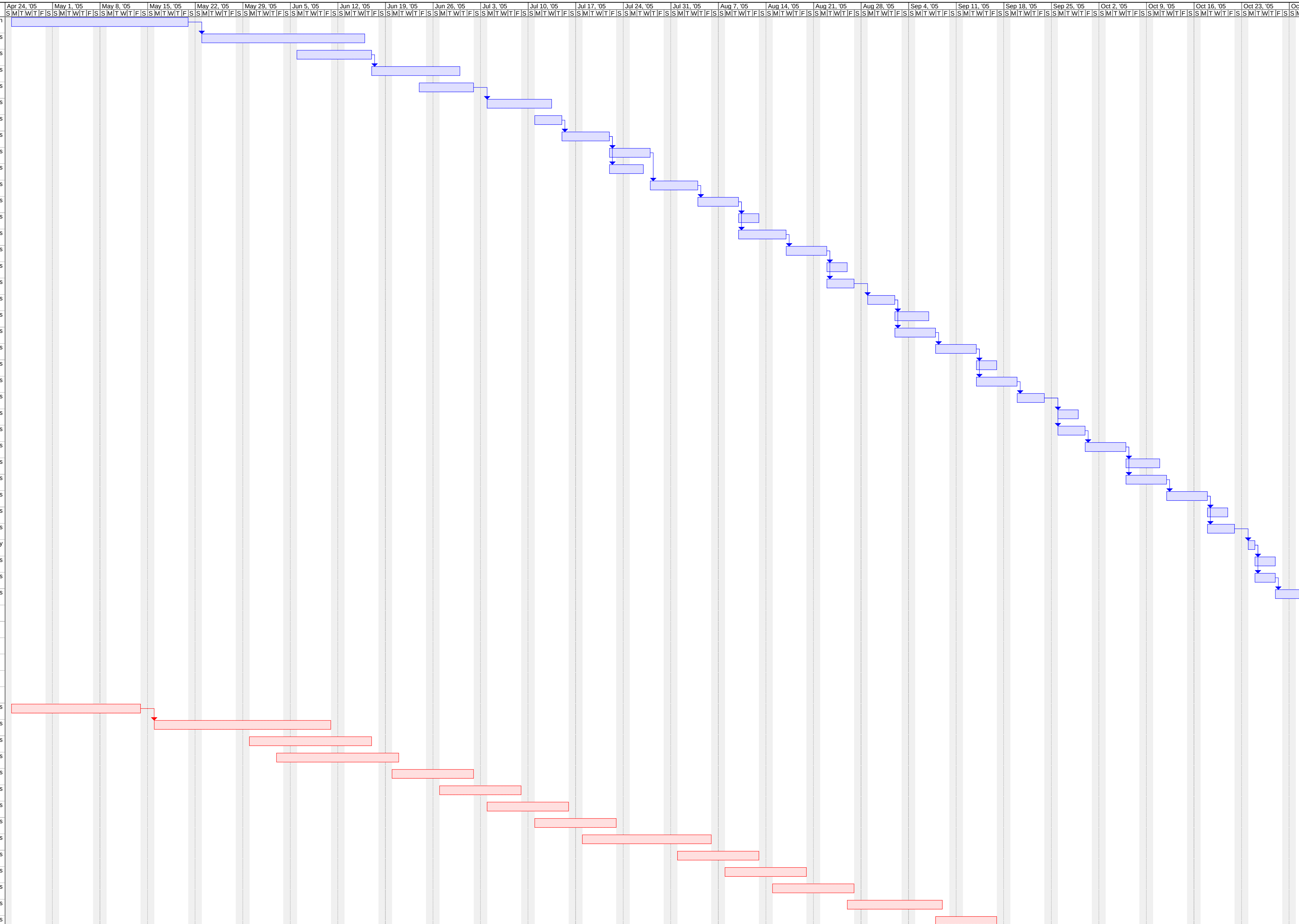
Appendix

ID	Task Name	Duration
1	Foundation, Walls, Columns, SOG	1 mon
2	Ground Floor FRP Slab	18 days
3	Ground FRP Columns	9 days
4	2nd Floor FRP Slab	9 days
5	2nd Floor FRP Columns	6 days
6	3rd FRP Slab	7.5 days
7	3rd Floor FRP Columns	4 days
8	4th Floor FRP Slab	5 days
9	4th Floor FRP Columns	4 days
10	Post Tensioning	3 days
11	5th Floor FRP Slab	5 days
12	5th Floor FRP Columns	4 days
13	Post Tensioning	3 days
14	6th Floor FRP Slab	5 days
15	6th Floor FRP Columns	4 days
16	Post Tensioning	3 days
17	7th FRP Slab	4 days
18	7th Floor FRP Columns	4 days
19	Post Tensioning	3 days
20	8th Floor FRP Slab	4 days
21	8th Floor FRP Columns	4 days
22	Post Tensioning	3 days
23	9th Floor FRP Slab	4 days
24	9th Floor FRP Columns	4 days
25	Post Tensioning	3 days
26	10 Floor FRP Slab	4 days
27	10th Floor FRP Columns	4 days
28	Post Tensioning	3 days
29	11th Floor FRP Slab	4 days
30	11th Floor FRP Columns	4 days
31	Post Tensioning	3 days
32	Penthouse Floor FRP Slab	4 days
33	Penthouse Floor FRP Columns	1 day
34	Post Tensioning	3 days
35	Penthouse Roof FRP Slab	3 days
36	Post Tensioning	2 days
37		
38		
39		
40		
41		
42		
43	Foundation, Walls, SOG	3 wks
44	Ground floor	20 days
45	2nd floor	14 days
46	3rd floor	12 days
47	4th floor	10 days
48	5th floor	10 days
49	6th floor	10 days
50	7th floor	10 days
51	8th floor	15 days
52	9th floor	10 days
53	10th floor	10 days
54	11th floor	10 days
55	penthouse	10 days
56	penthouse roof	7 days



Hilton Hotel Becoming LEED Certified

This survey will take about 5 minutes and will ask questions pertaining to your stay at hotels.

Survey Questions

1. How frequently do you travel and stay in hotels?

2. Are you familiar with the LEED rating system for buildings?

LEED rating system was developed and is administered by the United States Green Building Council (USGBS). It was designed to increase profitability of a building for all phases of its life (design, construction, and sustainability), while reducing the negative impact caused by these processes and promoting a healthier environment for its occupants and community. (USGBS website)

3. Given these aspects of LEED buildings, is this something that appeals to you? If so, what aspects do you find appealing?

4. Are you familiar with CO2 monitoring systems? Sensors will go off when high CO2 levels are obtained in the space. When this happens the mechanical system draws in more outside air to maintain a close to ambient level of CO2. Fresh air**!**

Are you aware of any hotels that do this?

5. If given the choice between a LEED rated hotel and a non-LEED rated hotel, which hotel would you stay at? Why would you choose to stay in that hotel?

6. **Consider the hotel that you stayed in most recently:**

   a. **What was the name of the hotel?**

   b. **Did you stay on a weekday or weekend night?**

   c. **How much did you pay per night for the room? (excluding taxes)**

   d. **If the room had been in a LEED-certified hotel, would you be willing to pay more for the room because it is LEED certified? If so, how much more would you be willing to pay?**

7. **Would a hotel advertising they are LEED rated effect your decision? If yes, how? What type of information would you want to learn from advertising about LEED certified hotels?**

8. **How does online marketing affect your hotel decision?**

Braced Frame Members

Level	Braced Frame #1			Braced Frame #3			Braced Frame #4		
Level	Column	Chevron	Beam	Column	Chevron	Beam	Column	Chevron	Beam
PH Roof			W10x26			W10x26			W10x26
Penthouse	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
11	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
10	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
9	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
8	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
7	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
6	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
5	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
4	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
3	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x106	2L6x6x1/2	W10x26
2	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26	W18x175	2L6x6x1/2	W10x26
G	W18x175	2L6x6x5/8	W10x26	W18x175	2L6x6x5/8	W10x12	W18x175	2L6x6x1/2	W10x12
Foundation	W18x175	2L6x6x1/2		W18x175	2L6x6x1/2		W18x175	2L6x6x1/2	

Level	Braced Frame #6			Braced Frame #8		
Level	Column	Chevron	Beam	Column	Cross	Beam
PH Roof			W10x26			
Penthouse	W18x106	2L6x6x1/2	W10x26			W10x12
11	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
10	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
9	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
8	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
7	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
6	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
5	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
4	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
3	W18x106	2L6x6x1/2	W10x26	W14x82	HSS 7x4x1/2	W10x12
2	W18x175	2L6x6x1/2	W10x26	W14x145	HSS 7x4x1/2	W10x12
G	W18x175	2L6x6x1/2	W10x12	W14x145	HSS 7x5x5/8	W10x12
Foundation	W18x175	2L6x6x1/2		W14x145	HSS 7x4x1/2	

	Braced Frame #11			Braced Frame #2			Braced Frame #5			
Level	Column	Chevron	Beam	Column	Cross	Beam	Columns	Chevron	Crosss	Beam
PH Roof						W10x12				W10x19
Penthouse				W18x175	HSS 8x8x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x30
11				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
10				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
9				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
8				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
7				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
6				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
5				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x3/8	HSS 7x4x3/8	W10x22
4				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x1/2	HSS 7x4x3/8	W10x22
3				W18x175	HSS 8x4x5/8	W10x12	W18x106	HSS 7x4x1/2	HSS 7x4x3/8	W10x22
2			W12x40	W18x175	HSS 8x4x5/8	W10x12	W18x175	HSS 7x5x1/2	HSS 7x5x1/2	W10x30
G	W12x96	HSS 8x8x1/2	W12x40	W18x175	HSS 8x4x5/8	W10x12	W18x175	HSS 7x7x1/2	HSS 7x5x1/2	W10x30
Foundation	W12x96	HSS 8x8x1/2		W18x175	HSS 8x4x5/8	W10x12	W18x175	HSS 7x4x3/8	HSS 7x4x3/8	

