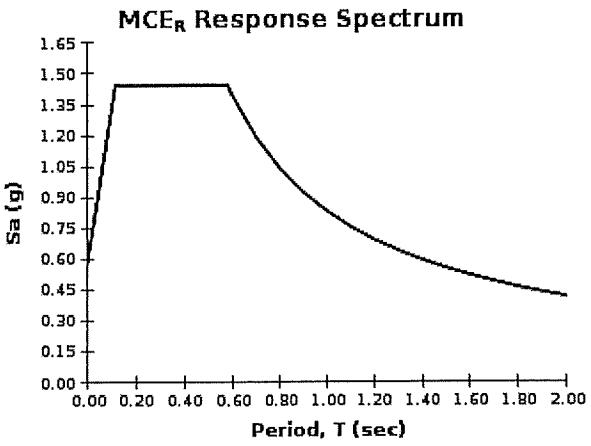
Report Title	Rudolf Res. Sat February 4, 2017 00:15:21 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	47.5561°N, 122.22481°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

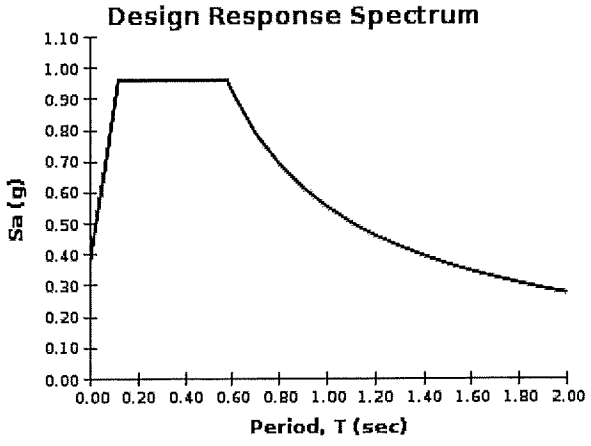


USGS-Provided Output

S_S =	1.442 g	S_{MS} =	1.442 g	S_{DS} =	0.961 g
S_1 =	0.554 g	S_{M1} =	0.831 g	S_{D1} =	0.554 g

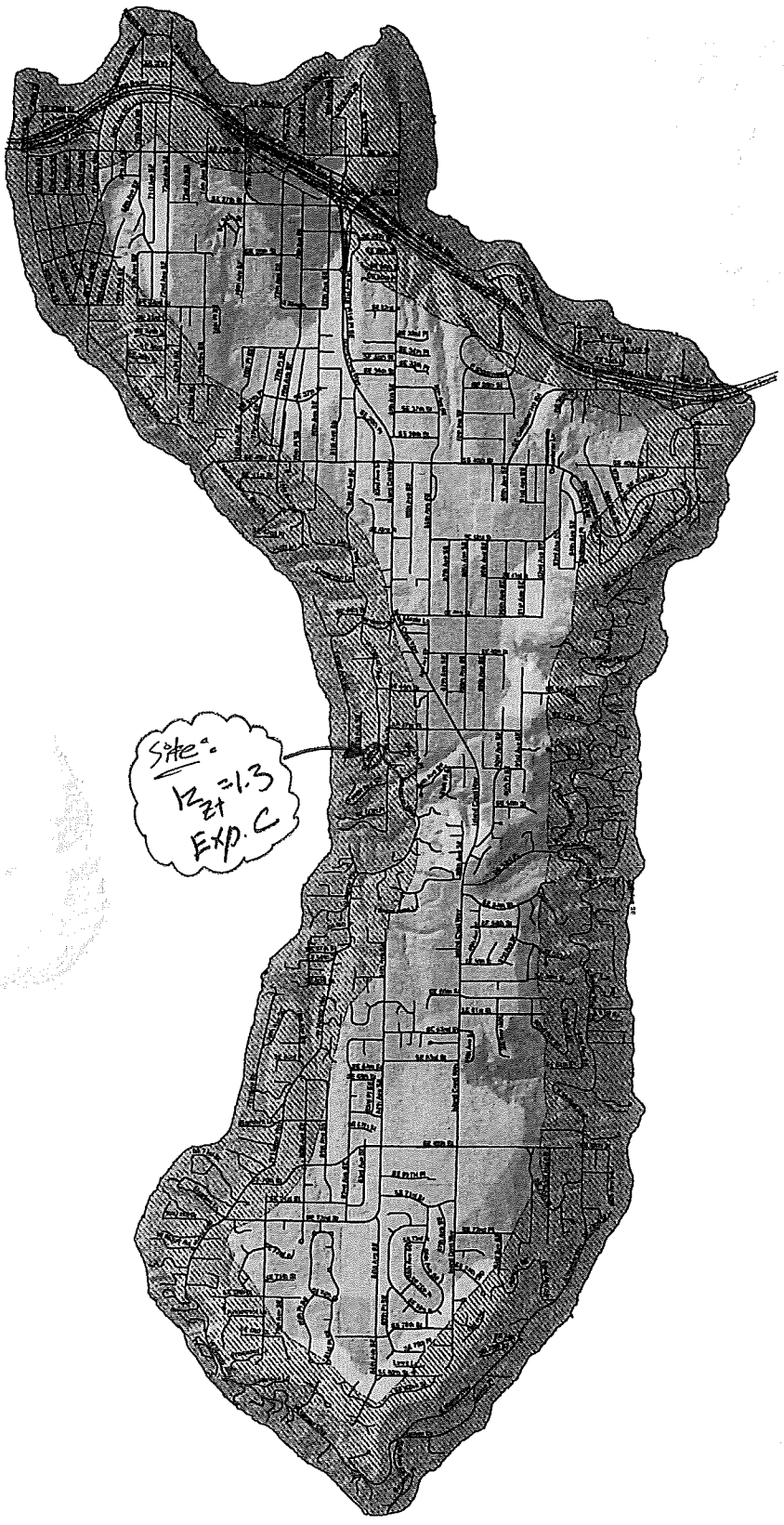
For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Mercer Island Wind Exposure and Wind Speed-Up (Topographic Effect)

by Development Services Group (DSG), City of Mercer Island
April 2009



0 0.5 1 Kilometers 1:12,000
0 0.5 1 Miles
TN

WIND EXPOSURE CATEGORIES & WIND SPEED-UP FACTORS
(ICC Section 1609 & ASCE 7-05 Chapter 6)

It is the responsibility of the Owner (or their Design Professional) to review site conditions and determine the Kzt factor to be utilized for each specific project. The Kzt factors and wind exposure categories indicated on this map are the minimum values accepted by the City of Mercer Island without requiring the design professional to submit additional calculations and supporting topographic documentation (to verify the values utilized in their wind load determination).

Please note – The Kzt values indicated on this map are approximations based upon periodic calculations of representative samplings around Mercer Island. These values are intended for City of Mercer Island's plan review purposes only.

WIND EXPOSURE CATEGORIES:

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

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

K_zt Factor
- K_zt = 1.0
- K_zt = 1.3
- K_zt = 1.6
- K_zt = 1.9

GENERAL NOTES FOR WIND EXPOSURE AND WIND SPEED-UP MAP

This map is the Wind Exposure Category and Wind Speed-up (Topographic Effects) Map for the City of Mercer Island. This map shows the minimum wind exposure category and the minimum wind speed-up, "K_zt" factor, which will be accepted without site specific documentation and calculation.

Other wind speed phenomena may occur on Mercer Island that is not specifically indentified on this map. It is the responsibility of the Owner (or their Design Professional) to review site conditions and determine the appropriate design wind speed and exposure category for their specific project and location.

This map is for the sole use of the staff of the City of Mercer Island's Development Services Group (DSG) for the purposes of permit application evaluation. This map provides DSG staff a general assessment of Wind Exposure Category and Wind Speed-up (Topographic Effects). All areas have not been specifically evaluated and there may be locations that are not correctly represented on this map. It is the responsibility of individual property owners and map users to evaluate risk associated with their proposed development. No site-specific assessment of risk is implied or otherwise indicated by the City of Mercer Island with this map.

Information about data used for the map, references, and data limitation are all described the associated "Read Me" document. The digital version of this map is accompanied by a meta data file containing pertinent information about map construction. This data map is available on the City of Mercer Island website.

The City of Mercer Island is using guidance provided within ICC Section 1609 & ASCE 7-05 Chapter 6 regarding definitions used when creating this map.

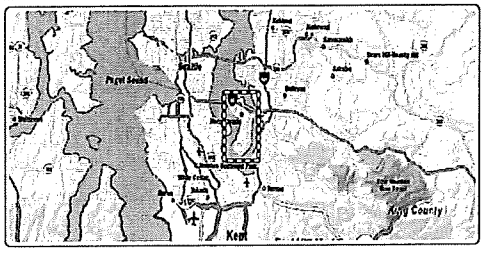
DEFINITIONS:

Kzt factor: The topographic effect of wind speed-up at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, that meet all of the conditions noted in ASCE 7-05 Minimum Design Loads for Buildings and Other Structures, Section 6.5.7.

