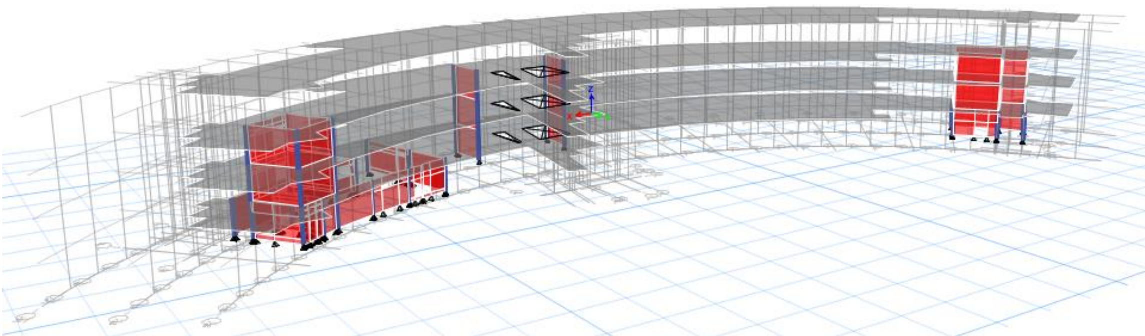
APPENDIX A

EXISTING STRUCTURAL ANALYSIS

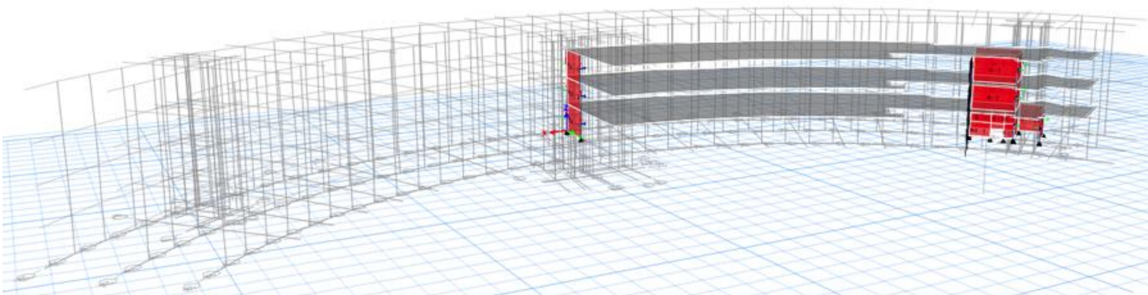
APPENDIX A.1 - EXISTING LATERAL SYSTEM MODELING

Evolution of the ETABS Model

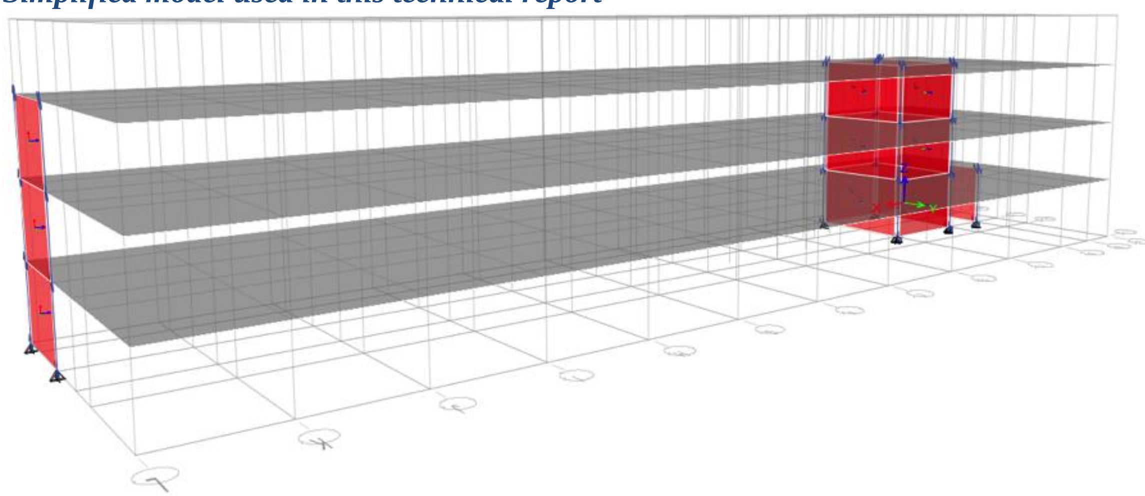
Model of entire building



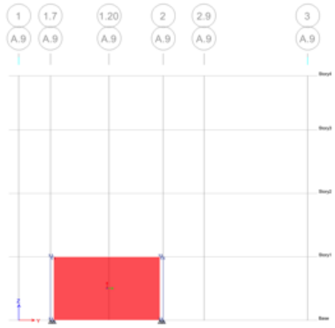
Model of half of the building, east side



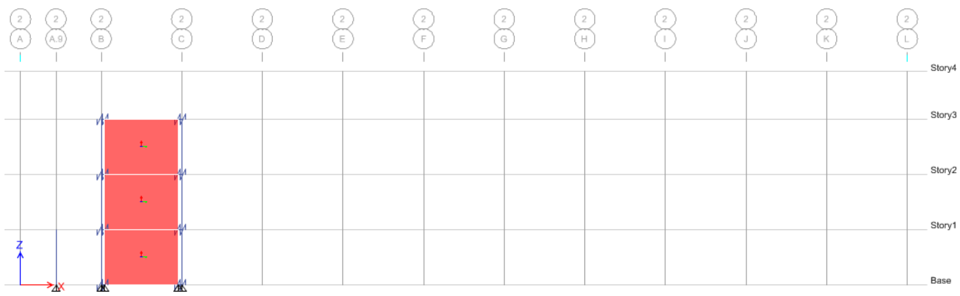
Simplified model used in this technical report



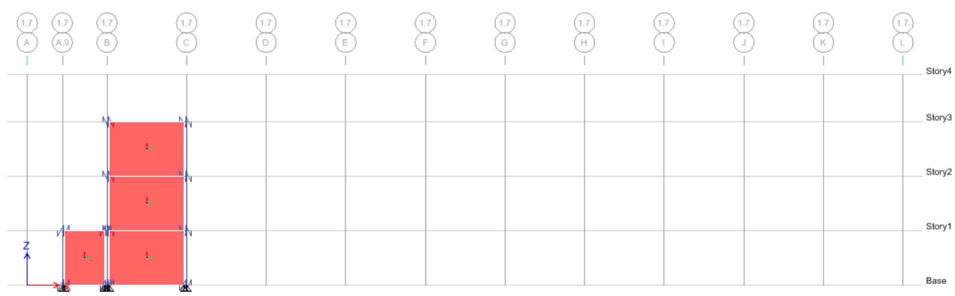
Elevations of Shear Walls



Shear Wall 1



Shear Wall 2, 4, 5, and 13@12



Shear Wall 3 and 3 (offset)

ETABS SPSW to Concrete Conversion

The steel plate shear wall lateral system was converted into an equivalent concrete shear wall system, using an effective stiffness method. This equates the stiffness of the steel plate shear wall to the stiffness of a concrete shear wall. This allows for an equivalent depth, of the concrete shear wall, to be solved for. It was found an equivalent depth of 2.98" would be used in the model.

Please find the calculations for the conversion of steel to effective concrete on the next page.

The steel plate shear wall lateral system was converted into an equivalent concrete shear wall system, using an effective stiffness method. This equates the stiffness of the steel plate shear wall to the stiffness of a concrete shear wall. This allows for the an equivlanet depth, of the concrete shear wall, to be solved for.

A36 STEEL

E =	29000	ksi
G =	11603	ksi
t =	0.375	in

CONCRETE

f'c =	4000
wc =	145
E =	3644
G =	1458

WALL DIMENSIONS				SPSW WALL PROPERTIES		SPSW STIFFNESS	UPDATED WALL PROPERITES		CONCRETE SHEAR WALL
h (ft)	h (in)	b (ft)	b (in)	Area (in2)	Inerita (in	k_{fixed} (k/in	Area (in2)	Inerita (in4)	$t_{effective}$ (in)
14	168	20	240	90	432000	4452	240	1152000	2.984901
14	168	25	300	112.5	843750	5862	300	2250000	2.984936
14	168	11.46	137.50	51.56	81230.87	1981	137.50	216615.65	2.984752
14	168	20.46	245.50	92.06	462363.75	4583	245.50	1232970.01	2.984905

EQUIVALENT DEPTH = 2.98 inches

Hand Calculation of SPSW to Concrete Conversion

PORTER-GILL | TECH REPORT 4 | EFFECTIVE SPSW

CONVERT SPSW TO EFFECTIVE THICKNESS OF CONCRETE:

$$K = \frac{1}{\frac{h^3}{12EI} + \frac{1.2h}{AG}} \qquad I = \frac{tb^3}{12} \qquad A = bt$$

SPSW

SPSW: $b = 20'$, $h = 14'$ $\quad E = 29000$ ksi, $G = 11603$ ksi $\quad t = 0.375''$

$$K_{SPSW} = \frac{1}{\frac{(14\times12)^3}{12(29000)\cdot\left(\frac{0.375(20\times12)^3}{12}\right)} + \frac{1.2(14\times12)}{(0.375(20\times12))(11603)}}$$

$= 4452.48 \sim 4453$ K/in FOR SPSW

CONC.

CONC: $b = 20'$, $h = 14'$ $\quad E = 3644$, $G = 1458$ $\quad t = $ (SOLVE)

$$4452.48 = \frac{1}{\frac{(14\times12)^3}{12\cdot3644\cdot\left(\frac{t\cdot(20\times12)^3}{12}\right)} + \frac{1.2(14\times12)}{(t\times(20\times12))(1458)}}$$

$t = 2.98''$ THICK CONCRETE WALL
FOR COMPARABLE
3/8'' THICK SPSW

Controlling Case Data Output

The controlling case for the building was found to be the earthquake loading in the y-direction.

PORTER-GILL TECH REPORT 4 SHEAR FORCES AND LOAD COMBINATIONS

Story 1 Shear Forces

Pier	Load Case/Combo	V2	Absolute Value
SW-1	QUAKE_X	-13.836	13.836
SW-1	QUAKE_X_REV	-3.049	3.049
SW-1	QUAKE_Y	-5.438	5.438
SW-1	QUAKE_Y_REVERSE	159.8	159.795
SW-1	C1_X	3.92	3.92
SW-1	C1_Y	34.435	34.435
SW-1	C2_X	5.784	5.784
SW-1	C2_Y	96.926	96.926
SW-1	C1_X	2.94	2.94
SW-1	C1_Y	37.213	37.213
SW-1	C4_X_COMBINED	4.262	4.262
SW-1	C4_Y_COMBINED	72.673	72.673

Max Shear = 159.795	for SW-1	
Controlling Load Case =	QUAKE_Y_REVERSE	
Tributary Area =	125	SF

Pier	Load Case/Combo	V2	Absolute Value
SW-13 (12)	QUAKE_X	25.141	25.141
SW-13 (12)	QUAKE_X_REV	3.689	3.689
SW-13 (12)	QUAKE_Y	546.4	546.403
SW-13 (12)	QUAKE_Y_REVERSE	217.8	217.797
SW-13 (12)	C1_X	-8.222	8.222
SW-13 (12)	C1_Y	90.98	90.98
SW-13 (12)	C2_X	-12.229	12.229
SW-13 (12)	C2_Y	208.07	208.07
SW-13 (12)	C1_X	-6.167	6.167
SW-13 (12)	C1_Y	97.827	97.827
SW-13 (12)	C4_X_COMBINED	-9.072	9.072
SW-13 (12)	C4_Y_COMBINED	155.52	155.524

Max Shear = 546.403	for SW-13 (12)	
Controlling Load Case =	QUAKE_Y	
Tributary Area =	215	SF

Pier	Load Case/Combo	V2	Absolute Value
SW-2	QUAKE_X	280.65	280.654
SW-2	QUAKE_X_REV	282.62	282.619
SW-2	QUAKE_Y	-40.502	40.502
SW-2	QUAKE_Y_REVERSE	-10.397	10.397
SW-2	C1_X	46.339	46.339
SW-2	C1_Y	-5.434	5.434
SW-2	C2_X	80.897	80.897
SW-2	C2_Y	-11.833	11.833
SW-2	C1_X	34.756	34.756
SW-2	C1_Y	-5.865	5.865
SW-2	C4_X_COMBINED	67.152	67.152
SW-2	C4_Y_COMBINED	-8.837	8.837

Max Shear = 282.619	for SW-2	
Controlling Load Case =	QUAKE_X_REV	
Tributary Area =	175	SF

Pier	Load Case/Combo	V2	Absolute Value
SW-3	QUAKE_X	299.17	299.17
SW-3	QUAKE_X_REV	297.85	297.85
SW-3	QUAKE_Y	27.798	27.798
SW-3	QUAKE_Y_REVERSE	7.573	7.573
SW-3	C1_X	47.395	47.395
SW-3	C1_Y	3.847	3.847
SW-3	C2_X	83.077	83.077
SW-3	C2_Y	8.428	8.428
SW-3	C1_X	35.548	35.548
SW-3	C1_Y	4.147	4.147
SW-3	C4_X_COMBINED	69.135	69.135
SW-3	C4_Y_COMBINED	6.295	6.295

Max Shear = 299.17	for SW-3	
Controlling Load Case =	QUAKE_X	
Tributary Area =	200	SF

PORTER-GILL TECH REPORT 4 SHEAR FORCES AND LOAD COMBINATIONS

Pier	Load Case/Combo	V2	Absolute Value
SW-3 (OFFSET)	QUAKE_X	155.27	155.267
SW-3 (OFFSET)	QUAKE_X_REV	154.53	154.534
SW-3 (OFFSET)	QUAKE_Y	14.225	14.225
SW-3 (OFFSET)	QUAKE_Y_REVERSE	2.997	2.997
SW-3 (OFFSET)	C1_X	24.213	24.213
SW-3 (OFFSET)	C1_Y	1.731	1.731
SW-3 (OFFSET)	C2_X	42.447	42.447
SW-3 (OFFSET)	C2_Y	3.692	3.692
SW-3 (OFFSET)	C1_X	18.161	18.161
SW-3 (OFFSET)	C1_Y	1.875	1.875
SW-3 (OFFSET)	C4_X_COMBINED	35.364	35.364
SW-3 (OFFSET)	C4_Y_COMBINED	2.756	2.756
Pier	**Load Case/Combo**	**V2**	**Absolute Value**
SW-4	QUAKE_X	-7.941	7.941
SW-4	QUAKE_X_REV	-0.615	0.615
SW-4	QUAKE_Y	39.025	39.025
SW-4	QUAKE_Y_REVERSE	151.24	151.243
SW-4	C1_X	2.925	2.925
SW-4	C1_Y	36.319	36.319
SW-4	C2_X	4.375	4.375
SW-4	C2_Y	97.667	97.667
SW-4	C1_X	2.194	2.194
SW-4	C1_Y	38.888	38.888
SW-4	C4_X_COMBINED	3.261	3.261
SW-4	C4_Y_COMBINED	73.191	73.191
Pier	**Load Case/Combo**	**V2**	**Absolute Value**
SW-5	QUAKE_X	-2.309	2.309
SW-5	QUAKE_X_REV	0.402	0.402
SW-5	QUAKE_Y	119.54	119.536
SW-5	QUAKE_Y_REVERSE	161.07	161.068
SW-5	C1_X	1.2	1.2
SW-5	C1_Y	44.728	44.728
SW-5	C2_X	1.819	1.819
SW-5	C2_Y	114.53	114.53
SW-5	C1_X	0.9	0.9
SW-5	C1_Y	47.967	47.967
SW-5	C4_X_COMBINED	1.37	1.37
SW-5	C4_Y_COMBINED	85.765	85.765

Max Shear = 155.267 for SW-3 (OFFSET)
Controlling Load Case = QUAKE_X
Tributary Area = 90 SF

Max Shear = 151.243 for SW-4
Controlling Load Case = QUAKE_Y_REVERSE
Tributary Area = 140 SF

Max Shear = 161.068 for SW-5
Controlling Load Case = QUAKE_Y_REVERSE
Tributary Area = 200 SF

OVERALL MAXIMUM SHEAR CONTROLLING = 546.4 kip

APPENDIX A.2 - EXISTING SEISMIC AND WIND ANALYSIS

Seismic Loading Calculations

add large page of calculations

Seismic Amplification Factor

The seismic amplification factor, A_x, was calculated for each story, for each earthquake loading. The worst case of a particular floor, for each case, was applied to calculate the total torsional moment and accidential torsional moment. 9.5.3.5.2 covers the amplification factor that must be applied to these moments.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad \text{(Eq. 9.5.3.5.2)}$$

QUAKE_X_REGULAR

Level	Maximum Displacement	Average Displacement	Amplification Factor	Updated Amplification Factor
Story3	1.650297	1.633777	0.708559251	1.0
Story2	0.888202	0.879394	0.708425206	1.0
Story1	0.295822	0.293091	0.707446301	1.0

QUAKE_X_REVERSE

Level	Maximum Displacement	Average Displacement	Amplification Factor	Updated Amplification Factor
Story3	1.637171	1.632025	0.698830707	1.0
Story2	0.881133	0.878502	0.698610216	1.0
Story1	0.293522	0.29287	0.697539891	1.0

QUAKE_Y_REGULAR

Level	Maximum Displacement	Average Displacement	Amplification Factor	Updated Amplification Factor
Story3	2.21301	1.271017	2.105239655	2.1
Story2	1.212938	0.691314	2.137784916	2.1
Story1	0.42688	0.229725	2.397908479	2.4

QUAKE_Y_REVERSE

Level	Maximum Displacement	Average Displacement	Amplification Factor	Updated Amplification Factor
Story3	1.358426	0.974261	1.350077945	1.4
Story2	0.744244	0.525585	1.392458523	1.4
Story1	0.262932	0.173467	1.59547742	1.6

Indicates controlling amplification factor

Wind Loading Calculations

Case 1

Case 1 - ASCE-7 1998

Case 1, X-Direction
East-West
L = 64 feet

Level	P_w	P_L	
1	10.91	-9.78	psf
2	11.04	-9.78	psf
3	12.57	-9.78	psf
4	13.60	-9.78	psf
Roof	14.76	-9.78	psf
Stair Tower	15.65	-9.78	psf

	P_w	P_L	
Stair Tower	15.65	-9.78	psf
Roof	14.76	-9.78	psf
4	13.60	-9.78	psf
3	12.57	-9.78	psf
2	11.04	-9.78	psf

	Force (k)	Force (k)	
Stair Tower	0.1409	-0.088	klf
Roof	0.2878	-0.191	klf
4	0.2380	-0.171	klf
3	0.1760	-0.137	klf
2	0.1545	-0.137	klf

Loads Applied to Diaphragm

	P_w	P_L	
lodified Roof	0.4286	-0.2788	klf
4	0.2380	-0.1712	klf
3	0.17601584	-0.136955	klf
2	0.1545	-0.1370	klf

Case 1, Y-Direction
North-South
L = 225 feet

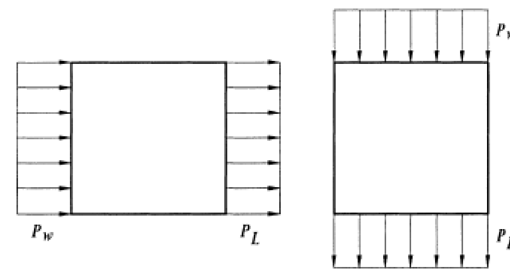
Level	P_w	P_L	
1	11.00	-3.95	psf
2	11.14	-3.95	psf
3	12.69	-3.95	psf
4	13.72	-3.95	psf
Roof	14.89	-3.95	psf
Stair Tower	15.79	-3.95	psf