	Braced Frame #7			Braced Frame #9			Braced Frame #10		
Level	Column	Chevron	Beam	Column	Chevron	Beam	Column	Chevron	Beam
PH Roof			W10x12						
Penthouse	W18x106	HSS 7x4x3/8	W12x14			W10x26			W10x26
11	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
10	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
9	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
8	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
7	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
6	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
5	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
4	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
3	W18x106	HSS 7x4x3/8	W12x14	W14x82	HSS 7x4x3/8	W10x26	W14x82	HSS 7x4x3/8	W10x26
2	W18x175	HSS 7x4x3/8	W12x14	W14x145	HSS 7x4x3/8	W10x26	W14x145	HSS 7x4x3/8	W10x26
G	W18x175	HSS 7x5x1/2	W12x14	W14x145	HSS 7x7x1/4	W10x26	W14x145	HSS 7x7x1/4	W10x26
Foundation	W18x175	HSS 7x4x3/8		W14x145	HSS 7x4x3/8		W14x145	HSS 7x4x3/8	

Wind Calculations

Exposure Class	B
Importance Factor I	1
Topographic Factor K_{zt}	1
Wind Directionality Factor K_d	0.85
Basic Wind Speed V (mph)	90
N-S Length of Bldg.	292.17
E-W Length of Bldg.	243.67
Ct factor in the N-S Direction	0.02
Ct factor in the E-W Direction	0.02

Level Heights(ft)	Level	hx	Kz	qz	Pressures					
					NS windward	NS leeward	NS Pressure	EW windward	EW leeward	EW Pressure
15.7	Penthouse	129.67	1.09	19.21	13.99	-5.24	19.23	13.06	-8.17	21.23
11	11	114	1.04	18.33	13.34	-4.37	17.72	12.46	-8.17	20.63
9	10	103	1.04	18.33	13.34	-4.37	17.72	12.46	-8.17	20.63
9	9	94	0.99	17.45	12.70	-4.37	17.07	11.87	-8.17	20.03
9	8	85	0.96	16.92	12.32	-4.37	16.69	11.51	-8.17	19.67
9	7	76	0.93	16.39	11.93	-4.37	16.30	11.15	-8.17	19.31
9	6	67	0.89	15.69	11.42	-4.37	15.79	10.67	-8.17	18.83
9	5	58	0.85	14.98	10.91	-4.37	15.28	10.19	-8.17	18.35
9	4	49	0.81	14.28	10.39	-4.37	14.76	9.71	-8.17	17.87
9	3	40	0.76	13.40	9.75	-5.24	15.00	9.11	-8.17	17.27
13	2	31	0.76	13.40	9.75	-5.24	15.00	9.11	-8.17	17.27
18	1	18	0.62	10.93	7.96	-5.24	13.20	7.43	-8.17	15.60
0	Ground	0	0.57	10.05	7.31	-5.24	12.56	6.83	-8.17	15.00

	acting at floor level					
	Forces (k)		Shears (k)		Overturning Moment	
Level	N/S	E/W	N/S	E/W	N/S	E/W
ph roof	11.60	23.95	11.60	23.95	1504.12	3105.1
ph floor	19.88	52.99	31.48	76.94	3588.84	8771.1
11th floor	15.06	52.81	46.54	129.75	4793.54	13364.5
10th floor	13.31	46.84	59.85	176.59	5625.53	16599.8
9th floor	12.91	45.74	72.76	222.33	6184.63	18898.0
8th flooor	12.62	44.91	85.38	267.24	6488.88	20310.0
7 floor	12.28	43.94	97.66	311.18	6542.97	20849.0
6 floor	11.88	42.84	109.54	354.02	6353.31	20532.9
5th floor	11.49	41.73	121.03	395.75	5930.51	19391.6
4th floor	11.38	42.04	132.41	437.79	5296.57	17511.7
3rd floor	20.16	52.44	152.58	490.24	4729.90	15197.3
2nd floor	43.38	72.00	195.95	562.24	3527.15	10120.2
				Total	60565.94	184651.4

Seismic Calculations

Seismic Design Criteria (Ch.11)	
S_s	0.15
S_1	0.053
Site Class	D
F_a	1.6
F_v	2.4
S_{MS}	0.240
S_{M1}	0.1272
S_{DS}	0.160
S_{D1}	0.0848
Ct	0.02
h_n (ft)	131.00
x	0.75
T_a	0.77
T_o	0.11
T_s	0.53
T_L	8.00
S_a	0.11
Occ. Category	II
Importance factor (I)	1.0
Seismic Design Category	B

$F_x = C_{vx} * V$	k = 1.135						
$C_{vx} = W_x H_x^k / \Sigma W_i H_i^k$							
LEVEL	W_x	H_x	$W_x H_x^{1.135}$	C_{VX}	F_X	V_X	Overturning Moment
PH Roof	319.2	127.67	78428.47	0.06	26.02	26.02	3322.55
Penthouse	872.30	112	184726.13	0.14	61.30	87.32	6865.24
11	941.70	101	177344.34	0.13	58.85	146.17	5943.58
10	938.10	92	158908.94	0.12	52.73	198.90	4851.16
9	939.00	83	141520.46	0.10	46.96	245.86	3897.69
8	939.80	74	124340.74	0.09	41.26	287.12	3053.20
7	939.90	65	107334.25	0.08	35.62	322.73	2315.05
6	941.80	56	90813.85	0.07	30.13	352.87	1687.52
5	943.70	47	74587.32	0.05	24.75	377.62	1163.25
4	943.70	38	58598.75	0.04	19.44	397.06	738.89
3	1862.10	29	85079.46	0.06	28.23	425.29	818.71
2	3198.50	18	85051.24	0.06	28.22	453.52	508.00
Total	13460.60		1366733.94	1.00	453.52		35164.85

Footing Take-off

Existing Footings

Type	Width (ft)	Length (ft)	Depth (ft)	Vol (ft^3)	Cubic yd.s	Quantity	Total Volume(ft^3)
F-3.0	3	3	1	9.00	0.33	1	0.33
F-3.5	3.5	3.5	1.17	14.29	0.53	6	3.18
F-4.0	4	4	1.33	21.33	0.79	7	5.53
F-4.5	4.5	4.5	1.67	33.75	1.25	4	5.00
F-5.0	5	5	1.83	45.83	1.70	9	15.28
F-5.5	5.5	5.5	2.00	60.50	2.24	1	2.24
F-6.0	6	6	2.08	75.00	2.78	1	2.78
F-6.5	6.5	6.5	2.25	95.06	3.52	1	3.52
F-7.0	7	7	2.42	118.42	4.39	4	17.54
F-7.5	7.5	7.5	2.58	145.31	5.38	3	16.15
F-8.0	8	8	2.75	176.00	6.52	5	32.59
F-8.5	8.5	8.5	2.92	210.73	7.80	12	93.66
F-9.0	9	9	3.00	243.00	9.00	10	90.00
F-9.5	9.5	9.5	3.17	285.79	10.58	2	21.17
F-10	10	10	3.33	333.33	12.35	5	61.73
F-12.5x9.0	12.6	9	3.50	396.90	14.70	1	14.70
F-3.0x5.0	3	5	1.50	22.50	0.83	1	0.83
F-5x8.5	5	8.5	2.25	95.63	3.54	1	3.54
TOTAL							389.77

Cost Calculations

Structual Steel

Lateral Frame Takeoff	Wt. (lbs)	Wt. (tons)		
Columns	315947	158.0		
Beams	71939	36.0		Includes construction
Braces	139933	70.0	Price per ton	$3,500
Total	527819	263.9	Cost	$923,683.25

Beam Takeoff				
Penthouse Roof	25403	12.7		
Penthouse	69539	34.8		
3rd	53244	26.6		
2nd	136742	68.4		
Ground	208910	104.5		
Gravity Column Takeoff				
all floors	388000	194.0		Includes construction
			Price per ton	$3,500
Total	881838	440.9	Cost	$1,543,216.50

Floors 4-11

Girder Slab				
DB8X40	140096	70.048		
DB8x35	5552	2.776	Price per Ton	$2,600
Total	145648	72.824	Cost	$189,342.40

Spandrel beams 4-11			Price per Ton	$2,300.00
Flrs. 4-11	162832	81.4	Cost	$187,256.80

Labor & eqipment per ton	Tons	cost		
$2,000	154.24	$308,480		
		Includes O&P	Total for G-Slab	$822,095.04

Total of steel $3,288,994.79

Includes Construction

Floor Area Takoff	Area (ft^2)	Material CIP $6.79/s.f.	Material Pre-cast plank $9.00/s.f	Material Fireproofing $1.68/s.f
Penthouse Roof	4928	$33,461.12		$8,279.04
Penthouse	15984	$108,531.36		$26,853.12
Flrs. 4-11	127872		$1,150,848.00	$214,824.96
3rd	17392	$118,091.68		$29,218.56
2nd	22120	$150,194.80		$37,161.60
Ground	49204	$334,095.16		$82,662.72
Total	237500	$744,374.12	$1,150,848.00	$399,000.00

Total of s.f costs $2,294,222.12

Grand Total $5,192,391.73

Floor Slabs

Floor	Volume (ft^3)	C.Y.	Material: Flat slab $550 per C.Y.	Flat plate $530 per C.Y.
Penthouse Roof	3080.00	114.1		$60,459.26
Penthouse	11988.00	444.0		$235,320.00
Flrs. 4-11	79920.00	2960.0		$1,568,800.00
3rd	13044.00	483.1	$265,711.11	
2nd	16590.00	614.4	$337,944.44	
Ground	36903.00	1366.8	$751,727.78	
Total			$1,355,383.33	$1,864,579.26
		Total Slab	$3,219,962.59	

