Hilton Baltimore Convention Center Hotel

Western Podium

Baltimore, MD

CHRIS SIMMONS

Structural Option

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Technical Report 1

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EXECUTIVE SUMMARY

The structural concepts and existing conditions report describes the structural system of the Hilton Baltimore Convention Center Hotel. This 21-story hotel, is located right in the heart of Baltimore next to Inner Harbor and Camden Yards, makes use of spread footings, caissons as a foundation, concrete flat plate slabs as well as concrete slabs with drop panels, reinforced concrete shear walls for the elevator shafts and stairwells, and a variety of sizes of concrete columns. Typical bay sizes are for the concrete columns are 26'-10" x 18'-8" for exterior bays and 26'-10" x 19'-7" for interior bays.

Gravity and lateral loads were calculated using ASCE 7-05 and compared to the loads determined by RTKL Associates. The controlling lateral load was found to be the seismic force with a base shear, V=1146.9k. The wind force in the North/South direction had a base shear, V=939.71k, while the East/West direction had a base shear, V=165k.

Spot checks were conducted on the beam 5-B16, and on the column L-11. These checks supported that the determination and accumulation of the gravity loads on this structure were comparable to those done by RTKL. The only columns that did not work while using ACI 318-08 equation 10-2 were the column on the first floor and the mezzanine floor. Reasons for this are assuming that the tributary area was the same for all floors for that column and certain design loads used varied from the design loads used by RTKL. It is to be remembered that lateral forces were not included in these calculations.

INTRODUCTION

The Hilton Baltimore Convention Center Hotel (HBCCH) is located right in downtown Baltimore next to the Baltimore Orioles stadium Camden Yards, and located blocks away from Inner Harbor. HBCCH is broken up into two podiums, East and West. The eastern podium is a 4-story building that houses a junior ballroom, meeting rooms and a multipurpose restaurant. The western podium is a 21-story building that houses the main hotel lobby, parking garage, grand ballroom with corresponding kitchen, meeting rooms, pool/health club, and 757 hotel rooms. The grand ballroom has moveable partitions located in the ceiling that allow multiple events to take place there. The western podium offers over 900,000 SF of hotel space. The structure of the western podium consists of concrete beams, columns and shear walls to resist lateral loading. The green roof above the grand ballroom is supported by special joists and while the pool above the grand ballroom is supported by steel beams.

Photo of Grand Ballroom. (Note Partition tracks in ceiling)

FOUNDATION SYSTEMS

The foundation of the western podium consists of caissons and spread footings. The spread footings will bear on firm natural soils and have a minimum bearing capacity of 4ksf. The drilled caissons will have straight shafts to bear on gneiss rock and have a minimum safe bearing capacity of 100ksf. The depths of the bottoms of the caissons vary from 14 feet all the way up to 32 feet below level B2's floor slab. The compressive strength of the drilled caissons and spread footings are 3500 psi, while the caisson caps that the columns bear on have a compressive strength of 4000 psi.

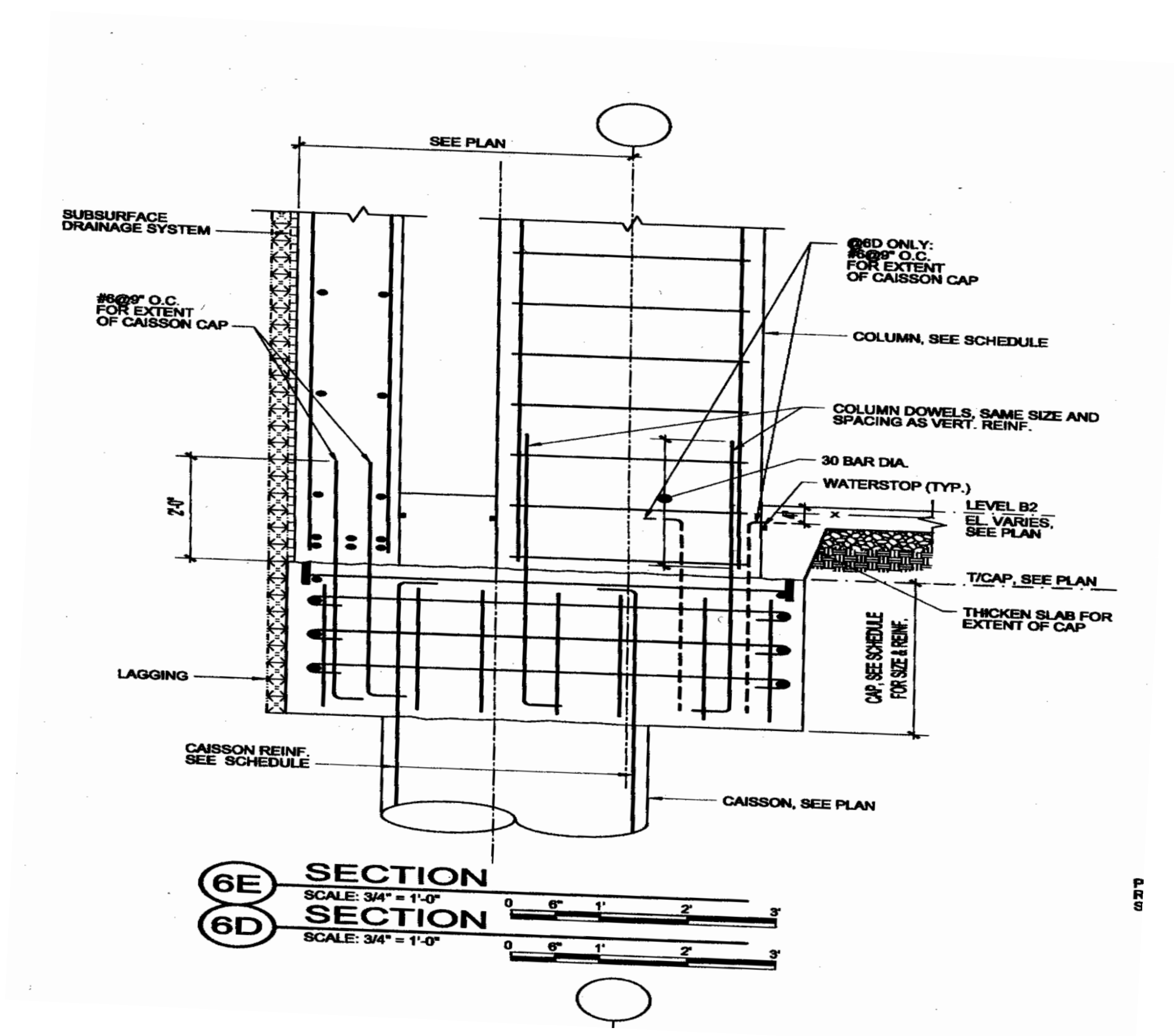


Fig. 1 – Typical Caisson Section

FLOOR SYSTEM

The floor system consists of two-way slabs whose thicknesses range from 8" thick on the floors with hotel rooms to 11" in the underground parking garage. The slabs shall are reinforced with 6x6-W1.4xW1.4 WWF, except for the slab-on-grade which is reinforced with 6x6-W2.1xW2.1 WWF as seen in Fig. 2. Drop panels are located on the B1, 1st, Mezzanine level, 2nd, 3rd, and 15th floors. The drop panels vary from 5" up to 11" in thickness. Typical spans for floors consisting of hotel rooms are 26'-10".