	P_w	P_L	
Stair Tower	15.79	-3.95	psf
Roof	14.89	-3.95	psf
4	13.72	-3.95	psf
3	12.69	-3.95	psf
2	11.14	-3.95	psf

	Force (k)	Force (k)	
Stair Tower	0.1422	-0.036	klf
Roof	0.1340	-0.077	klf
4	0.1235	-0.069	klf
3	0.1142	-0.055	klf
2	0.1002	-0.055	klf

Loads Applied to Diaphragm

	P_w	P_L	
Modified Roof	0.2762	-0.1125	klf
4	0.1235	-0.0691	klf
3	0.114185095	-0.05528166	klf
2	0.1002	-0.0553	klf



CASE 1

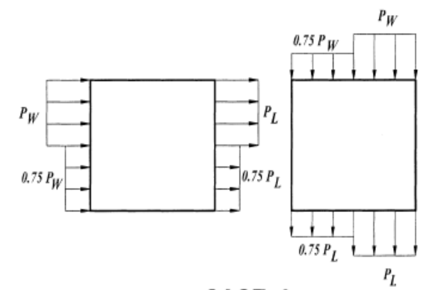
East-West, X

Point Load	Location X	Location Y
45.28		
71.46	112.5	32
20.03	112.5	32
18.65	112.5	32

North-South, Y

Point Load	Location X	Location Y
87.46		
130.80	112.5	32
38.13	112.5	32
34.99	112.5	32

Case 2



CASE 2

Case 2 - ASCE-7 1998

Case 2, X-Direction
East-West
L = 64 feet

Level	P_w	$0.75P_w$	P_L	$0.75P_L$	
1	10.91	8.18	-9.78	-7.34	psf
2	11.04	8.28	-9.78	-7.34	psf
3	12.57	9.43	-9.78	-7.34	psf
4	13.60	10.20	-9.78	-7.34	psf
Roof	14.76	11.07	-9.78	-7.34	psf
Stair Tower	15.65	11.74	-9.78	-7.34	psf

	P_w	$0.75P_w$	P_L	$0.75P_L$	
Stair Tower	15.65	11.74	-9.78	-7.34	psf
Roof	14.76	11.07	-9.78	-7.34	psf
4	13.60	10.20	-9.78	-7.34	psf
3	12.57	9.43	-9.78	-7.34	psf
2	11.04	8.28	-9.78	-7.34	psf

	Force (k)	Force (k)	Force (k)	Force (k)	
Stair Tower	0.1409	0.106	-0.0880	-0.066	klf
Roof	0.2878	0.216	-0.1908	-0.143	klf
4	0.2380	0.178	-0.1712	-0.128	klf
3	0.1760	0.132	-0.1370	-0.103	klf
2	0.1545	0.116	-0.1370	-0.103	klf

Loads Applied to Diaphragm

	P_w	$0.75P_w$	P_L	$0.75P_L$	
Modified Roof	0.4286	0.3215	-0.2788	-0.2091	klf
4	0.2380	0.1785	-0.1712	-0.1284	klf
3	0.1760	0.132	-0.1370	-0.103	klf
2	0.1545	0.1159	-0.1370	-0.1027	klf

Case 2, Y-Direction
North-South
L = 225 feet

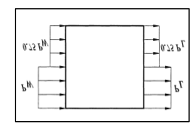
Level	P_w	$0.75P_w$	P_L	$0.75P_L$	
1	11.00	8.25	-3.95	-2.96	psf
2	11.14	8.35	-3.95	-2.96	psf
3	12.69	9.52	-3.95	-2.96	psf
4	13.72	10.29	-3.95	-2.96	psf
Roof	14.89	11.17	-3.95	-2.96	psf
Stair Tower	15.79	11.85	-3.95	-2.96	psf

	P_w	$0.75P_w$	P_L	$0.75P_L$	
Stair Tower	15.79	11.85	-3.95	-2.96	psf
Roof	14.89	11.17	-3.95	-2.96	psf
4	13.72	10.29	-3.95	-2.96	psf
3	12.69	9.52	-3.95	-2.96	psf
2	11.14	8.35	-3.95	-2.96	psf

	Force (k)	Force (k)	Force (k)	Force (k)	
Stair Tower	0.1422	0.107	-0.0355	-0.027	klf
Roof	0.2904	0.218	-0.0770	-0.058	klf
4	0.2402	0.180	-0.0691	-0.052	klf
3	0.1776	0.133	-0.0553	-0.041	klf
2	0.1559	0.117	-0.0553	-0.041	klf

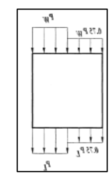
Loads Applied to Diaphragm

	P_w	$0.75P_w$	P_L	$0.75P_L$	
Modified Roof	0.4325	0.3244	-0.1125	-0.0844	klf
4	0.2402	0.1801	-0.0691	-0.0518	klf
3	0.1776	0.133	-0.0553	-0.041	klf
2	0.1559	0.1169	-0.0553	-0.0415	klf



East-West for 1.0P

Point Load	Location X	Location Y
45.28		
71.46	112.5	16
20.03	112.5	16
18.65	112.5	16



North-South for 1.0P

Point Load	Location X	Location Y
122.64		
192.22	56.25	32
52.40	56.25	32
47.52	56.25	32

East-West for 0.75P

Point Load	Location X	Location Y
33.96		
53.60	112.5	48
15.02	112.5	48
13.99	112.5	48

North-South for 0.75P

Point Load	Location X	Location Y
91.98		
144.17	168.75	32
39.30	168.75	32
35.64	168.75	32

Case 3

Case 3 - ASCE-7 1998

Case 3, X-Direction
East-West
L = 64 feet

Level	$0.75P_w$	$0.75P_L$
1	8.18	-7.34
2	8.28	-7.34
3	9.43	-7.34
4	10.20	-7.34
Roof	11.07	-7.34
Stair Tower	11.74	-7.34

	$0.75P_w$	$0.75P_L$	
Stair Tower	11.74	-7.34	psf
Roof	11.07	-7.34	psf
4	10.20	-7.34	psf
3	9.43	-7.34	psf
2	8.28	-7.34	psf

	Force (k)	Force (k)	
Stair Tower	0.1057	-0.066	klf
Roof	0.2158	-0.143	klf
4	0.1785	-0.128	klf
3	0.1320	-0.103	klf
2	0.1159	-0.103	klf

Loads Applied to Diaphragm

	$0.75P_w$	$0.75P_L$	
odified Roof	0.3215	-0.2091	klf
4	0.1785	-0.1284	klf
3	0.1320	-0.1027	klf
2	0.1159	-0.1027	klf

Case 3, Y-Direction
North-South
L = 225 feet

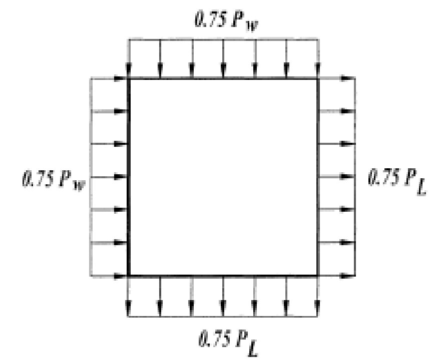
Level	$0.75P_w$	$0.75P_L$
1	8.25	-2.96
2	8.35	-2.96
3	9.52	-2.96
4	10.29	-2.96
Roof	11.17	-2.96
Stair Tower	11.85	-2.96

	$0.75P_w$	$0.75P_L$	
Stair Tower	11.85	-2.96	psf
Roof	11.17	-2.96	psf
4	10.29	-2.96	psf
3	9.52	-2.96	psf
2	8.35	-2.96	psf

	Force (k)	Force (k)	
Stair Tower	0.1066	-0.027	klf
Roof	0.2178	-0.058	klf
4	0.1801	-0.052	klf
3	0.1332	-0.041	klf
2	0.1169	-0.041	klf

Loads Applied to Diaphragm

	$0.75P_w$	$0.75P_L$	
Modified Roof	0.3244	-0.0844	klf
4	0.1801	-0.0518	klf
3	0.1332	-0.0415	klf
2	0.1169	-0.0415	klf



CASE 3

East-West

Point Load	Location X	Location Y
[redacted]		
53.60	112.5	32
15.02	112.5	32
13.99	112.5	32

North-South

Point Load	Location X	Location Y
[redacted]		
144.17	112.5	32
39.30	112.5	32
35.64	112.5	32

Case 4

Case 4 - ASCE-7 1998

Case 4, X-Direction
East-West
L = 64 feet

Level	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
1	8.18	6.11	-7.34	-5.48	psf
2	11.04	6.18	-7.34	-5.48	psf
3	12.57	7.04	-7.34	-5.48	psf
4	13.60	7.62	-7.34	-5.48	psf
Roof	14.76	8.26	-7.34	-5.48	psf
Stair Tower	15.65	8.77	-7.34	-5.48	psf

	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
Stair Tower	15.65	8.77	-7.34	-5.48	psf
Roof	14.76	8.26	-7.34	-5.48	psf
4	13.60	7.62	-7.34	-5.48	psf
3	12.57	7.04	-7.34	-5.48	psf
2	11.04	6.18	-7.34	-5.48	psf

	Force (k)	Force (k)	Force (k)	Force (k)	
Stair Tower	0.1409	0.079	-0.0660	-0.049	klf
Roof	0.2878	0.161	-0.1431	-0.107	klf
4	0.2380	0.133	-0.1284	-0.096	klf
3	0.1760	0.099	-0.1027	-0.077	klf
2	0.1545	0.087	-0.1027	-0.077	klf

Loads Applied to Diaphragm

	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
odified Roof	0.4286	0.2400	-0.2091	-0.1561	klf
4	0.2380	0.1333	-0.1284	-0.0959	klf
3	0.1760	0.099	-0.1027	-0.077	klf
2	0.1545	0.0865	-0.1027	-0.0767	klf

Case 4, Y-Direction
North-South
L = 225 feet

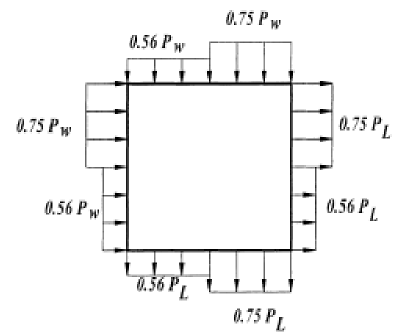
Level	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
1	8.25	6.16	-2.96	-2.21	psf
2	8.35	6.24	-2.96	-2.21	psf
3	9.52	7.10	-2.96	-2.21	psf
4	10.29	7.68	-2.96	-2.21	psf
Roof	11.17	8.34	-2.96	-2.21	psf
Stair Tower	11.85	8.85	-2.96	-2.21	psf

	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
Stair Tower	11.85	8.85	-2.96	-2.21	psf
Roof	11.17	8.34	-2.96	-2.21	psf
4	10.29	7.68	-2.96	-2.21	psf
3	9.52	7.10	-2.96	-2.21	psf
2	8.35	6.24	-2.96	-2.21	psf

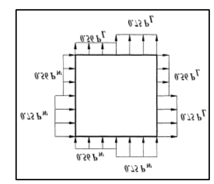
	Force (k)	Force (k)	Force (k)	Force (k)	
Stair Tower	0.1066	0.080	-0.0267	-0.020	klf
Roof	0.2178	0.163	-0.0577	-0.043	klf
4	0.1801	0.134	-0.0518	-0.039	klf
3	0.1332	0.099	-0.0415	-0.031	klf
2	0.1169	0.087	-0.0415	-0.031	klf

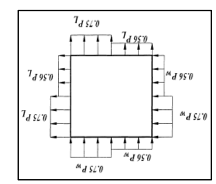
Loads Applied to Diaphragm

	0.75P_w	0.56P_w	0.75P_L	0.56P_L	
Modified Roof	0.3244	0.2422	-0.0844	-0.0630	klf
4	0.1801	0.1345	-0.0518	-0.0387	klf
3	0.1332	0.099	-0.0415	-0.031	klf
2	0.1169	0.0873	-0.0415	-0.0310	klf



CASE 4





East-West for 0.75P

Point Load	Location X	Location Y
40.81		
64.26	112.5	16
17.84	112.5	16
16.46	112.5	16

North-South for 0.75P

Point Load	Location X	Location Y
91.98		
144.17	56.25	32
39.30	56.25	32
35.64	56.25	32

East-West for 0.56P

Point Load	Location X	Location Y
25.35		
40.02	112.5	48
11.22	112.5	48
10.45	112.5	48

North-South for 0.56P

Point Load	Location X	Location Y
68.68		
107.64	168.75	32
29.35	168.75	32
26.61	168.75	32

APPENDIX B

REDESIGN OF GRAVITY SYSTEM

APPENDIX B.1 - TYPICAL OFFICE BEAM DESIGN

BEAM DESIGN (FRAMING INTO QUEEN POST)

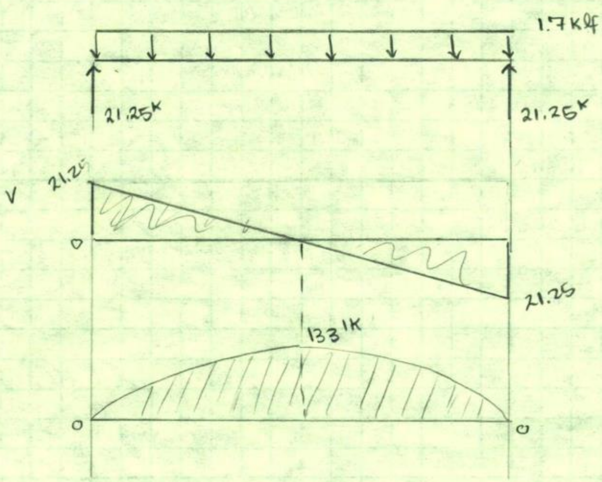
w = 164.5 PSF

TRIB. WIDTH : 9'-8" ≈ 9.67' → ROUND TO 10'

164.5 PSF × 10' = 1645 PLF or 1.7 KLF ON BEAMS

$$M = \frac{1.7(25')^2}{8} = 133'^K$$

$$R = \frac{1.7(25)}{2} = 21.25^K$$



$M_{max} = 133'^K$

GLULAM, PRIMARILY IN BENDING → TABLE 5A

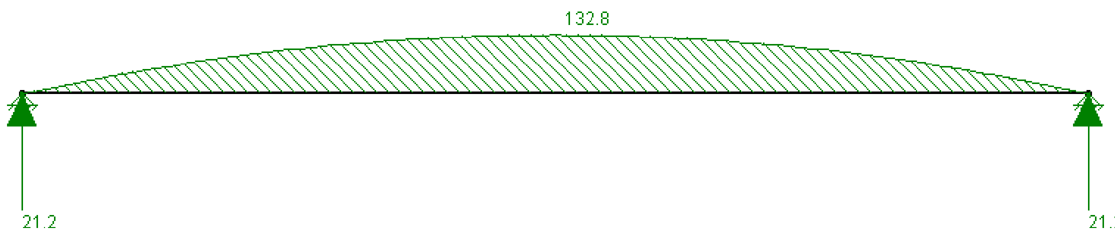
PLEASE SEE EXCEL SHEET FOR INTERPOLATED GLULAM SIZE...

Loading

Computer analysis loading

Flexure and Reactions

Computer analysis results, showing the maximum moment is 132.8 kip-ft or 133 kip-ft



Computer Analysis Data

Designer : **SIKANDAR PORTER-GILL**

BEAM ANALYSIS

February 11, 2014
12:09 PM
Checked By:_____

Member Data

Member Label	I Joint	J Joint	Area in^2	Moment of Inertia in^4	Elastic Modulus ksi	End Releases I-End	End Releases J-End	Length ft
M1	N1	N2	10	100	29000			25

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-1.7	-1.7	0	0

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	0	21.25	0
N2	0	21.25	0
Totals:	0	42.5	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	21.25	0
	2	0	10.625	99.609
	3	0	0	132.812
	4	0	-10.625	99.609
	5	0	-21.25	0

Member Sizing

Flexure in Beam

Moment 133 kip-ft

$F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_V \times C_{fu} \times C_c \times C_i$

Pick a size,

10-1/2" x 19-1/4""

10.5 x 19.25

where the $A_{provided}$ = 202.1 in^2

$S_{sect\ modulus}$ = 648.5 in^3

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_L =	0.987	calculated below	§5.3.5
C_V =	0.934	calculated below	§5.3.6
C_{fu} =	1.00	because not loaded parellel to wide faces of lamin.	§5.3.7
C_c =	1.00	because no curvature to beam	§5.3.8
C_i =	1.00	because no tapering of beam	§5.3.9

Pick a Visually Graded Southern Pine Stress Group — Table 5A

Group = 30F-2.1E SP

F_b = 3000 psi

Emin = 1110000 psi

Calculate C_L Adjustment Factor

l_u = 25.00 ft, the unbraced length of the girder

d = 19.25 in, choosen to be consistent with girder depth

l_u / d = 15.58

so now we can calculate l_e,

l_e = 552 in, or 46.00 ft

reliant on inequality on page 16, Supplem

R_B = 9.82

$$F_{bE} = \frac{1.20 E'_{min}}{(R_B)^2} = 13820.2$$

$F_b^* = F_b \times C_D \times C_M \times C_t \times C_c \times C_i =$ 3000 psi

$F_{bE} / F_b^* =$ 4.61

$$C_L = \frac{1 + {F_{bE}}/{F_b *}}{1.9} - \sqrt{\left(\frac{1 + {F_{bE}}/{F_b *}}{1.9}\right)^2 - \frac{{F_{bE}}/{F_b *}}{0.95}} = 0.987$$

Calculate C_V Adjustment Factor

$$C_V = \left(\frac{21}{L}\right)^{1/x} \left(\frac{12}{d}\right)^{1/x} \left(\frac{5.125}{b}\right)^{1/x} \leq 1.0$$

L =	25	ft	x =	20	for Southern Pine
d =	19.25	in			
b =	10.5	in			

$C_V =$ 0.934 < 1.0

Calculate F_b' Using the Minimum of C_V or C_L

min | $C_L =$ 0.987
$C_V =$ 0.934

$F'_b =$ 2802 psi

$f_b = \frac{M}{S} =$ 2461.1 psi < F'_b

Calculate f_b and Determine if Selected Beam Passes

$f_b =$ 2461 psi < $F_b' =$ 2802

Bending Passes

Use a 10-1/2" x 19-1/4"" for the beam

APPENDIX B.2 - QUEEN POST DESIGN HAND CALCULATION

QUEEN POST DESIGN

LOADS:

• SUPERIMPOSED DL
1.5 CARPET
12 COMPUTER
10 SDL (MECH + LTG + SPRINKLER)
10 FRAMING

33.5 PSF SDL

• LIVE LOADS
80 OFFICE + PARTITIONS

• DEAD LOAD
51 3VLI20 VULCRAFT DECKING

ASD → SDL + LL + DL = 33.5 + 51 + 80 = 164.5 PSF

BEAMS (INTERIOR)

TRIB. WIDTH = 9'-8" ~ 10'