Beams

Floor	Volume (ft^3)	C.Y.	Beams $1025 per C.Y
Penthouse Roof	748.16	27.7	$28,402.36
Penthouse	686.19	25.4	$26,049.82
Flrs. 4-11	1587.19	58.8	$60,254.44
3rd	606.38	22.5	$23,019.79
2nd	606.38	22.5	$23,019.79
Ground	519.75	19.3	$19,731.25
Total		176.1	$180,477.45

Columns

Floor	Volume (ft^3)	C.Y.	Columns $1475 per s.f
Penthouse Roof		`	
Penthouse	372.75	13.8	$20,363.19
Flrs. 4-11	16862.22	624.5	$921,176.95
3rd	1897.00	70.3	$103,632.41
2nd	2079.00	77.0	$113,575.00
Ground	4491.00	166.3	$245,341.67
Total		951.9	$1,404,089.22

Post Tension

Assume 1psf

Floor	Area	PT $1.89 per s.f
Penthouse Roof	4928	$9,313.92
Penthouse	15984	$30,209.76
Flrs. 4-11	127872	$241,678.08
Total		$281,201.76

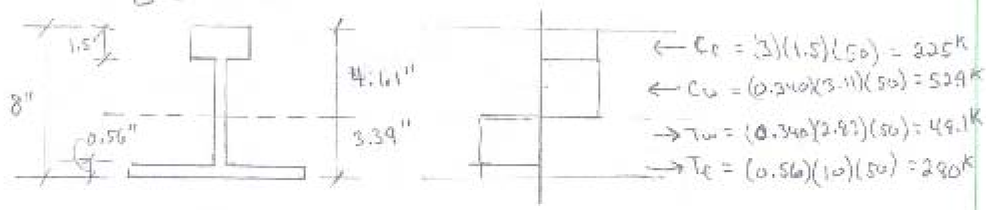
Shear Walls

	Volume (ft^3)	cubic yards	$365/per cubic yard
SW #1	3423.33	126.79	$46,278.40
SW #2	1580.00	58.52	$21,359.26
SW #3	3385.33	125.38	$45,764.60
SW #4	2793.50	103.46	$37,763.98
SW #5	4614.92	170.92	$62,386.84
SW #6	3002.00	111.19	$40,582.59
SW #7	1540.50	57.06	$20,825.28
SW# 8	1580.00	58.52	$21,359.26
SW # 9	1410.00	52.22	$19,061.11
SW # 10	3666.00	135.78	$49,558.89
SW #11	3284.13	121.63	$44,396.50
SW #12	1296.46	48.02	$17,526.20
		1169.49	$426,862.90

Grand Total	**$5,126,712.35**

Thomas Sabol

DB 8×40

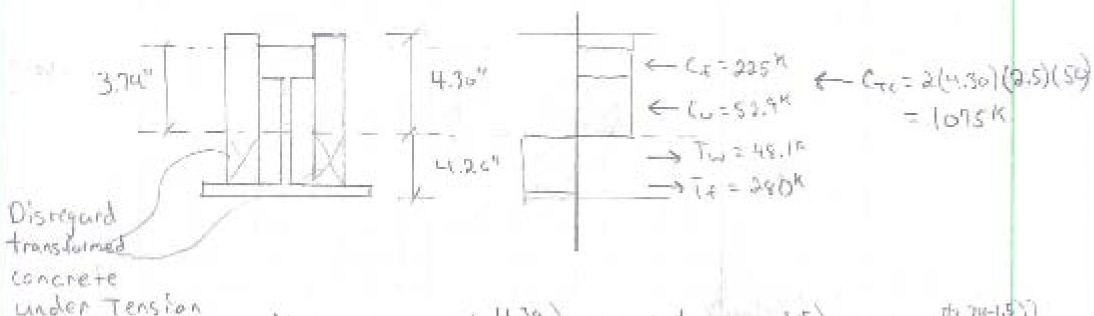


$$M_p = 225^k\left(4.61 - \left(\tfrac{1.5}{2}\right)\right) + 52.9^k\left[\frac{(4.61-1.5)}{2}\right] + 48.1^k\left[\frac{(3.39-0.56)}{2}\right] + 280^k\left[3.39 - \left(\tfrac{0.56}{2}\right)\right]$$

$$M_p = 1889.6^{''K} = 157.5^{'K}$$

W/o web $M_p = 1739.3^{''K} \Rightarrow \underline{144.9^{'K}}$

Transformed Section
DB8×40



$$M_p = 1075^K\left(\tfrac{4.30}{2}\right) + 225^K\left(3.74 - \tfrac{1.5}{2}\right) + 52.9^K\left[\frac{(3.74-1.5)}{2}\right] + 48.1^K\left[\frac{4.26-0.56}{2}\right] + 280\left[4.26 - \left(\tfrac{0.56}{2}\right)\right]$$

$$M_p = 4246.64^{''K} \Rightarrow 353.9^{'K}$$

W/o web $M_p = 4098.4^{''K} \Rightarrow \underline{341.53^{'K}}$

Transformed section carries 236 the amount of plastic moment capacity

Girder- Slab — Thomas Sabol — (1)

SDL = 10psf
LL = 40psf

Trib width 27'

16'

8'-4" Span Deck
self wt. = 58 psf

@ 27' Deck can carry 62psf > 58psf ∴ ok

DB 8 x 40
$I_t = 122\ in^4$
$S_t = 26.5\ in^3$
$S_b = 36.1\ in^3$
$t_w = 0.34\ in$

Transformed Section
$I_t = 287\ in^4$
$S_t = 67.2\ in^3$
$S_b = 67.9\ in^3$
$h = 3.5\ in$

Grout $f'_c = 4000psi$

Initial Pre-composite

$$M_{DL} = \frac{(58psf \cdot 27')(16)^2}{8} \Rightarrow 50112^{ft \cdot lb} = 50.1^{'k}$$

DB 8x40 Allowable Moment $\Rightarrow 66^{'k} > 50.1$ ∴ ok

Total Load Composite

$$M_{sup} = \frac{(0.010+0.040)(27')(16)^2}{8} \Rightarrow 43.2^{'k}$$

$$M_{TL} = 43.2^{'k} + 50.1^{'k} = 93.3^{'k}$$

$$S_{req} = \frac{93.3^{'k}(12^{in}/ft)}{(0.6)(50^{ksi})} = 37.3\ in^3 < 67.2\ in^3 \quad \therefore ok$$

$$\text{Allowable } \Delta_{LL} = L/360 = \frac{16'(12in/ft)}{360} = 0.53''$$

$$\Delta_{LL} = \frac{5(27')(0.04+0.010)(16')^4(1728)}{384(287 in^4)(29000^{ksi})}$$

$$= 0.28'' < L/360 = 0.53'' \quad \therefore ok$$

Superimposed Compressive Stress on Concrete

$$N = \frac{E_{steel}}{E_{conc.}} \quad \frac{29000000}{57000\sqrt{4000}} = 8.04$$

$$S_{tc} = N \cdot S \Rightarrow (8.04)(67.2\ in^3) = 540\ in^3$$

$$\text{Allowable } F_c = (0.45)(4ksi) = 1.8\ ksi$$

$$f_c = \frac{M_{sup}}{S_{tc}} = \frac{(43.2^{'k})(12^{in}/ft)}{540} = 0.96^{ksi}$$

$F_c = 1.8^{ksi} > 0.96^{ksi}$ ∴ ok

Girder - Slab Thomas Sabol ②

Bottom Flange Tension Stress (Total Load)

$$f_b = \frac{50.1(12\,in/ft)}{36.1\,in^3} + \frac{43.2^{\,ik}(12\,in/ft)}{67.9}$$

$$= 16.7\,ksi + 7.6\,ksi \Rightarrow 24.3\,ksi$$

$$F_b = 0.9(50\,ksi) = 45\,ksi > f_b = 24.3\,ksi$$

$\therefore$ ok

Shear Check

Total Load = 108 psf

ω = 2.92 klf

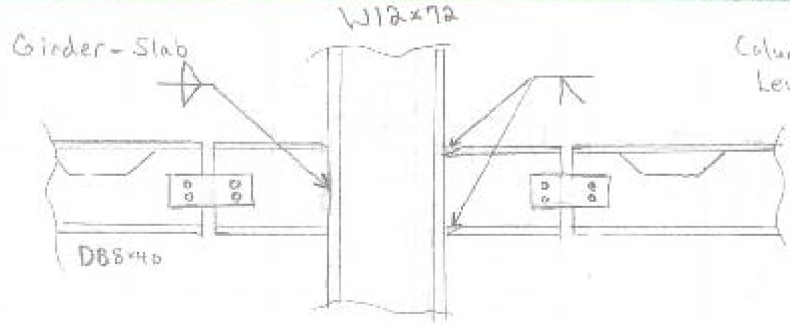
$$R = \frac{(2.92\,klf)(16')}{2} = 23.3^k$$

$$f_v = \frac{23.3^k}{(0.340)(3.5)} = 19.6\,ksi$$

$$F_v = 0.4(50\,ksi) = 20\,ksi > 19.6\,ksi \quad \therefore ok$$

Tree Connection | Thomas Sabol

Column Line 1-5.2
Level 4



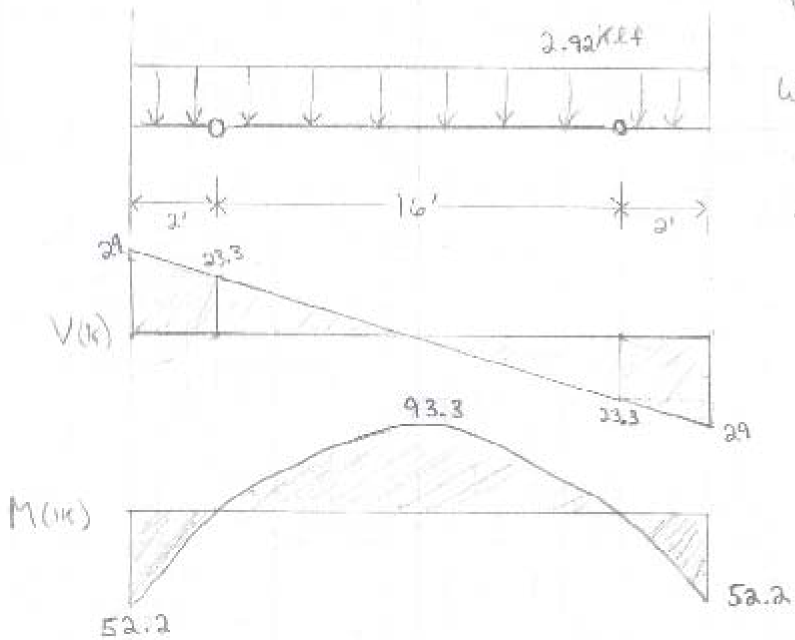
DB8x40

$d = 8"$
$t_w = 0.340"$

Trib width 27'