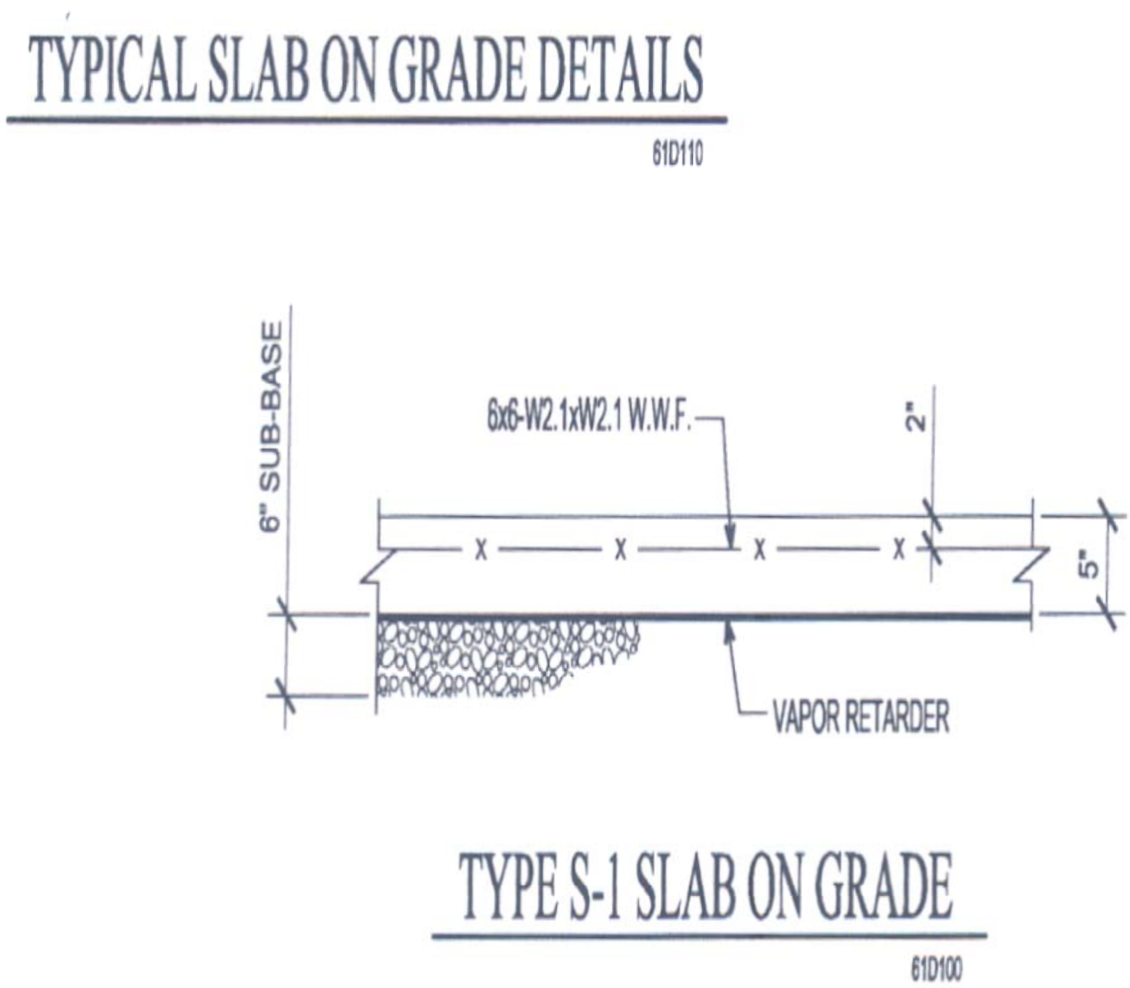


Fig. 2 Slab on Grade Detail

ROOF SYSTEM

As shown in Fig. 3, the roof system either type R-1 or R-2 roof construction. Type R-2 roof construction is used for the green roof above the grand ballroom and exercise room while type R-1 roof construction is used for the roofs located on the 15th and 21st floors. Fig. 4 shows the transition from the green roof assembly to the corresponding roof construction assembly.

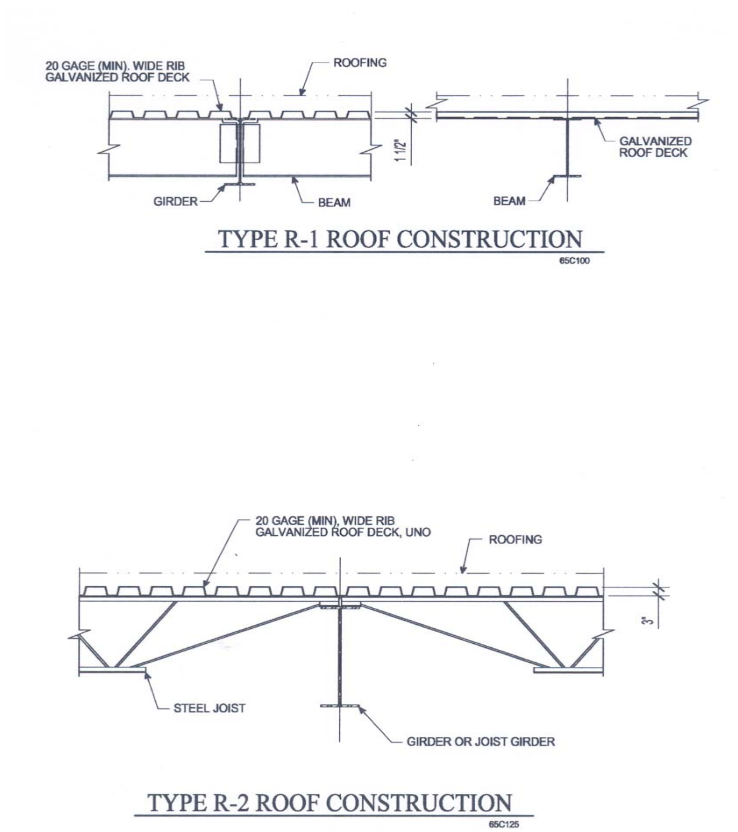


Fig. 3 Roof Construction Details

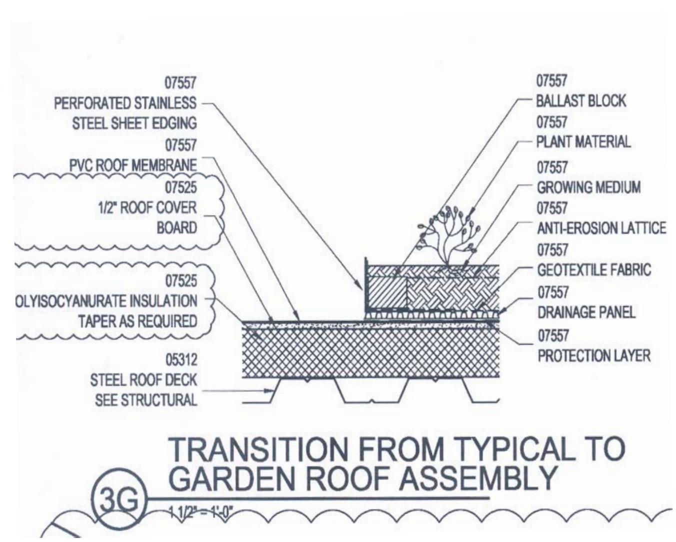


Fig. 4 Green Roof Transition

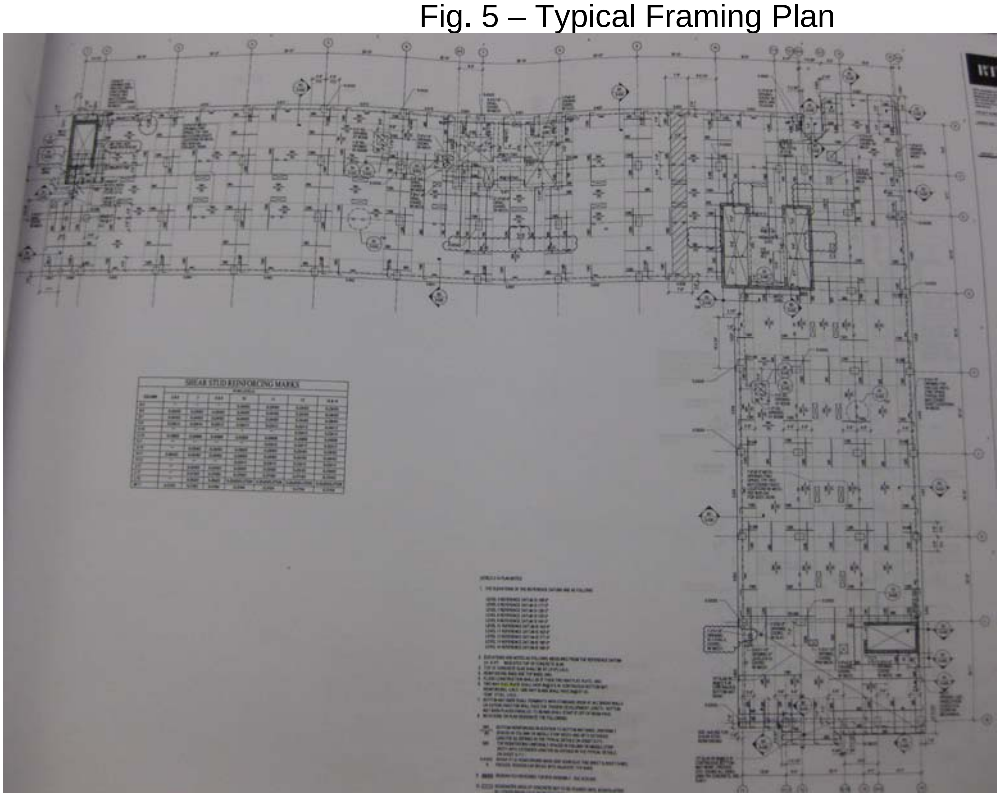
LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consist of a number of steel brace frames that are located between the 3rd floor and the 4th floor roof. The load that is from wind and seismic is also transferred from each individual floor to the concrete beams and then to the concrete columns. A shear wall system is used as well for resisting wind and seismic loads. Shear walls are located around the elevator shafts and stairwells.

COLUMN SYSTEM

The layout of the column system is a very uniform layout consisting of typical exterior bays of 26’-10” x 18’-8” and interior bays of 26’-10” x 19’-7”. All columns consist of either a gravity resisting member or a combination of lateral and gravity resisting members. Column sizes vary from 12” x 18” columns to 44” x 30” Columns. Sloped columns can be found on the second and third floors of the western podium. A typical framing plan is shown in fig. 5.

Fig. 5 – Typical Framing Plan



APPLICABLE CODE

Design Codes used for Original Design:

- International Building Code, 2000 Edition
- City of Baltimore Code
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, 13th Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-08

Code Substitutions/ Additional References used for Thesis Design:

- International Building Code, 2006 Edition
- Reinforced Concrete: Mechanics & Design, Fifth Edition
  - By: James K. Wight, James G. MacGregor

MATERIALS AND PROPERTIES

Steel:

W and WT Shapes	A992
Channels	A36
Angles	A36
Rectangulare & Square HSS	A500 Grade B
Round HSS	A500 Grade B
Steel Pipe	A53, Grade B
Steel Plates	A36
Steel Bars	A36
High Strength Bolts	A325 or A490
Anchor Bolts	F1154 Grade 36
Standard Fasteners	A307

Concrete:

Drilled Caissons	f'c = 3500 psi
Spread Footings	f'c = 3500 psi
Grade Beams & Caisson Caps	f'c = 4000 psi
Typical SOG	f'c = 4000 psi
Columns	f'c = 4000 psi
Shear Walls	f'c = 4000 psi
Normal Weight Slab on Metal Decc	f'c = 3500 psi

Reinforcement:

Composite Floor Deck	A611 Grade C Min. A653 Quality SS Grade 33
Roof Deck	A653 Quality SS Grade 33
Deformed Bars	A615 Grade 60
Deformed Bars (weldable)	A706
Welded Wire Fabric	A185

Note: Material strengths are based on American Society for Testing and Materials (ASTM) standard ratings.

DESIGN LOADS

All of the design loads for this technical report werr calculated referencing ASCE 7-05: *Minimum Design Loads for Buildings and Other Structures*. All dead, live and snow loads can be seen in Table 1 below.