164.5 PSF × 10' = 1645 PLF OR 1.7 KLF ON BEAMS

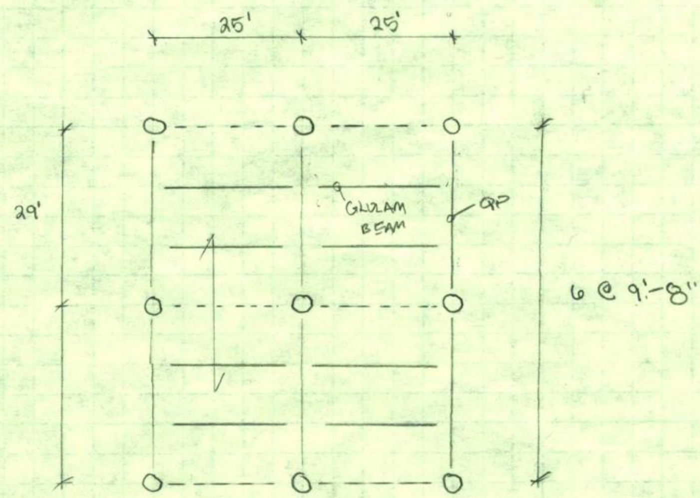
$M = \frac{wl^2}{8} = \frac{1.7(25')^2}{8} = 133^{1K}$ MAX ON BEAMS (INTERIOR)

$R = \frac{wl}{2} = \frac{1.7(25')}{2} =$ 21.25 REACTION AT ENDS

×2 b/c 2 BEAMS FALL ON EA. GIRDER SECTION

$= 42.5^K$

GENERAL FRAMING LAYOUT



DUE TO ASSUMPTION OF HINGE, ALL 42.5^k WOULD BE TRANSFERED INTO POST

TABULATED IN EXCEL PRINTOUTS

LET'S CHOOSE 9'-8" FROM BEAM END TO POST W/ 2.25' HIGH POST

$f_c = 181.37^k$ AXIAL LOAD (PARALLEL TO BEAM GRAIN)

$f_{c\perp} = 42.5^k$ SHEAR LOAD (PEPPENDICULAR TO BEAM GRAIN)

$f_b = \frac{M}{S}$ (BENDING)

SOUTHERN PINE WILL BE USED

COMPRESSION PARALLEL TO BEAM GRAIN

$F_c' = F_c \times C_M \times C_t \times C_P$ TABLE 53.1

ADJUSTMENT FACTORS

$C_D = 1.0$

$C_M = 1.0$

$C_t = 1.0$

$C_P = 0.65$ assumed

$F_c =$ 1900 psi FOR 47 VISUALLY GRADED SOUTHERN PINE
FOR 4 OR MORE LAMINATIONS

so, $F_c' = (2300) \times 1 \times 1 \times 1 \times 0.65$

$= 1235$ psi

we know $f_c = 181.37^{kips} \sim 182^{kips} / \text{AREA}$

$f_c \leq F'_c$

$\frac{182^{kips}}{\text{AREA}} \times 1000^{lb/k} \leq 1235 \text{ psi}$

$\text{AREA} = 147.36 \text{ IN}^2$

a $10\text{-}1/2" \times 15\text{-}1/8" \rightarrow A = 158.8 \text{ IN}^2$

CHECK $C_p = 0.65$ ASSUMPTION

$$C_p = \frac{1 + F_{cE}/F_c^*}{2c} - \sqrt{\left(\frac{1 + F_{cE}/F_c^*}{2c}\right)^2 - \frac{F_{cE}/F_c^*}{c}}$$

$$F_c^* = F_c \times C_D \times C_M \times C_t$$

$$= 1900 \times 1 \times 1 \times 1 = 1900 \text{ psi}$$

$$\frac{l_e}{d} = \frac{9.67' \times 12''/'}{10.5} = 11.05 < 50 \checkmark \rightarrow \text{CONTROLS}$$

$$= \frac{9.67' \times 12''/'}{15.125} = 7.67 < 50 \checkmark$$

$$E'_{min} = E_{min} \times C_m \times C_t$$

$$= (0.74 \times 10^6) \times 1.0 \times 1.0$$

$$= 0.74 \times 10^6$$

$$F_{cE} = \frac{0.822\,E'_{min}}{(l_e/d)^2} = \frac{0.822\,(0.74 \times 10^6)}{(11.05^2)}$$

$$= 4982$$

$$F_{cE} / F_c^* = 4982/1900 = 2.62 \text{ ratio}$$

$$C_p = \frac{1 + 2.62}{2 \cdot 0.9} - \sqrt{\left(\frac{1 + 2.62}{2 \cdot 0.9}\right)^2 - \frac{2.62}{0.9}}$$

$$C_p = 0.946 > 0.65 = C_{p,assumed}$$

So, member can be smaller...

Lets assume $C_p = 0.9$ instead

$F'_c = 1900 \times 1 \times 1 \times 1 \times 0.9 = 1710$ psi

$AREA = \dfrac{182^k \times 1000}{1710} = 106.43\ IN^2 \longrightarrow$ 8-1/2" × 13-3/4"
APROV = 116.9 IN²

Now CHECK, $C_p = 0.9$

$F_c^* = 1900$ psi (no change)

$l_e/d = \dfrac{9.67' \times 12''/1}{8.5} = 13.65 < 50$ ✓ → CONTROLS

$= \dfrac{9.67' \times 12''/1}{13.75} = 8.43 < 50$ ✓

$E'_{min} = 0.74 \times 10^6$ psi (no change)

$F_{cE} = \dfrac{0.822(0.74 \times 10^6)}{13.65^2} = 3264.6$ psi

$F_{cE}/F_c^* = 3265/1900 = 1.71$

$C_p = \dfrac{1 + 1.71}{2 \cdot 0.9} - \sqrt{\left(\dfrac{1+1.71}{2 \cdot 0.9}\right)^2 - \dfrac{1.71}{0.9}}$

$C_p = 0.9067$ ✓ GOOD ASSUMPTION

So, $F'_c = 1710 \times 116.9\ in^2 / 1000 = 199.89^k$ allowed

$f_c = \dfrac{182^k \times 1000}{116.9} = 1556$ psi

$f_c = 1556 < F'_c = 1710$ ✓ GOOD
8½ × 13-3/4

SELF WEIGHT OF SELECTED MEMBER

$$D = 62.4\left(\frac{G}{1 + G(0.009)(M.C.)}\right)\left(1 + \frac{M.C.}{100}\right)$$

G = 0.55 FOR 47 SOUTHERN PINE GLULAM

M.C. = 5% b/c INSIDE
OR 10%

$$D = 62.4\left(\frac{0.55}{1 + 0.55(0.009)(5)}\right)\left(1 + \frac{5}{100}\right)$$

or 10 / or 10

= 35.17 or 35.97 PCF

we have 8-1/2 x 13-3/4, A = 116.9 IN²

$$A = 116.9\ IN^2 \times \frac{1^2\ ft^2}{12^2\ IN^2} = 0.8118\ ft^2$$

$$35.97\ \frac{lb}{ft^3} \times 0.8118\ ft^2 = 29.2\ plf$$

SELF WEIGHT AND POINT LOADS CAUSE MOMENT (MAX) OF 8.819k ~ 8.9'k ← COMPUTER ANALYSIS

BENDING ANALYSIS

$F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_V \times C_{fu} \times C_c \times C_I$

$F_b = 1400$ psi (about x-x)

ADJUSTMENT FACTORS

$C_D = 1.0$

$C_M = 1.0$

$C_t = 1.0$

$C_L \neq 1.0$

$l_u = 9.67'$

$d = 13.75''$

$l_u/d = \dfrac{9.67' \times 12''/'}{13.75''} = 8.44$

$7 \leq l_u/d \leq 14.3$

$l_e = 1.63 l_u + 3d$

$= 1.63(9.67' \times 12) + 3(13.75'')$

$= 230.39''$ or $19.19'$

$R_B = \sqrt{\dfrac{l_e d}{b^2}} = \sqrt{\dfrac{(19.19 \times 12)(13.75)}{(8.5^2)}} = 6.62$

$F_{bE} = \dfrac{1.20\, E'_{min}}{R_B^2} = \dfrac{1.2(0.74 \times 10^6)}{(6.62^2)} = 20263$

$F_b^* = F_b \times C_D \times C_M \times C_t \times C_c \times C_I$

$= 1400 \times 1 \times 1 \times 1 \times 1 \times 1 = 1400$ psi

$F_{bE}/F_b^* = 20263/1400 = 14.47$

So,

$$C_L = \frac{1 + F_{bE}/F_b^*}{1.9} - \sqrt{\left(\frac{1 + F_{bE}/F_b^*}{1.9}\right)^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

$$C_L = 0.9963$$

$$C_V = \left(\frac{21}{29'}\right)^{1/20} \cdot \left(\frac{12}{13.75}\right)^{1/20} \cdot \left(\frac{5.125}{8.5}\right)^{1/20} = 0.953$$

$$\min \left| \begin{array}{l} C_L = 0.9963 \\ C_V = 0.953 \end{array} \right. \longrightarrow 0.953 \text{ CONTROLS}$$

$C_{fu} = 1.0$ b/c NOT loaded parallel to wide faces of laminations

$C_c = 1.0$

$C_I = 1.0$

Now, $F_b' = 1400 \times 1 \times 1 \times 1 \times 0.953 \times 1 \times 1 \times 1$

$= 1334.2$ psi

SELF+POINT LOADS

$$f_b = \frac{M}{S} = \frac{8.9^k \times 12}{267.8} = 0.3988\text{ksi} < 1334 \text{ psi allowed}$$

$$\downarrow$$

$$f_b = 398.8 \text{ psi} < 1334 \text{ psi} = F_b'$$

✓ Good

8-1/2" x 13-3/4"

COMBINED AXIAL AND LOADING

$$\left(\frac{f_c}{F'_c}\right)^2 + \frac{f_{b1}}{F'_{b1}(1 - f_c/F_{cE1})} \leq 1.0$$

$$\left(\frac{1556}{1710}\right)^2 + \frac{398.8}{1334(1 - 1556/8540)} = 1.191 \nleq 1.0$$

X FAILS
INTERACTION

where,

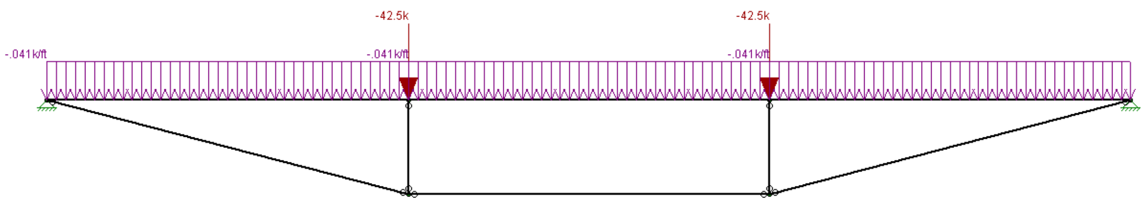
$$F_{cE} = \frac{0.822\, E'_{min}}{(l_{e1}/d_1)^2} = \frac{0.822(0.74 \times 10^6)}{\left(\frac{9.67' \times 12}{13.75}\right)^2} = 8540$$

AN EXCEL SHEET WILL BE USED TO INTERPOLATE THE
SIZE OF THE GLULAM BEAM REQUIRED...

APPENDIX B.3 - TYPICAL OFFICE QUEEN POST DESIGN

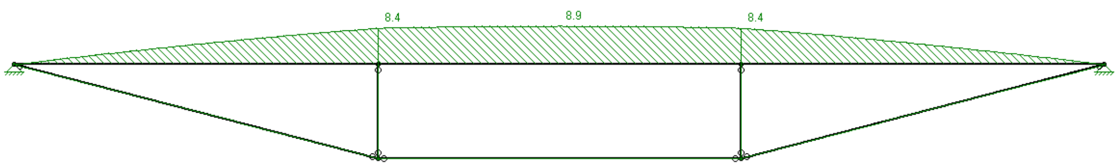
Loading

Computer analysis loading



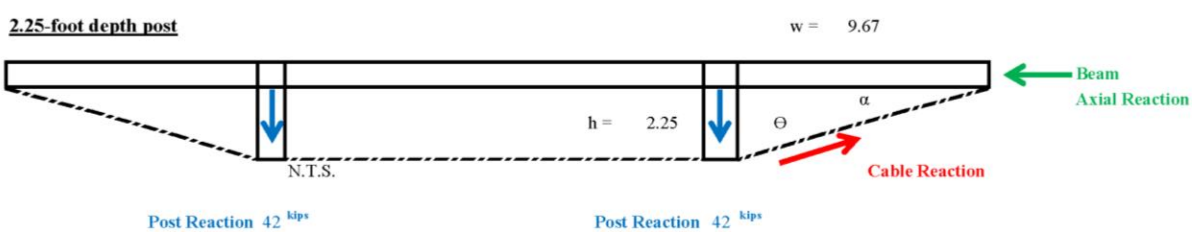
Flexure and Reactions

Computer analysis results, showing the maximum moment is 8.9 kip-ft



Axial Cable and Girder Forces

The assumption of the hinged queen post was used to determine the post reactions, cable tension and girder axial forces.



PRELIMINARY CALCULATIONS

Θ = tan-1(w/h) = 1.34 radians

α = 88.66 radians

CALCULATE RESULTANT FORCES IN CABLE AND BEAM

Cable Reaction 186.21 kips

Beam Axial Reaction 181.37 kips

Computer Analysis Data

Designer : **SIKANDAR PORTER-GILL** **GIRDER ANALYSIS** February 11, 2014 12:03 PM Checked By:_____

Member Data

Member Label	I Joint	J Joint	Area in^2	Moment of Inertia in^4	Elastic Modulus ksi	End Releases I-End	End Releases J-End	Length ft
M1	N1	N2	10	100	29000			9.67
M2	N2	N3	10	100	29000			9.66
M3	N3	N4	10	100	29000			9.67
M4	N1	N5	10	100	29000	PIN	PIN	9.988
M5	N5	N6	10	100	29000	PIN	PIN	9.66
M6	N6	N4	10	100	29000	PIN	PIN	9.988
M7	N2	N5	10	100	29000	PIN	PIN	2.5
M8	N3	N6	10	100	29000	PIN	PIN	2.5

Joint Loads/Enforced Displacements

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (k, k-ft, in, rad)
N2	L	Y	-42.5
N3	L	Y	-42.5

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-.041	-.041	0	0
M2	Y	-.041	-.041	0	0
M3	Y	-.041	-.041	0	0

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-162.565	43.095	0
N4	162.565	43.094	0
Totals:	0	86.189	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	1.066	0
	2	0	.967	2.458
	3	0	.868	4.676
	4	0	.769	6.655
	5	0	.67	8.394
M2	1	0	.198	8.394
	2	0	.099	8.752
	3	0	0	8.872
	4	0	-.099	8.752
	5	0	-.198	8.394
M3	1	0	-.67	8.394
	2	0	-.769	6.655
	3	0	-.868	4.676
	4	0	-.967	2.458
	5	0	-1.066	0
M4	1	-167.91	0	0
	2	-167.91	0	0
	3	-167.91	0	0

Designer : **SIKANDAR PORTER-GILL**

GIRDER ANALYSIS

February 11, 2014
12:03 PM
Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	-167.91	0	0
	5	-167.91	0	0
M5	1	-162.565	0	0
	2	-162.565	0	0
	3	-162.565	0	0
	4	-162.565	0	0
	5	-162.565	0	0
M6	1	-167.91	0	0
	2	-167.91	0	0
	3	-167.91	0	0
	4	-167.91	0	0
	5	-167.91	0	0
M7	1	42.028	0	0
	2	42.028	0	0
	3	42.028	0	0
	4	42.028	0	0
	5	42.028	0	0
M8	1	42.028	0	0
	2	42.028	0	0
	3	42.028	0	0
	4	42.028	0	0
	5	42.028	0	0

Top Chord Member Sizing

Compression Parallel to Beam Grain Axial Compression **181.37** kips

$F'_c = F_c \times C_D \times C_M \times C_t \times C_p$

Adjustment Factors

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_p =	0.92	assumed value	§3.7.1

Pick a Visually Graded Southern Pine Stress Group — Table 5B

Group =	50	
F_c =	2300	psi
Emin =	1000000	psi

so,

F'_c = 2116 psi allowable compression stress

now the required area would be,

A = 86 in^2 required area of glulam

Pick a,

8-1/2" x 19-1/4"

8.5 x 19.25

where the $A_{provided}$ = 163.6 in^2

Is the area greater than required area? Yes

Check the Assumption of the C_p Adjustment Factor

$$C_p = \frac{1 + {F_{CE}}/{F_c *}}{2c} - \sqrt{\left(\frac{1 + {F_{CE}}/{F_c *}}{2c}\right)^2 - \frac{{F_{CE}}/{F_c *}}{c}}$$

$F_C* = F_c \times C_D \times C_M \times C_t =$ 2300 psi

$l_e / d =$ 13.65 and 6.03 *where* *13.65* *controls*

< 50 < 50

$E_{min}' = E_{min} \times C_M \times C_t =$ 1000000 psi

$F_{CE} = \frac{0.822E'_{min}}{\left({l_e}/{d}\right)^2} =$ 4414 psi

$F_{CE} / F_C* =$ 1.92 c = 0.9

now the C_P adjustment factor can be calculated

$C_P =$ 0.92 < $C_{P,asummed}$

Calculate f_c and Determine if Selected Beam Passes

$f_c =$ 1108 psi < $F_C' =$ 2116

Compression Parallel to Grain Passes

Moment Induced by Self-Weight of Member

G = 0.55 Table 5B

M.C. = 5 % or 10 %
because interior beam in conditioned space

$D = 62.4\left(\frac{G}{1+G(0.009)(M.C.)}\right)\left(1+\frac{M.C.}{100}\right) =$ 35.17 pcf, or 35.97 pcf

we will take the maximum,
D = 35.97 pcf

we have a 8-1/2" x 19-1/4" *glulam beam with,*
A = 163.6 in^2

convert to square feet,
A = 1.1363 ft^2

now calculate the linear load created by its self weight, over a 29' span
w = 40.87 plf

Flexure in Queen Post Girder Moment 8.9 kip-ft

$F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_V \times C_{fu} \times C_c \times C_i$

Adjustment Factors

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_L =	0.994	calculated below	§5.3.5
C_V =	0.937	calculated below	§5.3.6
C_{fu} =	1.00	because not loaded parellel to wide faces of lamin.	§5.3.7
C_c =	1.00	because no curvature to beam	§5.3.8
C_i =	1.00	because no tapering of beam	§5.3.9

Pick a Visually Graded Southern Pine Stress Group Table 5B

Group =	50	
F_b =	2100	psi
Emin =	1000000	psi

Calculate C_L Adjustment Factor

l_u = 9.67 ft, the unbraced length of the girder

d = 19.25 in, depth choosen in compression parellel to grain calculation

l_u / d = 6.03

so now we can calculate l_e,

l_e = 246.83 in, or 20.57 ft

R_B = 8.11

$$F_{bE} = \frac{1.20E'_{min}}{(R_B)^2} = 18247$$

$F_b^* = F_b \times C_D \times C_M \times C_t \times C_c \times C_i$ = 2100 psi

F_{bE} / F_b^* = 8.69

$$C_L = \frac{1+{F_{bE}}/{F_b*}}{1.9} - \sqrt{\left(\frac{1+{F_{bE}}/{F_b*}}{1.9}\right)^2 - \frac{{F_{bE}}/{F_b*}}{0.95}} = 0.994$$

Calculate C_V Adjustment Factor

$$C_V = \left(\frac{21}{L}\right)^{1/x}\left(\frac{12}{d}\right)^{1/x}\left(\frac{5.125}{b}\right)^{1/x} \leq 1.0$$