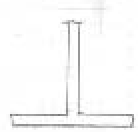
$W = DL + LL$
$68 + 40$
$= 108\,psf$

2916 plf



Need WT8 to resist 52.2 1k

Try WT8 x 22.5 w/o welded top bar 3x1.5



$S_x = 37.8\,in^3$

$d = 8.07"$
$t_w = 0.345"$
$b_f = 7.04"$
$t_f = 0.565"$

$$S_{req} = \frac{52.2^{1k}(12\,in/ft)}{(0.60 \times 50\,ksi)} = 21\,in^2 < 37.8\,in^3 \quad \therefore ok$$

Bottom flange Tension stress

$$\frac{52.2^{1k}(12\,in/ft)}{37.8\,in^3} = 16.6\,ksi < F_b = 45\,ksi$$

Shear

$f_v = 29^k / (0.345)(7.5)$
$= 11.2$

$F_v = 0.4(50\,ksi) = 20\,ksi > 11.2\,ksi$
$\therefore ok$

Tree Connection | Thomas Sabol | ②

Weld Design

Bevel weld Top & Bottom

Fillet on both sides
Table J2.5 AISC 13th Ed.
Tension normal to weld axis
Compression " " } strength of joint is controlled by base metal

Max size of fillet weld
1/4" thick
controlled web t_w of W8x28

Col. W12x72
$t_f = 0.670"$

Beam WT8x22.5
$t_w = 0.345$

Design of Shear Plate

A36 Steel Plate

=> 6" dim

$V = 29^K$

Table 10-9a Pg. 10-109

Use 2-Row bolts - 1" ø A325-N
7/16" plate => 32.4^K (ASD)

Use 9½" x 6½" x 7/16" Plate

Check Bearing & T.O. of DB $t_w = 0.340$ < WT $t_w = 0.345$

Bearing $2.4(65^{ksi})(1")(0.340) = 53^K$

Edge = $1.2(65^{ksi})[1.5 - 0.5(1" + 1/16)](0.340) = 25.7^K$

Other = $1.2(65^{ksi})[3 - 0.5(17/16)](0.340) = 64.5^K$

$R_n/\Omega = [53 + 25.7]/2.00 = 39^K > 29^K \therefore$ ok

Shear yielding of W8x28 Table 9-3b

$R_n/\Omega =$

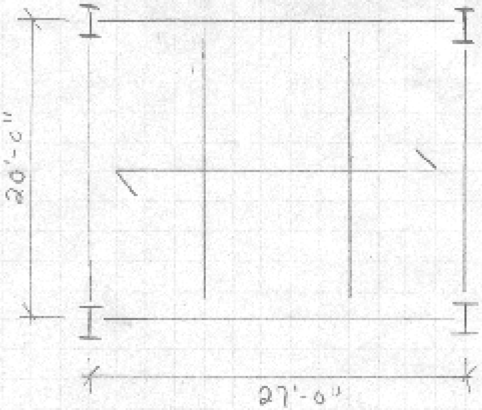
Shear Rupture of W8x28 Table 9-3c

$R_n/\Omega =$

2/21/07 | AE Thesis | Thomas Sabol | 1

2nd Floor Bay size 27' × 20'

A992 Steel
$F_y = 50$ ksi
Composite Beam
ASD



Live Load: 2nd floor
100 psf

Dead Load:
superimposed = 10 psf
Deck: 2.1 psf
concrete slab: 38 psf
beam: Assume 26 plf

Deck
USD 2" LOK-Floor
3" concrete normal wt. $f'_c = 3$ ksi
Max unshored 3 spans => 9.61' > 9' ∴ ok

Construction Loading: Beams spaning 20' ⊥ to Deck

Assume $\omega_{LL\,cons.} = 20$ psf

$$\omega_{DL} = (38\,psf + 2.1\,psf)(9'_{\text{Trib}}) + 26\,plf = 387\,plf$$

$$\omega_{Total} = 387\,plf + (20\,psf)(9') = 567\,plf$$

$$M = \frac{567(20)^2}{8} = 28350^{\#} = 28.4^{'K}$$

Try W12×26 $M_p/\Omega_b = 78.1^{'K}$ $I = 144\ in^4$

Check Deflection during construction

$$\Delta_{cons} = \frac{5}{384}\frac{\omega l^4}{EI} \Rightarrow \frac{5}{384}\frac{(0.567^{Klf})(20)^4(1728)}{(29000)(144)}$$

$$\Delta_{cons} = 0.49"$$

$$L/360 = \frac{20 \times 12}{360} = 0.67" > \Delta_{const} \therefore ok$$

Thesis | Thomas Sabol | 2

Composite Action: Beam

Assume $a = 1''$

$Y_2 = 5 - \frac{1}{2} = 4.5''$

$\omega_D = (10psf + 2.1psf + 39psf)(9') + 26plf$
$= 477plf$

Live Load Reduction

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$K_{LL} \Rightarrow$ Int beam $= 2$

$A_T = 4' \times 20'$
$= 180 ft^2$

$= (100psf)\left(0.25 + \frac{15}{\sqrt{(2)(180)}}\right)$

$L = (100psf)(1.04)$

Ignore Reduction

$\omega_{Total} = 477plf + (100psf)(9')$
$= 1377plf$

$M = \frac{(1377plf)(20)^2}{8} = 68850^{lb} \Rightarrow 68.9^{'k}$

W10x26 PNA BFL (5) $M_p/\Omega_b = 124^{'k}$

$b_{eff} \le 1/4$ span $= \frac{20 \times 12''}{4} = 60''$ ✓ controls

$b_{eff} \le$ Spacing $= 9' \times 12'' = 108''$

$C_{tot} = C_c + C_s$ but Q_n will control

$\Sigma Q_n = 127^K$

$a = \frac{127^K}{(0.85)(3)(60)} = 0.83 < 1''$

$Y_2 = 5 - \frac{0.83}{2} = 4.58$

Thesis — Thomas Sabol — (3)

Studs Table - 3-21

Weak studs per rib
1 - Ø = 3/4"

Total studs per beam

$$\frac{2\ \Sigma Q_n}{Q} = \frac{2(127^k)}{17.2^k} = 14.8^k$$

Use 16 - 3/4" Ø studs

Check LL Deflection $I_{LB} = 322 in^3$

$$\Delta_{LL} = \frac{5}{384}\frac{(0.100)(20)^4\ 1728}{(29000)(322)}$$

$= 0.039"$

$L/360 = 0.67" > \Delta_{LL} = 0.039"$

$$0.67 = \frac{5}{384}\frac{(0.100)(20)^4\ 1728}{29000\ I_{LB}}$$

For 27'0" x 20'-0" Bay

$\Rightarrow I_{LB} = 18.5\ in^4$ required to comply with Δ_{LL}

$M = 69^{'k}$

Actual Bay sizes vary 27' x (19'-22')

$L/240$

$I = 105\ in^4$ required for $\Delta_{const.}$

I_{LB} req for Δ_{Total} $= 171\ in^4$

W10x26 least Depth

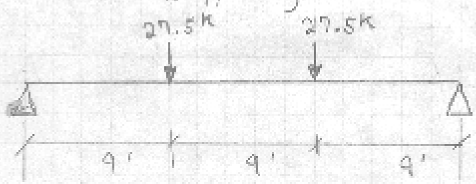
possibly W10x22 will work

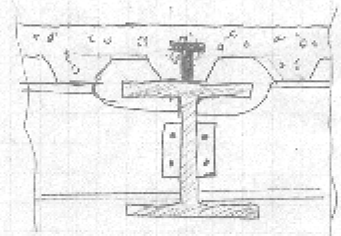
RAM Results:

		M_p/Ω_b (PNA-5)	I_{const}	I_{LB}
14 studs	W 12 x 14	87.8 'k ✓ok	88.6 x	253 in⁴ ✓ok
16 studs	W 12 x 19	114 'k ✓ok	130 in⁴ ✓ok	338 in⁴ ✓ok
10 studs	W 14 x 22	145 'k ✓ok	199 in⁴ ✓ok	473 in⁴ ✓ok