Exposure B: The wind exposure category that applies where the site in question is located a minimum of 1500 feet from the shoreline and the mean roof height is less than or equal to 30 feet per IBC 2006 section 1609.4.3.

Exposure C: The wind exposure category that applies where the site in question is located within 1500 feet from the shoreline per IBC 2006 section 1609.4.3.

Wind Speed: Minimum 85 mph 3-second gust per IRC Figure R301.2(4)



1 of 2

Seismic Design			2015 IBC	ASCE 7-10
Risk Category	Type	II	Table 1604.5	Table 1.5-1
Seismic Importance Factor	I_E=	1.00		Table 1.5-2
Site Class	S.C.=	D (Assumed)	Section 1613.3.2	Table 20.3-1
0.2s Spectral Response	S_S=	1.442	Figure 1613.3.1(1-6)	Figure 22-1
1.0s Spectral Response	S_1=	0.554	Figure 1613.3.1(1-6)	Figure 22-2
	(Source: http://geohazards.usgs.gov/designmaps/us/application.php)			
Site Coefficient (short period)	F_a=	1	Table 1613.3.3(1)	Table 11.4-1
Site Coefficient (1.0 second)	F_v=	1.5	Table 1613.3.3(2)	Table 11.4-2
$S_{MS}=F_a*S_s$	S_{MS}=	1.442	Section 1613.3.3	Section 11.4.3
$S_{M1}=F_v*S_1$	S_{M1}=	0.831	Section 1613.3.3	Section 11.4.3
$S_{DS}=2/3*S_{MS}$	**S_{DS}=**	**0.961**	Section 1613.3.4	Section 11.4.4
$S_{D1}=2/3*S_{M1}$	S_{D1}=	0.554	Section 1613.3.4	Section 11.4.4
Seismic Design Category	**SDC=**	**D**	Table 1613.3.3(1-2)	Section 11.6
Lateral Resisting System	Wood Shear Walls			
Response Modification Coeff.	R=	6.50		Table 12.2-1
System Overstrength Factor	Ω_0=	3.0		Table 12.2-1
Deflection Amplification Factor	C_d=	4.0		Table 12.2-1
Horizontal Irregularities	---	NA		Table 12.3-1
Vertical Irregularities	---	NA		Table 12.3-2
Analysis Procedure	---	ELF		Table 12.6-1
Equivalent Lateral Force Design				Section 12.8
Structure Type for Period	All Other Structural Systems			Table 12.8-2
Building Period Coefficient	C_t=	0.02		Table 12.8-2
Building Period Coefficient	x=	0.75		Table 12.8-2
Building Height	h_n=	35		
Building Period, $T_a=C_t h_n^x$	T_a=	0.288		Section 12.8.2.1
$C_s=S_{DS}/(R/I_E)$	C_S=	0.148		Eqn. 12.8-2
Max $C_s=S_{D1}/(T*R/I_E)$	Max C_S=	0.296		Eqn. 12.8-3
Min. $C_s=0.044*S_{DS}*I_E$	Min. C_S=	0.042		Eqn. 12.8-5
Min. $Cs=0.5*S_1/(R/I_E)$, SDC E&F	Min. C_S=	N.A.		Eqn. 12.8-6
Seismic Response Coeff., C_S	**C_S=**	**0.148** (LRFD)		Section 12.8.1.1

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

Description	By	Date
Seismic Summary	DTR	3/9/2017
	Checked	Date
	Scale: NTS	Sheet No.: 2-3
Project: Rudolf Res.	Job No.: 15227.10	

Seismic Forces - Vertical Distribution

Refer to ASCE 7-10 Section 12.8.3 — k = 1.0

Diaphragm Level	DL (psf)	Area (ft^2)	w_{DL} (kips)	Story Elev. (h)	$w_i \cdot h_i^k$ (k-ft)	$w_x \cdot h_x^k$ / $\Sigma w_i \cdot h_i^k$	Shear F_x	Sum F_x
Roof Framing	20	2752	55.0	32	1761	0.46	**8.9**	8.9
3rd Framing	25	2950	73.8	20.75	1530	0.40	**7.7**	16.6
2nd Framing	25	2225	55.6	10.16	565	0.15	**2.9**	19.5
		Σ =	184.415	-	3857	1.00	**19.5**	-

Base Shear (ULT)	27.3	kips
Base Shear (ASD)	**19.5**	**kips**

* note that all table forces are ASD

Seismic Forces - Vertical Distribution Including Rho

Refer to ASCE 7-10 Section 12.3.4.2

Diaphragm Level	Rho ρ	Shear F_x	Sum F_x
Roof Framing	1.0	**8.9**	8.9
3rd Framing	1.0	**7.7**	16.6
2nd Framing	1.0	**2.9**	19.5
	Σ =	**19.5**	-

Diaphragm Forces - Vertical Distribution

Refer to ASCE 7-10 Section 12.10.1.1

Diaphragm Level	w_i (kips)	Σw_i (kips)	F_i (kips)	ΣF_i (kips)	$\Sigma F_i \cdot w_{px}$ / Σw_i	F_{px} (Min) $0.2S_{DS}Iw_{px}$	F_{px} (Max) $0.4S_{DS}Iw_{px}$	F_{px} Govern
Roof Framing	55.0	55.0	8.9	8.9	8.9	7.6	15.1	**8.9**
3rd Framing	73.8	128.8	7.7	16.6	9.5	10.1	20.3	**10.1**
2nd Framing	55.6	184.4	2.9	19.5	5.9	7.6	15.3	**7.6**

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

Description	Seismic & Diaphragm Force Distribution	By	DTR	Date	02/28/18
		Checked		Date	
		Scale	NTS	Sheet No.	
Project	Rudolf Res.	Job No.			
			15227.10		2-4

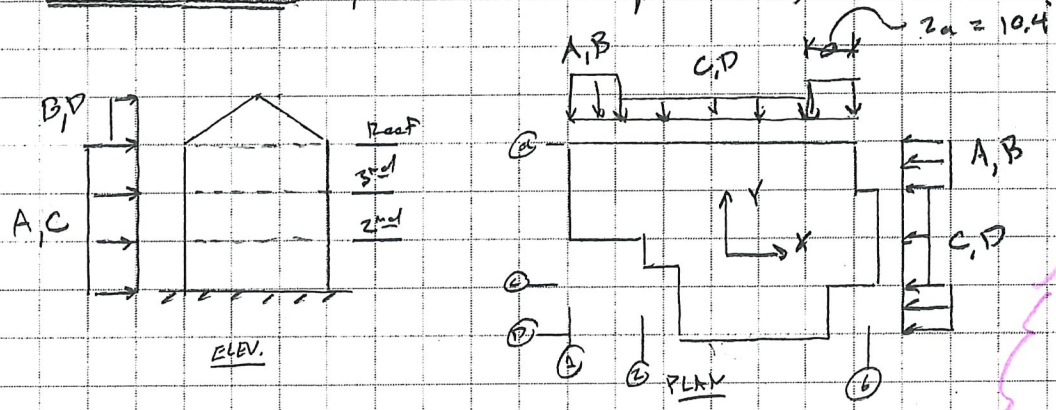
Wind Design (ASCE 28.5 Enclosed Simple Diaphragm) | **2015 IBC** | **ASCE 7-10**

Building Exposure	Exp.= C	Section 1609.4	Section 26.7.3
Basic Wind Speed	V= 110		Per Jurisdiction
Risk Category	I_w= II		Table 1.5-1
Top of Roof Height (feet)	h= 33.5		
Mean Roof Height (feet)	h_{mean}= 29.5		
Building Length (feet)	L= 78		
Building Width (feet)	W= 52		
End Zone Width, a (feet)	a= 5.2		Figure 28.6-1
Roof Angle	Angle= 36.5		
Design Wind Pressure, p_{s30A}	p_{s30A}= 21.6		Figure 28.6-1
Design Wind Pressure, p_{s30B}	p_{s30B}= 14.8		Figure 28.6-1
Design Wind Pressure, p_{s30C}	p_{s30C}= 17.2		Figure 28.6-1
Design Wind Pressure, p_{s30D}	p_{s30D}= 11.8		Figure 28.6-1
Design Wind Pressure, p_{s30E}	p_{s30E}= 1.7		Figure 28.6-1
Design Wind Pressure, p_{s30F}	p_{s30F}= -13.1		Figure 28.6-1
Design Wind Pressure, p_{s30G}	p_{s30G}= 0.6		Figure 28.6-1
Design Wind Pressure, p_{s30H}	p_{s30H}= -11.3		Figure 28.6-1
Design Wind Pressure, p_{s30EOH}	p_{s30EOH}= -7.6		Figure 28.6-1
Design Wind Pressure, p_{s30GOH}	p_{s30GOH}= -8.7		Figure 28.6-1
Height/Exposure Adjustment, λ	λ_{max}= 1.40		
Topo. Effect Coeff., K_{zt}	K_{zt}= 1.30		