L =	29	ft	x =	20	for Southern Pine
d =	19.25	in			
b =	8.5	in			

C_V = 0.937 < 1.0

Calculate F_b' Using the Minimum of C_V or C_L

	C_L =	0.994	Section Modulus (x) =	525	in^3
min	C_V =	0.937			

F'_b = 1968 psi

$f_b = \frac{M}{S} =$ 203.4 psi < F'_b

Calculate f_b and Determine if Selected Beam Passes

f_b = 203 psi < F_b' = 1968

Bending Passes

Combined Axial and Bending Loading Interaction

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left(1 + {f_c}/{F_{CE1}}\right)} \leq 1.0$$

§3.9.2

f_c =	1108	psi	E_{min}' =	1000000	psi
F_c' =	2116	psi			
f_{b1} =	203	psi			
F_{b1} =	1968	psi			

$$F_{CE1} = \frac{0.822E_{min}'}{\left({l_{e1}}/{d_1}\right)^2} = 22621.3 \text{ psi}$$

where,

l_{e1} = 9.67 ft

d_1 = 19.25 in

$f_c < F_{CE1}$	True

0.274 + 0.099 = 0.373 < 1.0

Combined Axial and Bending Pass

Use a,

8-1/2" x 19-1/4"

for the glulam queen post

With a,

Southern Pine Group of 50

Tension Cable Sizing

QUEEN POST - TENSION CABLE

186.21 kips ⟶ 190 kips

MACALLOY 460 BAR SYSTEM

	MIN YIELD	MIN BREAK
M56	205 k	271.8 k
M64	270.7 k	358.8 k

M56 205 k > 190 k TOTAL ✓ GOOD

DESIGN CALLS FOR TWO CABLES
✓ FOR STABILITY
✓ ADDED SAFETY

$\boxed{\text{(2) M56 MACALLOY 460 BARS FOR TENSION WILL BE USED}}$

4

Macalloy 460 Bar System

Table 1 - Tendon Capacities for Carbon Macalloy 460

Thread	mm	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
	inch	3/8	1/2	5/8	3/4	1	1 1/4	1 3/8	1 5/8	2	2 1/4	2 1/2	3	3 3/8	3 1/2	4
Nominal Bar Dia.	mm	10	11	15	19	22	28	34	39	45	52	60	72	82	87	97
	inch	0.39	0.43	0.59	0.75	0.87	1.1	1.34	1.54	1.77	2.05	2.36	2.83	3.23	3.43	3.82
Min. Yield Load	kN	25	36	69	108	156	249	364	501	660	912	1204	1756	2239	2533	3172
	kip	5.6	8.1	15.5	24.3	35.1	56	81.8	112.6	148.4	205	270.7	394.7	503.3	569.4	713.1
Min. Break Load	kN	33	48	91	143	207	330	483	665	875	1209	1596	2329	2969	3358	4206
	kip	7.4	10.8	20.5	32.1	46.5	74.2	108.6	149.5	196.7	271.8	358.8	523.6	667.4	754.9	945.5
Design Resistance to EC3	kN	24	35	66	103	149	238	348	479	630	870	1149	1677	2138	2418	3029
	kip	5.4	7.87	14.84	25.16	33.5	53.5	78.23	107.7	141.63	195.58	258.31	377	480.64	543.59	680.95
Nominal Bar Weight	(kg/m)	0.5	0.75	1.4	2.2	3	4.8	7.1	9.4	12.5	16.7	22.2	32	41.5	46.7	58
	(lb/ft)	0.34	0.5	0.94	1.48	2.02	3.23	4.77	6.32	8.4	11.22	14.92	21.5	27.89	31.38	38.97

Macalloy 460 in Application

Engineers all over the world have used Macalloy systems in the most diverse of applications. Among these are bridges, government buildings, stadia, airports, and hotels, to name just a few. The longevity and design again reflect the level of innovation and quality, which have become firm components of Macalloy products.

Macalloy 460 Carbon Bars

Macalloy 460 is a manufactured carbon steel, with excellent mechanical properties. The thread is rolled, rather than cut. This gives rise to the use of smaller diameter bars for a given metric thread, resulting in material cost saving. The carbon Macalloy 460 is also a weldable steel with a maximum carbon equivalent of 0.55%. Arc welding may be carried out using standard techniques and low hydrogen rods.

The Macalloy 460 bar has the following mechanical properties:

Minimum Yield Stress	460 N/mm²
Minimum Breaking Stress	610 N/mm²
Minimum Elongation	19%
Minimum charpy Impact Value	27J @ -20°C
Young's Modulus	205 kN/mm²

Minimum Yield Stress	66,700 psi
Minimum Breaking Stress	88,400 psi
Minimum Elongation	19%
Minimum charpy Impact Value	20 ft-lb @ 4°F
Young's Modulus	29,700 ksi

The standard diameter range for this system is from M10 (3/8") to M100 (4"). In addition, other diameters can be supplied but are subject to longer lead times. Tendons up to and including M16 (5/8") diameters can be supplied in lengths of 6m (19'8"). For larger diameters, lengths of up to 11.95m (39'2") are available. Greater lengths are possible using couplers and turnbuckles. These fittings are designed to take the full load of the bar.

Adjustment

Adjustments within each fork or spade are:
M10 to M56: +/- ½ thread diameter
M64 to M100: +/-25mm / 1"

Turnbuckles give additional adjustments of:
M10 to M24: +/-25mm / 1"
M30 to M100: +/-50mm / 2"

Special turnbuckles, with a greater adjustment, are available on request.

Fatigue

Threads are rolled on to the bar and are therefore more resistant to fatigue. Testing a range of diameters has been carried out over 2 million cycles, the results of which are available from the Macalloy technical department.

Corrosion Protection

Macalloy tendons can be supplied in plain carbon steel, primed, or hot dip galvanized finish. If requested at the time of order, hot dip galvanizing can be applied to tendons after the threading process. The threads are then brushed to remove any excess zinc.

Length permitting, galvanized bars are delivered pre-assembled. This procedure ensures that threads are 100% operational. Connected bars, greater than 11.95m (39' 2"), are delivered part assembled. Please note that hot dip galvanizing is not comparable with a paint finish. The visual appearance of forks and spades may differ in appearance from that of the bar, by virtue of the different material compositions.

Paint

For architectural purposes, it is recommended a painted finish is applied to the galvanizing. The corrosion resistance of the bar can then be enhanced.

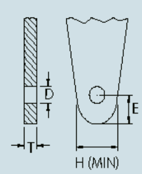
Macalloy offers any kind of paint finish (primer, paint or fire protection) for hot dip galvanized, or self color tendons. These finishes will be sourced from certified suppliers.

European Approval

The Macalloy 460 system has European CE approval under the ETA number 07/0215 for all standard diameters from M10 (3/8") and M100 (4"). When specifying, always ask for CE approved systems.

Table 2 - Macalloy 460 Gusset Plate Dimensions

Thread	mm	M10	M12	M16	M20	M24	M30	M36	M42	M48	M56	M64	M76	M85	M90	M100
	inch	3/8	1/2	5/8	3/4	1	1 1/4	1 3/8	1 5/8	2	2 1/4	2 1/2	3	3 3/8	3 1/2	4
T (Thickness)	mm	10	10	12	15	20	22	30	35	40	45	55	70	70	80	85
	inch	0.39	0.39	0.47	0.59	0.79	0.87	1.18	1.38	1.57	1.77	2.17	2.76	2.76	3.15	3.35
D	mm	11.5	13	17	21.5	25.5	31.5	37.5	43.5	49.5	57.5	65.5	78.5	91.5	96.5	111.5
	inch	0.45	0.51	0.67	0.85	1	1.24	1.48	1.71	1.95	2.26	2.58	3.09	3.6	3.8	4.39
E	mm	18	22	30	37	43	56 2.2	64	74	84	101	112	132	160	166	196
	inch	0.71	0.87	1.18	1.46	1.69		2.52	2.91	3.31	3.98	4.41	5.2	6.3	6.54	7.72
H (min)	mm	28	34	48	60	68	90	103	118	135	163	180	211	259	266	317
	inch	1.1	1.34	1.89	2.36	2.68	3.54	4.06	4.65	5.31	6.42	7.09	8.31	10.2	10.47	12.48

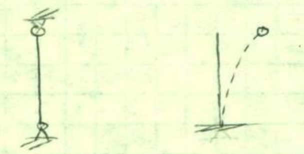


EXPERIENCE INNOVATION QUALITY

Steel Square HSS Sizing

QUEEN POST - STEEL POST DESIGN

assuming $l_e = 28" \times 1$ or $\times 2.10$ = 28" or 58.8"



$K_e = 1.0$ $\quad$ $K_e = 2.10$

let's use the more conservative value, assuming the cable offers a "free" constraint.

$l_e =$ 58.8" or 4.9'

CHOOSE 3-½" × 3-½" × 3/8" SQUARE HSS

l_e	ASD P (kips)
4	102
4.9	97.2
5	96.7

→ INTERPOLATED VALUE (pg. 4-64 OF STL. MANUAL)

$97.2^K > 42.5^K$ MAX. AXIAL FORCE

✓ GOOD

$\therefore$ USE 3-½ × 3-½" × 3/8" SQUARE HSS

FOR EACH POST

Deflection Check

DEFLECTION CHECK - TYP. OFFICE BAY

TYP. OFFICE - BEAM

$$\frac{5wl^4}{384EI} = \frac{5(1.97)(25^4)}{384(2.1E6)(6292)} \times 12^3 = 0.0011 < \frac{l}{600} = \frac{25\times12}{600} = 0.5''$$

30F-2.1ESF   10-1/2" x 19-1/4"

✓ GOOD

TYP OFFICE - QP

$$\frac{5wl^4}{384EI} = \frac{5(6.04087)(29^4)}{384(1.9E6)(5053)} \times 12^3 = 0.000067'' \text{ due to selfweight}$$

TYPE 50SP   8 1/2" x 19-1/4"

$$\frac{Pl^3}{28EI} = \frac{42.5(29^4)}{28(19e6)(5053)} \times 12^3 = 0.193'' \text{ due to point loads}$$

TWO POINT loads

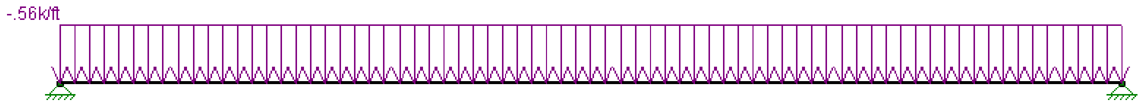
$$0.193067'' < \frac{l}{600} = \frac{29\times12}{600} = 0.58''$$

✓ GOOD

APPENDIX B.4 - ROOF BEAM DESIGN

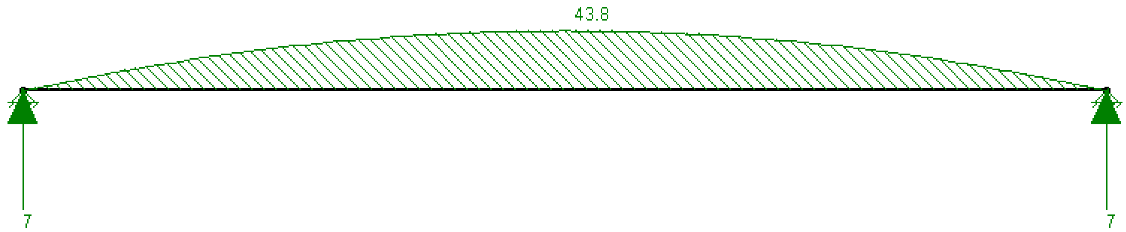
Loading

Computer analysis loading



Flexure and Reactions

Computer analysis results, showing the maximum moment is 43.8 kip-ft, or 44 kip-ft



Computer Analysis Data

Designer : **SIKANDAR PORTER-GILL**

BEAM ANALYSIS

February 11, 2014
12:18 PM
Checked By:_____

Member Data

Member Label	I Joint	J Joint	Area in^2	Moment of Inertia in^4	Elastic Modulus ksi	End Releases I-End	End Releases J-End	Length ft
M1	N1	N2	10	100	29000			25

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-.56	-.56	0	0

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	0	7	0
N2	0	7	0
Totals:	0	14	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	7	0
	2	0	3.5	32.812
	3	0	0	43.75
	4	0	-3.5	32.813
	5	0	-7	0

Member Sizing

ROOF BEAM DESIGN

LOADS

- SUPERIMPOSED DL

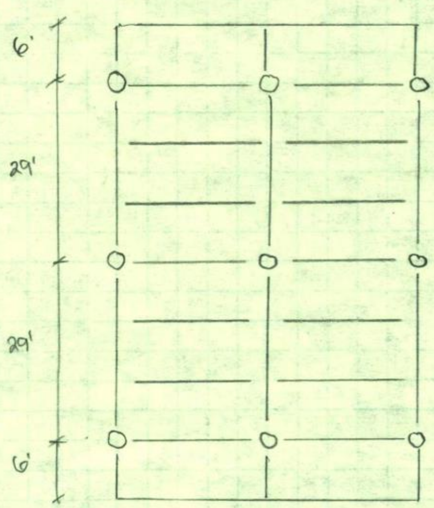
0.29	ROOF MEMBRANE
6	4" RIGID INSULATION
7.3	T&G WOOD DECK
9	3" WOOD NAILER
4	MECH.
4	LTG.
3	SPRINKLER
1	MISC SDL

34.59 PSF ~ 36 PSF DL

- LIVE LOADS

20 PSF ROOF (PER DRAWINGS) > 10 PSF ASCE-10 SNOW FOR ARKANSAS

ASD → LL + DL = 36 + 20 = 56 PSF



FRAME IN STEEL

56 PSF x (10') = 560 PLF

APPROX. TRIB WIDTH

$$M = \frac{0.56(25')^2}{8} = 43.75 \text{ kip ft}$$

USING ESTABLISHED EXCEL DESIGN TABLE,
USE 8-1/2" x 12-3/8" BEAM
GROUP 30F-2.1E SP

$$R = \frac{0.56(25')}{2} = 7^{\text{KIPS}} \times 2 \longrightarrow 14^{\text{KIPS}}$$

APPLIED TO
QP GIRDER

Flexure in Beam - Roof

Moment 44 kip-ft

$F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_V \times C_{fu} \times C_c \times C_i$

Pick a size,

8-1/2" x 12-3/8""

8.5 x 12.375

where the $A_{provided}$ = 105.2 in^2

$S_{sect\ modulus}$ = 216.9 in^3

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_L =	0.987	calculated below	§5.3.5
C_V =	0.965	calculated below	§5.3.6
C_{fu} =	1.00	because not loaded parellel to wide faces of lamin.	§5.3.7
C_c =	1.00	because no curvature to beam	§5.3.8
C_i =	1.00	because no tapering of beam	§5.3.9

Pick a Visually Graded Southern Pine Stress Group — Table 5A

Group = 30F-2.1E SP

F_b = 3000 psi

Emin = 1110000 psi

Calculate C_L Adjustment Factor

l_u = 25.00 ft, the unbraced length of the girder

d = 12.375 in, choosen to be consistent with girder depth

l_u / d = 24.24

so now we can calculate l_e,

l_e = 552 in, or 46.00 ft

reliant on inequality on page 16, Supplem

R_B = 9.72

$$F_{bE} = \frac{1.20E'_{min}}{(R_B)^2} = 14088.3$$

$F_b{*} = F_b \times C_D \times C_M \times C_t \times C_c \times C_i =$ 3000 psi

$F_{bE} / F_b{*} =$ 4.70

$$C_L = \frac{1 + {F_{bE}}/{F_b *}}{1.9} - \sqrt{\left(\frac{1 + {F_{bE}}/{F_b *}}{1.9}\right)^2 - \frac{{F_{bE}}/{F_b *}}{0.95}} = 0.987$$

<u>Calculate C_V Adjustment Factor</u>

$$C_V = \left(\frac{21}{L}\right)^{1/x} \left(\frac{12}{d}\right)^{1/x} \left(\frac{5.125}{b}\right)^{1/x} \leq 1.0$$

L =	25	ft	x =	20	for Southern Pine
d =	12.375	in			
b =	8.5	in			

$C_V =$ 0.965 < 1.0

<u>Calculate F_b' Using the Minimum of C_V or C_L</u>

min | $C_L =$ 0.987
$C_V =$ 0.965

$F'_b =$ 2895 psi

$f_b = \frac{M}{S} =$ 2434.3 psi < F'_b

<u>Calculate f_b and Determine if Selected Beam Passes</u>

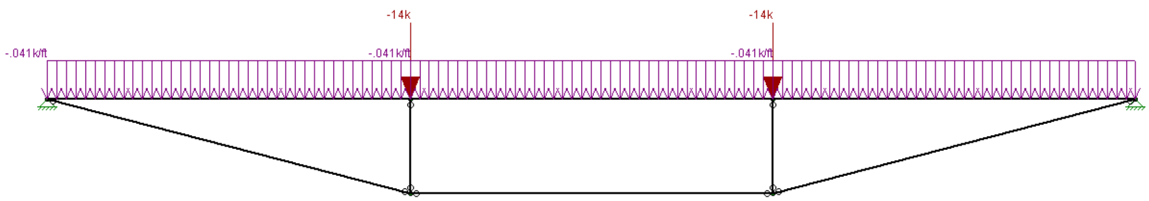
$f_b =$ 2434 psi < $F_b' =$ 2895

Bending Passes
Use a 8-1/2" x 12-3/8"" for the beam

APPENDIX B.5 - ROOF QUEEN POST DESIGN

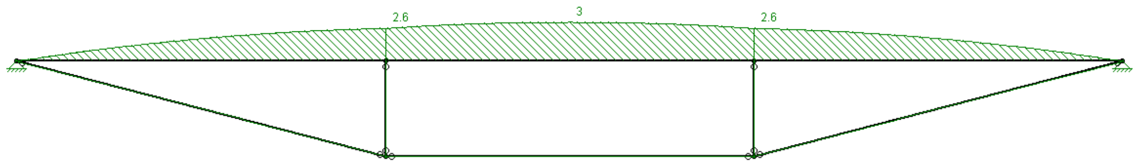
Loading

Computer analysis loading



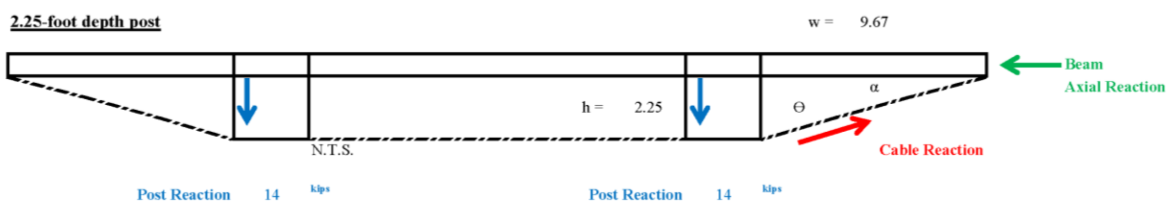
Flexure and Reactions

Computer analysis results, showing the maximum moment is 3 kip-ft, or 3.1 kip-ft



Axial Cable and Girder Forces

The assumption of the hinged queen post was used to determine the post reactions, cable tension and girder axial forces.