Thesis | Thomas Sabol | 4

Girder supporting beams





$b_{eff} \leq \frac{1}{4}(27' \times 12) = 81''$ ✓ controls

$b_{eff} \leq (20' \times 12) = 240''$

$M_{max} = (27.5^k)(9') = 248^{'k}$

Solve for Deflection and then check strength

$$\Delta_{Total} = \frac{P l^3}{28 E I}$$

$$L/240 = 1.35''$$

$$1.35'' = \frac{27.5^k (27' \times 12'')^3}{28(29000^{ksi})\, I}$$

$$I_{\Delta Total} = 853.2\ in^4$$

$$I_{LB} \geq 853\ in^4$$

Assume $a = 1''$

$Y_2 = 4.5''$

Try 16 x 36 PNA BFL(5) $M_p/\Omega_b = 257^{'k}$ $I_{LB} = 940\ in^4$

$\Sigma Q_n = 228^k$

$$a = \frac{228^k}{(0.85)(3)(60)} = 1.49$$

$Y_2 = 5 - \frac{1.49}{2} = 4.25''$ use $Y_2 = 4 \Rightarrow M_p = 251^{'k}$ $I_{LB} = 901 in^4$

\# of studs = 26

RAM Results for Girder — 70% composite $\Rightarrow \Sigma Q_n = 360$, PNA-4

$Y_2 = 3.5$	M_p/Ω (PNA4)	I_{LB}
W18x35	$282^{'k}$ ok	$1120\ in^4 > 853\ in^4$

$a = \frac{360}{(0.85)(3)(60)} \Rightarrow 2.35$ $Y_2 = 5 - \frac{2.35}{2} = 3.825$ use 3.5

Girder Thomas Sabol

3rd Floor

RAM Result

Check W16x31 $M_{max} = 157^{'K}$ Span 27'

25.3% composite

$\Sigma Q_n = 0.253(456^K) \Rightarrow 115.4^K \Rightarrow$ PNA 7

$a = \frac{115.4}{0.85(3)(60)} = 0.75$

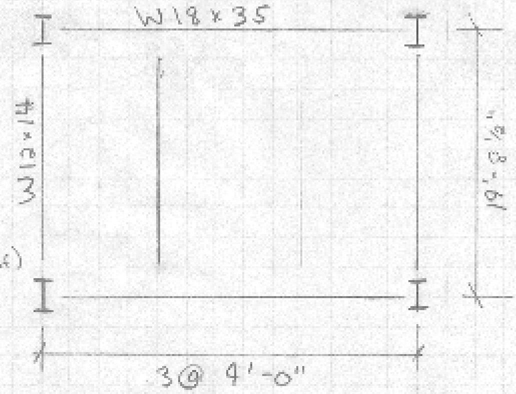
$Y_2 = 5 - \frac{0.75}{2} = 4.625 \Rightarrow$ Use 4.5

$M_n/\Omega = 194^{'K} > M_{max} = 157^{'K}$

AISC Tables RAM Result

Vibration Thesis Thomas Sabol ①

2nd Floor Office
Bay J-K - 3-4



Live Load = 11 psf
Super DL = 4 psf

Concrete $w_c = 145$ pcf
$f'_c = 3000$ psi

Floor thickness
= 3 in + 2 in ribs (2.1 psf)
= 5 in

Slab + Deck wt = 38 psf

Beam Properties:
W12x14
$A = 4.16 in^2$
$I_x = 88.6 in^4$
$d = 11.9$ in

Girder Properties:
W18x35
$A = 10.30 in^2$
$I_x = 510 in^4$
$d = 17.70 in^4$

Beam mode Properties

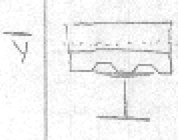
✓ Effective Slab width $108" = 9' > 0.4L_j = 0.4(19.71') = 7.88'$
$= 94.6"$

$E_c = w^{1.5}\sqrt{f'_c} = 145^{1.5}\sqrt{3} = 3024$ ksi

$n = E_s / 1.35 E_c = 29000/(1.35 \times 3024) = 7.10$

$$\bar{Y} = \frac{(4.16 in^2)(2 + 11.9/2) - (94.6"/7.10)(3")(3"/2)}{4.16 + (94.6/7.10)(3)}$$

$= -0.609$ in occur above top of form deck



$$I_j = 88.6 + 4.16(2 + 11.9/2 + 0.609)^2 + (94.6/7.1)(3)^3/12 + (94.6/7.1)(3)[(3 - 0.609)/2]^2$$

$= 88.6 + 304.7 + 29.98 + 57.1$
$= 480 in^4$

$w_j = 9'(11 + 38 + 4 + 14/9) = 491$ plf

$$\Delta_j = \frac{5(491 plf)(19.71)^4(1728)}{384(29000000)(480 in^4)} = 0.12 \text{ in}$$

Vibration | Thesis | Thomas Sabol | 2

$f_j = 0.18\sqrt{\frac{g}{\Delta_j}} = 0.18\sqrt{\frac{386}{0.12}} = 10.2\ Hz$

Avg conc thickness = 4 in

$D_s = 12 d_e^{12}/(12n) = 12(4)^3/(12\times 7.10) = 9.01\ in^4/ft$

$D_j = I_j/S \Rightarrow 480\ in^4/9 = 53.3\ in^4/ft$

$C_j = 2.0$

$B_j = C_j(D_s/D_j)^{1/4} L_j = 2.0(9.01/53.3)^{1/4}(19.71)$
$= 25.4\ ft$

✓ 2/3 width $\Rightarrow \frac{2}{3}(3\times 27')$
$= 54 > 25.4\ ft \quad \therefore B_j = 25.4\ ft$

$W_j = 1.5(w_j/S)B_j L_j = 1.5(491/9)(25.4\times 19.71)$
$= 40968.5\ lb \Rightarrow 40.9\ K$

Girder Mode

Effective slab width
$0.4 L_g = 0.4\times 27'\times 12 = 129.6'' < L_j = 236.5''$

Avg conc. depth = 4 in

$$\bar{y} = \frac{10.30(1+17.7/2) - (129.6/7.10)(4)(4/2)}{10.30 + (129.6/7.10)(4)}$$

$= -0.535$ above eff slab

$$I_g = 510 + 10.30(1 + 17.7/2 + 0.535)^2 + (129.6/7.10)(4)^3/12 + (129.6/7.10)(4)[(4-0.535)/2]^2$$
$= 510 + 1110.8 + 97.4 + 219.1$
$= 1937\ in^4$

$W_g = L_j(w_j/S) + \text{girder weight}$
$= 19.71(491/9') + 35 = 1110\ plf$

$$\Delta_g = \frac{5(1110)(27)^4(1728)}{384(29000000)(1937)} = 0.236$$

Vibration | Thesis | Thomas Sabol | (3)

$f_g = 0.18\sqrt{\frac{386}{0.236}} = 7.28\ Hz$

$D_j = 53.3\ in^4/ft$

$D_g = I_g/L_j = 1937in^4/19.71ft \Rightarrow 98.3\ in^4/ft$

$C_g = 1.8$

$B_g = C_g(D_j/D_g)^{1/4} L_g = (1.8)(53.3/98.3)^{1/4}(27') \Rightarrow 41.7\ ft$

✓ $2/3(3 \times 19.71') = 39.4' < 41.7\ ft$ use 39.4 ft

$W_g = (w_g/L_j) B_g L_g = (1110/19.71)(39.4')(27')$
$= 59910\ lb \Rightarrow 59.9K$

Girder Span = 27' > beam panel width = 25.4'

$f_n = 0.18\sqrt{g/(\Delta_j + \Delta_g)} = 0.18\sqrt{386/(0.12+0.236)} \Rightarrow \underline{5.93\ Hz}$

Equiv. Panel Wt.

$W = \frac{\Delta_j}{\Delta_j+\Delta_g} W_j + \frac{\Delta_g}{\Delta_j+\Delta_g} W_g \Rightarrow \left(\frac{0.12}{0.356}\right)40.9^k + \left(\frac{0.236}{0.356}\right)59.9^k$

$W_{EQ} = 53.5^K$

For office occupancy with full Ht. Partitions $\Rightarrow \beta = 0.05$

$\beta W = 53.5^K \times 0.05 = 2.67^k$

office $P_o = 65\ lbs$

$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \Rightarrow \frac{65 \exp(-0.35 \times 5.93)}{2670}$

$= 0.0031 \Rightarrow 0.31$ percent g

∴ Satisfactory

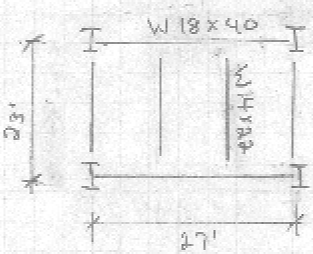
Reccomended for offices 0.5% g

Ballroom Vibration

LL => $w_p = 12.5$ psf

Super DL = 4 psf

$K = 1.3$ for dancing



$f = 3$ Hz (Dancing first Harmonic)

$\frac{a_o}{g} = 2.5\%$ (Between frequency 4-8)

$\alpha_1 = 0.5$ (Table 2.1 Dancing)

$$f_n (\text{req'd}) = f\sqrt{1 + \frac{K}{a_o/g}\,\frac{\alpha_1 w_p}{w_t}}$$

$$= 3\sqrt{1 + \left(\frac{1.3}{0.025}\right)\left(\frac{0.5 \times 12.5}{67.8}\right)}$$

$$= 7.22 \text{ Hz}$$

$$w_t = 4\left(\frac{22\text{plf}}{9'}\right) + 2\left(\frac{40\text{plf}}{23'}\right) + 38\text{psf} + 4\text{psf} + 12.5\text{psf} = \underline{67.8\text{ psf}}$$