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

	ULT	ASD
	$p_s=\lambda*Kzt*p_{s30}$	$p_s=\lambda*Kzt*ps30*0.6$
p_{s30A}=	39.2	23.5
p_{s30B}=	26.8	16.1
p_{s30C}=	31.2	18.7
p_{s30D}=	21.4	12.8
p_{s30E}=	3.1	1.8
p_{s30F}=	-23.8	-14.3
p_{s30G}=	1.1	0.7
p_{s30H}=	-20.5	-12.3
p_{s30EOH}=	-13.8	-8.3
p_{s30GOH}=	-15.8	-9.5

Wind Loads (per ASCE7-10 Chapter 28.5)



NOTE: SHEAR WALLS HAVE BEEN REVISED AFTER THESE CALCS. HOWEVER, THE DIMENSIONS OF THE HOUSE HAVE NOT CHANGED, THUS WIND BASE SHEARS ARE STILL VALID

A = 23.5 psf B = 16.1 psf C = 18.7 psf D = 12.8 psf

$$F_{a,roof} = (10.4')\left[8.25' \times 16.1\ psf + \tfrac{1}{2}(9') \times 23.5\ psf\right] + (7.6')\left[8.25' \times 12.8\ psf + \tfrac{1}{2}(9') \times 18.7\ psf\right]$$

$$= 3.9^{k}$$

$$F_{c,roof} = \left(\tfrac{16'}{2}\right)\left[8.25' \times 16.1 + \tfrac{1}{2}(9') \times 23.5\right] + \left(\tfrac{36'}{2}\right)\left[8.25' \times 12.8 + \tfrac{1}{2}(9') \times 18.7\ psf\right]$$

$$= 5.4^{k}$$

$$F_{D,roof} = \left(\tfrac{16'}{2}\right)\left[8.25' \times 16.1 + (9'/2)\,23.5\right] = 1.9^{k}$$

SUM x-Dir. Roof = 11.2 kips

$$F_{1,roof} = (10.4')\left[4.25' \times 16.1 + 9'/2 \times 23.5\right] + (2')\left[4.25' \times 12.8 + 9'/2 \times 18.7\right] = 2.1^{k}$$

$$F_{2,roof} = (12.25')\left[4.25' \times 12.8 + 9'/2 \times 18.7\right] + \left(\tfrac{54.5'}{2}\right)\left[8.25' \times 12.8 + 9'/2 \times 18.7\right] = 6.9^{k}$$

$$F_{B(roof)} = (10.4')\left[8.25' \times 16.1 + \tfrac{9'}{2} \times 23.5\right] + 11'\left[8.25' \times 12.8 + 9'/2 \times 18.7\right] = 5.8^{k}$$

SUM Y-Dir Roof = 14.8 k

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250 4th Ave. South
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Description: Lateral Analysis		By: DTR	Date: 3/15/17
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Res.		Job No. 15227.10	2-6

Wind Loads (cont...)

$F_{A,3rd} = (10.4')\left[\left(\frac{9'+11.25'}{2}\right)23.5\,psf\right] + (7.6')\left[\left(\frac{9'+11.25'}{2}\right)18.7\,psf\right] = 4.0^{k}$

$F_{C,3rd} = \left(\frac{16'}{2}\right)\left[\left(\frac{9'+11.25'}{2}\right)(23.5)\right] + \left(\frac{34'}{2}\right)\left[\left(\frac{9'+11.25'}{2}\right)(18.7)\right] = 5.4^{k}$

$F_{D,3rd} = \left(\frac{16'}{2}\right)\left[\left(\frac{9'+11.25'}{2}\right)(23.5)\right] = 1.7^{k}$

SUM X-DIR, 3rd Floor = 11.3 k

$F_{1,3rd} = (10.4')\left[\left(\frac{9'+11.25'}{4}\right)(23.5\,psf)\right] = 2.5^{k}$

$F_{2,3rd} = (12.25')\left[\left(\frac{9'+11.25'}{2}\right)(18.7\,psf)\right] + \left(\frac{54.5'}{2}\right)\left[\left(\frac{9'+11.25'}{2}\right)(18.7)\right] = 7.6^{k}$

$F_{6,3rd} = (10.4')\left[\left(\frac{9'+11.25'}{2}\right)(23.5)\right] + 17'\left[\left(\frac{9'+11.25'}{2}\right)(18.7)\right] = 5.8^{k}$

SUM Y-Dir, 3rd Floor = 15.9 k

$F_{A,2nd} = (10.4')\left[\left(\frac{11.25'+10.25'}{2}\right)(23.5)\right] + (7.6')\left[\left(\frac{11.25'+10.25'}{2}\right)(18.7)\right] = 4.0^{k}$

$F_{C,2nd} = \left(\frac{16'}{2}\right)\left[\left(\frac{11.25'+10.25'}{2}\right)(23.5)\right] + (18')\left[\left(\frac{11.25'+10.25'}{2}\right)(18.7)\right] = 5.4^{k}$

$F_{D,2nd}$ = @ grade, no additional loads

SUM X-DIR, 2nd Floor = 9.4 k

$F_{1,2nd}$ = @ grade, no additional loads

$F_{2,2nd} = \left(\frac{54.5'}{2}\right)\left[\left(\frac{11.25'+10.25'}{2}\right)(18.7)\right] = 5.2^{k}$

$F_{6,2nd} = (10.4')\left[\left(\frac{11.25'+10.25'}{2}\right)(23.5)\right] + (17')\left[\left(\frac{11.25'+10.25'}{2}\right)(18.7)\right] = 5.8^{k}$