PRELIMINARY CALCULATIONS

$\Theta = \tan^{-1}(w/h) =$ 1.34 radians

$\alpha =$ 88.66 radians

CALCULATE RESULTANT FORCES IN CABLE AND BEAM

Cable Reaction 61.78 kips

Beam Axial Reaction 60.17 kips

Computer Analysis Data

Designer : **SIKANDAR PORTER-GILL** **GIRDER ANALYSIS** February 11, 2014 12:04 PM Checked By:_____

Member Data

Member Label	I Joint	J Joint	Area in^2	Moment of Inertia in^4	Elastic Modulus ksi	End Releases I-End	End Releases J-End	Length ft
M1	N1	N2	10	100	29000			9.67
M2	N2	N3	10	100	29000			9.66
M3	N3	N4	10	100	29000			9.67
M4	N1	N5	10	100	29000	PIN	PIN	9.988
M5	N5	N6	10	100	29000	PIN	PIN	9.66
M6	N6	N4	10	100	29000	PIN	PIN	9.988
M7	N2	N5	10	100	29000	PIN	PIN	2.5
M8	N3	N6	10	100	29000	PIN	PIN	2.5

Joint Loads/Enforced Displacements

Joint Label	[L]oad or [D]isplacement	Direction	Magnitude (k, k-ft, in, rad)
N2	L	Y	-14
N3	L	Y	-14

Member Distributed Loads

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M1	Y	-.041	-.041	0	0
M2	Y	-.041	-.041	0	0
M3	Y	-.041	-.041	0	0

Reactions

Joint Label	X Force (k)	Y Force (k)	Moment (k-ft)
N1	-54.658	14.595	0
N4	54.658	14.594	0
Totals:	0	29.189	

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
M1	1	0	.464	0
	2	0	.365	1.001
	3	0	.266	1.763
	4	0	.166	2.285
	5	0	.067	2.568
M2	1	0	.198	2.568
	2	0	.099	2.927
	3	0	0	3.046
	4	0	-.099	2.927
	5	0	-.198	2.568
M3	1	0	-.067	2.568
	2	0	-.166	2.285
	3	0	-.266	1.763
	4	0	-.365	1.001
	5	0	-.464	0
M4	1	-56.455	0	0
	2	-56.455	0	0
	3	-56.455	0	0

Designer : **SIKANDAR PORTER-GILL** **GIRDER ANALYSIS**
February 11, 2014
12:04 PM
Checked By:_____

Member Section Forces

Member Label	Section	Axial (k)	Shear (k)	Moment (k-ft)
	4	-56.455	0	0
	5	-56.455	0	0
M5	1	-54.658	0	0
	2	-54.658	0	0
	3	-54.658	0	0
	4	-54.658	0	0
	5	-54.658	0	0
M6	1	-56.455	0	0
	2	-56.455	0	0
	3	-56.455	0	0
	4	-56.455	0	0
	5	-56.455	0	0
M7	1	14.131	0	0
	2	14.131	0	0
	3	14.131	0	0
	4	14.131	0	0
	5	14.131	0	0
M8	1	14.131	0	0
	2	14.131	0	0
	3	14.131	0	0
	4	14.131	0	0
	5	14.131	0	0

Top Chord Member Sizing

Compression Parallel to Beam Grain

Axial Compression **60.17** kips

$F'_c = F_c \times C_D \times C_M \times C_t \times C_p$

Adjustment Factors

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_p =	0.92	assumed value	§3.7.1

Pick a Visually Graded Southern Pine Stress Group — Table 5B

Group =	50	
F_c =	2300	psi
Emin =	1000000	psi

so,

F'_c = 2116 psi — allowable compression stress

now the required area would be,

A = 28 in^2 — required area of glulam

Pick a,

8-1/2" x 12-3/8"

8.5 x 12.375

where the $A_{provided}$ = 105.2 in^2

Is the area greater than required area? Yes

Check the Assumption of the C_p Adjustment Factor

$$C_p = \frac{1+ {F_{CE}}/{F_c*}}{2c} - \sqrt{\left(\frac{1+ {F_{CE}}/{F_c*}}{2c}\right)^2 - \frac{{F_{CE}}/{F_c*}}{c}}$$

$F_C* = F_c \times C_D \times C_M \times C_t =$ 2300 psi

$l_e / d =$ 13.65 and 9.37 *where* *13.65* *controls*

< 50 < 50

$E_{min}' = E_{min} \times C_M \times C_t =$ 1000000 psi

$$F_{CE} = \frac{0.822 E'_{min}}{\left({l_e}/{d}\right)^2} = 4414 \text{ psi}$$

$F_{CE} / F_C* =$ 1.92 c = 0.9

now the C_P adjustment factor can be calculated

$C_P =$ 0.92 < $C_{P,asummed}$

Calculate f_c and Determine if Selected Beam Passes

$f_c =$ 572 psi < $F_C' =$ 2116

Compression Parallel to Grain Passes

Moment Induced by Self-Weight of Member

G = 0.55 Table 5B

M.C. = 5 % or 10 %
because interior beam in conditioned space

$D = 62.4\left(\frac{G}{1+G(0.009)(M.C.)}\right)\left(1+\frac{M.C.}{100}\right)$ = 35.17 pcf, or 35.97 pcf

we will take the maximum,
D = 35.97 pcf

we have a 8-1/2" x 12-3/8" *glulam beam with,*
A = 105.2 in^2

convert to square feet,
A = 0.7305 ft^2

now calculate the linear load created by its self weight, over a 29' span
w = 26.28 plf > 0.041 klf assumed in maximum moment calculation

Flexure in Queen Post Girder - Roof

Moment 3.1 kip-ft

$F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_V \times C_{fu} \times C_c \times C_i$

Adjustment Factors

C_D =	1.00	because live load controls	§2.3.2
C_M =	1.00	because interior beam in conditioned space	§5.3.3
C_t =	1.00	because interior beam in conditioned space	§5.3.4
C_L =	0.996	calculated below	§5.3.5
C_V =	0.958	calculated below	§5.3.6
C_{fu} =	1.00	because not loaded parellel to wide faces of lamin.	§5.3.7
C_c =	1.00	because no curvature to beam	§5.3.8
C_i =	1.00	because no tapering of beam	§5.3.9

Pick a Visually Graded Southern Pine Stress Group — Table 5B

Group = 50

F_b = 2100 psi

Emin = 1000000 psi

Calculate C_L Adjustment Factor

l_u = 9.67 ft, the unbraced length of the girder

d = 12.375 in, depth choosen in compression parellel to grain calculation

l_u / d = 9.37

so now we can calculate l_e,

l_e = 226.205 in, or 18.85 ft

R_B = 6.22

$$F_{bE} = \frac{1.20 E'_{min}}{(R_B)^2} = 30972.2$$

$F_b^* = F_b \times C_D \times C_M \times C_t \times C_c \times C_i$ = 2100 psi

F_{bE} / F_b^* = 14.75

$$C_L = \frac{1 + {F_{bE}}/{F_b *}}{1.9} - \sqrt{\left(\frac{1 + {F_{bE}}/{F_b *}}{1.9}\right)^2 - \frac{{F_{bE}}/{F_b *}}{0.95}} = 0.996$$

Calculate C_V Adjustment Factor

$$C_V = \left(\frac{21}{L}\right)^{1/x}\left(\frac{12}{d}\right)^{1/x}\left(\frac{5.125}{b}\right)^{1/x} \leq 1.0$$

L =	29	ft	x =	20	for Southern Pine
d =	12.375	in			
b =	8.5	in			

C_V = 0.958 < 1.0

Calculate F_b' Using the Minimum of C_V or C_L

min | C_L = 0.996
| C_V = 0.958

Section Modulus (x) = 267.8 in^3

F'_b = 2012 psi

$f_b = \frac{M}{S} =$ 138.9 psi < F'_b

Calculate f_b and Determine if Selected Beam Passes

f_b = 139 psi < F_b' = 2012

Bending Passes

Combined Axial and Bending Loading Interaction

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left(1 + {f_c}/{F_{CE1}}\right)} \leq 1.0$$

§3.9.2

f_c =	572	psi	E_{min}' =	1000000	psi
F_c' =	2116	psi			
f_{b1} =	139	psi			
F_{b1} =	2012	psi			

$$F_{CE1} = \frac{0.822 E_{min}'}{\left({l_{e1}}/{d_1}\right)^2} = 9348.6 \text{ psi}$$

where,

l_{e1} = 9.67 ft

d_1 = 12.375 in

$f_c < F_{CE1}$	True

0.073 + 0.065 = 0.138 < 1.0

Combined Axial and Bending Pass

Use a,

8-1/2" x 12-3/8"

for the glulam queen post

With a,

Southern Pine Group of 50

Member Summary, Tension Cable and Steel Square HSS Sizing

ROOF QUEEN POST DESIGN

CONSERVATIVELY ASSUME 0.041 K/FT SELF WEIGHT
↳ REFERENCING TYP. FLOOR QP

FROM COMPUTER ANALYSIS: M_{max} = 1.615 K-ft → 1.7 K-ft
qp
girder

AN EXCEL TABLE WILL BE USED TO CALCULATE
THE SUFFICIENT QP GIRDER SIZE...
↳ AFTER DESIGN SUMMARY

8-1/2" x 12-3/8"
GLULAM QP, GROUP 50 SP

TENSION CABLE

61.78K → 62 Kips

	Min. Yield	Min. Break
M16	69	91
M20	108	143

M16 69K > 62K TOTAL ✓GOOD

DESIGN CALLS FOR TWO CABLES
✓FOR STABILITY
✓ADDED SAFETY

(2) M16 MACALLOY 460 BARS
FOR TENSION WILL BE USED

QP POST (following previously sized post)

l_e = 58.8" or 4.9' with 14^k axial

CONSTRUCTIBILITY

USE SAME POST, 3-1/2" x 3-1/2" x 3/8" SQUARE HSS

97.2^k > 14^k MAX. AXIAL FORCE

✓ GOOD

∴ USE 3-1/2" x 3-1/2 x 3/8"
FOR EACH POST

ROOF QUEEN POST DESIGN SUMMARY

8-1/2" x 12-3/8", GLULAM QP
GROUP 50 SP

WITH (2) 460 MACALLOY BARS

WITH 3-1/2" x 3-1/2" x 3/8" POST/S

Deflection Check

DEFLECTION CHECK - ROOF BAY

BEAM

$$\frac{5(0.56)(25^4)}{384(2.166)(1342)} \times 12^3 = 0.0017 < \frac{l}{600} = \frac{25' \times 12}{600} = 0.5''$$

8 1/2 x 12 3/8

30F.-2.1E SP

✓ GOOD

QUEEN POST

$$\frac{5(0.04087)(29^4)}{384(1.986)(1342)} \times 12^3 = 0.00044$$

TYPE 50

8 1/2 x 12 3/8

$$\frac{14(29^4)}{28(1.986)(1342)} \times 12^3 = 0.23$$

$$= 0.2304'' < \frac{l}{600} = 0.58''$$

✓ GOOD

APPENDIX B.6 - SUMMARY OF BEAM SIZES

- The typical office beam will be specified as a 10 ½" x 19 ¼" 30F-2.1E Southern Pine.
- The typical office queen post will be specified as an 8 ½" x 19 ¼" Stress Class 50 Visual Southern Pine, with 3 ½" x 3 ½" x ⅜" Square HSS Post and (2) M56 Macalloy 460 Bars
- The typical roof beam will be specified as a 8 ½" x 12 ⅜" 30F-2.1E Southern Pine.
- The typical roof queen post will be specified as a 8 ½" x 12 ⅜" Stress Glass 50 Visual Southern Pine, with 3 ½" x 3 ½" x ⅜" Square HSS Post and (2) M16 Macalloy 460 Bars

APPENDIX B.7 - TYPICAL OFFICE PERIMETER BEAM

TYPICAL OFFICE - PERIMETER BEAM

FROM PREV. TECH REPORT II: 280 PLF ON EDGE

TRIB. = $\frac{9.67'}{2}$ + 2' = 6.84' ~ 7'-0" x 164.5 PSF OFFICE

edge

= 1151.5 PLF ~ 1.2 KLF

+ 0.28 KLF

1.48 KLF ~ 1.5 KLF

$M = \frac{1.5(25^2)}{8} = 117.18^{\text{kft}} \sim 117.2^{\text{'k}}$

$R = \frac{1.5(25)}{2} = 18.75^{\text{k}}$

GLULAM:

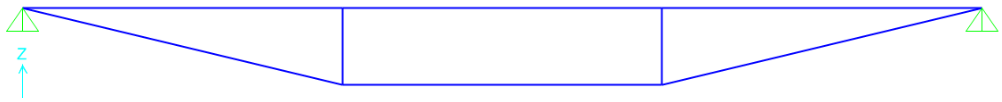
SIZE 10-1/2" x 17-7/8", 30F-2.1E SP d = 19.25"

STEEL SIZE W14X22, SAME AS ACTUAL DESIGN d = 13.7"
(STL MANUAL, pg. 3-26)

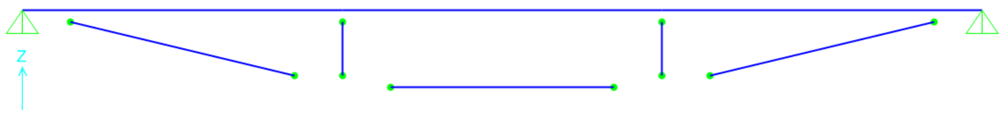
* IT SHOULD BE NOTED THAT THE USE OF
STEEL PERIMETER BEAMS WILL CHANGE
IBC 2009 CONSTRUCTION TYPE.

APPENDIX B.8 - SAP2000 QUEEN POST MODEL

Original Model



Member Releases



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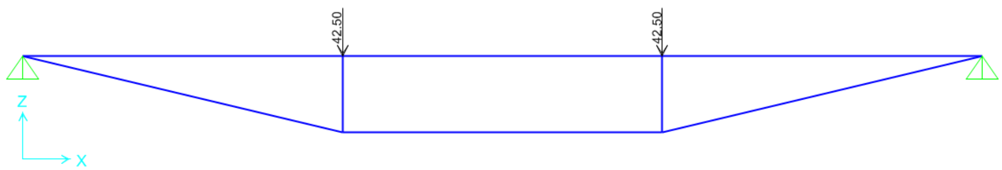
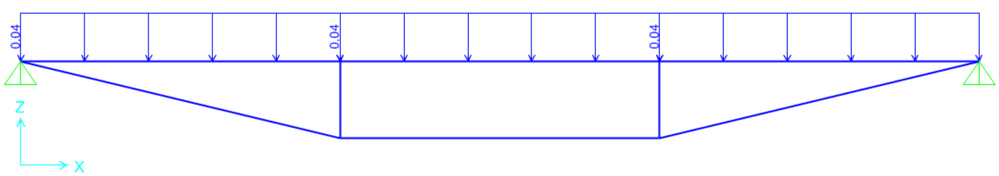
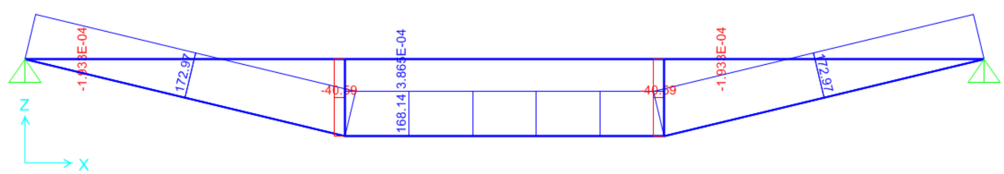




Axial Loading



Member	Force	Percent Error (from actual)
Cable	172.97	7.1%
Cable	168.141	9.7%
Cable	172.97	7.1%
Post	-40.586	3.4%
Post	-40.586	3.4%

APPENDIX B.9 – COLUMN SIZING

COLUMN CONFIRMATION

• FOR EASE OF CONNECTION AND AESTHETICS, THE SAME COLUMNS WILL BE USED FROM EXISTING BUILDING DESIGN

→ 24" DIAMETER, MIN THICKNESS = 0.438" @ 14'-0" effective

PERIMETER BEAM 18.75^k × 2/ FLOOR

QUEEN POST (OFF) 43^k

QUEEN POST (ROOF) 14.5^k

CONSERVATIVE VALUE ASSUMED

$$P_u = 3 \times (18.75(2) + 43) + 1 \times (14.5 + 18.75(2))$$

$$= 293.5^k < ASD_{axial} = 691^k$$

@ 14'

for HSS20x20x0.5

(STL. MANUAL 4-68)

HSS 24x24x0.5 > HSS 20x20x0.5 ✓ GOOD

APPENDIX C

REDESIGN OF LATERAL SYSTEM

APPENDIX C.1 – HSS24X0.5 COLUMN

The Master Steel Table for RAM SS was modified to account for the larger HSS24x0.5 used in the Heifer International Center (American Institute of Steel Construction, 2011).