Use this req'd f_n to solve for Δ_j

$$f = 0.18\sqrt{\frac{g}{\Delta_j}}$$

$$\Rightarrow \Delta_j = \frac{g}{\left(\frac{f}{0.18}\right)^2} \Rightarrow \frac{386 \text{ in/sec}^2}{\left(\frac{7.22}{0.18}\right)^2} = 0.24''$$

$$\Delta_j = \frac{5 w_j L_j^4}{384 E_s I_j} \Rightarrow I_j = \frac{5 w_j L_j^4}{384 E_s \Delta_j}$$

$$= \frac{5(512.5)(23 ft)^4(1728)}{384(29000000)(0.24'')}$$

$$\underline{I_{j\,\text{req'd}} = 463.6 \text{ in}^4}$$

I_j for a W12x14 previously calculated $= 480 \text{ in}^4$

Beams used in Ballroom:

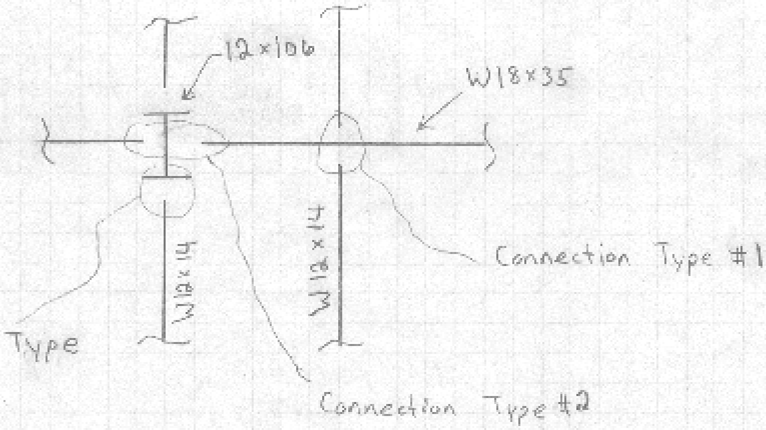
W 14x22

W 12x19

Both will have larger I_j values ∴ Vibration is within acceptable limits

Connections | Thomas Sabol | 1

Typical Bay
2nd Floor



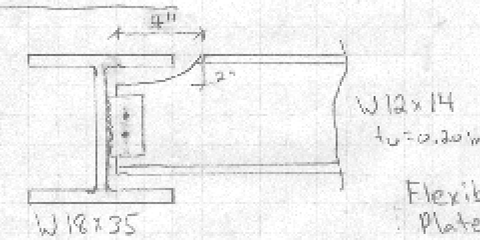
Conn. Type #1 => Shear Tab

Type #2 => Welded (to column) / bolted Single Angle

Type #3 => Bolted (Beam) / welded (Girder) Double Angle

Type #1 Shear Tab

$b_f = 6$ in
$t_w = 0.3$ in



Bolts: 3/4" ϕ A325-N
Plate: A36

$V = 14^K$

Flexible: Girder web with Plate on one side

Table 10-9a AISC Manual 13th Edition Pg 10-107

Use 2 - 3/4" - ϕ A325 N bolts
1/4" plate => 16.3^K (ASD)
L = 5 1/2" a = 3 1/2" $L_{eh} - 2d_b = 1.5"$ => 5 1/2" × 5" PL
weld size 3/4 t_p => 3/16" min web size = 0.286" < 0.3" ∴ ok

ASD Coped Beam Flexural Strength:

Buckling

$\frac{c}{d} = \frac{4}{11.9} = 0.37$

$f = 2(0.37) = 0.67$

$c/h_o = 4/9.9 = 0.41 \le 1.0$

$K = 2.2\left(\frac{9.9}{4}\right)^{1.65} = 9.81$

$F_{cr}/\Omega = \frac{26210}{1.67}\left(\frac{0.20}{9.9}\right)^2 (0.37)(9.81) => 23.2^{ksi}$

$S_{net} = 4.71 in^3$ Table 9-2 pg 9-24

$\frac{50}{1.67} = 29.9^{ksi} > F_{cr} = 23.2^{ksi}$ ∴ ok

$M_n/\Omega = \left(\frac{1}{2.00}\right)(65^{ksi})(4.71 in^3) = 153^{in \cdot K}$

$M_a = (14^K)(4.5") = 63^{in \cdot K}$ ✓ ok

Connections — Thomas Sabol — (2)

Block Shear of beam

$R_n/\Omega = 0.6\,F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6\,F_y A_{gv} + U_{bs} F_u A_{nt}$

$\Omega = 2.00$ (ASD)

$t_w = 0.2''$

3/4" ⌀ bolt

$L_{eh\,beam} = a - 0.5'' \Rightarrow 3''$

Table 9-3a Pg. 9-31 Tension Rupture

$\dfrac{F_u A_{nt}}{t\Omega} = 83.3\ ^{K}/_{in}$

Table 9-3b Shear yielding

$n = 2$

$L_{ev} = 1\,7/8$

$\dfrac{0.6\,F_y A_{gv}}{t\Omega} = 73.1\ ^{K}/_{in}$

Table 9-3c Shear Rupture

$\dfrac{0.6\,F_u A_{nv}}{t\Omega} = 69.5\ ^{K}/_{in}$

$R_n/\Omega = 69.5\ ^{K}/_{in}(0.2'') + 83.3\ ^{K}/_{in}(0.2'') = 30.6^{K} > 14^{K}$

$\therefore$ ok

Bearing $2.4\,F_u d_b t = 2.4(65^{ksi})(3/4)(0.2'') = 23.4^{K}$

Edge: $1.2\,F_u L_c t = 1.2(65^{ksi})[1.5 - 0.5(3/4 + 1/16)](0.2) = 17.1^{K}$

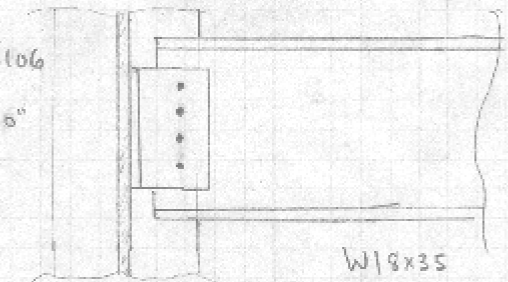
other: $1.2\,F_u L_c t = 1.2(65^{ksi})(3 - 13/16)(0.2) = 34.1^{K}$

$R_n/\Omega = [23.4 + 17.1]/2.00 = 20.2^{K} > 14^{K}$

$\therefore$ ok

Connections | | Thomas Sabol | 3

Type #2 Welded / Bolted Single Angle



W12x106

$t_w = 0.610''$

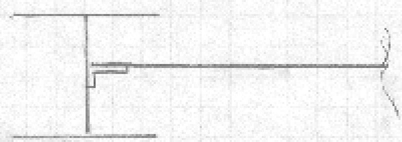
End reaction + Self wt. of beam

$V = 28.5^K$

A36 Steel for Angle

W18x35

$t_w = 0.300''$

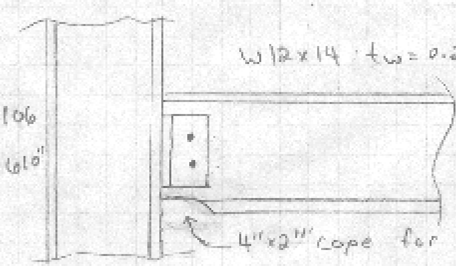


Eccentric Weld Table
K = 0.26
x = 0.026 Interpolation
$x\ell = 0.24$
$a\ell = 3 - 0.20 + 0.15 = 2.86$
a = 0.25
D = 2.18 ∴ 3/16" is ok

Table 10-11 pg. 10-137
Use 4 - 3/4" ø A-325 N bolts => 42.4^K (ASD)
L 4 × 3 × 3/8 - 17 1/2" long
Weld (70 ksi)
3/16" => 37.7^k min web thickness = 0.256 < 0.610" col web

weld strength controls

Type #3 Bolted / Welded Double Angle



W12x14 $t_w = 0.2''$

$V = 14^K$

W12x106

$t_w = 0.610''$

4"x2" cope for constructability

Coped Beam flexural strength checked on connection Type #1

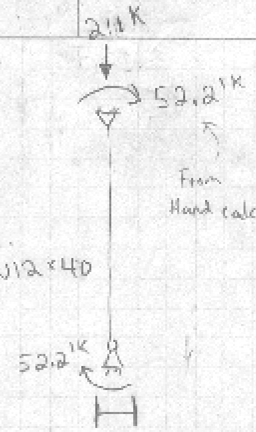
Table 10-1 Bolt Design
Use 2 rows 3/4 ø" - 325 N bolts => 32.6^k (ASD)
1/4" thick angle
L 3 1/2" x 3 1/2 x 1/4" - 6" Long

Table 10-2 Weld Design
Welds B (70 ksi)
size 5/16" -> $R_n/\Omega = 18.2^K$ min support thickness 0.238" < 0.610" t_w = 12x106

3 1/2" Angle width

(2)

Column F-5.2 Tree connection column



10th Floor

$K = 1.0 \qquad L_b = 9'$

$P = 210.5^k$

$M_{NT} = 52.2'^k$

$M_{LT} = 0$

RAM col. Design W12x40

W12x40

$P_n/\Omega = 279^k$

$L_b = 9' < \quad M_n/\Omega = 134'^k$

$$B_1 = \frac{C_m}{1 - \alpha P_r/P_{e1}} \geq 1$$

$$C_m = 0.6 - 0.4(M_1/M_2)$$

$$M_1 = M_2$$

$$\therefore C_m = 0.6$$

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 (29000^{ksi})(307 \text{ in}^4)}{(1.0 \times 9' \times 12")^2}$$

$$= 7533^k$$

$P_r = 140^k \qquad \alpha = 1.60 \text{ ASD}$

$$B_1 = \frac{0.6}{\left[1 - \left[(1.6)(211^k)\right]/7533^k\right]} = 0.63 \quad \therefore B_1 = 1.0$$