SUM Y-DIR, 2nd Floor = 11.0 k

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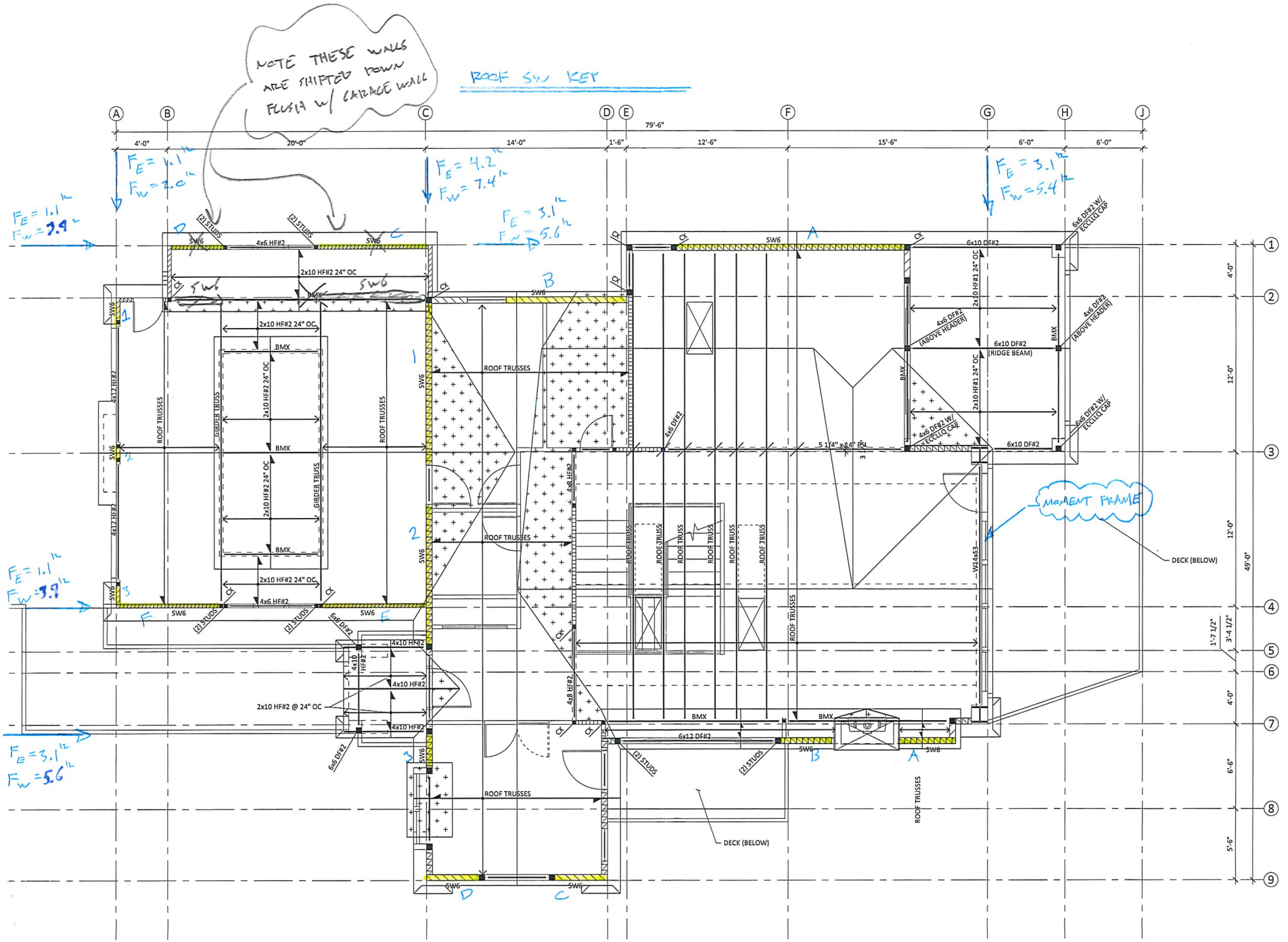
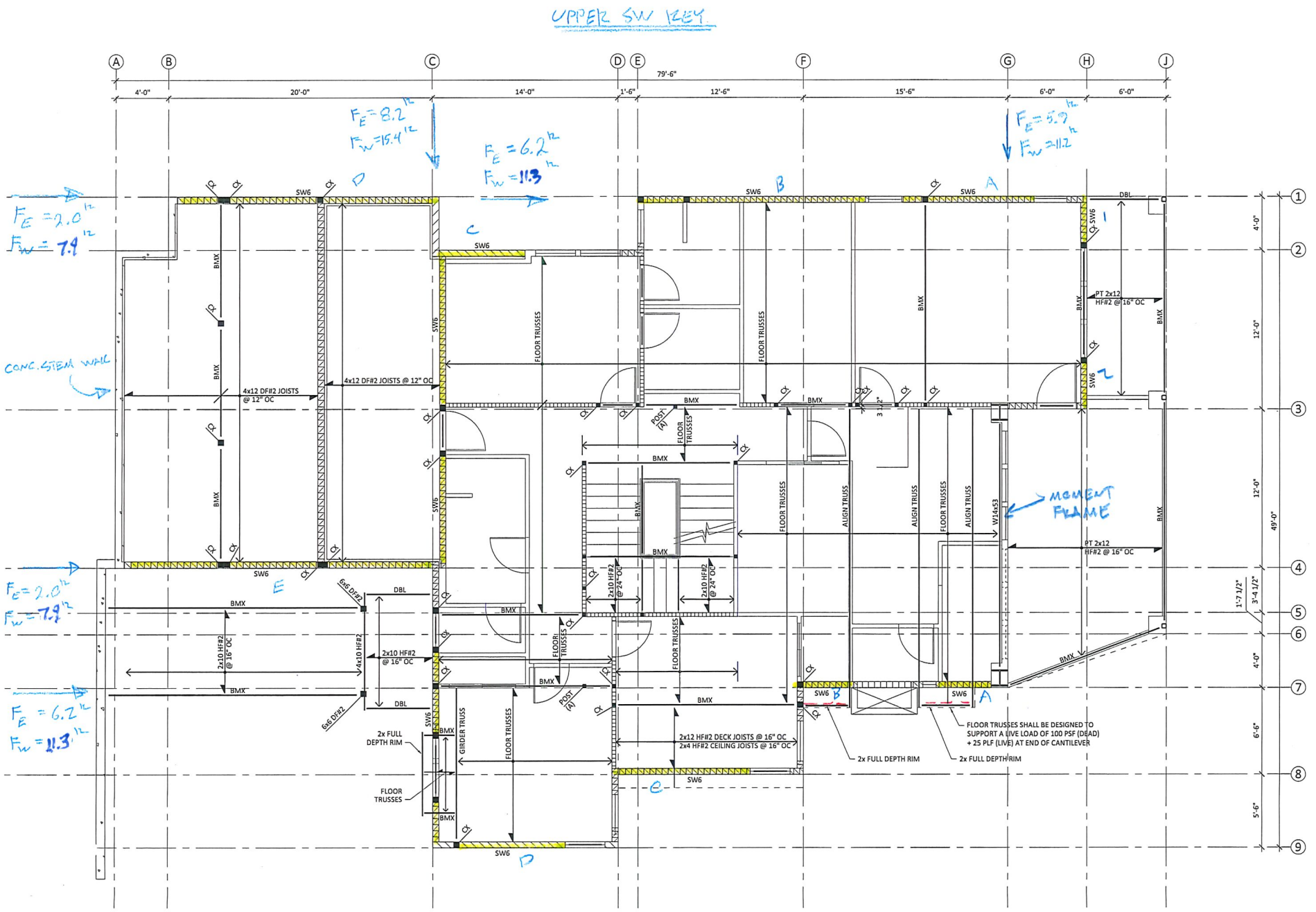
Description: Lateral Analysis		By: PTR	Date: 3/15/17
		Checked	Date
		Scale	Sheet No. 2-7
Project: Rudolf Res		Job No. 15227.10	


NOTE THESE WALLS ARE SHIFTED DOWN FLUSH W/ GARAGE WALL
ROOF SW KEY
A
B
C
D
E
F
G
H
J
1
2
3
4
5
6
7
8
9
79'-6"
4'-0"
20'-0"
14'-0"
1'-6"
12'-6"
15'-6"
6'-0"
6'-0"
$F_E = 1.1^{k/ft}$
$F_W = 2.0^{k/ft}$
$F_E = 4.2^{k/ft}$
$F_W = 7.4^{k/ft}$
$F_E = 3.1^{k/ft}$
$F_W = 5.4^{k/ft}$
$F_E = 1.1^{k/ft}$
$F_W = 1.9^{k/ft}$
$F_E = 3.1^{k/ft}$
$F_W = 5.6^{k/ft}$
$F_E = 1.1^{k/ft}$
$F_W = 1.9^{k/ft}$
$F_E = 3.1^{k/ft}$
$F_W = 5.6^{k/ft}$
SW6
[2] STUDS
4x6 HF#2
2x10 HF#2 24" OC
BMX
ROOF TRUSSES
GIRDER TRUSS
4x12 HF#2
6x10 DF#2
6x6 DF#2 W/ ECCLUD CAP
4x6 DF#2 (ABOVE HEADER)
6x10 DF#2 (RIDGE BEAM)
5 1/4" x 9 1/4" PSL
MOMENT FRAME
DECK (BELOW)
W14x53
6x12 DF#2
4x10 HF#2
4x8 HF#2
2x10 HF#2 @ 24" OC
4'-0"
12'-0"
12'-0"
4'-0"
1'-7 1/2"
3'-4 1/2"
4'-0"
6'-6"
5'-6"
49'-0"


UPPER SW KEY
A B C D E F G H J
4'-0" 20'-0" 14'-0" 1'-6" 12'-6" 15'-6" 6'-0" 6'-0"
79'-6"
1 2 3 4 5 6 7 8 9
4'-0" 12'-0" 12'-0" 4'-0" 1'-7 1/2" 3'-4 1/2" 4'-0" 6'-6" 5'-6"
49'-0"
$F_E = 8.2^{k}$ $F_W = 15.4^{k}$
$F_E = 6.2^{k}$ $F_W = 11.3^{k}$
$F_E = 5.9^{k}$ $F_W = 11.2^{k}$
$F_E = 2.0^{k}$ $F_W = 7.9^{k}$
$F_E = 2.0^{k}$ $F_W = 7.9^{k}$
$F_E = 6.2^{k}$ $F_W = 11.3^{k}$
CONC. STEM WALL
SW6
BMX
4x12 DF#2 JOISTS @ 12" OC
4x12 DF#2 JOISTS @ 12" OC
FLOOR TRUSSES
POST (A)
2x10 HF#2 @ 24" OC
ALIGN TRUSS
W14x53
MOMENT FRAME
PT 2x12 HF#2 @ 16" OC
DBL
6x6 DF#2
4x10 HF#2
2x10 HF#2 @ 16" OC
2x FULL DEPTH RIM
GIRDER TRUSS
2x12 HF#2 DECK JOISTS @ 16" OC
2x4 HF#2 CEILING JOISTS @ 16" OC
FLOOR TRUSSES SHALL BE DESIGNED TO SUPPORT A LIVE LOAD OF 100 PSF (DEAD) + 25 PLF (LIVE) AT END OF CANTILEVER
2x FULL DEPTH RIM
2x FULL DEPTH RIM

6-2


MAIN FLOOR SW 12EV
MANBEAM FRAME
FLOOR TRUSSES
BMX
SW6
HUC HANGER
FOUND. HEIGHT CONC. BASEMENT WALL
FRONT WINDOW

Upper Floor Shear Walls - Walls Below the Roof Framing

X - Direction Walls

Fx (EQ) =	8.9	kips (Story Shear)
Fx (wind) =	11.2	kips (Story Shear)