Table 1 - Gravity Loads			
Description	ASCE 7-05	RTKL Value	Design Value
DEAD (DL)			
Concrete	150pcf	150pcf	150pcf
Ceiling, Mech, Ducts, etc.	20-25psf	5-20psf	20psf
LIVE (LL)*			
Private Hotel Rooms	40psf	40psf	40psf
Ballroom	100psf	100psf	100psf
Corridors (first floor, main lobby)	100psf	100psf	100psf
Corridors (serving private hotel rooms)	100psf	40psf	100psf
Aerobic/Exercise Rooms	100psf	100psf	100psf
Pool Deck	75psf	80psf	75psf
Green Roof	100psf	100psf	100psf
Exterior Balconies (East Tower)	100psf	100psf	100psf
Roofing	20psf	30psf	20psf
SNOW (S)			
Snow	20psf	20psf	20psf
* Live load reductions were not taken into consideration in the design.			

Almost all lateral loads were calculated by hand and inserted into the tables on the following pages. The hand calculations can be found on pages 31-35 in the Appendix. After calculating the lateral loads it was concluded that Seismic controlled over Wind.

WIND LOADS

Wind loads were determined using ASCE 7-05 Section 6.5. Section 6.5 describes Method 2-Analytical Procedure. The variables used in this analysis are located below in Table 2a and these values are supported by base calculations which are located in the Appendix on pages 32 and 33.

Table 2a- Wind Variables			ASCE References
Basic Wind Speed	V	90 mph	Fig. 6-1
Directionality Factor	k_d	0.85	Table 6-4
Importance Factor	I	1.15	Table 6-1
Exposure Category		B	Sec. 6.5.6.3
Topographic Factor	K_{zt}	1	Sec. 6.5.7.1
Velocity Pressure Exposure Coefficient evaluated at Height z	K_z	Varies	Table 6-3
Velocity Pressure at Height z	q_z	Varies	Eq. 6-15
Velocity at Mean Roof Height	q_h	24.522	Eq. 6-15
Gust Effect Factor (North/South)	G_f	0.827069	Eq. 6-8
Gust Effect Factor (East/West)	G_f	0.894305	Eq. 6-8
External Pressure Coefficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coefficient (N/S Leeward)	C_p	-0.5	Fig. 6-6
External Pressure Coefficient (E/W Leeward)	Cp	-0.2	Fig. 6-6

Table 2b was developed to determine the wind pressures in the North/South direction as well as the wind pressure on the parapet. Figure 6 shows both the windward and leeward pressures on the building.

Table 2b									
Wind Loads (North/South Direction) B=293'-2 1/2", L=60'-6 1/2									
Floor	Level (ft)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Total Pressure(k)	Story Shear Total(k)	Moment Total (ft-k)
				Windward	Leeward				
1	0	0.57	11.55	0	0	0	0	939.71	0
Mezzanine	10	0.57	11.55	12.06	-14.55	26.61	39.02	939.71	9397.08
2	20	0.62	12.57	12.06	-14.55	26.61	39.02	900.69	18013.84
3	38	0.75	15.19	13.22	-14.55	27.78	73.30	861.68	32743.69
4	56	0.84	16.97	14.78	-14.55	29.34	77.41	788.37	44148.79
5	65	0.87	17.71	15.77	-14.55	30.32	40.01	710.96	46212.26
6	74	0.91	18.38	16.25	-14.55	30.80	40.64	670.95	49650.09
7	83	0.94	18.99	16.68	-14.55	31.23	41.21	630.31	52315.55
8	92	0.96	19.56	17.08	-14.55	31.63	41.73	589.10	54197.00
9	101	0.99	20.09	17.44	-14.55	32.00	42.22	547.36	55283.70
10	110	1.02	20.58	17.79	-14.55	32.34	42.67	505.14	55565.69
11	119	1.04	21.05	18.11	-14.55	32.67	43.10	462.47	55033.67
12	128	1.06	21.49	18.42	-14.55	32.97	43.50	419.37	53678.91
13	137	1.08	21.92	18.71	-14.55	33.26	43.89	375.86	51493.14
14	146	1.10	22.32	18.98	-14.55	33.54	44.25	331.98	48468.53
15	155	1.12	22.70	19.25	-14.55	33.80	44.60	287.73	44597.61
16	164	1.14	23.07	19.50	-14.55	34.05	44.93	243.13	39873.28
17	173	1.16	23.43	19.74	-14.55	34.29	45.25	198.20	34288.71
18	182	1.17	23.77	19.97	-14.55	34.53	45.56	152.95	27837.35
19	191	1.19	24.10	20.20	-14.55	34.75	45.85	107.40	20512.90
Roof	203	1.21	24.52	20.43	-14.55	34.98	61.55	61.55	12493.92
∑Story Shear(Total) =			939.71	k	∑Moment (Total) =			805805.7007	ft-k

PARAPET on North/South Elevations only			
Flowchart 5.7			
Top height=222'-7-3/4"	Kp	1.24	
	qp	25.18	
	Parapet Height	19' 7-3/4"	19.65
GCpn	Wind.	1.5	
	LeeW.	-1	
pp	Wind.	37.7668	
	LeeW.	-25.1779	
F	Wind.(plf)	741.9602583	
	LeeW.(plf)	-494.6408271	
	Total(plf)	1236.601085	

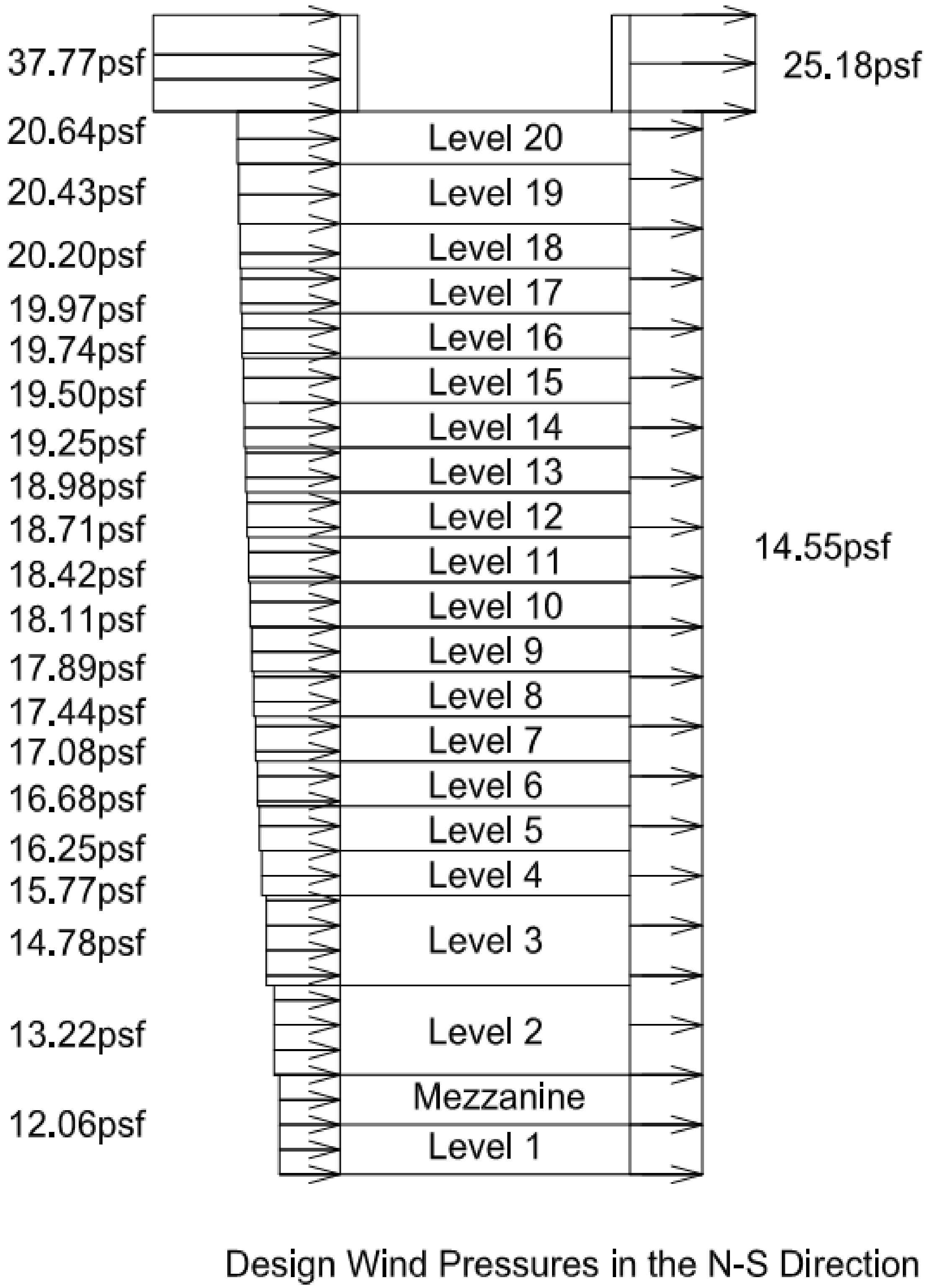


Fig. 6 – North/South Wind Pressures

Table 2c was developed to determine the wind pressures in the East/West direction. Figure 7 shows the windward and leeward pressures on the building.