RAMAISC.TAB - Notepad

File Edit Format View Help

HSS4X2X3/16	R	4	0.174	2	0.174	1.89	3.66	1.83	2.34	1.22	1.22	1.43	3.08
HSS4X2X1/8	R	4	0.116	2	0.116	1.30	2.65	1.32	1.66	0.898	0.898	1.02	2.20
HSS3.5X3.5X3/8	R	3.5	0.349	3.5	0.349	4.09	6.49	3.71	4.69	6.49	3.71	4.69	11.2
HSS3.5X3.5X5/16	R	3.5	0.291	3.5	0.291	3.52	5.84	3.34	4.14	5.84	3.34	4.14	9.89
HSS3.5X3.5X1/4	R	3.5	0.233	3.5	0.233	2.91	5.04	2.88	3.50	5.04	2.88	3.50	8.35
HSS3.5X3.5X3/16	R	3.5	0.174	3.5	0.174	2.24	4.05	2.31	2.76	4.05	2.31	2.76	6.56
HSS3.5X3.5X1/8	R	3.5	0.116	3.5	0.116	1.54	2.90	1.66	1.93	2.90	1.66	1.93	4.58
HSS3.5X2X1/4	R	3.5	0.233	2	0.233	2.21	3.17	1.81	2.36	1.30	1.30	1.58	3.16
HSS3.5X2X3/16	R	3.5	0.174	2	0.174	1.71	2.61	1.49	1.89	1.08	1.08	1.27	2.55
HSS3.5X2X1/8	R	3.5	0.116	2	0.116	1.19	1.90	1.09	1.34	0.795	0.795	0.912	1.83
HSS3.5X1.5X1/4	R	3.5	0.233	1.5	0.233	1.97	2.55	1.46	1.98	0.638	0.851	1.06	1.79
HSS3.5X1.5X3/16	R	3.5	0.174	1.5	0.174	1.54	2.12	1.21	1.60	0.544	0.725	0.867	1.49
HSS3.5X1.5X1/8	R	3.5	0.116	1.5	0.116	1.07	1.57	0.896	1.15	0.411	0.548	0.630	1.09
HSS3X3X3/8	R	3	0.349	3	0.349	3.39	3.78	2.52	3.25	3.78	2.52	3.25	6.64
HSS3X3X5/16	R	3	0.291	3	0.291	2.94	3.45	2.30	2.90	3.45	2.30	2.90	5.94
HSS3X3X1/4	R	3	0.233	3	0.233	2.44	3.02	2.01	2.48	3.02	2.01	2.48	5.08
HSS3X3X3/16	R	3	0.174	3	0.174	1.89	2.46	1.64	1.97	2.46	1.64	1.97	4.03
HSS3X3X1/8	R	3	0.116	3	0.116	1.30	1.78	1.19	1.40	1.78	1.19	1.40	2.84
HSS3X2.5X5/16	R	3	0.291	2.5	0.291	2.64	2.92	1.94	2.51	2.18	1.74	2.20	4.34
HSS3X2.5X1/4	R	3	0.233	2.5	0.233	2.21	2.57	1.72	2.16	1.93	1.54	1.90	3.74
HSS3X2.5X3/16	R	3	0.174	2.5	0.174	1.71	2.11	1.41	1.73	1.59	1.27	1.52	3.00
HSS3X2.5X1/8	R	3	0.116	2.5	0.116	1.19	1.54	1.03	1.23	1.16	0.931	1.09	2.13
HSS3X2X5/16	R	3	0.291	2	0.291	2.35	2.38	1.59	2.11	1.24	1.24	1.58	2.87
HSS3X2X1/4	R	3	0.233	2	0.233	1.97	2.13	1.42	1.83	1.11	1.11	1.38	2.52
HSS3X2X3/16	R	3	0.174	2	0.174	1.54	1.77	1.18	1.48	0.932	0.932	1.12	2.05
HSS3X2X1/8	R	3	0.116	2	0.116	1.07	1.30	0.867	1.06	0.692	0.692	0.803	1.47
HSS2.5X2.5X5/16	R	2.5	0.291	2.5	0.291	2.35	1.82	1.46	1.88	1.82	1.46	1.88	3.20
HSS2.5X2.5X1/4	R	2.5	0.233	2.5	0.233	1.97	1.63	1.30	1.63	1.63	1.30	1.63	2.79
HSS2.5X2.5X3/16	R	2.5	0.174	2.5	0.174	1.54	1.35	1.08	1.32	1.35	1.08	1.32	2.25
HSS2.5X2.5X1/8	R	2.5	0.116	2.5	0.116	1.07	0.998	0.799	0.947	0.998	0.799	0.947	1.61
HSS2.5X2X1/4	R	2.5	0.233	2	0.233	1.74	1.33	1.06	1.37	0.930	0.930	1.17	1.90
HSS2.5X2X3/16	R	2.5	0.174	2	0.174	1.37	1.12	0.894	1.12	0.786	0.786	0.956	1.55
HSS2.5X2X1/8	R	2.5	0.116	2	0.116	0.956	0.833	0.667	0.809	0.589	0.589	0.694	1.12
HSS2.5X1.5X1/4	R	2.5	0.233	1.5	0.233	1.51	1.03	0.822	1.11	0.449	0.599	0.764	1.10
HSS2.5X1.5X3/16	R	2.5	0.174	1.5	0.174	1.19	0.882	0.705	0.915	0.390	0.520	0.636	0.929
HSS2.5X1.5X1/8	R	2.5	0.116	1.5	0.116	0.840	0.668	0.535	0.671	0.300	0.399	0.469	0.687
HSS2.5X1X3/16	R	2.5	0.174	1	0.174	1.02	0.646	0.517	0.713	0.143	0.285	0.360	0.412
HSS2.5X1X1/8	R	2.5	0.116	1	0.116	0.724	0.503	0.403	0.532	0.115	0.230	0.274	0.322
HSS2X2X1/4	R	2	0.233	2	0.233	1.51	0.747	0.747	0.964	0.747	0.747	0.964	1.31
HSS2X2X3/16	R	2	0.174	2	0.174	1.19	0.641	0.641	0.797	0.641	0.641	0.797	1.09
HSS2X2X1/8	R	2	0.116	2	0.116	0.840	0.486	0.486	0.584	0.486	0.486	0.584	0.796

PIPE

HSS24.000X0.500	R	24.000	0.465	34.400	2381.400	198.400	257.600
HSS20.000X0.500	R	20.000	0.465	28.500	1360.000	136.000	177.000
HSS20.000X0.375	R	20.000	0.349	21.500	1040.000	104.000	135.000
HSS18.000X0.500	R	18.000	0.465	25.600	985.000	109.000	143.000
HSS18.000X0.375	R	18.000	0.349	19.400	754.000	83.800	109.000
HSS16.000X0.625	R	16.000	0.581	28.100	838.000	105.000	138.000
HSS16.000X0.500	R	16.000	0.465	22.700	685.000	85.700	112.000
HSS16.000X0.438	R	16.000	0.407	19.900	606.000	75.800	99.000
HSS16.000X0.375	R	16.000	0.349	17.200	526.000	65.700	85.500
HSS16.000X0.312	R	16.000	0.291	14.400	443.000	55.400	71.800
HSS16.000X0.250	R	16.000	0.233	11.500	359.000	44.800	57.900
HSS14.000X0.625	R	14.000	0.581	24.500	552.000	78.900	105.000
HSS14.000X0.500	R	14.000	0.465	19.800	453.000	64.800	85.200
HSS14.000X0.375	R	14.000	0.349	15.000	349.000	49.800	65.100
HSS14.000X0.312	R	14.000	0.291	12.500	295.000	42.100	54.700
HSS14.000X0.250	R	14.000	0.233	10.100	239.000	34.100	44.200
HSS12.750X0.500	R	12.750	0.465	17.900	339.000	53.200	70.200
HSS12.750X0.375	R	12.750	0.349	13.600	262.000	41.000	53.700
HSS12.750X0.250	R	12.750	0.233	9.160	180.000	28.200	36.500
HSS10.750X0.500	R	10.750	0.465	15.000	199.000	37.000	49.200
HSS10.750X0.375	R	10.750	0.349	11.400	154.000	28.700	37.800
HSS10.750X0.250	R	10.750	0.233	7.700	106.000	19.800	25.800
HSS10.000X0.625	R	10.000	0.581	17.200	191.000	38.300	51.600
HSS10.000X0.500	R	10.000	0.465	13.900	159.000	31.700	42.300
HSS10.000X0.375	R	10.000	0.349	10.600	123.000	24.700	32.500
HSS10.000X0.312	R	10.000	0.291	8.880	105.000	20.900	27.400
HSS10.000X0.250	R	10.000	0.233	7.150	85.300	17.100	22.200
HSS10.000X0.188	R	10.000	0.174	5.370	64.800	13.000	16.800
HSS9.625X0.500	R	9.625	0.465	13.400	141.000	29.200	39.000
HSS9.625X0.375	R	9.625	0.349	10.200	110.000	22.800	30.000

APPENDIX C.2 – SEISMIC AND WIND LOADING

Seismic ASCE 7-10

General Programming Input

Risk Category II

For ordinary reinforced concrete shear walls, Classification 1.2 of §12.2-1

$C_d = 4.0$

$R = 4.0$

Please review the Summary and Detailed Report on the next page for the following values (U.S. Geological Survey, 2013):

$S_S = 0.410g$

$S_1 = 0.165g$

$TL = 12\ sec$

Site Class C

The Structure Period, T_a:

Value calculated by RAM SS using the Standard Equation

$C_t = 0.020$ was used for “all other structural systems” per Table 12.8-2

Orthogonal Effects Considered at 100%/30%

(American Society of Civil Engineers, ASCE-7 10, Minimum Design Loads for Buildings and Other Structures, 2010)

U.S. Geological Survey Report

USGS Design Maps Summary Report

User–Specified Input

Report Title Heifer International Center –2010
Thu February 20, 2014 17:15:04 UTC

Building Code Reference Document ASCE 7–10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.74492°N, 92.25781°W

Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

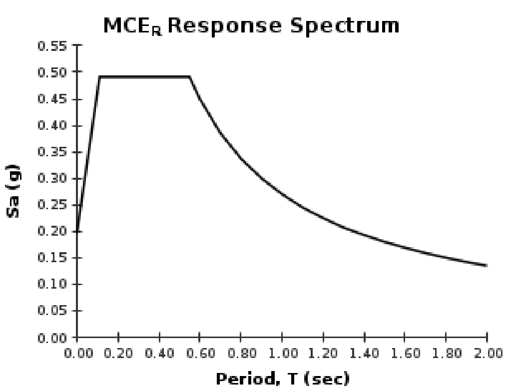
Risk Category I/II/III

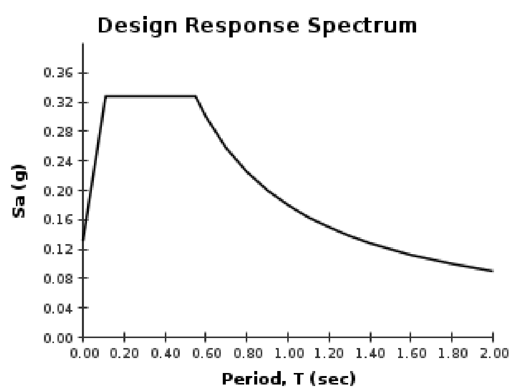
USGS–Provided Output

$S_S =$ 0.410 g $S_{MS} =$ 0.491 g $S_{DS} =$ 0.328 g

$S_1 =$ 0.165 g $S_{M1} =$ 0.270 g $S_{D1} =$ 0.180 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.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.74492°N, 92.25781°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 0.410 g
From Figure 22-2 [2]	S_1 = 0.165 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index PI > 20, • Moisture content w ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_s = 0.410 g, F_a = 1.200

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.165 g, F_v = 1.635

2/20/14 Design Maps Detailed Report

Equation (11.4–1):	$S_{MS} = F_aS_S = 1.200 \times 0.410 = 0.491$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 1.635 \times 0.165 = 0.270$ g

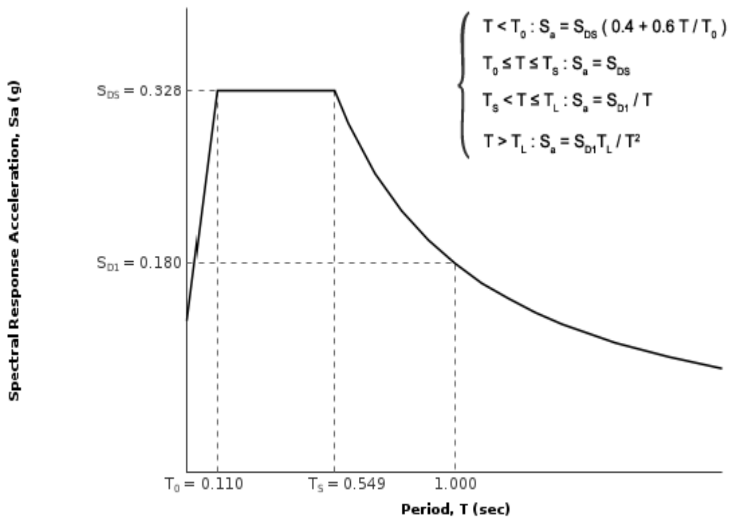
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 0.491 = 0.328$ g
Equation (11.4–4):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 0.270 = 0.180$ g

Section 11.4.5 — Design Response Spectrum

From Figure 22-12 [3]	$T_L = 12$ seconds

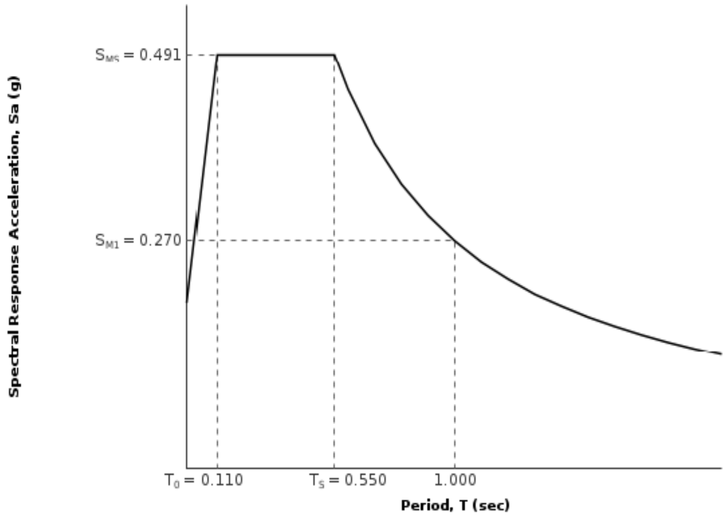
Figure 11.4–1: Design Response Spectrum



geohazards.usgs.gov/designmaps/us/report.php?template=minimal&latitude=34.7449152&longitude=-92.2578128&siteclass=2&riskcategory=0&edition=asce-2... 3/6

Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.213

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.187 \times 0.213 = 0.253$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.213 g, F_{PGA} = 1.187

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] $C_{RS} = 0.829$

From Figure 22-18 [6] $C_{R1} = 0.816$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.328 g, Seismic Design Category = B

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.180 g, Seismic Design Category = C

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = C

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

Seismic Story Drift

Seismic Story Drift - West End

C_d = 4
I = 1

West Side

X-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.3799	1.5196	14	0.6464	3.36	PASS
Story2	0.2183	0.8732	14	0.5816	3.36	PASS
Story1	0.0729	0.2916	14	0.2916	3.36	PASS

@ RAM Frame Location EX A @ (-156.198, -393.277), trace Location 1

X-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.2436	0.9744	14	0.4084	3.36	PASS
Story2	0.1415	0.566	14	0.3784	3.36	PASS
Story1	0.0469	0.1876	14	0.1876	3.36	PASS

@ RAM Frame Location EX B @ (-379.546, -319.250), trace Location 3

Y-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.4542	1.8168	14	0.7776	3.36	PASS
Story2	0.2598	1.0392	14	0.674	3.36	PASS
Story1	0.0913	0.3652	14	0.3652	3.36	PASS

@ RAM Frame Location EX A @ (-156.198, -393.277), trace Location 1

Y-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.1035	0.414	14	0.1736	3.36	PASS
Story2	0.0601	0.2404	14	0.1744	3.36	PASS
Story1	0.0165	0.066	14	0.066	3.36	PASS

@ RAM Frame Location EX B @ (-379.546, -319.250), trace Location 3

Please refer to Appendix C.6 – Trace Locations for a visual location of EX A and EX B

Seismic Story Drift - East End

C_d = 4

I = 1

East Side

X-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.2051	0.8204	14	0.3948	3.36	PASS
Story2	0.1064	0.4256	14	0.38	3.36	PASS
Story1	0.0114	0.0456	14	0.0456	3.36	PASS
@ RAM Frame Location		EX C @ (-365.149, -844.326), trace location 4				

X-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.4083	1.6332	14	0.8188	3.36	PASS
Story2	0.2036	0.8144	14	0.74	3.36	PASS
Story1	0.0186	0.0744	14	0.0744	3.36	PASS
@ RAM Frame Location		EX D @ (-556.445, -926.789), trace location 5				

Y-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	0.2524	1.0096	14	0.5116	3.36	PASS
Story2	0.1245	0.498	14	0.4592	3.36	PASS
Story1	0.0097	0.0388	14	0.0388	3.36	PASS
@ RAM Frame Location		EX C @ (-365.149, -844.326), trace location 4				

Y-direction Seismic Loading

Level	δ, Actual Displacement (in)	δ_x, Modified Displacement (in)	Story Height (ft)	Δ, Design Story Drift (in)	Δ_a, Allowable Story Drift (in)	Pass
Story3	-0.219	-0.876	14	-0.472	3.36	PASS
Story2	-0.101	-0.404	14	-0.376	3.36	PASS
Story1	-0.007	-0.028	14	-0.028	3.36	PASS
@ RAM Frame Location		EX D @ (-556.445, -926.789), trace location 5				

Please refer to Appendix C.6 – Trace Locations for a visual location of EX C and EX D

Wind ASCE 7-10

Exposure C

Mean roof height = 65’-0” (conservatively assumed)

$k_{zt} = 0$ due to no hills near building

Use calculated n for x and y for natural frequency

$V = 115\ mph$ for basic wind speed

$G = 0.85$ (conservatively assumed)

(American Society of Civil Engineers, ASCE-7 10, Minimum Design Loads for Buildings and Other Structures, 2010)

Wind Building Drift

Wind Building Drift - West End

$h_{building}$ = 65 ft

X-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.211	1.95	PASS
W2	0.067	1.95	PASS
W3	0.139	1.95	PASS
W4	0.178	1.95	PASS
W5	0.093	1.95	PASS
W6	0.007	1.95	PASS
W7	0.208	1.95	PASS
W8	0.108	1.95	PASS
W9	0.109	1.95	PASS
W10	0.203	1.95	PASS
W11	0.034	1.95	PASS
W12	0.128	1.95	PASS

Y-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.109	1.95	PASS
W2	0.346	1.95	PASS
W3	0.032	1.95	PASS
W4	0.131	1.95	PASS
W5	0.369	1.95	PASS
W6	0.149	1.95	PASS
W7	0.340	1.95	PASS
W8	-0.178	1.95	PASS
W9	0.136	1.95	PASS
W10	0.375	1.95	PASS
W11	-0.253	1.95	PASS
W12	-0.014	1.95	PASS

EX A @ (-156.198, -393.277), trace Location 1

Wind Building Drift - West End

$h_{building}$ = 65 ft

X-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.159	1.95	PASS
W2	-0.001	1.95	PASS
W3	0.126	1.95	PASS
W4	0.113	1.95	PASS
W5	-0.015	1.95	PASS
W6	0.014	1.95	PASS
W7	0.119	1.95	PASS
W8	0.120	1.95	PASS
W9	0.105	1.95	PASS
W10	0.073	1.95	PASS
W11	0.105	1.95	PASS
W12	0.074	1.95	PASS

Y-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	-0.048	1.95	PASS
W2	0.142	1.95	PASS
W3	-0.008	1.95	PASS
W4	-0.065	1.95	PASS
W5	0.043	1.95	PASS
W6	0.169	1.95	PASS
W7	0.070	1.95	PASS
W8	-0.142	1.95	PASS
W9	0.121	1.95	PASS
W10	-0.016	1.95	PASS
W11	-0.038	1.95	PASS
W12	-0.175	1.95	PASS

EX B @ (-379.546, -319.250), trace Location 3

Wind Building Drift - East End

$h_{building}$ = 65 ft

X-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.072	1.95	PASS
W2	0.009	1.95	PASS
W3	0.056	1.95	PASS
W4	0.052	1.95	PASS
W5	0.000	1.95	PASS
W6	0.013	1.95	PASS
W7	0.061	1.95	PASS
W8	0.048	1.95	PASS
W9	0.052	1.95	PASS
W10	0.039	1.95	PASS
W11	0.042	1.95	PASS
W12	0.030	1.95	PASS

Y-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.051	1.95	PASS
W2	0.059	1.95	PASS
W3	0.010	1.95	PASS
W4	0.067	1.95	PASS
W5	0.144	1.95	PASS
W6	-0.056	1.95	PASS
W7	0.083	1.95	PASS
W8	-0.006	1.95	PASS
W9	-0.035	1.95	PASS
W10	0.158	1.95	PASS
W11	-0.101	1.95	PASS
W12	0.092	1.95	PASS

EX C @ (-365.149, -844.326), trace location 4

Wind Building Drift - East End

$h_{building}$ = 65 ft

X-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	0.125	1.95	PASS
W2	-0.071	1.95	PASS
W3	0.065	1.95	PASS
W4	0.123	1.95	PASS
W5	0.048	1.95	PASS
W6	-0.155	1.95	PASS
W7	0.041	1.95	PASS
W8	0.147	1.95	PASS
W9	-0.067	1.95	PASS
W10	0.129	1.95	PASS
W11	0.013	1.95	PASS
W12	0.208	1.95	PASS

Y-direction, Wind Loading

Load Case	Total Building Displacement (in)	Maximum Building Drift Allowed (in)	Pass
W1	-0.072	1.95	PASS
W2	0.244	1.95	PASS
W3	-0.011	1.95	PASS
W4	-0.097	1.95	PASS
W5	0.032	1.95	PASS
W6	0.334	1.95	PASS
W7	0.129	1.95	PASS
W8	-0.237	1.95	PASS
W9	0.242	1.95	PASS
W10	-0.049	1.95	PASS
W11	-0.032	1.95	PASS
W12	-0.323	1.95	PASS