Wx =	8897	PLF seismic
Wx =	11200	PLF wind

Story HT =	8.5
Wall HT =	8.5
Max h/w	3.5
S_{DS} =	0.98

Wall Line	Wall Mark	SW Length	Trib Width	EQ 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 $(0.6-0.14S_{DS})DL$ 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
1	a	18.5	0.3699	1.0	119	144	SW6	18.0	1.0	1.8	1.3	1.3	1.7	1.7	0.1	0.1	None	None	3.3	5.6
	b	9.3	-	1.0	119	144	SW6	8.8	1.1	1.8	1.3	1.3	1.5	1.5	0.3	0.3	None	None	-	-
1	c	9.1	0.1301	1.0	85	205	SW6	8.6	0.8	2.6	0.6	0.6	0.8	0.8	1.8	1.8	HDU5	HDU5	1.2	3.9
	d	4.5	-	1.0	85	205	SW6	4.0	0.8	2.7	0.3	0.3	0.4	0.4	2.3	2.3	HDU5	HDU5	-	-
7	a	4.33	0.3699	1.0	185	224	SW6	3.8	1.8	3.0	0.3	0.3	0.4	0.4	2.6	2.6	MST48	MST48	3.3	5.6
	b	4.5	-	1.0	185	224	SW6	4.0	1.8	3.0	0.3	0.3	0.4	0.4	2.6	2.6	MST48	MST48	-	-
	c	4.5	-	1.0	185	224	SW6	4.0	1.8	3.0	0.3	0.3	0.4	0.4	2.6	2.6	MST48	MST48	-	-
	d	4.5	-	1.0	185	224	SW6	4.0	1.8	3.0	0.3	0.3	0.4	0.4	2.6	2.6	MST48	MST48	-	-
4	e	8.33	0.1301	1.0	69	167	SW6	7.8	0.6	2.1	0.6	0.6	0.7	0.7	1.4	1.4	HDU5	HDU5	1.2	3.9
	f	8.33	-	1.0	69	167	SW6	7.8	0.6	2.1	0.6	0.6	0.7	0.7	1.4	1.4	HDU5	HDU5	-	-
		Σ	1.00																8.9	19.0

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.	700	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

ENGINEERING

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

Description: Upper Floor Shear Walls X-Direction		By: DTR	Date: 02/28/18
		Checked	Date
		Scale: NTS	Sheet No.: 2-11
Project: Rudolf Res.		Job No.: 15227.10	

Upper Floor Shear Walls - Walls Below the Roof Framing

Y - Direction Walls

Fy (EQ) =	8.9	kips (Story Shear)
Fy (wind) =	14.8	kips (Story Shear)

Wy =	8897	PLF	seismic
Wy =	14800	PLF	wind

Story HT =	8.5
Wall HT =	8.5
Max h/w	3.5
S_{DS} =	0.96

REFER TO PORTAL FRAME CALCS

Wall Line	Wall Mark	SW Length	Trib Width	EQ 2w/h	EQ Shear	Wind Shear	SW Callout	Reduced HD Length	EQ Gross Uplift	Wind Gross Uplift	EQ ($(0.6-0.14S_{DS})D$E) 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
A	a	2	0.13663	0.7	456	361	SW2	1.5	2.4	5.7	0.1	0.1	0.2	0.2	5.5	5.5	HDU8	HDU8	1.2	2.0
	b	2	-	0.7	456	361	SW2	1.5	2	5.7	0.1	0.1	0.2	0.2	5.5	5.5	HDU8	HDU8	-	-
C	a	13	0.5	1.0	180	214	SW6	12.5	1.6	2.7	0.9	0.9	1.2	1.2	1.5	1.5	MST37	MST37	4.4	7.4
	b	11.67	-	1.0	180	214	SW6	11.2	1.6	2.7	0.8	0.8	1.1	1.1	1.6	1.6	MST37	MST37	-	-
G	1	3	0.36337	0.7	665	557	2SW4	2.5	4.8	8.0	0.2	0.2	0.3	0.3	7.7	7.7	0.0	0.0	1.4	2.3
																			1.8	3.0
		Σ	1.0																8.9	14.8

Walls at garage have 6'-0" height

3-Story Moment Frame, refer to additional calculations. Moment Frame supports full load from floor above in addition to tributary area on this floor. The remaining load is supported by (2) wall piers to north

Input Cell

Input Cell w/ Formula

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

Description: Upper Floor Shear Walls		By: DTR	Date: 02/28/18
Y-Direction		Checked	Date
		Scale: NTS	Sheet No.: 2-12
Project: Rudolf Res.		Job No.: 15227.10	

Main Floor Shear Walls - Walls Below the Upper Floor Framing

X - Direction Walls

Fx (EQ) =	7.7	kips (Story Shear)
Fx (wind) =	11.3	kips (Story Shear)

Wx =	7730	PLF seismic
Wx =	11300	PLF wind

Story HT =	10.33
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.96

Wall Line	Wall Mark	SW Length	Trib Width	Line Load From Above EQ	Line Load From Above Wind	EQ 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 $(0.6-0.14S_{DS})DL$ 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
1	a	10.0	0.38823	3.3	5.6	1.0	185	236	SW6	9.5	2.0	3.6	0.7	0.7	0.9	0.9	0.0	0.0	2.7	2.7	MST48	MST48	6.3	11.3
	b	17.5		-	-	1.0	185	236	SW6	17.0	2.0	3.5	1.2	1.2	1.6	1.6	0.0	0.0	1.9	1.9	MST37	MST37	-	-
	c	6.5		-	-	1.0	185	236	SW6	6.0	2.1	3.7	0.8	0.8	0.9	0.9	0.0	0.0	2.8	2.8	MST48	MST48	-	-
1	e	19.83	0.11177	1.2	3.9	1.0	102	285	SW4	19.3	1.1	4.2	1.9	1.9	2.3	2.3	1.8	1.8	3.7	3.7	MST72	MST72	2.0	7.9
7	a	4	0.38823	3.3	5.6	0.9	256	290	SW4	3.5	2.7	4.8	1.1	1.1	1.2	1.2	2.6	2.6	6.3	6.3	MST72	MST72	6.3	11.3
	b	3.67	-	-	-	0.8	279	290	SW4	3.2	2.7	4.9	1.1	1.1	1.1	1.1	2.6	2.6	6.3	6.3	MST72	MST72	-	-
	c	10	-	-	-	1.0	227	290	SW4	9.5	2.5	4.4	0.7	0.7	0.9	0.9	0.0	0.0	3.5	3.5	MST48	MST48	-	-
	d	10	-	-	-	1.0	227	290	SW4	9.5	2.5	4.4	0.7	0.7	0.9	0.9	2.6	0.0	6.1	3.5	MST72	MST48	-	-
4	a	23.33	0.11177	1.2	3.9	1.0	87	242	SW6	22.8	0.9	3.6	2.1	2.1	2.6	2.6	1.4	1.4	2.3	2.3	HDU5	HDU5	2.0	7.9
		Σ	1.0	8.9	19.0																		16.6	38.3

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.	700	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
MST72	(2)-2x HF	6.5	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	-	6.5	kips

Input Cell

Input Cell w/ Formula

ENGINEERING

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

Description: Main Floor Shear Walls		By: DTR	Date: 02/28/18
X-Direction		Checked	Date
		Scale: NTS	Sheet No.
Project: Rudolf Res.		Job No.: 15227.10	2-13

Main Floor Shear Walls - Walls Below the Upper Floor Framing

Y - Direction Walls

Fy (EQ) =	7.7	kips (Story Shear)
Fy (wind) =	15.9	kips (Story Shear)

Wy =	7730	PLF seismic
Wy =	15900	PLF wind

Story HT =	10.33
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.96

Wall Line	Wall Mark	SW Length	Trib Width	Line Load From Above EQ	Line Load From Above Wind	EQ 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 (0.6-0.14S_{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
A			0.13663	1.2	2.0																		2.3	4.2
			-																					
			-																					
			-																					
C	a	11.5	0.5	4.4	7.4	1.0	274	362	SW3	11.0	3.0	5.5	0.8	0.8	1.0	1.0	1.5	1.5	5.9	5.9	MST72	MST72	8.3	15.4
	b	8.16	-	-	-	1.0	274	362	SW3	7.7	3.0	5.6	0.6	0.6	0.7	0.7	1.6	1.6	6.5	6.5	MST72	MST72	-	-
	c	6.83	-	-	-	1.0	274	362	SW3	6.3	3.1	5.6	0.5	0.5	0.6	0.6	0.0	0.0	5.0	5.0	MST72	MST72	-	-
	d	3.83	-	-	-	0.9	322	362	SW3	3.3	3.3	6.0	0.3	0.3	0.3	0.3	0.0	0.0	5.7	5.7	MST72	MST72	-	-
G		3.83	0.36337	1.4	2.3	0.9	188	235	SW6	3.3	1.9	3.9	0.3	0.3	0.3	0.3	7.7	7.7	11.2	11.2	HDU14	HDU14	1.2	2.5
		3.83	-			0.9	188	235	SW6	3.3	1.9	3.9	0.3	0.3	0.3	0.3	0.0	0.0	3.6	3.6	MST48	MST48		
																							3.0	5.6
																					0.8			
5																								
		Σ	1.0	7.1	11.8																		14.8	27.7