Table 2c
Wind Loads (East/West Direction) B=60'-6 1/2", L=293'-2 1/2"

Floor	Level (ft)	K_z	q_z	Wind Pressure (psf)		Total Pressure (psf)	Force of Total Pressure(k)	Story Shear Total(k)	Moment Total (ft-k)
				Windward	Leeward				
1	0	0.57	11.55	0	0	0	0	164.97	0
Mezzanine	10	0.57	11.55	12.68	-8.80	21.48	6.50	164.97	1649.67
2	20	0.62	12.57	12.68	-8.80	21.48	6.50	158.46	3169.29
3	38	0.75	15.19	13.94	-8.80	22.74	12.39	151.96	5774.57
4	56	0.84	16.97	15.62	-8.80	24.42	13.31	139.57	7816.02
5	65	0.87	17.71	16.69	-8.80	25.49	6.95	126.26	8207.15
6	74	0.91	18.38	17.21	-8.80	26.01	7.09	119.32	8829.58
7	83	0.94	18.99	17.68	-8.80	26.48	7.21	112.23	9315.35
8	92	0.96	19.56	18.11	-8.80	26.91	7.33	105.02	9661.86
9	101	0.99	20.09	18.50	-8.80	27.30	7.44	97.69	9866.72
10	110	1.02	20.58	18.88	-8.80	27.68	7.54	90.25	9927.69
11	119	1.04	21.05	19.23	-8.80	28.03	7.64	82.71	9842.70
12	128	1.06	21.49	19.56	-8.80	28.36	7.73	75.08	9609.80
13	137	1.08	21.92	19.87	-8.80	28.67	7.81	67.35	9227.15
14	146	1.10	22.32	20.17	-8.80	28.97	7.89	59.54	8693.00
15	155	1.12	22.70	20.45	-8.80	29.25	7.97	51.65	8005.68
16	164	1.14	23.07	20.72	-8.80	29.52	8.04	43.68	7163.60
17	173	1.16	23.43	20.98	-8.80	29.78	8.11	35.64	6165.24
18	182	1.17	23.77	21.24	-8.80	30.04	8.18	27.52	5009.14
19	191	1.19	24.10	21.48	-8.80	30.28	8.25	19.34	3693.89
Roof	203	1.21	24.52	21.73	-8.80	30.53	11.09	11.09	2251.41
Σ Story Shear(Total) =			164.97	k	Σ Moment (Total) =			143879.5271	ft-k

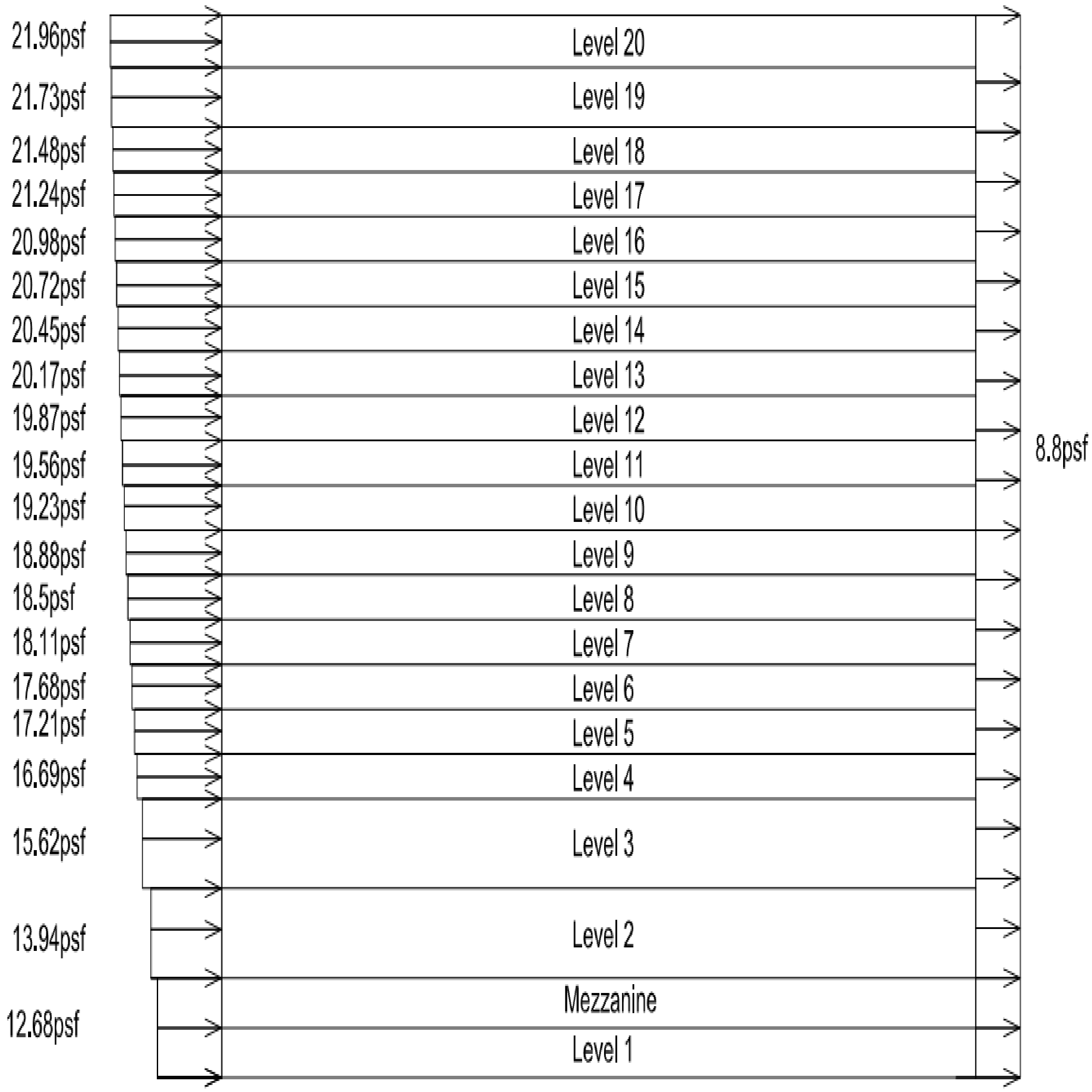


Fig. 7 East/ West Wind Pressures

SEISMIC LOADS

In order to calculate the seismic forces on HBCCH, Chapters 11 and 12 were referenced from ASCE 7-05. HBCCH was assumed to have a rigid diaphragm which allowed for the use of the Equivalent Lateral Force Procedure found in Section 12.8 of ASCE 7-05. The variables used in this procedure are located in Table 3a.

Table 3a - Seismic Design Variables			ASCE Reference
Soil Classification		B	Table 20.3-1
Occupancy		II	Table 1-1
Importance Factor		1.00	Table 11.5-1
Structural System		Building Frame System: Ordinary Reinforced Concrete Moment Frames	Table 12.2-1
Spectral Response Acceleration, short	S_s	0.169	USGS
Spectral Response Acceleration, 1 s	S_1	0.051	USGS
Site Coefficient	F_a	1.2	Table 11.4-1
Site Coefficient	F_v	1.7	Table 11.4-2
MCE Spectral Response Acceleration, short	S_{MS}	0.2028	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 s	S_{M1}	0.0867	Eq. 11.4-2
Design Spectral Acceleration, short	S_{DS}	0.1352	Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{D1}	0.0578	Eq. 11.4-4
Seismic Design Category	S_{DC}	A	Table 11.6-2
Response Modification Coefficient	R	5	Table 12.2-1
Approximate Period Parameter	C_t	0.016	Table 12.8-2
Building Height (above grade)	h_n	203 ft	
Approximate Period Parameter	x	0.9	Table 12.8-2
Calculated Period Upper Limit Coefficient	C_u	1.7	Table 12.8-1
Approximate Fundamental Period	T_a	1.9s	Eq. 12.8-7
Fundamental Period	T	1.9s	Sec. 12.8.2
Long Period Transition Period	T_L	8.00s	Fig. 22-15
Seismic Response Coefficient	C_s	0.01	Eq. 12.8-2
Structure Period Exponent	k	2	Sec. 12.8.3

The seismic design variables Z, I, S, R_w, h_n, and C can be found in the Appendix on pages 34 and 35. Figure 8 shows the story forces and the story shear found during seismic design.

Table 3b				
Floor	Fx(kips)	hx	Story Shear, Vx (kips)	Moment (k-ft)
20(Roof)	117.9019	203	-	-
19	99.4425	191	117.90	23934.09496
18	93.4499	182	217.34	18993.51465
17	83.7862	173	310.79	17007.8849
16	88.6247	164	394.58	14495.00432
15	99.8772	155	483.21	14534.44983
14	92.3279	146	583.08	15480.96431
13	81.2959	137	675.41	13479.87914
12	70.9655	128	756.71	11137.5363
11	61.3607	119	827.67	9083.58616
10	52.4098	110	889.03	7301.922678
9	44.1845	101	941.44	5765.081968
8	36.6609	92	985.63	4462.636902
7	29.8390	83	1022.29	3372.802515
6	23.7187	74	1052.13	2476.6333
5	18.4380	65	1075.85	1755.183753
4	28.4069	56	1094.28	1198.471032
3	18.2595	38	1122.69	1590.787444
2	4.9817	20	1140.95	693.8591252
Mezzanine	0.9710	10	1145.93	99.63328606
Base	1146.90	-	1146.90	9.709846175
			Overturning Moment(k-ft)	166873.6364