EX D @ (-556.445, -926.789), trace location 5

APPENDIX C.3 – TORSIONAL IRREGULARITY AND SEISMIC AMPLIFICATION FACTOR

Amplification Factor - West Side of Heifer International Center

X-direction Seismic Loading

Level	δ EX A + Ext A (in)	δ EX B + Ext B (in)	δ Average (in)	δ Maximum (in)	A_x	1.2(δ Average	1.4(δ Average	Irregularity Type 1a (Table 12.3-1)	Irregularity Type 1b (Table 12.3-1
Story3	0.380	0.244	0.312	0.380	1.03	0.374	0.436	NA	Type 1b
Story2	0.218	0.142	0.180	0.218	1.02	0.216	0.252	NA	Type 1b
Story1	0.073	0.047	0.060	0.073	1.03	0.072	0.084	NA	Type 1b
Controlling Case	E5								

EX A @ (-156.198, -393.277), trace Location 1
EX B @ (-379.546, -319.250), trace Location 3

Level	A_x	V_i (kips)	e (ft)	M_z (k-ft)	V_{apply} (kips
Story3	1.03	186.15	11.26	2162	191.97
Story2	1.02	283.64	11.26	3266	290.03
Story1	1.03	331.55	11.26	3840	341.03

Eccentrictity calculated by RAM Frame

Shear only from x-direction of case E5, conservative assumption

Amplification Factor - West Side of Heifer International Center

Y-direction Seismic Loading

Level	δ EY A + Ext A (in)	δ EY B + Ext B (in)	δ Average (in)	δ Maximum (in)	A_x	1.2(δ Average	1.4(δ Average	Irregularity Type 1a (Table 12.3-1)	Irregularity Type 1b (Table 12.3-1
Story3	0.454	0.104	0.279	0.454	1.84	0.335	0.390	NA	NA
Story2	0.260	0.060	0.160	0.260	1.83	0.192	0.224	NA	NA
Story1	0.091	0.017	0.054	0.091	1.99	0.065	0.075	NA	NA
Controlling Case	E9								

EY A @ (-156.198, -393.277), trace Location 1
EY B @ (-379.546, -319.250), trace Location 3

Level	A_x	V_i (kips)	e (ft)	M_z (k-ft)	V_{apply} (kips
Story3	1.00	185.64	7.88	1463	185.64
Story2	1.00	282.97	7.88	2230	282.97
Story1	1.00	331.21	7.88	2610	331.21

Eccentricity calculated by RAM Frame

Shear only from y-direction of case E9, conservative assumption

Amplification Factor - East Side of Heifer International Center

X-direction Seismic Loading

Level	δ EX C + Ext C (in)	δ EX D + Ext D (in)	δ Average (in)	δ Maximum (in)	A_x	1.2(δ Average	1.4(δ Average	Irregularity Type 1a (Table 12.3-1)	Irregularity Type 1b (Table 12.3-1
Story3	0.205	0.408	0.307	0.408	1.23	0.368	0.429	NA	Type 1b
Story2	0.106	0.204	0.155	0.204	1.20	0.186	0.217	NA	Type 1b
Story1	0.011	0.019	0.015	0.019	1.07	0.018	0.021	NA	Type 1b
Controlling Case	E21								

EX C @ (-365.149, -844.326), trace location 4
EX D @ (-556.445, -926.789), trace location 5

Level	A_x	V_i (kips)	e (ft)	M_z (k-ft)	V_{apply} (kips
Story3	1.23	180.16	9.87	2188	221.73
Story2	1.20	274.77	9.87	3249	329.23
Story1	1.07	325.55	9.87	3431	347.62

Eccentrictity calculated by RAM Frame

Shear only from x-direction of case E21, conservative assumption

Amplification Factor - East Side of Heifer International Center

Y-direction Seismic Loading

Level	δ EY C + Ext C (in)	δ EY D + Ext D (in)	δ Average (in)	δ Maximum (in)	A_x	1.2(δ Average	1.4(δ Average	Irregularity Type 1a (Table 12.3-1)	Irregularity Type 1b (Table 12.3-1
Story3	0.252	-0.219	0.236	0.252	1.00	1.252	0.744	NA	Type 1b
Story2	0.125	-0.101	0.113	0.125	1.00	1.125	0.619	NA	Type 1b
Story1	0.010	-0.007	0.008	0.010	1.00	1.010	0.509	NA	Type 1b
Controlling Case	E21								

EX C @ (-365.149, -844.326), trace location 4
EX D @ (-556.445, -926.789), trace location 5

Level	A_x	V_i (kips)	e (ft)	M_z (k-ft)	V_{apply} (kips
Story3	1.00	180.16	5.63	1014	180.16
Story2	1.00	274.77	5.63	1547	274.77
Story1	1.00	325.55	5.63	1833	325.55

Eccentricity calculated by RAM Frame

Shear only from y-direction of case E21, conservative assumption

APPENDIX C.4 – BUILDING OVERTURNING CHECK

Overturning Moment and Base Shear - West End

Building Effective Weight = 4022.94 kips

Wind Base Shear and Overturning Moment

Load Case	Level	Elevation (ft)	Base Shear		Overturning Moment	
			Vx (kip)	Vy (kip)	Mx (kip-ft)	My (kip-ft)
Wind X	Level 3	42	35.04	0		
	Level 2	28	67.36	0		
	Level 1	14	63.31	0	4,244.10	-
Wind Y	Level 3	42	0	53.91		
	Level 2	28	0	103.94		
	Level 1	14	0	98.15	-	6,548.64

Seismic Base Shear and Overturning Moment

Load Case	Level	Elevation (ft)	Base Shear		Overturning Moment	
			Vx (kip)	Vy (kip)	Mx (kip-ft)	My (kip-ft)
Seismic X	Level 3	42	184.71	0		
	Level 2	28	96.62	0		
	Level 1	14	48.55	0	11,142.88	-
Seismic Y	Level 3	42	0	184.49		
	Level 2	28	0	96.67		
	Level 1	14	0	48.72	-	11,137.42

Maximum moment =	11,142.88	kip-ft	experienced by building (assume worst case moment in either direction)
Resisting Moment =	60,800.03	kip-ft	from the weight of the building (assuming smallest moment arm and factor of safety of 1.5)

Factor of Safety = 5.5

Overturning Passes

	Weight (kips)	Mass (k-s2/ft)	Center of Mass X	Center of Mass Y	Building Edge X	Building Edge Y
Story 3	1560.4	48.46	-260.6	-385.83	-155.803	-392.882
Story 2	1226.43	38.088	-264.46	-383.47	-155.803	-392.882
Story 1	1236.11	38.388	-265.33	-382.93	-155.803	-392.882

Distance for Measurement
104.797
108.657
109.527

Distance to Edge	
COM to N	COM to S
23.2	34.6
23.08	35.27
22.67	33.56

Overturning Moment and Base Shear - East End

Building Effective Weight = 3966.09 kips

Wind Base Shear and Overturning Moment

Load Case	Level	Elevation (ft)	Base Shear Vx (kip)	Base Shear Vy (kip)	Overturning Moment Mx (kip-ft)	Overturning Moment My (kip-ft)
Wind X	Level 3	42	35.04	0		
	Level 2	28	67.36	0		
	Level 1	14	63.31	0	4,244.10	-
Wind Y	Level 3	42	0	47.25		
	Level 2	28	0	91.1		
	Level 1	14	0	86.02	-	5,739.58

Seismic Base Shear and Overturning Moment

Load Case	Level	Elevation (ft)	Base Shear Vx (kip)	Base Shear Vy (kip)	Overturning Moment Mx (kip-ft)	Overturning Moment My (kip-ft)
Seismic X	Level 3	42	178.93	0		
	Level 2	28	93.81	0		
	Level 1	14	52.48	0	10,876.46	-
Seismic Y	Level 3	42	0	178.93		
	Level 2	28	0	93.81		
	Level 1	14	0	52.48	-	10,876.46

Maximum moment =	10,876.46	kip-ft	experienced by building (assume worst case moment in either direction)
Resisting Moment =	40,136.83	kip-ft	from the weight of the building (assuming smallest moment arm and factor of safety of 1.5)

Factor of Safety = 3.7

Overturning Passes

	Weight (kips)	Mass (k-s2/ft)	Center of Mass X	Center of Mass Y	Building Edge X	Building Edge Y
Story 3	1487.45	46.194	-471.97	-875.62	-364.201	-844.61
Story 2	1169.75	36.328	-468.17	-874.49	-364.201	-844.61
Story 1	1308.89	40.649	-465.25	-875.26	-364.201	-844.61

Distance for Measurement		Distance to Edge COM to N	Distance to Edge COM to S
	107.769	15.18	43.36
	103.969	15.25	44.19
	101.049	16.93	42.03

APPENDIX C.5 – LATERAL SYSTEM HAND CHECKS

LATERAL SYSTEM HAND CHECK

WHAT IS CONTROLLING LOAD COMBINATION?

(5) $1.2D + 1.0E + L + 0.2S$

(7) $0.9D + 1.0E$

where,

$D = 113.98^k$

$W = 138.77^k$ worst case

$E = Q_E = 406.05^k$ → PER § 12.4.2 $E = E_h \pm E_v$

(+) → (5)

(−) → (7)

$L = 2.75^k$

$E_h = \rho Q_E = (1.0)(406.05^k) = 406.05^k$

$\rho = 1.0$ b/c SDC C PER § 12.3.4.1

$E_v = 0.2 S_{DS} D = 0.2(0.328)(113.98) = 7.477^k$

$S_{DS} = 0.328$ FROM U.S.G.S

So, (5) $P_u = 1.2D + 1.0E_v + L + 0.2S$

$= 1.2(113.98) + 1.0(7.477) + 2.75 + \cancel{0.2S}$ DISCONT. FROM ROOF

$= 147.003^k$

$V_u = 1.0E_h$

$= 1.0(406.05)$

$= 406.05^k$

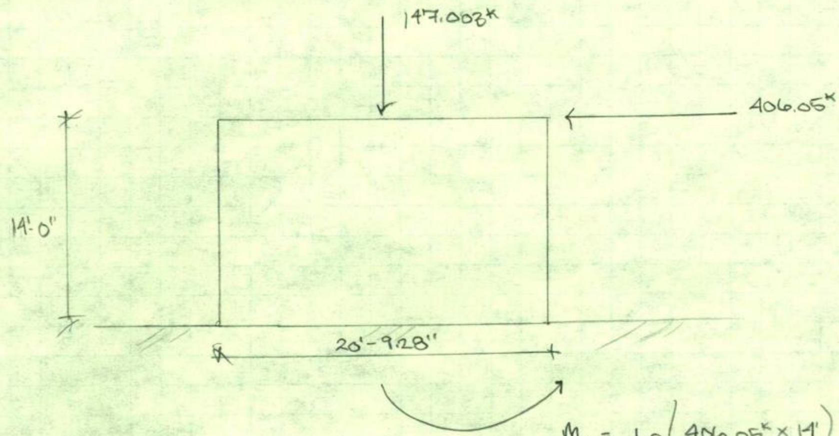
(7) $P_u = 0.9D + 1.0E_v$

$= 0.9(113.98) + 1.0(7.477)$

$= 110.059^k$

$V_u = 1.0E_h = 406.05^k$ (FROM ABOVE)

thus, CASE 5 CONTROLS



$M_U = 1.0(406.05^k \times 14')$

$= 5684.7'^k = M_U$

CHECK CONCRETE SHEAR WALL

ENSURE, $\phi V_n > V_u = 406.05^k$

What is V_c?

PER ACI 318-11, CHAPTER 11

$$V_c = 3.3\lambda\sqrt{f'_c}\cdot h\cdot d + \frac{N_u \cdot d}{4 l_w} \quad \text{(eq. 11-27)}$$

$\lambda = 1.0$
$f'_c = 4000$ psi
$h = 8''$
$d = 0.8 l_w = 0.8(20 + {}^{9.28}/_{12}) \times 12 = 199.424''$
$N_u = 1.2(113.98) = 136.776^k$

$$\text{so, } V_c = \left[3.3(1)\sqrt{4000}\cdot 8 \cdot 199.424 + \left(\frac{136.776^k \cdot 199.424''}{4(20\times12+9.28)}\right)\right]/1000$$

$$= 333.00^k$$

ALSO CHECK,

$$V_c = \left[0.6\lambda\sqrt{f'_c} + \frac{l_w\cdot\left(1.25\lambda\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]\cdot h\cdot d \quad \text{(eq. 11-28)}$$

$$= \left[0.6(1)\sqrt{4000} + \frac{199.424\cdot\left(1.25(1)\sqrt{4000} + 0.2\cdot\frac{136.776}{(20\times12+9.28)(8)}\right)}{\frac{5689.7\times12''/'}{406.05} - \frac{(20\times12+9.28)}{2}}\right]\times\frac{8\times199.424}{1000}$$

$$= 785.77^k$$

so, $V_c = \min \begin{cases} 333 \\ 786^k \end{cases} \rightarrow V_c = 333^k$

$\phi V_n = 0.75(333) = 249.75^k < 406.05^k$

X INSUFFICIENT

what is V_s?

$V_s = \dfrac{A_v f_y d}{s}$ HORIZONTAL REINF: #4 @ 18" O.C.

$A_v = \left(\dfrac{14' \times 12''/'}{18''} + 1\right)(0.2)(2) = 4.13$

$f_y = 60$

$d = 199.424''$

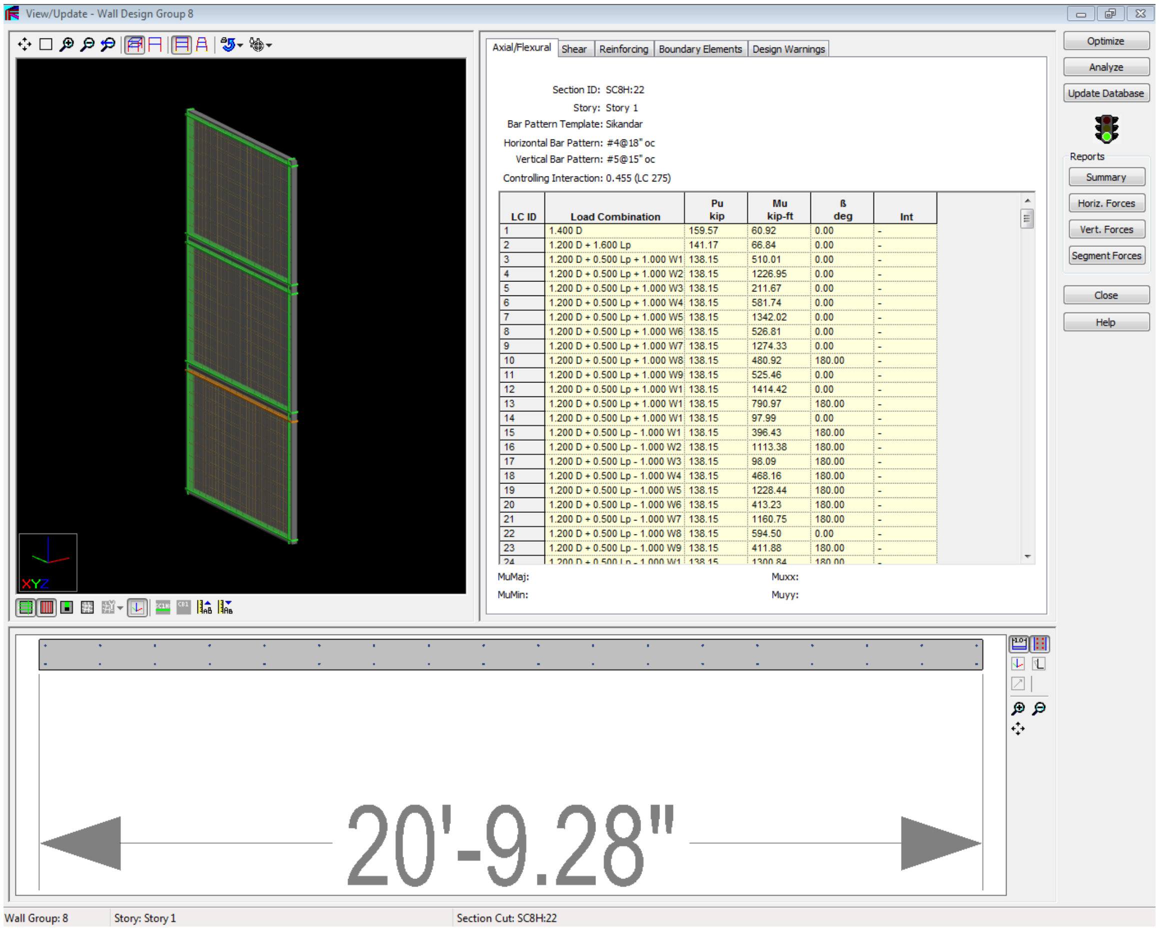
so, $V_s = \dfrac{4.13(60)(199.424)}{18''} = 2747.6^k$

now, $\phi V_n = 0.75(333 + 2747.6) = 2310^k > 406.05^k$

✓ GOOD

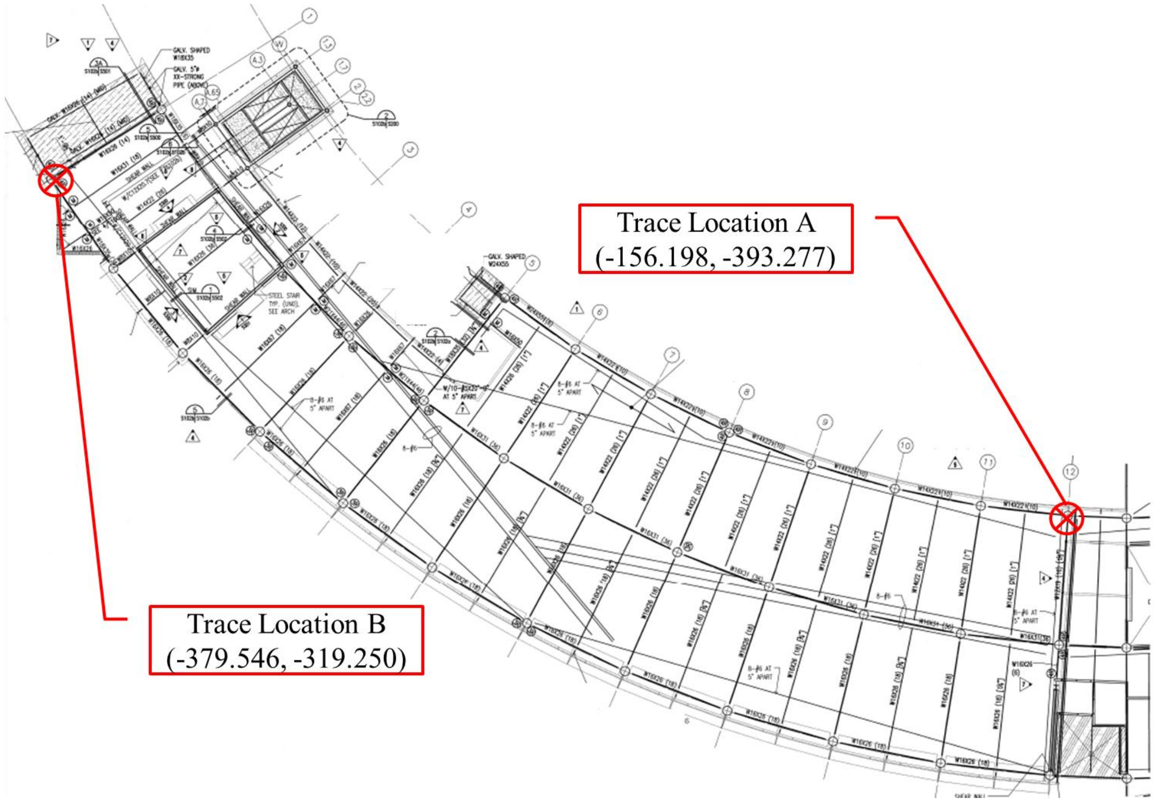
DESIGN GENERATED
BY RAM SS ARE
SUFFICIENT...