No Shearwalls, at grade

3-Story Moment Frame, refer to additional calculations. Moment Frame supports full load from floor above in addition to tributary area on this floor. The remaining load is supported by (2) wall piers to north

Input Cell
Input Cell w/ Formula

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

Description: Main Floor Shear Walls		By: DTR	Date: 02/28/18
Y-Direction		Checked	Date
		Scale: NTS	Sheet No.: 2-14
Project: Rudolf Res.		Job No.: 15227.10	

Lower Floor Shear Walls - Walls Below the Main Floor Framing

X - Direction Walls

Fx (EQ) =	2.9	kips (Story Shear)
Fx (wind) =	9.4	kips (Story Shear)

Wx =	2855	PLF seismic
Wx =	9400	PLF wind

Story HT =	10.1
Wall HT =	9
Max h/w	3.5
S_{DS} =	0.96

Wall Line	Wall Mark	SW Length	Trib Width	Line Load From Above EQ	Line Load From Above Wind	EQ 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 (0.6-0.14S_{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
1	a	4.0	0.5	0.3	11.3	0.9	378	495	SW2	3.5	3.9	8.0	0.3	0.3	0.4	0.4	2.7	0.0	10.3	7.6	HDU14	HDU11	7.7	16.0
	b	5.5		-	-	1.0	336	495	SW2	5.0	3.7	7.7	0.4	0.4	0.5	0.5	0.0	0.0	7.2	7.2	HDU11	HDU11	-	-
	c	4.5		-	-	1.0	336	495	SW2	4.0	3.8	7.9	0.3	0.3	0.4	0.4	0.0	0.0	7.5	7.5	HDU11	HDU11	-	-
	d	9.0		-	-	1.0	336	495	SW2	8.5	3.6	7.4	0.6	0.6	0.8	0.8	0.0	0.0	6.6	6.6	HDU8	HDU8	-	-
1																								
7	a	5.5	0.5	0.3	11.3	1.0	233	343	SW4	5.0	2.6	5.3	0.4	0.4	0.5	0.5	6.3	6.3	11.1	11.1	HDU14	HDU14	7.7	16.0
	b	3.67	-	-	-	0.8	285	343	SW4	3.2	2.7	5.6	0.3	0.3	0.3	0.3	6.3	6.3	11.6	11.6	HDU14	HDU14	-	-
	c	10	-	-	-	1.0	233	343	SW4	9.5	2.5	5.1	0.7	0.7	0.9	0.9	3.5	3.5	7.7	7.7	HDU11	HDU11	-	-
	d	14	-	-	-	1.0	233	343	SW4	13.5	2.4	5.0	1.0	1.0	1.3	1.3	6.1	3.5	9.9	7.3	HDU14	HDU11	-	-
4																								
	Σ		1.0	12.6	22.5																		15.4	31.9

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.	700	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

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

Description: Lower Floor Shear Walls		By: DTR	Date: 02/28/18
X-Direction		Checked	Date
		Scale: NTS	Sheet No.: 2-15
Project: Rudolf Res.		Job No.: 15227.10	

Lower Floor Shear Walls - Walls Below the Main Floor Framing

Y - Direction Walls

Fy (EQ) =	2.9	kips (Story Shear)
Fy (wind) =	11.0	kips (Story Shear)

Wy =	2855	PLF seismic
Wy =	11000	PLF wind

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

Wall Line	Wall Mark	SW Length	Trib Width	Line Load From Above EQ	Line Load From Above Wind	EQ 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 (0.6-0.14S_{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
A																								
C			0.5	8.3	15.4																		9.7	20.9
			-																					
			-																					
			-																					
G	1	3.83	0.5	1.2	2.5	0.9	283	458	SW2	3.3	2.8	7.4	0.3	0.3	0.3	0.3	11.2	11.2	18.3	18.3	0.0	0.0	1.8	4.9
	2	3.83	-			0.9	283	458	SW2	3.3	2.8	7.4	0.3	0.3	0.3	0.3	3.6	3.6	10.7	10.7	HDU14	HDU14		
				3.0	5.6																		3.8	8.7
4																								
5																								
		Σ	1.0	12.5	23.5																		15.4	34.5

Full-Height Concrete Basement Wall

3-Story Moment Frame, refer to additional calculations. Moment Frame supports full load from floor above in addition to tributary area on this floor. The remaining load is supported by (2) wall piers to north

Input Cell
Input Cell w/ Formula

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

Description: Lower Floor Shear Walls		By: DTR	Date: 02/28/18
Y-Direction		Checked	Date
		Scale: NTS	Sheet No.: 2-16
Project: Rudolf Res.		Job No.: 15227.10	

GARAGE PORTAL FRAME Design Loads Seismic = 1.2^1k wind = 2.0^1k Capacity = 1,675^12 = 2,345^12

USE PORTAL FRAME @ GARAGE

Table 1. Recommended Allowable Design Values for APA Portal Frame Used on a Rigid-Base

Minimum Width (in.)	Maximum Height (ft)	Allowable Design (ASD) Values per Frame Segment		
		Shear[e,f] (lbf)	Deflection (in.)	Load Factor
16	8	850	0.33	3.09
	10	625	0.44	2.97
24	8	1,675	0.38	2.88
	10	1,125	0.51	3.42

Foundation for Wind or Seismic Loading[a,b,c,d]

(a) Design values are based on the use of Douglas-fir or Southern pine framing. For other species of framing, multiply the above shear design value by the specific gravity adjustment factor = (1 – (0.5 – SG)), where SG = specific gravity of the actual framing. This adjustment shall not be greater than 1.0.

(b) For construction as shown in Figure 1.

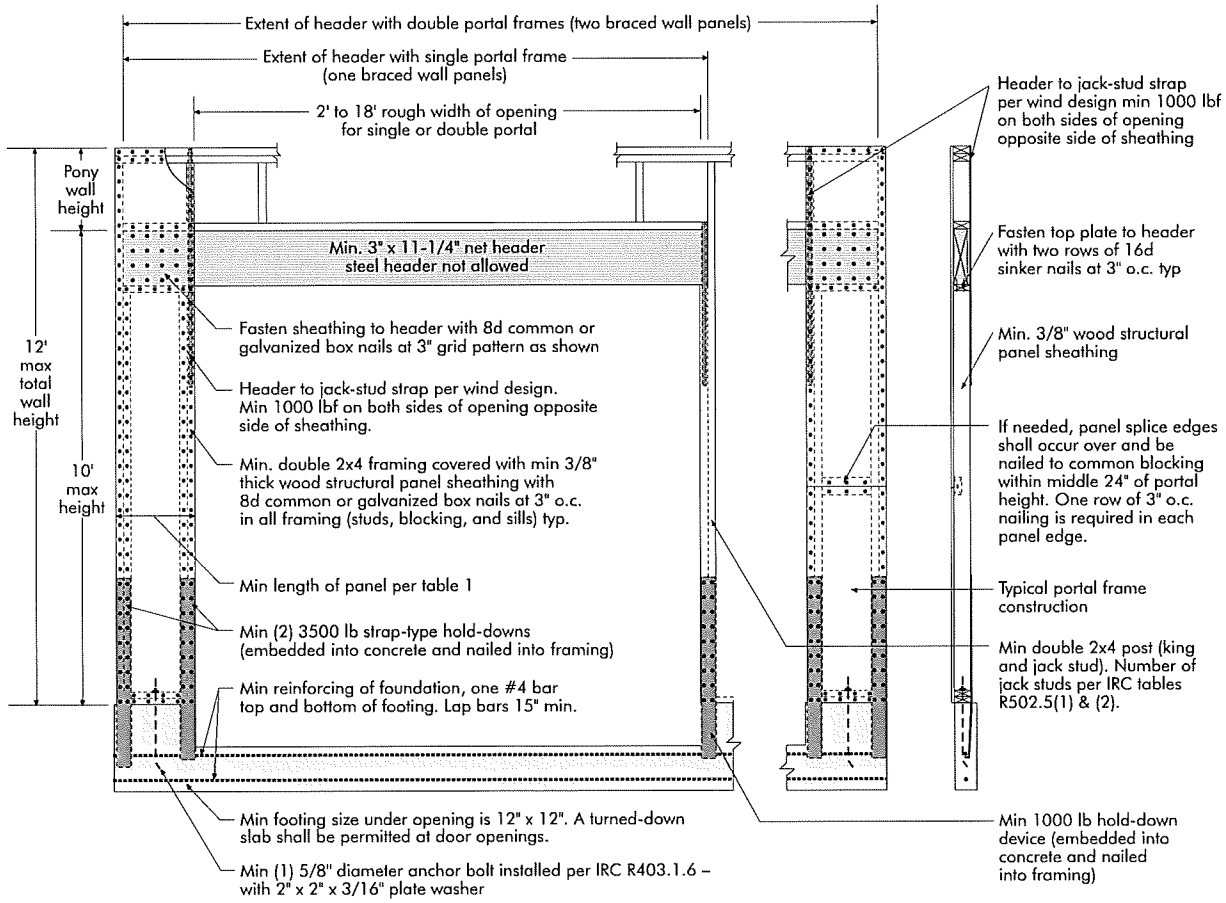
(c) Values are for a single portal-frame segment (one vertical leg and a portion of the header). For multiple portal-frame segments, the allowable shear design values are permitted to be multiplied by the number of frame segments (e.g., two = 2x, three = 3x, etc.).