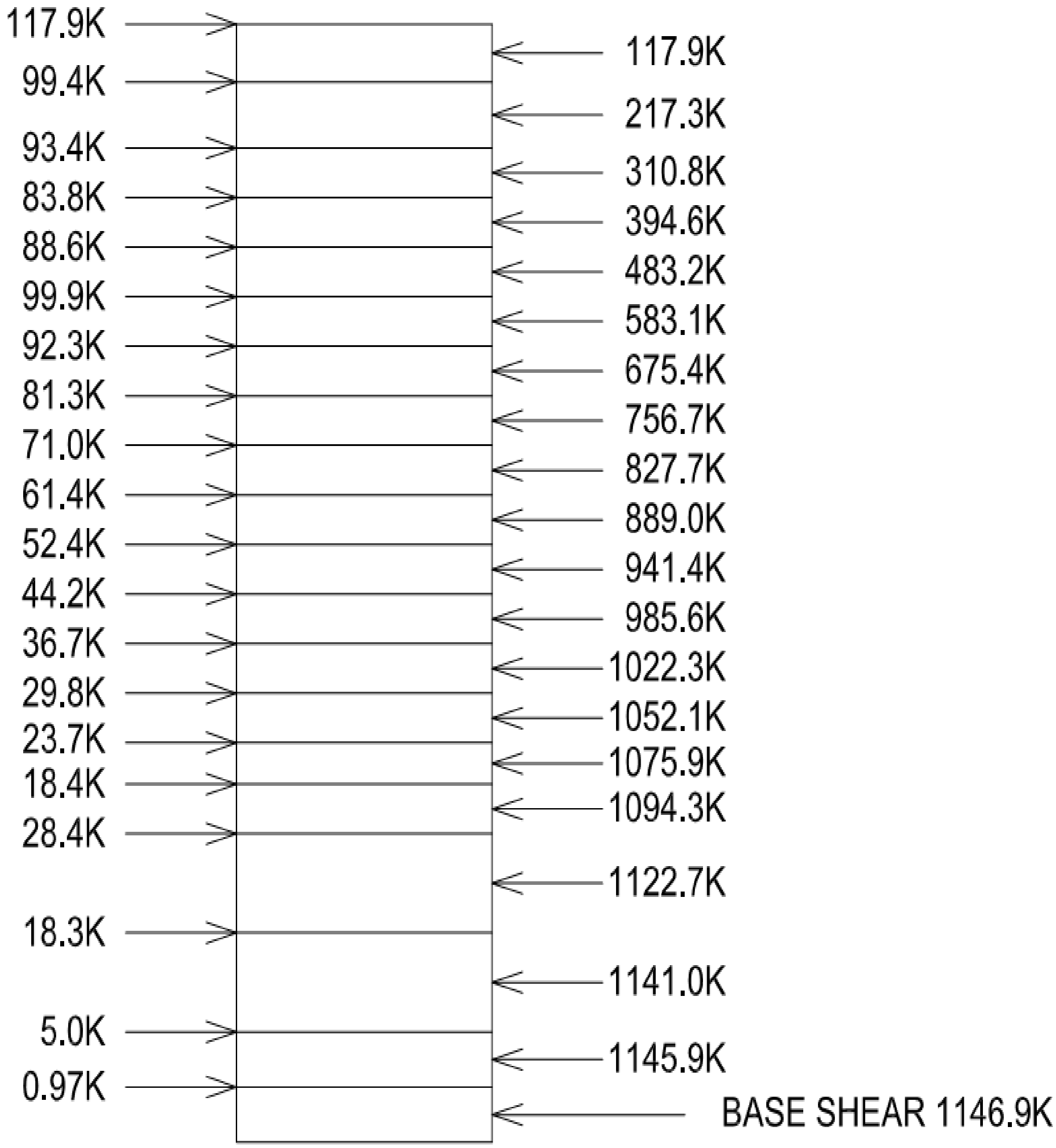


Fig. 8 Seismic Design Diagram

SPOT CHECKS

In order to verify that the loads determined via Technical Report 1 were reasonable and adequate, spot checks of typical framing were conducted. These spot checks were imperative in being able to compare the calculations done in this report to the design of HBCCH by RTKL. Only gravity loads were applied during these calculations and therefore at least some variation could be attributed to the fact that lateral loads will also be present and require inclusion in analysis. The typical framing elements were taken from Floor 5 and include a check of beam 5-B16 (Figure 9) and column L-11 (Figure 10).

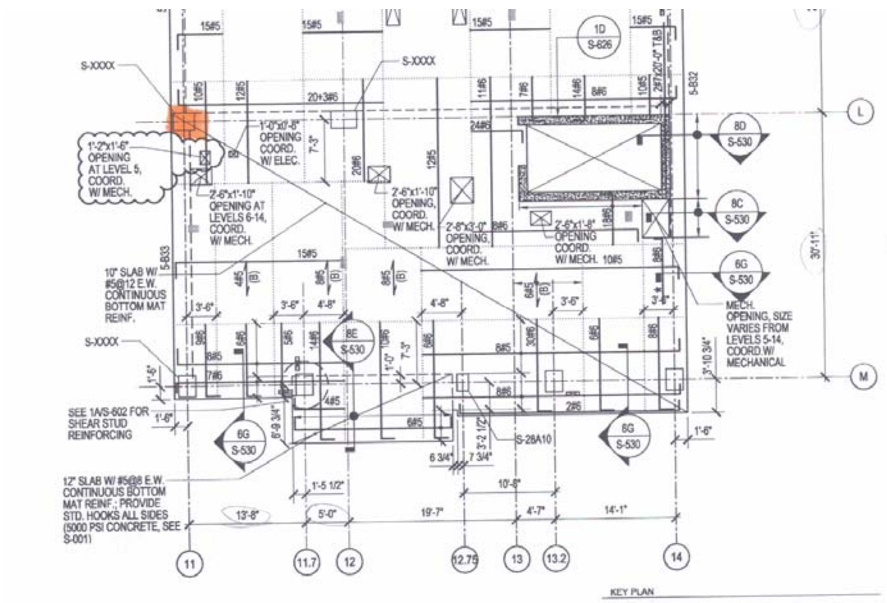


Fig. 10 Column L-11
(Column is highlighted)

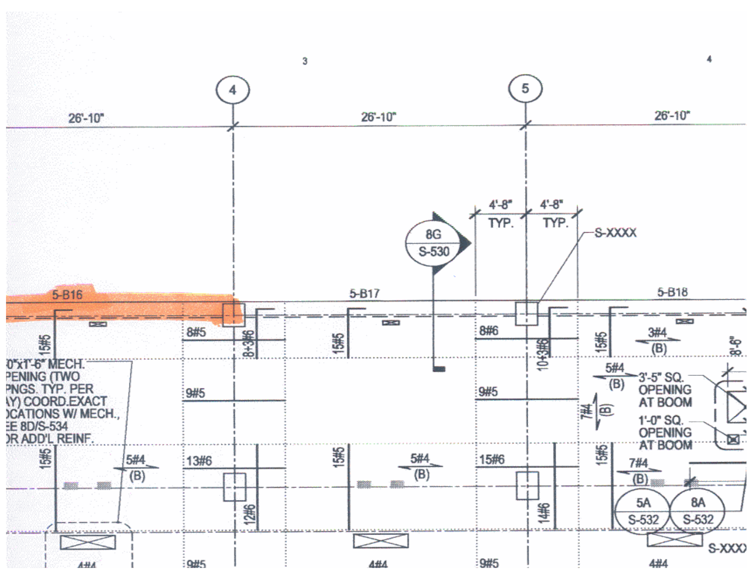


Fig. 9 Beam 5-B16
(Beam is highlighted)

Table 4a - Spot Check - Column L-11							
Level Supported	Tributary Area (sf)	Dead Load (psf)	Live Load (psf)	Snow Load (psf)	Total Load (1.4DL) kips	Total Load (1.2DL+1.6LL +0.5S) kips	Total Load (1.2DL+1.6S+LL) kips
14	312.8	120	140	0	52.55	115.11	88.84
13	312.8	120	140	0	52.55	115.11	88.84
12	312.8	120	140	0	52.55	115.11	88.84
11	312.8	120	140	0	52.55	115.11	88.84
10	312.8	120	140	0	52.55	115.11	88.84
9	312.8	120	140	0	52.55	115.11	88.84
8	312.8	120	140	0	52.55	115.11	88.84
7	312.8	120	140	0	52.55	115.11	88.84
6	312.8	120	140	0	52.55	115.11	88.84
5	312.8	120	140	0	52.55	115.11	88.84
4	312.8	120	40	0	52.55	65.06	57.56
3	312.8	132.5	100	0	58.02	99.78	81.02
2	312.8	132.5	100	0	58.02	99.78	81.02
Mezzanine	312.8	132.5	100	0	58.02	99.78	81.02
1	312.8	132.5	100	0	58.02	99.78	81.02

Table 4b Column Load Take Down			
Level Supported	Total Load (1.4DL) kips	Total Load (1.2DL+1.6LL +0.5S) kips	Total Load (1.2DL+1.6S+LL) kips
14	52.55	115.11	88.84
13	105.10	230.22	177.67
12	157.65	345.33	266.51
11	210.20	460.44	355.34
10	262.75	575.55	444.18
9	315.30	690.66	533.01
8	367.85	805.77	621.85
7	420.40	920.88	710.68
6	472.95	1035.99	799.52
5	525.50	1151.10	888.35
4	578.05	1216.17	945.91
3	636.08	1315.95	1026.92
2	694.10	1415.73	1107.94
Mezzanine	752.13	1515.52	1188.95
1	810.15	1615.30	1269.97

CONCLUSION

After examination of the existing structural system and calculations of various gravity and lateral loads of the Hilton Baltimore Convention Center Hotel, it was found that the hotel was adequately designed to withstand these forces. Through the use of ASCE 7-05 to calculate wind loads via Method 2 and seismic loads via the Equivalent Lateral Force Procedure, the controlling force was Seismic with a base story shear, V=1146.90k.

Spot checks done on beam 5-B16 and the exterior column L-11 on floor five also proved that the determination and accumulation of loads done within this report were comparable to those completed by RTKL. These two components were found to be satisfactorily designed. The only discrepancy found was for the first floor and mezzanine column L-11. This could've been because of differences in values for loads or because of a difference in tributary area calculated for those two columns. Also due to not including lateral forces in the calculations larger moments would be added to the columns and beams. As research continues for Hilton Baltimore Convention Center Hotel lateral forces will be taken into account and will have an effect on the framing, shear walls and foundations.