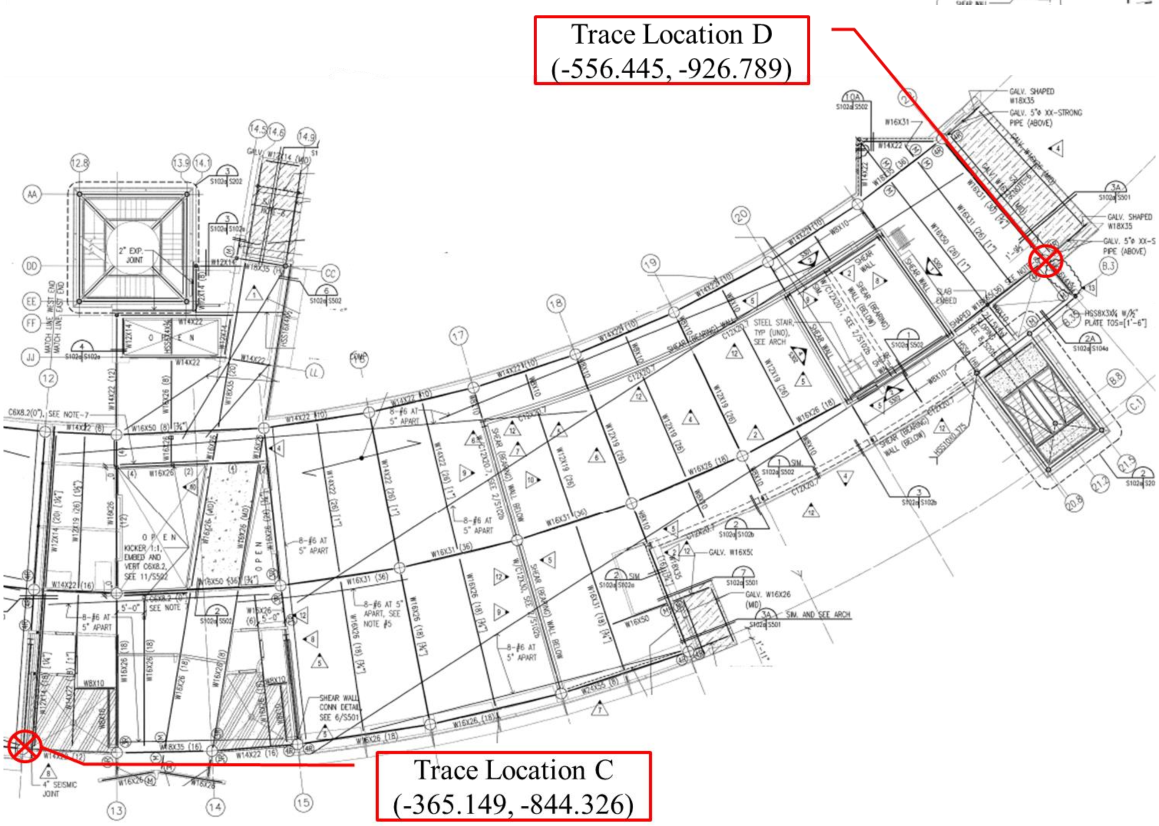
RAM Concrete was used in the design of the shear walls for the Heifer International Center. SW 13 @ column 12 is shown below.



LC ID	Load Combination	Pu kip	Mu kip-ft	ß deg	Int
1	1.400 D	159.57	60.92	0.00	-
2	1.200 D + 1.600 Lp	141.17	66.84	0.00	-
3	1.200 D + 0.500 Lp + 1.000 W1	138.15	510.01	0.00	-
4	1.200 D + 0.500 Lp + 1.000 W2	138.15	1226.95	0.00	-
5	1.200 D + 0.500 Lp + 1.000 W3	138.15	211.67	0.00	-
6	1.200 D + 0.500 Lp + 1.000 W4	138.15	581.74	0.00	-
7	1.200 D + 0.500 Lp + 1.000 W5	138.15	1342.02	0.00	-
8	1.200 D + 0.500 Lp + 1.000 W6	138.15	526.81	0.00	-
9	1.200 D + 0.500 Lp + 1.000 W7	138.15	1274.33	0.00	-
10	1.200 D + 0.500 Lp + 1.000 W8	138.15	480.92	180.00	-
11	1.200 D + 0.500 Lp + 1.000 W9	138.15	525.46	0.00	-
12	1.200 D + 0.500 Lp + 1.000 W1	138.15	1414.42	0.00	-
13	1.200 D + 0.500 Lp + 1.000 W1	138.15	790.97	180.00	-
14	1.200 D + 0.500 Lp + 1.000 W1	138.15	97.99	0.00	-
15	1.200 D + 0.500 Lp - 1.000 W1	138.15	396.43	180.00	-
16	1.200 D + 0.500 Lp - 1.000 W2	138.15	1113.38	180.00	-
17	1.200 D + 0.500 Lp - 1.000 W3	138.15	98.09	180.00	-
18	1.200 D + 0.500 Lp - 1.000 W4	138.15	468.16	180.00	-
19	1.200 D + 0.500 Lp - 1.000 W5	138.15	1228.44	180.00	-
20	1.200 D + 0.500 Lp - 1.000 W6	138.15	413.23	180.00	-
21	1.200 D + 0.500 Lp - 1.000 W7	138.15	1160.75	180.00	-
22	1.200 D + 0.500 Lp - 1.000 W8	138.15	594.50	0.00	-
23	1.200 D + 0.500 Lp - 1.000 W9	138.15	411.88	180.00	-
24	1.200 D + 0.500 Lp - 1.000 W1	138.15	1300.84	180.00	-

APPENDIX C.6 – TRACE LOCATIONS



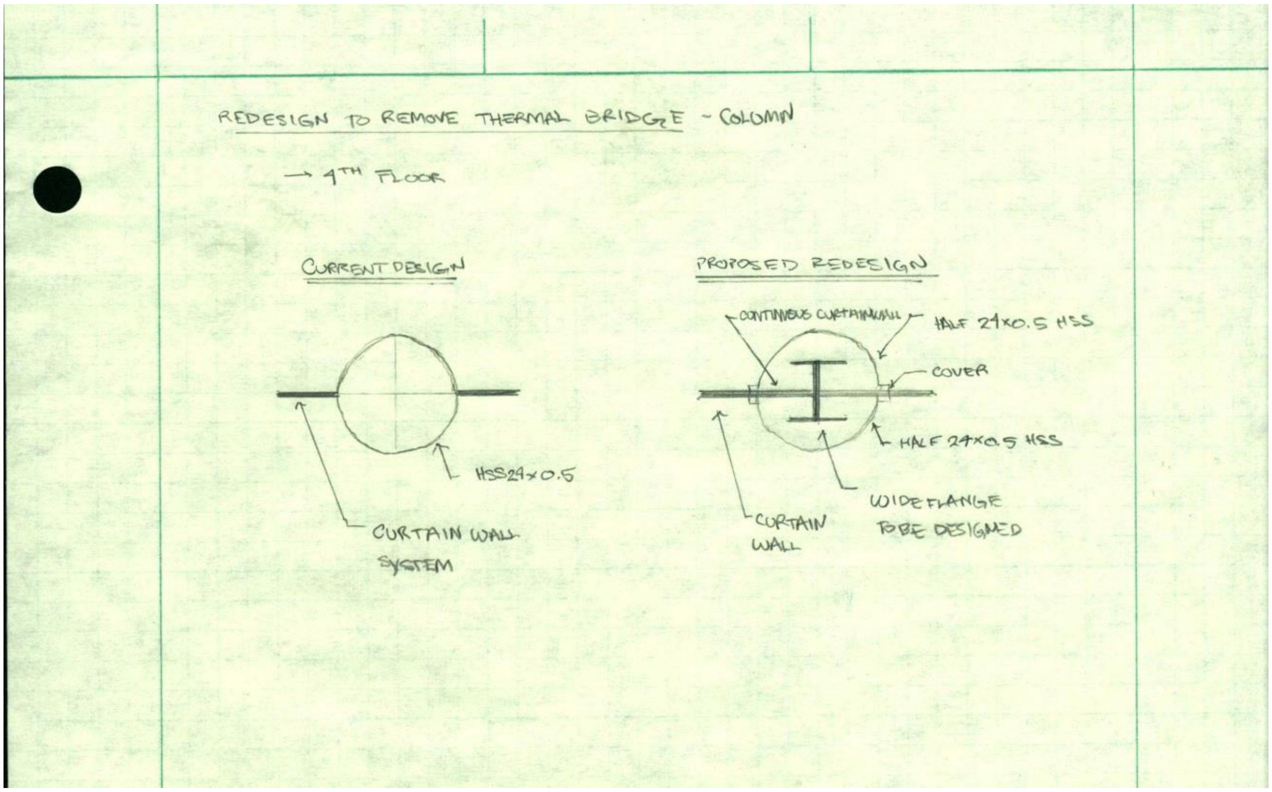


APPENDIX D

MECHANICAL AND ENVELOPE BREADTH

APPENDIX D.1 – THERMAL BRIDGE STUDY

Column Design



FROM PREVIOUS CALCS

PERIMETER BEAM 18.75^K $\times$ 2 / FLOOR

QUEEN POST (ROOF) 14.5^K

CONSERVATIVE CANTILEVERED AREA USED OF ROOF see below

CANTILEVER ROOF LOADS (ASD)

30 PSF DL
20 PSF LL (SNOW) $\longrightarrow$ 36 + 20 = 56 PSF

18.65' FOM ℄ COL. TO ℄ COL. → TRIB WIDTH ~19'

10' FOR LENGTH OF CANTILEVER

SO, 56 PSF × 19' = 1064 PLF ON CANTILEVER

R = V = wx = 1064 PLF × 10' = 10640 lb OR 10.64^K FROM CANTILEVERED SECTION

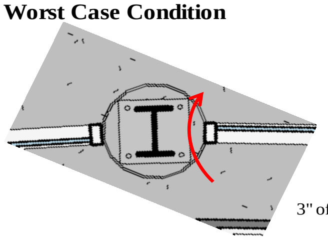
NOW, $18.75^K + 14.5^K + 10.64^K = 43.89^K \sim 44^K$ WITH AN UNBRACED LENGTH 10'

PER TABLE 4-1, W10x33 OR W12x40 @ 10' = l_c
220^K (ASD) 265^K (ASD)

WOULD BE SUFFICIENT → SMALLER WIDE FLANGES BUCKLE MORE EASILY AND WERE NOT CONSIDERED...

Worst Case Thermal Gradient

Worst Case Condition



3" of insulation assumed

Let's say:

T_i = 70
T_o = 10
$T_i - T_o$ = 60
T_{dp} = 14.69

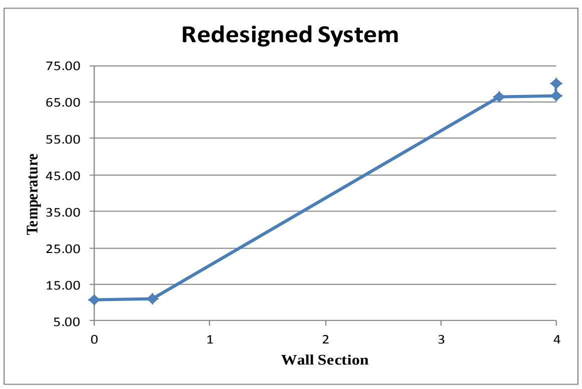
Redesigned System

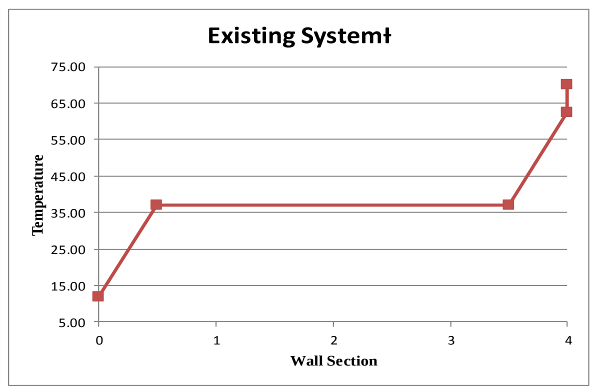
	Material	Depth (in)	R (BTU-in/h-ft^2-°F)	U (1/R)	$\sum R_{o-x}$	T_x	Reference
0	Outside Air Film	-	0.17	5.88	0.17	10.82	
0.5	Aluminum Composite	0.5	0.06	15.86	0.23	11.12	Almaxco - Aluminum Compsite Panels
3.5	Batt Insulation	3	11.45	0.09	11.69	66.41	Owens Corning Insulation Systems, LLC
4	Aluminum Composite	0.5	0.06	15.86	11.75	66.72	Almaxco - Aluminum Compsite Panels
4	Inside Air Film	-	0.68	1.47	12.43	70.00	
		Sum	12.43	0.08			

Existing Systemǂ

	Material	Depth (in)	R (BTU-in/h-ft^2-°F)	U (1/R)	$\sum R_{o-x}$	T_x	Reference
0	Outside Air Film	-	0.17	5.88	0.17	11.91	
0.5	HSS Steel	0.5	2.24	0.45	2.41	37.12	Wolfram Alpha, LLC
3.5	Air	23	0.00125	802.57	2.41	37.14	Wolfram Alpha, LLC
4	HSS Steel	0.5	2.24	0.45	4.65	62.35	
4	Inside Air Film	-	0.68	1.47	5.33	70.00	Wolfram Alpha, LLC
		Sum	5.33	0.19			

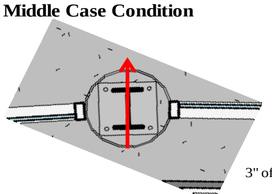
ǂthis is really a thermal bridge





Middle Case Thermal Gradient

Middle Case Condition



3" of insulation assumed

Let's say:

T_i =	70
T_o =	10
$T_i - T_o$ =	60
T_{dp} =	14.69

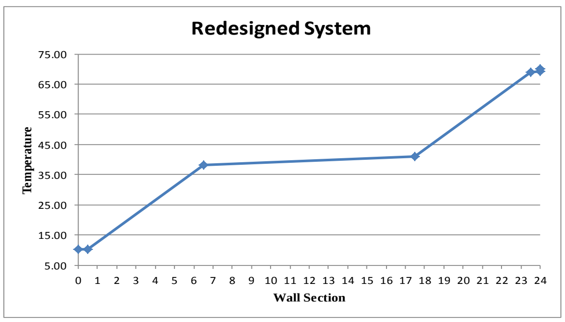
Redesigned System

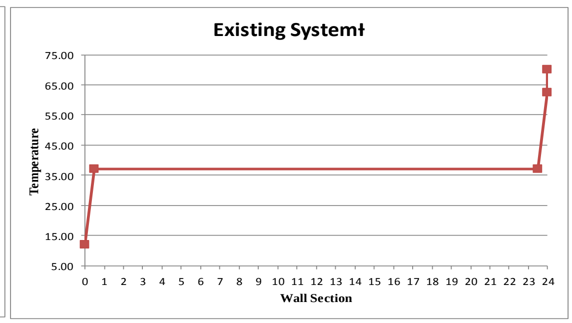
	Material	Depth (in)	R (BTU-in/h-ft^2-°F)	U (1/R)	$\sum R_{o-x}$	T_x	Reference
0	Outside Air Film	-	0.17	5.88	0.17	10.21	
0.5	Aluminum Composite	0.5	0.06	15.86	0.23	10.29	Almaxco - Aluminum Compsite Panels
6.5	Batt Insulation	6	22.91	0.04	23.14	38.32	Owens Corning Insulation Systems, LLC
17.5	Wide Flange	11	2.24	0.45	25.38	41.06	
23.5	Batt Insulation	6	22.91	0.04	48.29	69.09	
24	Aluminum Composite	0.5	0.06	15.86	48.35	69.17	Almaxco - Aluminum Compsite Panels
24	Inside Air Film	-	0.68	1.47	49.03	70.00	
		Sum	49.03	0.02			

Existing System[ǂ]

	Material	Depth (in)	R (BTU-in/h-ft^2-°F)	U (1/R)	$\sum R_{o-x}$	T_x	Reference
0	Outside Air Film	-	0.17	5.88	0.17	11.91	
0.5	HSS Steel	0.5	2.24	0.45	2.41	37.12	Wolfram Alpha, LLC
23.5	Air	23	0.00125	802.57	2.41	37.14	Wolfram Alpha, LLC
24	HSS Steel	0.5	2.24	0.45	4.65	62.35	
24	Inside Air Film	-	0.68	1.47	5.33	70.00	Wolfram Alpha, LLC
		Sum	5.33	0.19			

[ǂ]this is really a thermal bridge





ACADEMIC VITA

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EDUCATION

The Pennsylvania State University **University Park, PA**
Integrated Bachelor and Master of Architectural Engineering *Graduation, May 2014*
Structural Option | Five-year professional degree | ABET accredited
EIT Certified upon graduation | Schreyer Honors College

The Tsinghua University **Beijing, China**
Summer School for International Construction *Summer 2012*

The University of Hong Kong **Pokfulam, Hong Kong**
The Department of Real Estate and Construction *Summer 2012*

WORK EXPERIENCE

Penn State University | Research and Education Institute **University Park, PA**
Indeterminate Analysis and Computer Modeling Teaching Assistant August 2013 – present

- Provide feedback to students by evaluating homework and exams

Holbert Apple Associates, Inc. | Structural Engineer Consultants **Washington D.C. Area**
Engineering Intern May 2013 – August 2013

- Performed analysis on existing framing and design of new framing, with limited site visits
- Reviewed shop drawings for concrete reinforcing and structural steel
- Prepared construction documents using AutoCAD and Revit
- Worked with RAM SBeam, spSlab, PROFIS, Enercalc and Decon STDesign

Penn State University | Research and Education Institute **University Park, PA**
Undergraduate Researcher | *Laboratory Scholar* January 2010 – May 2013

- Tested Structural Insulated Panels for intended wide spread use
- Examined formaldehyde reduction in buildings using gypsum dry board
- Engineered working models in RISA, ETABS and SAP 2000
- Managed ordering of project
- Conceptualized fuel cell productivity using printed biofilms on the electrode and examined immobilization of biofilms using latex substance

Sustainable Design Group | Design and Construction Firm **Gaithersburg, MD**
Intern May 2011 – August 2011

- Prepared construction documents for residential and business projects
- Evaluated sustainable design research for developing countries
- Oversaw permitting application for counties in Maryland and Virginia
- Worked with Graphisoft ArchiCAD 13/15, Photoshop CS.5, and Google Sketchup

PUBLICATIONS

Wagner, R., Porter-Gill, S. "Immobilization of anode-attached microbes in a microbial fuel cell." AMB Express. 2012.
Porter-Gill, S. "Overview of the Causes and Remediation of Sinkholes." The American Society of Civil Engineers TCFE. 2013.