(d) Interpolation of design values for heights between 8 and 10 feet, and for portal widths between 16 and 24 inches, is permitted.

(e) The allowable shear design value is permitted to be multiplied by a factor of 1.4 for wind design.

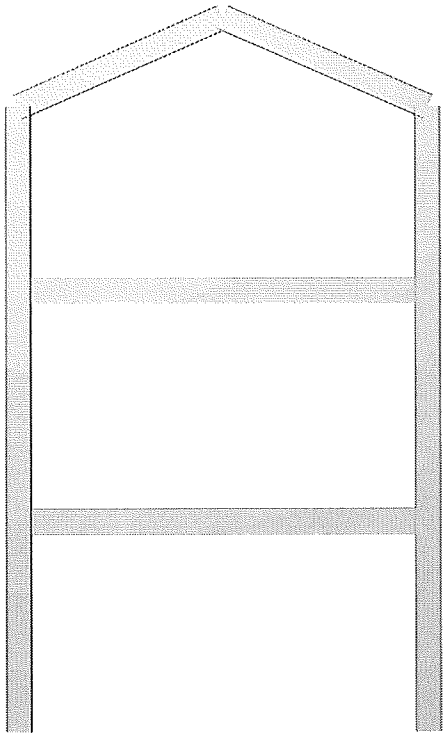
(f) If story drift is not a design consideration, the tabulated design shear values are permitted to be multiplied by a factor of 1.15. This factor is permitted to be used cumulatively with the wind-design adjustment factor in Footnote (e) above.

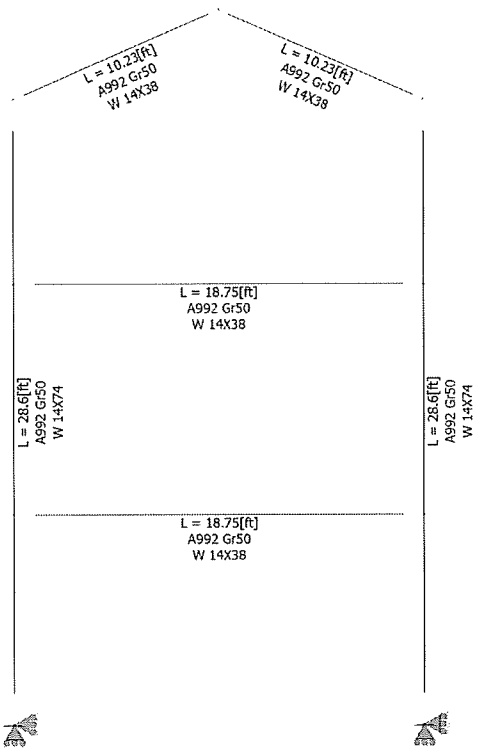
Figure 1. Construction Details for APA Portal-Frame Design with Hold Downs



2-17

MOMENT FRAME DESIGN



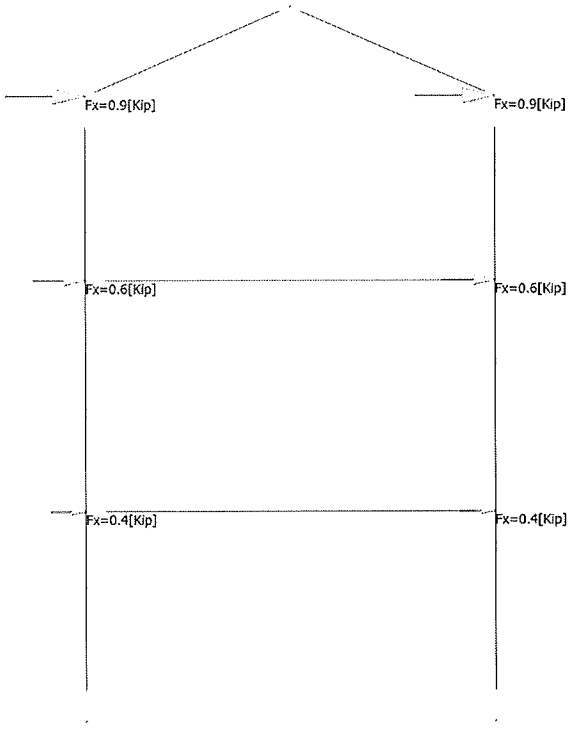

L = 10.23[ft]
A992 Gr50
W 14X38
L = 10.23[ft]
A992 Gr50
W 14X38
L = 18.75[ft]
A992 Gr50
W 14X38
L = 28.6[ft]
A992 Gr50
W 14X74
L = 28.6[ft]
A992 Gr50
W 14X74
L = 18.75[ft]
A992 Gr50
W 14X38


Y
Z
X

Loads

Concentrated - Nodes

Seismic Forces



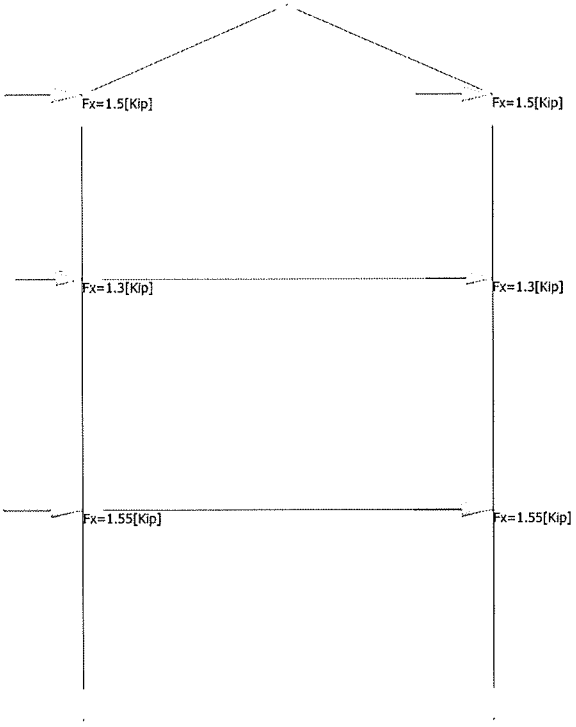
Bentley **HP Inc.**

Current Date: 2/28/2018 1:48 PM
Units system: English
File name: R:\2015 Projects\15227.20 Rudolf Residence Civil Design_Structural\Engineering\Lateral\New Lateral Analysis\Moment Frame\Moment Frame.etz\
Load condition: WL=Wind

Loads

Concentrated - Nodes

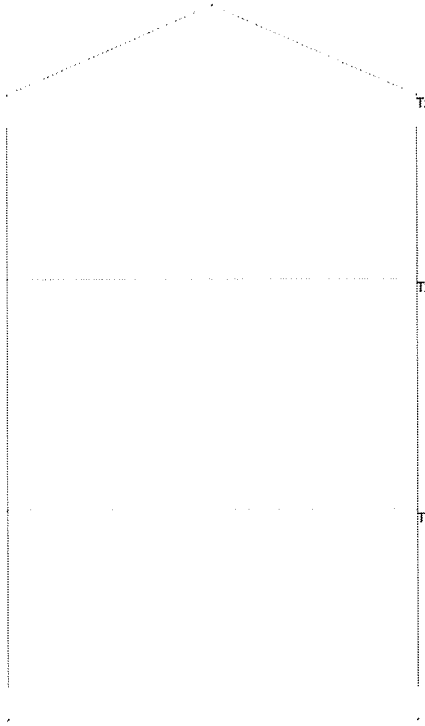
Wind Forces



Bentley **HP Inc.**

Current Date: 2/28/2018 1:49 PM
Units system: English
File name: R:\2015 Projects\15227.20 Rudolf Residence Civil Design_Structural\Engineering\Lateral\New Lateral Analysis\Moment Frame\Moment Frame.etz\
Load condition: id4=1.2DL+0.5LL+0.2SL+1.43EQ

Seismic Deflection (Strength Level)