APPENDIX

DEAD WEIGHT CALCULATIONS

Assume Beams are 29' in length, 24" in Width and 32" in depth

Dead Loads

MEP = 15 psf - includes rooftop mechanical units

Green Roof = 109 psf

Roofing and Insulation = 15 psf

Walls = 20 psf

Perimeters (ft)	Areas (sf)
Floors Mez 2 (33)	
258'-8" + 293'-2½" + 258'-8" + 293'-2½" = 1103'-9"	1103'-9" × 33 = 36423.75 sf
Floor 3 (18)	
(202'-1¾") × 2 + (288'-8") × 2 = 981'-4½"	981'-4½" × 18' = 17669.25 sf
Floors 4-14 (9)	
212'-6⅝" + 60'-6½" + 141'-5" + 233'-1⅜" + 60'-6½" + 289'-3⅞" ≈ 997'6"	997'6" × 9 × 11 floors = 98752.5 sf
Floors 15-16 (9)	
(293'-7⅞" + 60'-6½") × 2 = 708'-4¾"	708'-4¾" × 9 × 2 = 12751.125 sf
Floors 17-19 (9)	
(261'-11⅝") × 2 + (62'-4¾") × 2 = 649'2¾"	649'-2¾" × 9 × 2 = 11686.125 sf

Green Roof Area Lvl 4

144'-2" × (58'-2" + 58'2" + 29'3¼") = 28271.46 sf

Roof Level Area

(26'-10" + 14'-7" + 18'-8" + 10'-⅞") × (214'10" + 10'6½" + 20'-6" + 8'3" + 7'6⅝" + 6'9" + 18'8") = 20141.42 sf

Roof Level Area Lvl 15

(130'-4 1/4") x 60'-6 1/2" = 7891.86 sf

MEP Area

258'8" x 293'2 1/2" = 78843.22 sf

Beam Weight $(29 \times \frac{24}{12} \times \frac{32}{12}) = 154.67$ ft³ x 150 = 23200

Mezzanine

\# of beams = 102 102 beams x 154.67 x 150 pcf = 2366400 lbs = 2366.4^k

Floor 2

\# of beams = 140 140 beams x 154.67 x 150 = 3248000 lbs = 3248^k

Floor 3

\# of beams = 163 163 beams x 23200 = 3781600 lbs. = 3781.6^k

Floor 4

\# of beams = 84 84 x 23200 = 1948800 lbs. = 1948.8^k

Floor 5-14

\# of beams = 47 47 x 23200 = 1090400 lbs. = 1090.4^k

Floor 15

\# of beams = 45 45 x 23200 = 1044000 lbs. = 1044^k

Floor 16

\# of beams = 46 46 x 23200 = 1067200 lbs. = 1067.2^k

Floor 17, 19

\# of beams = 37 37 x 23200 = 858400 lbs. = 858.4^k

Floor 18

\# of beams = 38 38 x 23200 = 881600 lbs. = 881.6^k

Floor 20

\# of beams = 31 31 x 23200 = 719200 lbs. = 719.2^k

Floor 21

\# of beams = 6 6 x 23200 = 139200 lbs. = 139.2^k

Building Dead Weight

Floor slabs

Total weight = 69181.08 kips

Drop Panels

Total weight = 144.18 kips

Walls

Total weight = (36423.75 + 17669.25 + 98752.5 + 12751.125 + 11686.125) x 20 psf

= 3656.03 kips

Roofs

Green Roof Lvl 4 = 28271.46 x 109 psf = 3081.59 kips

Roof Lvl 20 = 20141.42 x 15 psf = 302.12 kips

Note: concrete slab for roofs already accounted for in slab calcs.

Roof Lvl 15 = 7891.86 x 15 psf = 119.43 k

Note: concrete slab already accounted for in slab calcs.

MEP

78843.22 sf x 15 = 1182.65 kips

Columns

$\frac{1}{2}(916.23) + 777.58 + 1825.88 + 4327.54 + 209.85 + 130.07 = 7729.04^k$

Beams

= 2366.4 + 3248 + 3781.6 + 1948.8 + 10(1090.4) + 1044 + 1067.2 + 2(858.4) + 881.6 + 719.2 + 139.2

= 27816.8 k

Total Dead Weight = 113102.85 k

Table 5

	Height (ft)	12x18 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	18x18 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
Level 20	10.5	1	1.5	15.75	2.36	1	2	24	4
Level 19	12	1	1.5	18	2.70	1	2	27	4
Levels 18-4	9	15	22.5	202.5	30.38	15	34	304	46
Levels 3-2	18	0	0	0	0	12	27	486	73
Level Mezz.	10	0	0	0	0	5	11	113	17
Level 1	10	0	0	0	0	8	18	180	27
Level B1	16	0	0	0	0	7	16	252	38
Level B2	10	0	0	0	0	6	14	135	20
Total		17	26	236	35	55	124	1520	228

	Height (ft)	16x24 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	22x24 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
Level 20	10.5	0	0	0	0	0	0	0	0
Level 19	12	0	0	0	0	0	0	0	0
Levels 18-4	9	12	32	288	43.2	0	0	0	0
Levels 3-2	18	0	0	0	0	3	11	198	29.7
Level Mezz.	10	0	0	0	0	0	0	0	0
Level 1	10	0	0	0	0	0	0	0	0
Level B1	16	0	0	0	0	0	0	0	0
Level B2	10	0	0	0	0	0	0	0	0
Total		12	32	288	43.2	3	11	198	29.7

	Height (ft)	30x30 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	30x36 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
Level 20	10.5	0	0	0	0	0	0	0	0
Level 19	12	0	0	0	0	0	0	0	0
Levels 18-4	9	0	0	0	0	0	0	0	0
Levels 3-2	18	0	0	0	0	1	7.5	0	0
Level Mezz.	10	0	0	0	0	2	15	0	0
Level 1	10	0	0	0	0	1	7.5	0	0
Level B1	16	1	6.25	100	15	4	30	30	4.5
Level B2	10	1	6.25	62.5	9.38	4	30	30	4.50
Total		2	12.5	162.5	24.38	12	90	60	9.00

	Height (ft)	44x30 Columns No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
Level 20	10.5	0	0	0	0
Level 19	12	0	0	0	0
Levels 18-4	9	0	0	0	0
Levels 3-2	18	0	0	0	0
Level Mezz.	10	1	9.17	91.67	13.75
Level 1	10	1	9.17	91.67	13.75
Level B1	16	0	0	0	0
Level B2	10	0	0	0	0
Total		2	18.33	183.33	27.50

Table 5 (con't)

18x24 Columns				18x30 Columns				18x36 Columns			
No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
21	63	662	99	0	0	0	0	0	0	0	0
31	93	1116	167	0	0	0	0	0	0	0	0
415	1245	11205	1681	0	0	0	0	0	0	0	0
9	27	486	73	3	11	203	30	0	0	0	0
23	69	690	104	4	15	150	23	5	23	380	57
21	63	630	95	0	0	0	0	11	50	1337	200
26	78	1248	187	0	0	0	0	16	72	2722	408
26	78	780	117	0	0	0	0	17	77	1549	232
572	1716	16817	2522	7	26	353	53	49	221	5987	898

24x24 Columns				24x30 Columns				24x36 Columns			
No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
1	4	42	6.3	0	0	0	0	0	0	0	0
2	8	96	14.4	0	0	0	0	0	0	0	0
244	976	8784	1317.6	73	365	3285	492.75	35	210	1890	283.5
11	44	792	118.8	56	280	5040	756	46	276	4968	745.2
5	20	200	30	28	140	1400	210	36	216	2160	324
5	20	200	30	20	100	1000	150	43	258	2580	387
3	12	192	28.8	18	90	1440	216	44	264	4224	633.6
3	12	120	18	16	80	800	120	45	270	2700	405
274	1096	10426	1563.9	211	1055	12965	1944.75	249	1494	18522	2778.3

36x36 Columns				12x71 Columns				12x137 Columns			
No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
0	0	0	0	2	11.83	124.25	18.64	0	0	0	0
0	0	0	0	2	11.83	142	21.3	0	0	0	0
0	0	0	0	8	47.33	426	63.9	24	274	2466	369.9
0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0	0	0
1	9	90	13.5	0	0	0	0	0	0	0	0
4	36	576	86.4	0	0	0	0	0	0	0	0
4	36	360	54.00	0	0	0	0	0	0	0	0
9	81	1026	153.90	12	71.00	692.25	103.84	24	274	2466	369.9

	Total Columns			
Level	No.	Total Area (sf)	Volume (ft^3)	Weight (kips)
Level 20	26	82.58	867.13	130.07
Level 19	37	116.58	1399.00	209.85
Levels 18-4	841	3205.58	28850.25	4327.54
Levels 3-2	141	683.75	12172.50	1825.88
Level Mezzanine	109	517.92	5183.85	777.58
Level 1	111	534.17	6108.17	916.23
Level B1	123	604.00	10783.60	1617.54
Level B2	122	602.25	6536.63	980.49
Total	1510	6347	71901	10785