$M_r = 52.2'^k$

$P_r = 211^k$

$$\frac{P_r}{P_c} = \frac{211^k}{279^k} = 0.76 \geq 0.2 \quad \therefore (H1-1a)$$

$$0.76 + \frac{8}{9}\left(\frac{52.2}{134}\right) = 1.10$$

$\therefore$ No Good

G. Col. Thomas Sabol (3)

Column Line F-5.2

Try W12x45

$L_b = 9'$ $P_n/\Omega = 314^k$ $M_{nt} = 52.2^{'k}$

$M_n/\Omega = 152^{'k}$ $P = 140^k$

$$\frac{P_r}{P_c} = \frac{211^k}{314^k} = 0.67 \geq 0.2 \quad (H1\text{-}1a)$$

$$0.67 + \left(\frac{8}{9}\right)\left(\frac{52.2}{152}\right) = 0.96 \quad \therefore ok$$

Change Ram results to W12x45

10 - PR roof

Check Next splice at Floor 7 $L_b = 9'$

W12x53 $P = 355^k$ $M_{nt} = 52.2^{'k}$

$P_n/\Omega = 406^k$ $M_n/\Omega = 193^{'k}$

$C_m = 0.6$

$$P_{e1} = \frac{\pi^2 (29000)(425\,in^4)}{(1.0 \times 9' \times 12'')^2} = 10429^k$$

$$B_1 = \frac{0.6}{\left[1 - \left[1.6 \times 355 / 10429\right]\right]} = 0.63 \quad \therefore B_1 = 1.0$$

$$\frac{P_r}{P_c} = \frac{355}{406} = 0.87 \geq 0.2 \quad \therefore H1\text{-}1a$$

$$0.86 + \frac{8}{9}\left(\frac{52.2}{193}\right) = 1.1 \quad \therefore \text{No Good}$$

Try W12x58

$P_n/\Omega = 446^k$ $M_n/\Omega = 216^{'k}$

$$\frac{P_r}{P_c} = \frac{355}{446} = 0.79 \geq 0.2$$

$$0.79 + \frac{8}{9}\left(\frac{52.2}{216}\right) = 1.0 \quad \therefore ok$$

Change Ram results Fl. 7-9 to W12x58

4

Check Next Splice at Floor 4

Changed from W 12x65
↳ Noncompact for flexure

$L_b = 9'$

Try W 12 x 72

$P_n/\Omega = 577^k$

$M_n/\Omega = 270^{'k}$

$M_{nt} = 52.2^{'k}$

$P = 495^k$

$$P_{e1} = \frac{\pi^2 (29000)(597 in^4)}{(1.0 \times 9' \times 12)^2} = 14650^k$$

$$B_1 = 1.0$$

$$\frac{P_r}{P_c} = \frac{495}{577} = 0.86$$

$$0.86 + \frac{8}{9}\left(\frac{52.2}{577}\right) = 0.94 \leq 1.0 \therefore ok$$

$\boxed{\text{Update Ram to W 12x72}}$

Check Floor Ground for strictly Axial

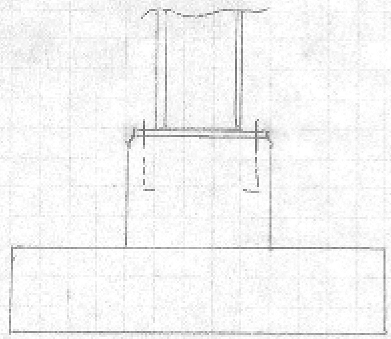
$L_b = 12'$ Ram Size W12x106 => $798^k < 654^k \therefore ok$

$L_b = 18'$ 2nd Floor => $656^k > 596^k \therefore ok$

FTG Design | Thesis | Thomas Sabol | 1

Bearing capacity $\Rightarrow$ $q_a = 12000\,psf$

Col Line F-5.2



Load at Base of Column $= 675^{K}$

Col: W12 x 106
$f'_c = 3^{ksi}$
A36 Base Plate

Base Plate Design: Assume $A_1 = A_2$

$P_p/\Omega_c > P_a$ $\quad P_p = P_a\,\Omega_c$

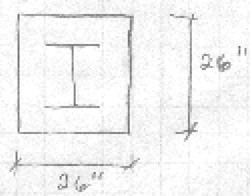
$P_p = 0.85 f'_c A_1$ $\quad \Omega_c = 2.50$

$$A_1 = \frac{675^{K}(2.50)}{0.85(3^{ksi})} \Rightarrow 662\,in^2 \approx (26\,in)^2$$

Try 26" x 26" Plate

W12 x 106
$b_f = 12.2"$

$d = 12.9"$



$$m = \frac{N - 0.95d}{2} \Rightarrow \frac{26" - 0.95(12.9")}{2} = 6.87"$$

$$n = \frac{B - 0.8b_f}{2} \Rightarrow \frac{26" - 0.8(12.2")}{2} = 8.12"$$

$$n' = \frac{\sqrt{d\,b_f}}{4} = \frac{\sqrt{(12.9)(12.2)}}{4} = 3.14"$$

Assume $\lambda = 1.0$
conservative

$$t_{min} = l\sqrt{\frac{3.33 P_a}{F_y B N}}$$

$$= 8.12"\sqrt{\frac{3.33(675^{K})}{(36)(26)(26)}}$$

$t_{min} = 2.48\,in$

Use 2 1/2" Base Plate 26in x 26in

Pier 26in x 26in x

FTG Design | Thesis | Thomas Sabol | 8

Footing Design: $q_a \geq \dfrac{P}{A_{Footing}} \Rightarrow 12\ ksf \geq \dfrac{675^K}{B^2}$

$B \geq 7.5' \Rightarrow$ use $B = 8\ ft$

crit. face $\ell = \dfrac{8' - 26''\left(\frac{1'}{12''}\right)}{2} = 2.92'$

$$M_a = \frac{12\ ksf\,(2.92)^2}{2} = 51^{1K}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} \Rightarrow \frac{A_s\, 60}{0.85(3)(12)}$$

$$a = 1.96\, A_s$$

$\Omega = 1.67$ $\quad M_a = M_n/\Omega = \frac{1}{\Omega} A_s f_y \left(d - \frac{a}{2}\right)$

$$51^{1K}\left(12\ in/ft\right) = \left(\frac{1}{1.67}\right) A_s (60) \left(24.8 - \frac{1.96 A_s}{2}\right)$$

$\Omega = 2.00$ $\quad V_c = \frac{1}{\Omega}\, 4 \sqrt{f'_c}$

$$= \frac{1}{2.00}(4)\sqrt{3000\ psi}$$

$$= 109.5\ psi$$

2-way Shear $\quad d^2\left(V_c + \frac{q}{4}\right) + d\left(V_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$

$$\Rightarrow d^2\left(110 + \frac{83.3}{4}\right) + d\left(110 + \frac{83.3\ psi}{2}\right)26 = \frac{83.3}{4}\left[(96\ in)^2 - (26\ in)^2\right]$$

$$130.8\, d^2 + 3942.9\, d - 177{,}845.5 = 0$$

✓ $d = 24.8''$

$$h = d + 3'' + d_b$$

$$= 24.8 + 3'' + 0.625'' = 28.4 \Rightarrow \text{Use } 29''$$

$$17 = 24.8 A_s - 0.98 A_s^2$$

$$A_s = 0.71\ in^2$$

Use #8 @ 12 in o.c. $\quad A_s = 0.79\ in^2$

Thomas Sabol (3)

$\rho = \frac{A_s}{bh} = \frac{0.711 in^2}{(12")(29")} = 0.0020 \geq 0.0018 \therefore ok$

$a = 1.46(0.79/2) = 1.55$

$c = \frac{1.55}{0.85} = 1.82"$

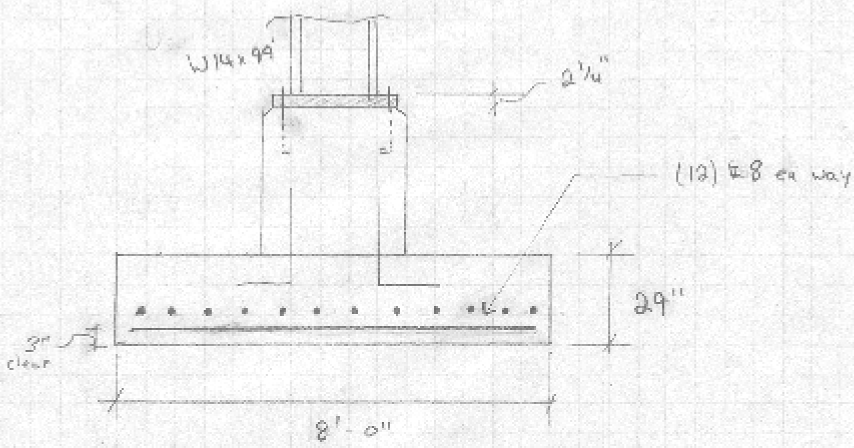
$\varepsilon_s = \frac{0.003}{1.82"}(24.8 - 1.82") = 0.038 \geq 0.005 \therefore ok$

Use (12) #8 ea. way

$B_n/\Omega = 1/\Omega \; 0.85 f'_c A_1$
$= (1/2.50)(0.85)(3ksi)(26")^2$
$= 689.5^k$