Drift Requirements (per ASCE7)

$$\Delta_2 = \frac{\delta_2 - \delta_1}{I_E} C_d < \Delta_a$$

$$(\delta_2 - \delta_1)_{roof} = \frac{0.02(9'X12'')}{5.5} = 0.372''$$

$\Delta = 0.443'' - 0.365'' = 0.08''$ OK

$$(\delta_2 - \delta_1)_{3-1} = \frac{0.02(10'X12'')}{5.5} = 0.436''$$

$\Delta = 0.366 - 0.213 = 0.153''$ OK

$$(\delta_2 - \delta_1)_{2-1} = \frac{0.02(10.2'X12'')}{5.5} = 0.445''$$

$\Delta = 0.213''$ OK

Frame is adequate in deflection

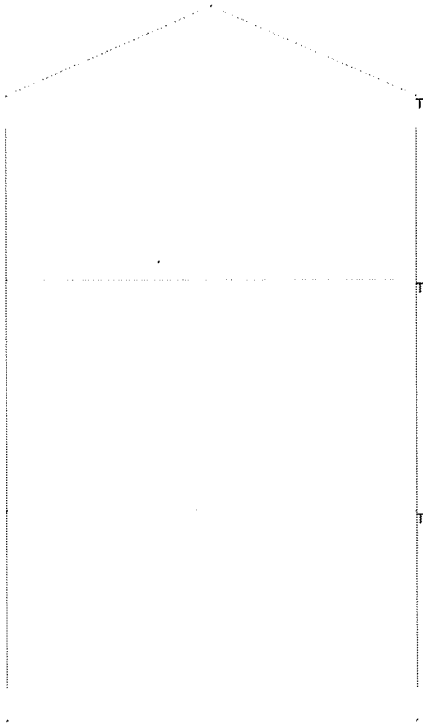
Bentley **HP Inc.**

Current Date: 2/28/2018 1:49 PM
Units system: English
File name: R:\2015 Projects\15227.20 Rudolf Residence Civil Design_Structural\Engineering\Lateral\New Lateral Analysis\Moment Frame\Moment Frame.etz\
Load condition: id7=DL+0.5LL+0.43WL

Wind Deflections (per ASCE7-10 App. C)

- Note optional check for serviceability
- Based on 72 mph wind (10-year reoccurance)

$\left(\frac{72\ mph^2}{110\ mph^2}\right)$(wind Pressure) = 0.43



- ASCE recommends limit of 1/600

$28.6' \times 12 / 600 = 0.572" > 0.28"$

Frame is inadequate in deflection

Bentley HP Inc.

Current Date: 2/28/2018 1:50 PM
Units system: English
File name: R:\2015 Projects\15227.20 Rudolf Residence Civil Design_Structural\Engineering\Lateral\New Lateral Analysis\Moment Frame\Moment Frame.etz\

Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

id0=1.4DL
id1=1.2DL+1.6LL+0.5SL
id2=1.2DL+1.6LL+0.5WL
id3=1.2DL+0.5LL+0.5SL+1.67WL ← Governing Load Case
id4=1.2DL+0.5LL+0.2SL+1.43EQ
id5=0.9DL+1.67WL
id6=0.6835DL+1.43EQ

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
Beam	***W 14X38***	**6**	id3 at 100.00%	**0.35**	**OK**	Eq. H1-1b
		9	id3 at 100.00%	0.21	OK	Eq. H1-1b
		12	id5 at 0.00%	0.09	OK	Eq. H1-1b
		13	id3 at 100.00%	0.13	OK	Eq. H1-1b
Column	***W 14X74***	**1**	id5 at 33.33%	0.15	OK	Eq. H1-1b
		2	id3 at 33.33%	**0.17**	**OK**	Eq. H1-1b

All members are adequate in strength

Design of Special Moment Frames (Reduced Beam Section)

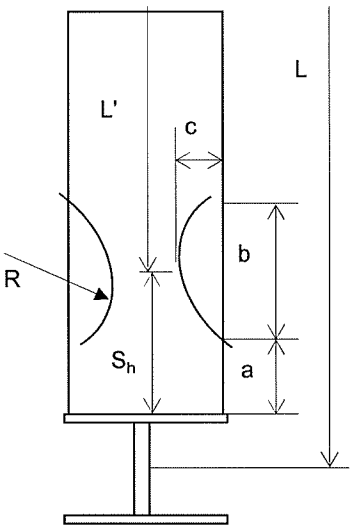
Per AISC Seismic Design Manual 2012

Beam Properties for W14x38

L=	228.4	*in*
d=	14.1	*in*
t_w=	0.31	*in*
b_f=	6.77	*in*
t_f=	0.515	*in*
Z_x=	61.5	in^3
R_y=	1.1	in^2
r_y=	1.55	*in*
F_y=	50	*ksi*
F_u=	65	*ksi*

Column Properties for W14x74

A_g=	21.8	in^2
d=	14.2	*in*
t_w=	0.45	*in*
b_f=	10.1	*in*
t_f=	0.785	*in*
k_{det}=	1.625	*in*
Z_x=	126	in^3



Beam Limitations (per AISC 538 5.3-1)

Depth ≤ W36

Weight ≤ 300 lbs/ft

t_f ≤ 1.75"

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

Width-to Thickness Ratios per Seismic Provisions

Lateral Bracing per Seismic Provisions

Protected Zone....

a = .6*bf =	4.1	*in (Eq. 5.8-1)*
b = .75*d =	10.6	*in (Eq. 5.8-2)*
c = .2*bf =	1.4	*in (Eq. 5.8-3)*

(refer to RBS detail sheets by CG Eng)

Compute Probable Maximum Moment @ Center of RBS

$Z_{RBS}=Z_x-2ct_f(d-t_f)$ =	42.6	in^3	(AISC 358 Eq. 5.8-4)
$C_{pr}=(F_y+F_u)/2F_y \le 1.2$ =	1.15		(AISC 358 Eq. 2.4.3-2)
$M_{pr}=C_{pr}R_yF_yZ_{RBS}$ =	2692	*kip-in*	(AISC 358 Eq. 5.8-5)

Compute Shear Force @ Center of RBS

S_h = a + b/2 =	9.3	*in*
V_{RBS} =	5.30	*kips (factored shear due to gravity loads at S_h per RAM model)*

Compute Maximum Moment @ Face of Column

$M_f = M_{pr}+V_{RBS}S_h$ =	2741.1	*kip-in*	(AISC 358 Eq. 5.8-6)

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

$M_{pe}=R_yF_yZ_x$ =	3382.5	*kip-in*	(AISC 358 Eq. 5.8-7)
$M_f \le \phi_d M_{pe}$	OKAY		(AISC 358 Eq. 5.8-8)

(for ϕ_d = 1.0)

ENGINEERING

250 4th Ave South
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Edmonds, WA 98020

Description: Reduced Beam Section Analysis		By	Date
		Checked	Date
		Scale	Sheet No.
Project: Rudolf Res.		Job No.	

Design of Special Moment Frames (Reduced Beam Section)

Per AISC Seismic Design Manual 2012

Determine the Required Shear Strength of the Beam

L_h =	195.5 *in*		
$V_{gravity}$ =	5.3 *kips*	(factored shear at face of column per RAM model)	
$V_u = 2*M_{pr}/L_h + V_{gravity}$ =	32.78 *kips*		(AISC 358 Eq. 5.8-9)
ϕVn (web)=	195.0 *kips*	OKAY	(per AISC Section G)
ϕVn (weld)=	195.0 *kips*	OKAY	(per AISC Section J)

Check Column-Beam Relationship Limitations

$M_{uv} = V_{RBS}*(a+b/2+d_c/2)$ =	87 *kip-in*		
$\Sigma M^*_{pb} = \Sigma(M_{pr}+M_v)$	2779 *kip-in*		(Seismic Provisions Eq. E3-3a)
P_{uc} =	18.27 *kips*	(factored axial demand per RAM output)	
$\Sigma M^*_{pc} = \Sigma Z_{xc}(F_{yc}-P_{uc}/A_g)$	6194 *kip-in*		(Seismic Provisions Eq. E3-2a)
$\Sigma M^*_{pc}/\Sigma M^*_{pb} \geq 1.0$ =	2.23 OKAY		

Check Column Panel Zone Shear Strength

$R_u = \Sigma M_f/(d-t_f)$ =	201.8 *kips*		
$.75*F_y*A_g$ =	817.5 *kips*	------->	USE J10-11
ϕRn =	231 *kips*		OKAY! DOUBLER PLATE NOT REQ'D

Check Continuity Plate Requirements

$t_{cf} \geq .4\sqrt{(1.8b_{bf}t_{bf}F_{yb}R_{yb}/(F_{yc}*R_{yc}))}$ =	1.002	REQUIRED
$t_{cf} \geq b_{bf}/6$ =	1.128	REQUIRED

If required, continuity plates shall match thickness of beam flanges

ENGINEERING 250 4th Ave South Suite 200 Edmonds, WA 98020	Description: Reduced Beam Section Analysis	By	Date
		Checked	Date
		Scale	Sheet No.
	Project: Rudolf Res.	Job No.	