Table 6								
Level	Thickness (in)	Thickness(ft)	Length	Width	Area(sf)	Dry Unit Wgt(pcf)	Total Load(lbs)	Total Load (kips)
Mezzanine	9	0.75	256.61|188.875	253.75	56520.90938	150	6358602.30	6358.60
2	9	0.75	261.6666667	291.5833333	76297.63889	150	8583484.38	8583.48
3	9	0.75	257.4583333	289.4583333	74523.46007	150	8383889.26	8383.89
4.1	8	0.666666667	59.91666667	288.09375	17261.61719	150	1726161.72	1726.16
4.2	8	0.666666667	56.1	208.0416667	11671.1375	150	1167113.75	1167.11
4.3	8	0.666666667	287.75	42.6	12258.15	150	1225815.00	1225.82
5.1	8	0.666666667	59.9	233.1	13962.69	150	1396269.00	1396.27
5.2	8	0.666666667	299.8125	54.97	16480.69313	150	1648069.31	1648.07
6.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
6.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
7.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
7.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
8.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
8.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
9.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
9.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
10.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
10.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
11.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
11.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
12.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
12.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
13.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
13.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
14.1	8	0.666666667	59.91666667	211.9895833	12701.7092	150	1270170.92	1270.17
14.2	8	0.666666667	228.1770833	76.22604167	17393.03586	150	1739303.59	1739.30
15.1	8	0.666666667	59.91666667	212.0416667	12704.82986	150	1270482.99	1270.48
15.2	8	0.666666667	228.1770833	67.125	15316.38672	150	1531638.67	1531.64
16	8	0.666666667	294.625	67.125	19776.70313	150	1977670.31	1977.67
17	8	0.666666667	254.25	67.125	17066.53125	150	1706653.13	1706.65
18	8	0.666666667	254.25	67.125	17066.53125	150	1706653.13	1706.65
19	8	0.666666667	254.25	67.125	17066.53125	150	1706653.13	1706.65
20	8	0.666666667	254.25	67.125	17066.53125	150	1706653.13	1706.65
Total					665893.0464		69181079.75	69181.08
Note: Floors 4-15 were broken into multiple areas for ease of calculation								

Table 7								
Level	Thickness(in)	# of Panels	Area/Panel (sf)	Total Area (SF)	Volume (ft^3)	Unit Weight (pcf)	Weight (lbs)	Weight (kips)
Mezzanine	5	1	66.5	66.5	27.71	150	4156.25	4.16
	9	2	78	156	117.00	150	17550.00	17.55
2	6	2	84	168	84.00	150	12600.00	12.60
	9	1	162.65	162.65	121.99	150	18298.44	18.30
	9	4	100	400	300.00	150	45000.00	45.00
	11	2	100	200	183.33	150	27500.00	27.50
3	6	1	138	138	69.00	150	10350.00	10.35
	6	1	65.16666667	65.16666667	32.58	150	4887.50	4.89
	6	1	27.19444444	27.19444444	13.60	150	2039.58	2.04
15	4	1	36	36	12.00	150	1800.00	1.80
						Total	144181.77	144.18

Table 8								
Floor Dead Weight								
Level	Walls	MEP	Columns	Beams	Slabs	Drop Panels	Roof	Total
Mezzanine	331.13	59.13	1235.69	2366.4	6358.60	21.71	0	10372.66
2	397.35	59.13	912.94	3248	8583.48	103.40	0	13304.30
3	353.39	59.13	912.94	3781.6	8383.89	17.28	0	13508.22
4	179.55	59.13	288.50	1948.8	4119.09	0	3081.59	9676.67
5	179.55	59.13	288.50	1090.4	3044.34	0	0	4661.92
6	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
7	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
8	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
9	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
10	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
11	179.55	59.13	288.50	1090.4	3009.47	1.80	0	4628.86
12	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
13	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
14	179.55	59.13	288.50	1090.4	3009.47	0	0	4627.06
15	127.51	59.13	288.50	1044	2802.12	0	119.73	4441.00
16	127.51	59.13	288.50	1067.2	1977.67	0	0	3520.02
17	77.91	59.13	288.50	858.4	1706.65	0	0	2990.60
18	77.91	59.13	288.50	881.6	1706.65	0	0	3013.80
19	77.91	59.13	209.85	858.4	1706.65	0	0	2911.94
20	0	59.13	130.07	858.4	1706.65	0	302.12	3056.37
Total	3545.66	1182.65	7729.02	27816.80	69181.08	144.18	3503.44	113102.83

Snow Load Calcs (ASCE 7-05)

- Determine Ground Snow Load, p_g
  - → Figure 7.1 Baltimore, MD ⇒ $\boxed{p_g = 25\text{ psf}}$
- Determine Snow Density
  - $\gamma = .13p_g + 14 \leq 30\text{ pcf}$
  - $\gamma = .13(25) + 14 = 17.3\text{ pcf} \leq 30 \therefore \text{ok}$
  - $\boxed{\gamma = 17.3\text{ pcf}}$
- Importance Category: II
- Importance Factor = 1.0
- Thermal Factor = 1.0 = C_t
- Exposure Factor = 1.0 = C_e ← partially exposed due to parapet
- Sloped Roof Factor = 1.0 = C_s

- Flat Roof Snow Load
  - $p_f = 0.7C_eC_tIp_g$
  - $p_f = .7(1.0)(1.0)(1.0)(25) = 17.5\text{ psf}$
  - $p_{f_{min}} = 20(I)$ where $p_g > 20\text{ psf}$
  - $p_{f_{min}} = 20(1.0) = \boxed{20\text{ psf}}$

Design Roof Snow Load

$$p_s = p_f \cdot C_s = 20(1.0) = \boxed{20\text{ psf} = p_s}$$

N-S Direction

$n_1 = 100/H = 100/203 = .492 \le 1 \therefore$ flexible

$g_Q = g_v = 3.4$

$g_r = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 4.02$

$\bar{z} = 0.6h \ge z_{min} = 30$

$0.6(203) = 121.8 \ge 30 \therefore \bar{z} = 121.8$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{121.8}\right)^{1/6} = .241$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{121.8}{33}\right)^{1/3} = 494.5$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = .783$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 82.33$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.96$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = 0.07$

$R_h = \frac{1}{n_h} - \frac{1}{2n_h^2}\left(1 - e^{-2n_h}\right) = 0.163$

$n_h = 4.6 n_1 h / \bar{V}_{\bar{z}} = 5.59$

$R_B = \frac{1}{n_B} - \frac{1}{2n_B^2}\left(1 - e^{-2n_B}\right) = 0.116$

$n_B = 4.6 n_1 B / \bar{V}_{\bar{z}} = 8.07$

$R_L = \frac{1}{n_L} - \frac{1}{2n_L^2}\left(1 - e^{-2n_L}\right) = 0.163$

$n_L = 15.4 n_1 L / \bar{V}_{\bar{z}} = 5.58$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + .47 R_L)} = 0.201$

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) = 0.827$

$c = 0.30$ Table 6-2

$\ell = 320$ Table 6-2

$\bar{\epsilon} = 1/3.0$ Table 6-2

$\bar{b} = 0.45$ Table 6-2

$\bar{\alpha} = 1/4.0$ Table 6-2

$B = 293' 2.5''$

$L = 60' 6.5''$

$V = 90$ mph

$\beta = 0.02$

E-W Direction ($B = 60'6.5''$, $L = 293'2.5''$)

$n_1 = .492$ ∴ flexible

$g_r = 4.02$ $g_Q = g_v = 3.4$

$\bar{z} = 121.8$

$I_{\bar{z}} = .241$

$L_{\bar{z}} = 494.5$

$Q = .838$

$\bar{V}_{\bar{z}} = 82.33$

$N_1 = 2.96$

$R_n = 0.07$

$R_h = 0.163$

$n_h = 5.59$

$R_B = .426$

$n_B = 1.67$

$R_L = 0.036$

$n_L = 27.02$

$R = .366$

$G_f = 0.894$

Flowchart 6.8

$S_S = 0.169$

$S_1 = 0.051$

$S_{DS} = .1352$

$S_{D1} = .0578$

SDC = Category A

$R = 3$

$I = 1.0$

$T_a = 1.9s = T$

$C_u = 1.7$

$C_u T_a = 1.9(1.7) = 3.23 > 1.9 \therefore T = 1.9s$

$T_L = 8s$

$T < T_L$

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

$$= \frac{.0578}{1.9\left(\frac{3}{1}\right)} \leq \frac{.1352}{\left(\frac{3}{1}\right)}$$

$$= .0101 \leq .045 \therefore ok$$

$C_S = .0101$

Weight

$W = 102361.64^k$

Base Shear

$V = C_S W$

$= .0101(113102.85^k) = 1146.9^k$

$K = 2$

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$

Seismic Ground Motion Values

$S_S = 0.169$
$S_1 = 0.051$

Site Classification of Soil

Site Class C

S_{MS} and S_{M1} $F_a = 1.2$ $F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.169) = .2028$
$S_{M1} = F_v S_1 = 1.7(0.051) = .0867$

S_{DS} and S_{D1}

$S_{DS} = 2S_{MS}/3 = 2(.2028)/3 = .1352$
$S_{D1} = 2S_{M1}/3 = 2(.0867)/3 = .0578$

Occupancy Category

II

Not Permitted by 12.14

Seismic Design Category

Category A

$T_S = S_{D1}/S_{DS} = .0578/.1352 = .4275$

$T_a = C_t h_n^x = 0.016 \cdot (203)^{0.9} = 1.9$

$x = 0.9$
$C_t = 0.016$
$h_n = 203'$

$T < 3.5T_s$
$1.9 < 3.5(.4275)$
$1.9 \not< 1.49$

∴ Use
• Modal Response spectrum
• Seismic Response History Procedures
• Equivalent Lateral Force Procedure
· Simplicity Reasons

$$b_{eff} \le \begin{cases} b_w + 6h_f & 21 + 6(8) = 69'' \\ b_w + \frac{1}{12}\text{span dist} & 21 + \frac{(26\times12)+10}{12} = 47.83'' \approx 47'' \end{cases}$$

span (26'-10'') (10'-4'')