$B_n/\Omega \geq P = 675^k$

$A_{s_{min}} = 0.005 A_{pier}$
$= 0.005(26)^2$
$= 3.38 in^2 \therefore ok$



8' x 8' FTG Vol. = 155 ft³

Original size => 10' x 10' x 40" => Vol. = 333.3 ft³

Vol. ratio = $\frac{155}{333.3} \times 100 = 47\%$

FTG volume decreased by 53%

FTG Design | | Thomas Sabol | 4

FTG Col line D-3 => 134.4^K

$$12^{ksf} > \frac{134.4^K}{B^2}$$

$$B \geq 3.3' \Rightarrow \text{Use } 4'$$

Pier Size

$$A_1 = \frac{134.4K\,(2.50)}{0.85(3^{ksi})} = 131.8\,in^2 \approx (12in)^2$$

$$v_c = 110\,psi$$

$$d^2\left(110 + \frac{83.3}{4}\right) + d\left(110 + \frac{83.3}{2}\right)12 = \frac{83.3}{4}\left[(48)^2 - (12)^2\right]$$

$$\Rightarrow 130.8d^2 + 1819.8d - 44982 = 0$$

$$d = 12.85''$$

$$h = 12.85'' + 3'' + 0.625 = 16.48'' \Rightarrow \text{Use } 17''$$

FTG size = 4' x 4' x 17" => 22.7 ft³
Original size = 5' x 5' x 22" => 45.83 ft³

$$\text{Vol ratio} = \frac{22.7}{45.83} \times 100 = 50\%$$

Vol Saving of 50%

FTG Col line J-6 => 508.5^K

$$B = \sqrt{\frac{508.5^K}{12^{ksf}}} = 6.5'$$

Pier size

$$A_1 = \frac{508.5^K(2.50)}{(0.85)(3^{ksi})} = 498.5\,in^2 \approx (23in)^2$$

$$d^2\left(110 + \frac{83.3}{4}\right) + d\left(110 + \frac{83.3}{2}\right)23 = \frac{83.3}{4}\left[(78)^2 - (23)^2\right]$$

$$\Rightarrow 130.8d^2 + 3487.9d - 115821.8 = 0$$

$$d = 19.3''$$

$$h = 19.3 + 3'' + 0.625'' = 22.925 \Rightarrow \text{Use } 23''$$

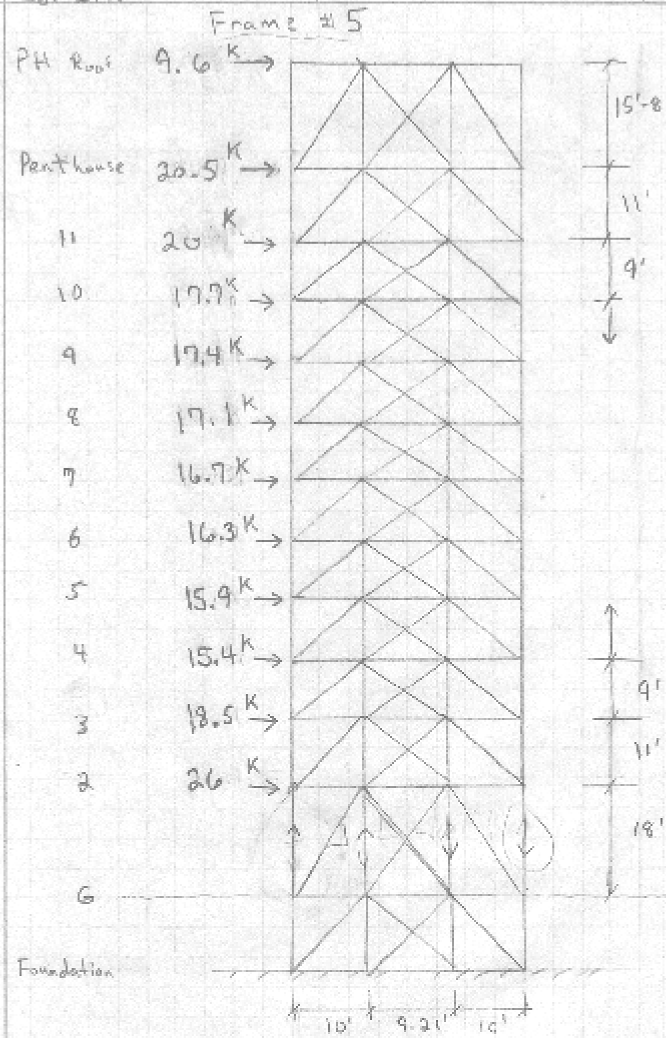
FTG size = 6'-6" x 6'-6" x 23" => 81 ft³
original size = 8'-6" x 8'-6" x 35" => 210.7 ft³

$$\text{Vol ratio} = \frac{81}{210.7} \times 100 = 38\%$$

Savings of 62%

Lateral — Thomas Sabol — 1

Frame #5

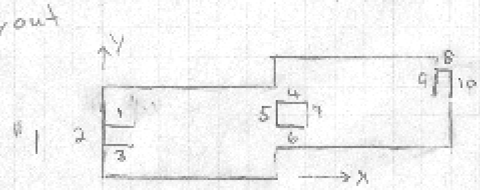


Wind Loads (Kips) are Wind Case 1 Y-Dir

Beams and Braces are Pin-Pin

Wind Forces from RAM x Relative rigidity hand calced

Frame layout



Δ from RAM Advance

Frame	1K Δ (in)	Rigidity (1/Δ)
2	0.12132	8.24
5	0.02103	47.6
7	0.06101	16.4
9	0.03905	25.6
10	0.03905	25.6
11	—	—

Relative Rigidity Frame #5

$$\Rightarrow \frac{47.6}{123.5} = 0.39$$

Lateral | Thomas Sabol | 2

D + W

D + 0.75W + 0.75L

Column: G - (between 6.3 & 5.2)
155.2 ft - 119.12 ft

at 2nd Level
DL = 223 K
LL = 69.8 K

Wind Axial = 499.3 K

Size W 18 x 175 $I = 3450 in^4$
$L_b = 18'$

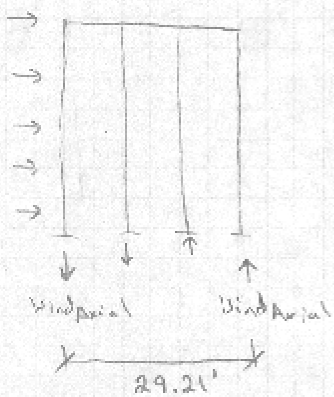
$$F_e = \frac{\pi^2 (29000)}{\left(\frac{0.7(18 \times 12)}{8.2}\right)^2} = 842 \text{ Ksi}$$

$F_e \geq 0.44 F_y$

$$F_{cr} = \left[0.658^{\frac{50}{842}}\right] 50 = 48.8 \text{ Ksi}$$

$P_n/\Omega = F_{cr} A_g/\Omega \Rightarrow 48.8(51.3 in^2)/1.67 = 1499 K$

$M_n/\Omega = 408 'K$



Moment = 14585 'K (From excel spreadsheet)

$$\text{Wind}_{Axial} = \frac{14585 'K}{29.21'} = 499.3 K$$

D + W = 223 K + 499.3 K = 722.3 K ✓ controls

D + 0.75 + 0.75W ⇒ 223 + 0.75 (69.8) + 0.75 (499.3) = 649.8 K

$$M = \frac{26K \times 18'}{4} = 117 'K$$

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}}$$

$$\Sigma P_{e2} = R_m \frac{\Sigma H L}{\Delta H}$$

$$= 1.0 \left[\frac{211K (18' \times 12)}{0.077}\right]$$

= 591 896

$$B_2 = \frac{1}{1 - \frac{1.0(722.3)}{591\,896}} = 1.00$$

Lateral Thomas Sobol (3)

$M_r = 117^{'k}$

$P_r = 722.3^{k}$

$\frac{P_r}{P_c} = \frac{722.3^{k}}{1499^{k}} = 0.48 > 0.2 \quad (H1-1a)$

$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\frac{M_{rx}}{M_{cx}} \Rightarrow 0.48 + \left(\frac{8}{9}\right)\frac{117}{908} = 0.59 < 1$

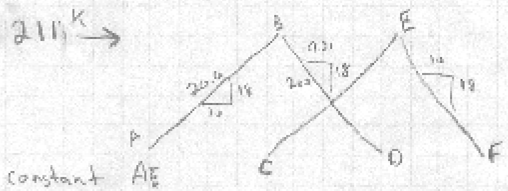
$\therefore$ ok

RAM output 0.68 ok

Lateral Columns D + W controls.

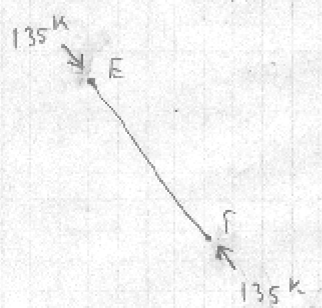
Lateral — Thomas Sabol — 6

Load combos: D + L
D + 0.75 W + 0.75 L



constant AE

	AB	BD	CE	EF
X component	$\frac{10}{20.6}$ ⇓ 0.49	$\frac{9.21}{20.2}$ ⇓ 0.46	$\frac{9.21}{20.2}$ ⇓ 0.46	$\frac{10}{20.6}$ ⇓ 0.49
Force Dist. (X-Direction)	$\frac{0.49}{1.9}$ ⇓ 0.26	$\frac{0.46}{1.9}$ ⇓ 0.24	$\frac{0.46}{1.9}$ ⇓ 0.24	$\frac{0.49}{1.9}$ ⇓ 0.26
	54.9k	50.6k	50.6	54.9



D+L controls

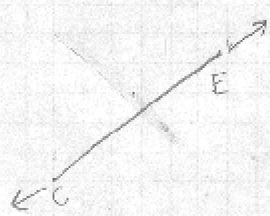
Axial Force from wind ⇒ $\frac{54.9}{0.49} = 112^K$

$DL \approx 11^K\left(\frac{20.6}{10}\right) \Rightarrow 22.7^K$

$D + W = 135^K$

RAM output 141^K

HSS 7×7×1/2 ⇒ $P_n/\Omega = 173^K$ in compression
$L_b = 20.0'$
∴ ok



RAM output
$D + W = 80.4^K$

HSS 7×5×½

P_n/Ω @ 21' ⇒ 84.3^K ∴ ok