$b_w = 21$ in

$h_f = 8$ in

$h = 13.75$ in

$d = 11.5$ in

$b_{eff} = 47''$

$\text{slab} = \frac{8(150)}{12} = 100 \times (10.33) = 1033 \text{ lb/ft}$

$\text{Beam} = \frac{21(13.75-8)(150)}{144} = 125.8 \text{ lb/ft}$

Live Load $= 40 \text{ psf} \times 10.33 = 413.2$ plf

$w_u = 1.2(1033 + 125.8) + 1.6(413.2) = 2051.68$

$M_u = \frac{w_u l^2}{8} = \frac{2051.68(26.88)^2}{8} = 184.6$ k-ft

$M_{n,T\text{-}Bm} = \phi \cdot 0.85 f'_c \cdot b \cdot h_f (d - h_f/2)$

$= 0.65 \cdot (.85)(4) \cdot (47) \cdot (8)(11.5 - 8/2) = 519$ k-ft

$M_u < M_{n,T\text{-}BM}$ Design as rectangular beam

$a = 8''$ $\beta_1 = .85$ $c = 9.4''$

$\varepsilon_t = \frac{.003}{9.4}(11.5 - 9.4) = .0006 < .005$

$\therefore \phi = .65$

$\rho_{max\,\phi} = 0.85\,\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + .005}$

$= 0.85(.85)\frac{4}{60}\left(\frac{.003}{.008}\right) = .0181$

$A_{s_1} = 0.0181(47)(11.5) = 9.76 \text{ in}^2$

$a = \frac{A_s \cdot f_y}{0.85 f'_c \cdot b} = \frac{9.76(60)}{.85(4)(47)} = 3.67''$

$c = 3.67/.85 = 4.31''$

$M_{n_1} = A_s f_y (d - \frac{a}{2}) = 9.76(60)(11.5 - \frac{3.67}{2}) = 471.65$ k-ft

$\varepsilon_t = 0.003\left(\frac{11.5 - 4.31}{4.31}\right) = .005 = .005 \quad \therefore \phi = 0.9$

$\phi M_n = 0.9(471.65) = 424^{k\text{-}ft} > 184.6^{k\text{-}ft}$ $\therefore$ singly-reinforced

$A_s = \frac{M_u}{4d} = \frac{184.6}{4(11.5)} = 4.01 \Rightarrow (4)\ \#9$

$a = \frac{4.01(60)}{0.85(4)(47)} = 1.5$

$c = 1.5/.85 = 1.77$

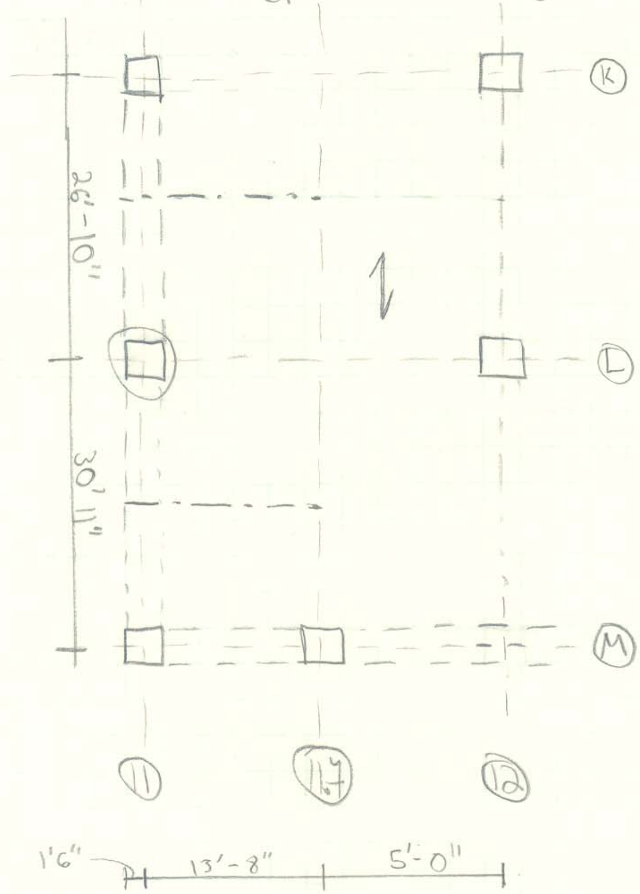
$\varepsilon_s \ge \varepsilon_y$

$\varepsilon_y = \frac{60}{29000} = .002$

$\frac{.003}{1.77}(11.5 - 1.77) = .016 > .002 \therefore$ assumptions verified

Column L-11 Spot Check

Typical Column layout



$$A_T = \left(1'\text{-}6" + \frac{18'\text{-}8"}{2}\right) \times \left(\frac{30'\text{-}11"}{2} + \frac{26'\text{-}10"}{2}\right) = 312.8 \text{ sf}$$

* Equation 10-2 – ACI 318-08

$$\phi P_{n,max} = 0.8\phi\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right]$$

* All loads are located in Table 4b (using Load Combination 1.2DL + 1.6LL + 0.5S)

Level Supported 14

As designed by RTKL = 24"x18" $A_g = 432 in^2$
4#10
$f'_c = 4 ksi$ $f_y = 60 ksi$ $A_{st} = 4(1.27) = 5.08 in^2$

$\phi P_n = 0.8(0.65)[0.85(4)(432-5.08) + 60(5.08)] = 913.3^k$

$P_u = 115.11^k < \phi P_{n,max}$ ∴ okay

Level Supported 13

As designed by RTKL = 24"x18" $A_g = 432 in^2$
4#10
$f'_c = 4 ksi$ $A_{st} = 5.08 in^2$
$f_y = 60 ksi$

$\phi P_{n,max} = 913.3^k$

$P_u = 230.22^k < 913.3^k$ ∴ ok

Level Supported 12

As designed by RTKL = 24"x18"
4#10 $A_g = 432 in^2$
$f'_c = 4 ksi$ $A_{st} = 5.08 in^2$
$f_y = 60 ksi$

$\phi P_{n,max} = 913.3^k$

$P_u = 345.33^k < 913.3^k$ ∴ ok

Level Supported 11

As designed by RTKL = 24"x18"
4#10 $A_g = 432 in^2$
$f'_c = 4 ksi$ $A_{st} = 5.08 in^2$
$f_y = 60 ksi$

$\phi P_{n,max} = 913.3^k$

$P_u = 460.44^k < 913.3^k$ ∴ ok

Level Supported 10

As designed by RTKL = 24"x18" $A_g = 432\,in^2$
4#10
$f'_c = 4\,ksi$ $A_{st} = 5.08\,in^2$
$f_y = 60\,ksi$

$\phi P_{n,max} = 913.3^k$

$P_u = 575.55^k < 913.3^k \therefore$ okay

Level Supported 9

As designed by RTKL = 24"x18" $A_g = 432\,in^2$
4#10
$f'_c = 4\,ksi$ $A_{st} = 5.08\,in^2$
$f_y = 60\,ksi$

$\phi P_{n,max} = 913.3^k > 690.66^k = P_u \therefore$ ok

Level Supported 8

As designed by RTKL = 24"x18" $A_g = 432\,in^2$
4#10
$f'_c = 4\,ksi$ $A_{st} = 5.08\,in^2$
$f_y = 60\,ksi$

$\phi P_{n,max} = 913.3^k > 805.77^k = P_u \therefore$ ok

Level Supported 7

As designed by RTKL = 24"x18"
4#11 $A_g = 432\,in^2$
$f'_c = 4\,ksi$
$f_y = 60\,ksi$ $A_{st} = 4 \times 1.56 = 6.24\,in^2$

$\phi P_{n,max} = 0.8(.65)[0.85(4)(432-6.24)+60(6.24)] = 947.4^k$

$P_u = 920.88^k < 947.4^k \therefore$ ok

Level Supported 6

As designed by RTKL = 24"x24"
4 #11
$f'_c = 4$ ksi
$f_y = 60$ ksi

$A_g = 24 \times 24 = 576$ in²
$A_{st} = 6.24$ in²

$\phi P_{n,max} = 0.8(.65)[0.85(4)(576-6.24)+60(6.24)] = 1202^k$

$P_u = 1035.99^k < 1202^k$ ∴ ok

Level Supported 5

As designed by RTKL = 24"x24"
8 #9
$f'_c = 4$ ksi
$f_y = 60$ ksi

$A_g = 576$ in²
$A_{st} = 8(1.00) = 8$ in²

$\phi P_{n,max} = 0.8(.65)[0.85(4)(576-8)+60(8)] = 1253.8^k$

$P_u = 1151.1^k < 1253.8^k$ ∴ ok

Level Supported 4

As designed by RTKL = 24"x24"
8 #9
$f'_c = 4$ ksi
$f_y = 60$ ksi

$A_g = 576$ in²
$A_{st} = 8$ in²

$\phi P_{n,max} = 1253.8^k$
$P_u = 1216.17^k < 1253.8^k$ ∴ ok

Level Supported 3

As designed by RTKL = 30"x24"
8 #9
$f'_c = 4$ ksi
$f_y = 60$ ksi

$A_g = 30 \times 24 = 720$ in²
$A_{st} = 8$ in²

$\phi P_{n,max} = 0.8(.65)[0.85(4)(720-8)+60(8)] = 1508.4^k$

$P_u = 1315.95^k < 1508.4^k$ ∴ ok

Level Supported 2

As designed by RTKL = 30" × 24"　　$A_g = 720\ in^2$
8 #9
$f'_c = 4\ ksi$　　$A_{st} = 8\ in^2$
$f_y = 60\ ksi$

$\phi P_{n,max} = 1508.4^k$
$P_u = 1415.73^k < 1508.4^k$ ∴ OK

Level Supported Mezzanine

As designed by RTKL = 30" × 24"
8 #9　　$A_g = 720\ in^2$
$f'_c = 4\ ksi$
$f_y = 60\ ksi$　　$A_{st} = 8\ in^2$

$\phi P_{n,max} = 1508.4^k$

$P_u = 1515.52^k > 1508.4^k$ ∴ Not OK

Level Supported 1

As designed by RTKL = 30" × 24"
8 #9　　$A_g = 720\ in^2$
$f'_c = 4\ ksi$
$f_y = 60\ ksi$　　$A_{st} = 8\ in^2$

$\phi P_{n,max} = 1508.4^k$

$P_u = 1615.30^k > 1508.4^k$ ∴ Not OK